EURO:TUN 2013

Computational Methods in Tunneling

and Subsurface Engineering
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Preface

The intelligent use of the underground space opens perspectives and new opportunities to meet a number of challenging requirements of modern society. The increasing demand for infrastructure in urban areas as well as the need for efficient, environmentally friendly and economically viable high performance transportation routes worldwide has been fostering the construction of underground facilities and tunnels. New projects are being developed in increasingly challenging geological conditions and within sensitive environments that would have been regarded impossible years ago.

Computational models and methods, together with advanced exploration and monitoring techniques are well established in the field of tunneling and subsurface engineering. Yet, the limited availability of information on the subsurface space imposes an inherent uncertainty to be considered in the design and construction of underground facilities. Furthermore, new technologies in subsurface engineering add their share to the need for developing new computational methods to capture the manifold interactions involved in underground construction.

EURO:TUN 2013 is the third conference of a series of successful conferences started in 2007 in Vienna, Austria. EURO:TUN 2013 expands the range of topics from the specific area of simulation models for tunneling towards computational models and methods for related areas of subsurface engineering such as mining, caverns and subsurface storage facilities. Like the previous conferences, EURO:TUN 2013 aims to provide a forum for the discussion, assessment and review of latest advancements in research, new developments and applications of computational models and methods in tunneling and subsurface engineering. Furthermore it will provide an overview of the current state of the research and future perspectives of numerical modeling and computational methods in underground construction.
The proceedings of EURO:TUN 2013 provide a collection of current research and ambitious projects and are pointing towards the future development of computational engineering sciences in the area of underground construction. The organizers of EURO:TUN 2013 would like to express their gratitude to the members of the Scientific Advisory Board, to the European Community on Computational Methods in Applied Sciences (ECCOMAS) and to the Collaborative Research Center “Interaction Modeling in Mechanized Tunneling” (SFB 837). Special thanks are due to the members of the local organization committee of EURO:TUN 2013 (T. Barciaga, M. Breyer, S. Kunter, J. Stascheit, S. Schützner and G. Vollmann) for their assistance in the preparation and realization of the conference.

Last but not least, the editors would like to express their thanks to all authors and delegates of the conference who are the foundation of a successful, inspiring and enjoyable event.

G. Meschke, J. Eberhardsteiner, T. Schanz, K. Soga and M. Thewes

Conference Chairmen

Bochum, Cambridge, Vienna
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Innovations in Tunnelling Construction Management: Applications of Simulation

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Abstract

Simulation-based innovations in construction management as it applies to tunnel construction are discussed in this paper. To outline these innovations, the paper first discusses a discrete event simulation environment useful for modelling construction operations, 4D modelling, and a distributed simulation framework. Secondly the paper overviews specific application in construction management of tunnel projects which make use of these innovations.

Simphony can be used to develop special purpose simulation tools as well as develop generic process interaction models of construction operations using generic modelling elements. To demonstrate the practicality and usefulness of this environment we provide an overview of a special purpose simulator for tunnel construction that was developed and implemented using this approach.

COSYE is a distributed environment which is useful for integrating various simulation components and approaches. We demonstrate the effectiveness of COSYE by integrating various construction management applications which we highlight in the paper including scenario-based planning, operation analysis during construction, collecting as-built information, and facilitating 4D modelling.

Keywords: Simulation of tunnel operations, construction management, project planning.
OVERVIEW OF ADVANCEMENTS IN SIMULATION TOOLS FOR CONSTRUCTION MANAGEMENT WITH APPLICATIONS TO TUNNELLING

Simulation is a powerful decision-support technique for construction management; an accurate construction process model can aid the development of better alternatives and optimization of involved resources [5]. Different simulation tools can be applied to tunnelling projects for 1) project planning, 2) identifying bottlenecks in tunnel operations, 3) examining productivity improvements and optimizing resource utilization and 4) quick comparison of alternative tunnelling scenarios [16]. Over the years, advancements have been made in construction management simulation tools, with applications to tunnelling: Simphony [3] Special Purpose Templates [11], 4D modelling methods and Construction Synthetic Environment (COSYE) [2]. These are outlined in the paper. Additional innovations were presented in Einstein et.al. [9], and Haas and Einstein [10] (amongst other publications) where an innovative simulation system for tunnel construction simulation named DAT (Decision Aid for Tunnelling) is described. Ioannou [13] also presented a geologic prediction model for tunnelling and risk reduction modelling as well as planning and simulation approaches to augment those predictions.

The application of simulation to manage tunnel projects from an operational perspective is outlined through several examples in this paper.

1.1 Overview of Simphony

Simphony is a Windows-based integrated simulation environment developed for the purpose of experimenting with general purpose simulation models of construction operations and for efficiently deploying Special Purpose Simulation (SPS) tools to model construction systems in a consistent, standard and intelligent manner. Simphony can be used to develop flexible, user-friendly simulation tools that support graphical, hierarchical, modular, integrated modelling, in a short period of time. The tools deployed through Simphony are referred to as simulation templates. There are two basic categories of templates that are continually maintained and updated at the University of Alberta including: (1) the general purpose modelling template (a standard modelling template for process interaction – a system similar to GPSS) and the general purpose modelling approach of CYLONE [12]; (2) special purpose templates including Earth Moving and Land Reclamation, Tunnelling, Fabrication plants, and numerous others.
Figure 1 depicts a high-level overview of the Simphony environment. Developers can utilize Microsoft Visual Studio.NET to develop the SPS Templates using the provided object models and Simphony services. The process is streamlined and fairly simple to follow for those familiar with Visual Studio (the reader can refer to Simphony Documentation for more information).

SPS Templates are a collection of modelling elements targeted for a single domain, which can be stored in Simphony as “add-on” libraries that other modellers can use to create special purpose models in an intuitive manner without having to know simulation languages [11].

Figure 1: Simphony environment

1.2 Overview of Special Purpose Template Development

The Special Purpose Simulation (SPS) approach enables a practitioner who is knowledgeable in a given domain, but not necessarily in simulation, to easily model a project within that domain using visual modelling tools that have a high degree of resemblance to the actual construction system [3].

The Tunnelling template is a special purpose template, developed within Simphony simulation environment, to simplify planning and analysis of tunnel construction projects. This template has evolved over the years to cope with the emerging needs of researchers and practitioners in the tunnelling domain, and advancements in technology in tunnel construction and computing. Currently, there are two versions of the template, one that can participate in a distributed simulation system and the other that is limited to executing as a standalone simulation.
The template is comprised of modelling elements, most of which represent the different physical components and resources that exist within a typical tunnelling project, for example a shaft element, tunnel element, crane (site) element and TBM element. The other modelling elements (in older versions of the template) represent project management processes that support main tunnelling operations such as a work shift element. This element has been replaced by a calendar defined within the property grid at the scenario level in the current version of the template. These modelling elements constitute building blocks for the template and are developed in Visual Studio .NET using Simphony services. They have a visual appearance similar to the constructs that they represent in a real-life setting to make their use easy for practitioners not proficient in simulation. Each modelling element plays a critical role in the modelling process of the tunnelling operation by either mimicking the construction process through the capture of resources, scheduling of events and the subsequent release of these resources, through the collection of statistics or through control of work and non-work times. A hierarchical approach was adopted in the design and implementation of this template because of the complex nature of the operation. These templates have a total of two modelling elements at a parent level and eleven elements at a child level. These modelling elements can be easily dragged from the templates panel and dropped onto the modelling surface, then setup in a logical manner that mimics the actual tunnel construction sequence and processes (see Figure 2 for a typical model layout). A modeller using this template for experimentation has provisions for customizing their model to the project by inputting project information such as the work method (hand excavation, TBM excavation), the project site conditions (depth of tunnel, tunnel diameter, tunnel length, geotechnical conditions along the shafts and tunnel, penetration rates in various ground conditions) and resource details (number of trains, number of carts). Other details that can be defined include weather and work shift configurations (work calendar). Modelling elements within the template allow for input of information before the simulation and provide results after simulation. Figure 2 shows sample screen shots of the input/output interface. The template simulates the construction of the tunnel based on prior modelling information defined and outputs different results such as the costs, project duration, resource utilization, waiting times, daily advance rates, and volumes of earth excavated and handled.

The template also models different dynamics and uncertainties experienced in a typical tunnelling project such as equipment breakdown, bad weather interruptions and the details of work shifts – work times, breaks, overtime, weekends and holidays.
1.3 Overview of 4D Modelling

4D modelling has gained recognition in construction management over the past ten years, especially after improvements in CAD software and advancements in BIM. A 4D model is essentially a 3D model of the facility being constructed, simulated in its progression using a schedule that relates the components being built to the CAD elements being displayed. The approach provides a visual simulation of how the facility is being built.

Figure 2: Special purpose simulation template for tunnel construction
Recently, Zhang et al. [17] demonstrated a distributed simulation visualization framework (DSVF) which integrates computer visualization and geographic information technologies with discrete-event simulation (DES) under the High Level Architecture (HLA) to fully analyze a tunnel project in 4D. The approach provides a framework within which visualization components (federates) can be easily reused and integrated with other construction simulation components. The federates, built with COSYE (described in the next section), include simulation federates, visualization supportive federates, and visualization federates, which work together through the Internet or an intranet, sharing and exchanging data with the COSYE server. They are designed in a loose-coupling style for easy reuse and for unlimited selection of graphics APIs. This visualization method could greatly improve understanding of simulation results in a collaborative environment, as demonstrated in Figure 3.

![3D animation of tunnel construction](image)

**Figure 3:** 3D animation of tunnel construction

### 1.4 Overview of COSYE

COSYE (COnstruction SYnthetic Environment) is an application programming interface that supports development of large-scale distributed synthetic simulation environments, and is based on the High Level Architecture (IEEE 1516) standard [2]. The HLA standards are guidelines that ensure distributed simulation systems are developed in an interoperable, reusable and consistent manner. The HLA standard is comprised of three components: the rules, the Object Model Template (OMT) and the Interface specifications. The COSYE framework is made up of a Run Time Infrastructure (RTI), an OMT editor, a federate host, and federate form components,
which allows creation of distributed simulation systems. These distributed simulation systems, also known as federations, are made up of separate simulation components are known as federates. A typical distributed simulation system will be represented by one federation and a number of different federates that are responsible for its simulation behaviour. Distributed simulation models, or federations, represent large-scale construction projects, as depicted in Figure 4.

![Figure 4: Comprehensive model of a construction project in COSYE](image)

2 APPLICATION OF SIMULATION TO MANAGEMENT OF TUNNEL CONSTRUCTION PROJECTS

2.1 Using Proven Technologies to Improve Tunnel Construction Projects

2.1.1 Scenario-based planning using special purpose simulation templates

The special purpose simulation tunnelling templates were used for scenario-based planning in a City of Edmonton project, construction of the Mill Woods Double Barrel for Flood Reduction in Mill Woods [6]. Using the simulation based approach for planning, numerous alternatives can be evaluated.

Using the scenario-based simulation planning tool for tunnelling construction, the total cost, productivity, and start and finish dates were found for each scenario (see Table 1 for an example). Results could then be compared to find the optimal scenario for the project.
Table 1: Summary of scenarios productivity figures

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<td>8.8</td>
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2.1.2 Risk analysis of cost and schedule

“Monte Carlo simulation is a computerized mathematical technique that allows people to account for risk in quantitative analysis and decision making. In construction we are constantly faced with uncertainty, ambiguity, and variability, and even though we have unprecedented access to information, we cannot accurately predict the future. Monte Carlo Simulation allows us to see all the possible outcomes of potential decisions and assess the impact of risk, allowing for better decision making under uncertainty.” [15].

Figure 5: Scen. 1a two-way tunnelling

Figure 6: Scen. 1a simulation model
Monte Carlo simulation methods have been successfully applied to numerous construction project estimates and schedules and as an integral part of a risk analysis of capital construction projects. When applied to costs and schedule analysis, Monte Carlo simulation is used to determine the degree of uncertainty associated with the base estimate or plan. The simulation results is a distribution of likely project costs, giving the project analyst a better understanding of the uncertainty inherent in the estimate or plan (demonstrated in Figure 7 and 8). The mainstream adoption of Monte Carlo simulation is manifested in a recent standard from the Construction Industry Institute on risk analysis where the method is described as level 3 - probabilistic risk analysis [8].

![Figure 7: A typical quantification of probability of risk and its impact](image)

2.1.3 Project control using 3D/4D models and visualized earned value analysis

Earned value analysis (EVA) is a project performance measurement approach that is widely used in construction. The approach is generally numeric in nature with results summarized in indexes for schedule (SPI) and cost (CPI). A typical EVA from an LRT project is shown in Figure 9.
2.1.4 Operational improvement and MPDM analysis

Productivity in tunnelling projects can be greatly impacted by issues such as equipment breakdowns, weather, surveying strategies, material handling methods, etc. The Method Productivity Delay Model (MPDM) [7], provides a simple, structured method for identifying key causes of delays in a project, which can be used to analyze productivity. MPDM is applied by 1) highlighting the construction cycle, and defining what might impact production during the cycle, then 2) collecting data for a number of cycles including the production, cycle time, delays and causes for the delays. Next, the overall production, as well as the production in non-delayed cycles, is calculated and compared. Finally, the severity and expected percentage of delay due to each cause is calculated. Information concluded from MPDM analysis can be

---

**Figure 8:** Final project costs determined from Monte Carlo simulation

**Figure 9:** EVA chart

**Figure 10:** EVA status in 3D
used by the project team to discover the type of delays that a project may face, and therefore, where they should concentrate their efforts to improve project productivity. Those approaches are amenable to integration within a simulation model to augment the interpretation of the results of given scenarios and help in guiding the improvement process.

Table 2: MPDM data collection sheet

<table>
<thead>
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<th>Date</th>
<th>Advance</th>
<th>Shift</th>
<th>Delay Type</th>
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<tbody>
<tr>
<td>31/5/12</td>
<td>0.6 m</td>
<td>8 hrs</td>
<td>100%</td>
<td>4 hrs</td>
</tr>
<tr>
<td>1/6/12</td>
<td>2.17 m</td>
<td>8 hrs</td>
<td>0%</td>
<td>0 hrs</td>
</tr>
<tr>
<td>4/6/12</td>
<td>1.04 m</td>
<td>8 hrs</td>
<td>100%</td>
<td>5 hrs</td>
</tr>
<tr>
<td>...</td>
<td>...</td>
<td>...</td>
<td>...</td>
<td>...</td>
</tr>
</tbody>
</table>

Table 3: Production summary

<table>
<thead>
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<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Non-Delayed Prod. Cycle</td>
<td>225.5 hrs</td>
<td>24</td>
<td>90.31 m</td>
<td>9.4</td>
<td>0.4</td>
</tr>
<tr>
<td>Overall Prod. Cycle</td>
<td>664 hrs</td>
<td>70</td>
<td>225.5 m</td>
<td>9.5</td>
<td>0.34</td>
</tr>
</tbody>
</table>

Table 4: Delay information

<table>
<thead>
<tr>
<th>Time Variance</th>
<th>TBM Elect.</th>
<th>TBM Mech.</th>
<th>Soil Cond.</th>
<th>Crane</th>
<th>Survey</th>
<th>Weather</th>
<th>Other</th>
</tr>
</thead>
<tbody>
<tr>
<td>No. occurrences</td>
<td>6</td>
<td>19</td>
<td>5</td>
<td>2</td>
<td>15</td>
<td>1</td>
<td>6</td>
</tr>
<tr>
<td>Total added time</td>
<td>15</td>
<td>81</td>
<td>15</td>
<td>7.5</td>
<td>32.5</td>
<td>0</td>
<td>39</td>
</tr>
<tr>
<td>Prob. occurrence</td>
<td>0.09</td>
<td>0.27</td>
<td>0.07</td>
<td>0.03</td>
<td>0.21</td>
<td>0.01</td>
<td>0.09</td>
</tr>
<tr>
<td>Relative severity</td>
<td>0.26</td>
<td>0.45</td>
<td>0.32</td>
<td>0.4</td>
<td>0.23</td>
<td>0</td>
<td>0.69</td>
</tr>
<tr>
<td>Expected % delay</td>
<td>2.26</td>
<td>12.2</td>
<td>2.26</td>
<td>1.13</td>
<td>4.89</td>
<td>0</td>
<td>5.87</td>
</tr>
</tbody>
</table>

2.1.5 Claims quantification

The process of mediating construction disputes, which arise from changes in project conditions, involves review of contractor estimates, analysing claim motives, and determining the responsibilities of the owner and contractor [1]. This process can be enhanced using simulation: simulation models are developed to estimate the cost of the operation as envisioned by the contractor at the time of bidding, and to estimate the cost of the operation after the change. This eliminates the need to rely on contractor estimates, which tend to be biased, in mediation. AbouRizk and Dozzi [1]
developed CYCLONE [12] models to analyse a claim between a public owner and a bridge contractor. Similarly they developed a Simphony simulation model to analyse a claim for changed conditions of a utility tunnel under the river in Edmonton (The Rossdale Tunnel — excavated using a TBM) [4]. In this tunnel, a cave-in took place as the tunnel vertical alignment mistakenly put it too close to the river bed. The contractor abandoned the site and completed the project using the retrieval shaft location, which was constrained. The simulation showed that the reduced productivities from the new setup were justified due to the imposed logistical constraints. SMA Consulting Ltd. is using similar strategies by building simulation models to analyse performance on the North Light Transit Tunnel in Edmonton (Excavated using SEM). In this case, the contractor was not able to execute the excavation strategy prescribed in the design as he deemed it to be unsafe to use. Using the conservative/safe approach of supporting the tunnel face and benching resulted in significant cost overruns. Simulation models were built that showed that while some of the productivity loss was justified, the contractors own methods were not efficient and its execution approach and shortcomings contributed to the productivity loss.

![Figure 11: NLRT project durations between categories found through simulation](image)

### 2.2 Using Advanced Simulation Technologies for Integrated Planning and Control of Tunnel Simulation Projects

The examples of innovation and advancements presented in the paper so far are generally isolated applications where the simulation models are created from scratch for each objective. Current research efforts at the University of Alberta are focused on providing integrated frameworks which facilitate collaboration in model development, reuse of model components and assimilation of multi-world views of simulation within one model.
2.2.1 Simulation-based framework for dynamically representing project information

Simulation models can be used to capture accurate and dynamic project information. A generic framework has been developed using distributed simulation concepts in COSYE, to facilitate integration of simulation components, to promote interoperability between the components, and to aid reusability of models in the future. The main objective of the framework is development of a generic framework for information representation that incorporates the process model, product model, and external factors of a project into one system to create a complete project chronology, including all changes to the original plan [14]. The system integrates the information and presents it to the user in a dynamic format, providing an overview of the project at every stage for comparison with the project as planned to help project managers control the project during execution and to compile a complete history of the project for future claims management, lesson-learned studies, or other uses. Further applications of this framework include the integration of information from an automated TBM guidance system that is installed in the tunnel. The automation system integrates: (1) a surveying robot (so called robotic total station) to track the advance of TBM in real-time; (2) on-board tablet computer for data processing; and (3) wireless on-site sensor network for remote data communication from the tunnelling face to the surface office. The precise coordinates of the TBM are surveyed automatically, so as to derive the real-time line and level deviations of the tunnel alignment and the advance rate of the tunnel operation. All of those parameters are transmitted to the COSYE RTI Server on the fly thus providing means for further simulations or for recording and documenting the as-built product and process.

3 CONCLUSIONS

The paper provided an overview of innovations resulting from the application of computer simulation concepts to the management of tunnel construction operations. Over the years, those innovations have matured, were tested and applied with varying degrees of success. Current efforts are directed at seemingly integrating different decision support systems, data collection systems, and information generated during tunnelling into one model where decisions can be optimized and information properly stored and documented.
Figure 12: Concept model of tunnel COSYE federation

REFERENCES


Interface Modelling Based on Levelsets

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Abstract

The level set method is applied to computationally simulate the propagation of spatial interfaces over time. Examples for such interfaces are ubiquitous in geomechanics and include reaction fronts, free surfaces in Eulerian formulations and infiltration and injection fronts. Techniques are presented for the treatment of the hyperbolic differential system at the core of the interface dynamics. These techniques are based upon an algorithm developed in the finite difference context, but are modified to take advantage of the robustness and flexibility of the finite element method. In this paper we apply the level set method to computationally simulate a particular infiltration problem, namely the propagation of porosity alteration fronts in porous media.

Keywords: Interface, Level Set, Instability, Finite Elements, Erosion
1 INTRODUCTION

Computational simulations are now firmly established as an effective tool to model and explore virtually all areas of geomechanical fluid dynamics. This coincides with an unprecedented increase in the computational capacity to solve large and complex problems. Accordingly the expectations in the community in regard to sophistication and model sizes have risen significantly. In this paper we present computational simulations of a hydraulically driven propagation of a porosity alteration front in a porous medium. For the numerical treatment of the governing model equations we use the partial differential equations (PDEs) solver Escript Gross et al. (2006).

In the following we consider porosity alterations due to hydraulic erosion, the dislodgement of fine particles from the pore walls or the pore space due to hydraulic drag forces (Scheuermann et. al. (1999), Vardoulakis (2004). Other mechanisms include melting (Mohajeri et. al. (2011), Muhlhaus et. al. (2012)), and chemical dissolution (Orteleva (1987)).

In the level set method a spatial surface $\gamma$ is advected by a velocity field that coincides at the zero level ($\gamma = 0$) of the level set function $\gamma$ with a physical velocity field, usually following from the physics of the problem at hand. In our case the velocity at the surface would be the normal velocity of the alteration front. The normal velocity itself is determined by the so called Stefan condition Fasano et. al. (2012). The level set method contains two major challenges: Firstly the level set function becomes distorted and even computationally intractable due to numerical effects; secondly we are facing the usual difficulties associated with the treatment of advection problems in the context of finite elements. The first problem is addressed by means of a PDE based re-initialisation procedure (Sussman et al. (1994), see outline in section 3 of this paper). Also in section 3 we give an outline of an algebraic upwinding procedure a flux reconstruction method addressing the numerical difficulties associated with the advection of the level set function, the reaction transport equation for the concentration of the dislodged particles and the fluid flux respectively. In the following section we give an outline of the governing equations. This is followed in section 4 by an outline of the level set method and its computational treatment. Results of the computational simulations are presented in section 5.
2 EROSION MODEL

In this section we give a brief outline of the governing equations of our model. The mass conservation of the pore fluid is described by

\[ q_{k,k} = 0 \]  

(1)

where Einstein summation tensor notation is used. In this equation \( q_k = \varphi u_k \cdot \varphi \) is the porosity, \( u_k \) is the pore fluid velocity and \( q_k \) is the pore fluid flux. The pore fluid flux is related to the fluid pressure \( p \) by means of a constitutive relationship commonly known as Darcy’s law as

\[ q_k = -\frac{\kappa(\varphi)}{\eta_F} \cdot p_k \]  

(2)

where \( \kappa \) denotes the permeability and \( \eta_F \) is the viscosity of the of the pore fluid. Mass transfer and transport of the concentration \( C \) of the particles suspended by hydraulic erosion is described by

\[ \frac{(\varphi C)_t}{\varphi} - (\varphi D \cdot C)_k + (q_k C)_k = \varphi_t \]  

(3)

where \( D(\varphi) \) is the diffusivity of the concentration. The system needs to be closed by a relationship describing the kinetics of the erosion process, the so called erosion equation. The general form of the erosion equation is \( \varphi_t = f(\varphi, C, q) \). Here we consider a simplified model in which we assume that the characteristic time scale of the erosion equation describing the erosion process is much shorter than the time scale associated with the fluid flow and the diffusion in Equ. 3. As in this case the erosion occurs instantaneously, it is characterized by a propagating discontinuity surface, separating the un-eroded part of the domain with porosity \( \varphi_0 \) from the eroded part with porosity \( \varphi_f \) behind the front. The Equ. 3 then reduces to

\[ -(\varphi D \cdot C)_k + q_k C_k = 0 \]  

(4)

The source term in Equ. 3 is replaced by a flux condition, the so called Stefan condition

\[ \varphi D \cdot C_n = -V \cdot (\varphi_f - \varphi_0) \]  

(5)

In Equ. 5 \( n_k \) defines the face normal of the interface with appropriate orientation, \( V \) the speed at which the front is moving along the face normal and \( \varphi_f - \varphi_0 \) is the
porosity contract across the front. We assume that the Peclet number

\[ Pe = \frac{H \cdot \|q\|}{\varphi \cdot D} \quad \text{with} \quad \|q\|^2 = q_k q_k \]  

(6)

is sufficiently small across the entire domain and therefore it is not required to stabilize the advective term \(q_k C_k\), as in Equ. (6) \(H\) denotes the thickness of the domain.

3 THE LEVEL SET METHOD

We assume that the two sub-domains \(\Omega_0\) and \(\Omega_f\) of the domain are filled with materials with distinct values for porosity \(\varphi\) and permeability \(\kappa\). To describe the interface between the two sub-domains we use the level set method. It is based upon an implicit representation of the interface by a smooth, scalar function \(\gamma\) which is called the level set function. The function usually takes the form of a signed distance to the interface, whereby the zero level surface \(\gamma(x) = 0\) represents the points \(x\) on the actual interface \(\Gamma\) between the two materials. Points \(x\) in \(\Omega_0\) can be characterised by \(\gamma(x) < 0\) while points \(x\) in \(\Omega_1\) can be characterised by \(\gamma(x) > 0\), or vice versa. At a given location in the domain the value of any parameter such as porosity \(\varphi\) and permeability \(\kappa\) can be set depending upon the sign of \(\gamma\) at that location.

In the presence of a velocity field \(v_k = V \cdot n_k\) the interface is transformed over time. In the Eulerian framework this is described by the advection equation:

\[ \gamma_t + v_k \gamma_k = 0 . \]  

(7)

where the interface normal field in the neighborhood of the interface is calculated at

\[ n_k = \frac{\gamma_k}{\sqrt{\gamma_k \gamma_k}} . \]  

(8)

It is desirable that over time the function \(\gamma\) maintains its initial character as a distance function. This can be expressed by the normalisation condition

\[ \gamma_k \gamma_k = 1 . \]  

(9)

One has to expect that the normalisation condition (9) for the level set function \(\gamma\) is not preserved during advection. It has however been shown, see Sussman et al. (1994), that, in order to obtain acceptable conservation of mass, it is critical that
the level set function $\gamma$ remains a distance function in regions close to the interface. Therefore, a reinitialization procedure is applied that transforms $\gamma$ back into a distance function $\psi$ but maintains the location of the interface. We are using the following computationally very light approach, see Sussman et al. (1994): With the artificial time $\tau$ we are solving the initial value problem

$$\psi, \tau = \text{sign}(\gamma) \left( 1 - \sqrt{\psi, i \psi, i} \right) \text{ and } \psi(0) = \gamma.$$  \hspace{1cm} (10)

This equation is solved until the solution has converged at least for locations close to the interface, typically after $5 - 10$ time steps.

4 SOLUTION METHODS

We apply standard finite element (FEM) Zienkiewicz & Taylor (2005) techniques to solve the model equations and the transition of the level set function $\gamma$. All equations are solved across the entire domain $\Omega$.

The transport equation for the level set function (see Equ. (7)) is solved using the flux-controlled transport finite element method (FCT-FEM) which can be applied directly to any first FEM discretization, see Gross et al. (2013). The reinitialization can be rewritten as, see Tornberg & Engquist (2000):

$$\psi, \tau = \text{sign}(\gamma) - w_i \psi, i \text{ with } w_i = \text{sign}(\gamma) \frac{\psi, k}{\sqrt{\psi, i \psi, i}}.$$ \hspace{1cm} (11)

This equation is stabilized by introducing artificial diffusion using the SUPG approach in the form

$$\psi, \tau = \text{sign}(\gamma) - w_i \psi, i + \left( \frac{dx}{2} \cdot w_i w_k \psi, k, i \right)$$ \hspace{1cm} (12)

where $dx$ denotes the local element size. We apply a forward Euler scheme with mass matrix lumping to integrate over time. In order to maintain stability a Courant-Friedrichs-Lewy condition (CFL condition) needs to be applied in order to bound the time step size $d\tau$. As $\|w\| = 1$ the condition takes here the simple form

$$d\tau \leq \chi \cdot dx$$ \hspace{1cm} (13)

where $\chi$ is an appropriate constant (e.g. $\chi = 0.1$). We notice that for the case of a homogeneous mesh with constant $dx$ Equ. (12) can be written in the form

$$\psi, \tau = \text{sign}(\gamma) \left( 1 - \sqrt{\psi, i \psi, i} \right) + \frac{dx}{2} \cdot \psi, kk.$$ \hspace{1cm} (14)
Under the assumption of a low Peclet number the Eqn. (4) for the concentration $C$ can easily be solved using standard FEM. When solving this equation the concentration is set to zero across the subdomain $\Omega_0$.

Pressure $p$ is calculated by inserting the flux definition (2) into Eqn. (1):

$$-(\kappa \cdot p_k)_k = 0.$$  \hspace{1cm} (15)

This equation can easily be solved using FEM. To calculate the flux $q_k$ we solve the differential equation

$$u_k - (\varepsilon \cdot dx \cdot q_{i,i})_k = -\kappa \cdot p_k$$  \hspace{1cm} (16)

with an an appropriate constant $\varepsilon > 0$. This global reconstruction of flux leads to a better flux approximation than simple gradient evaluation, see Gross et al. (2013).

Permeability $\kappa$ and porosity $\phi$ have a jump across the erosion front. In practice, it is an advantage to smooth the jump across the interface if the contrast is very large, see Gross et al. (2006). We set

$$\kappa(x) = \begin{cases} 
\kappa_0 & \text{if } \gamma(x) < -dx \\
\kappa_f & \text{if } \gamma(x) > dx \\
\frac{\kappa_0 - \kappa_f}{2} \cdot \gamma(x) + \frac{\kappa_0 + \kappa_f}{2} & \text{if } |\gamma(x)| \leq dx
\end{cases}.$$  \hspace{1cm} (17)

where $\kappa_0$ and $\kappa_f$ are the values of the permeability within the sub-domains $\Omega_0$ and $\Omega_f$, respectively. An analog approach is taken for the porosity $\phi$.

For an initial level set function a time integration step is as follows:

1. calculate permeability $\kappa$ and porosity $\phi$ from Eqn. (17).
2. calculate flux $q$ by solving Eqn. (15) and Eqn. (16)
3. calculate concentration $C$ by solving Eqn. (4).
4. front velocity $V$ from Eqn. (5)
5. update level set function with appropriate time step size solving Eqn. (7)
6. apply reinitialization Eqn. (14)

It is pointed out that the time step size of this scheme is controlled by the front velocity $V$ as we apply a CFL type condition in the FEM-FCT scheme to maintain concentration bounds. The algorithm has been implemented using the Escript PDE solver environment for Python, see Gross et. al. (2001).
5 EXAMPLES

5.1 Straight Erosion Front
In our first case we use a domain of height 4 and width 1 with periodical boundary conditions. Water is injected at the bottom of the domain at a rate of 0.1 while the top pressure is kept as zero. The concentration is kept at the constant value zero at the bottom and one at the top of the domain. The diffusivity is set to $D = 1$. Initially a straight, horizontal erosion front is assumed at 0.4 units below below the top of the domain. The porosity and permeability contrast across the front are set to $5 \cdot 10^{-6}$ and 0.3, respectively. As expected the simulation is creating a horizontal straight line which moving downwards. Figure 1 shows the location of the front as distance from the top of the domain over time and the velocity of the erosion as function of front location. The concentration decays from its maximum value one at the front to zero at the bottom of the domain. Consequently the slope of the concentration is becoming steeper as the front approaches the bottom of the domain. Due to the Stephan condition (5) the front is traveling at a higher velocity when it is closer to the boundary.

5.2 Perturbed Erosion Front
In the second example we use the same problem set-up but the initial erosion front is slightly disturbed away from a straight vertical front. In fact the left part is moved downwards while the right part is moved upwards. As a consequence the left part of the erosion front is moving downwards faster than the right part creating a fingering structure, see Figure 2.

6 CONCLUDING REMARKS
The unstable sensitivity to perturbations of the shape of the erosion front is quite strong for the model considered here (Figure 2). A related model was considered by Orteleva et al 1987. Here straight reaction fronts where unstable as well however the strength of the instability seemed much less by comparing the results presented.
In the Orteleva (1987) model the sign of the source term in (3) is negative and the boundary conditions are slightly different. In a forthcoming paper Muhlhaus et al. (2013) we will include linear instability analyses to explain the differences between the models and also include more details of the level set simulations including comparisons of the results of the two models.
ACKNOWLEDGMENT

This work has been supported by the AuScope National Collaborative Research Infrastructure Strategy, the Australian Geophysical Observatory System (AGOS) and the ARC Discovery grant DP120102188: Hydraulic erosion of granular structures.

REFERENCES


Figure 1: Front location versus time and front velocity $V$ versus front location for a straight erosion front in a domain of height 4 and width 1 with periodical boundary condition. The broken horizontal line indicates the analytical propagation speed for the infinite 1D domain.
Figure 2: Concentration distribution and erosion front for perturbed, vertical initial erosion with periodic vertical boundary conditions.
A Coupled Discontinuum-continuum Numerical Model for the Analysis of Face Instabilities in Blocky Rock Masses

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Abstract

Face instability phenomena in blocky rock masses may have a paramount importance in tunnel construction, especially when the excavation is performed by Tunnel Boring Machines (TBMs). Despite the primary importance of such a condition in TBM tunnelling, little research has been conducted so far about face instability mechanisms in blocky rock conditions. The aim of this article is going deeper into the process of development of a blocky face, by assessing the effect that the rock mass structure, in terms of joint frequency, and the in-situ state of stress may have on the face stability. To this purpose, 3D numerical simulations of a 10 m diameter circular tunnel driven in a jointed/heavily jointed rock mass, prone to face instabilities, have been run. In order to limit the computational effort but at the same time being able to reach large values of rock mass fracturing near the face, corresponding to small blocks, a coupled discontinuum-continuum model (CDCM) has been set-up.

Keywords: Blocky rock conditions, Discrete element method, Face instabilities
1 INTRODUCTION

Face instability phenomena in blocky rock masses may have a paramount importance in tunnel construction, especially when the excavation is performed by Tunnel Boring Machines (TBMs). Large breakouts (up to several cubic meters) may appear at the excavation face as the results of structurally controlled and stress driven failure mechanisms. Recent cases of tunnels driven in such a condition (Lötschberg Base Tunnel, Gotthard Base Tunnel, etc.) pointed out the extreme effect that a blocky/irregular excavation face may have on the normal TBM operations. These negative effects generally comprise increased maintenance requirements, caused by the increased wear of the cutters/cutterhead and muck transportation system, lower advance rates and a general increase of the construction time and costs [1].

The aim of this article is going deeper into the process of development of a blocky face, by assessing in a quantitative way the effect that the rock mass structure, in terms of joint frequency and orientation, and the in-situ state of stress may have on the face stability. Given the purely 3D conditions in the vicinity of the excavation face and the discontinuum nature of the medium being considered, a 3D discontinuum numerical study has been performed with the Three Dimensional Distinct Element Code 3DEC (Itasca®). The numerical model aims to simulate a 10 m diameter circular tunnel driven in a jointed/heavily jointed rock mass, prone to face instabilities. This means that at least one joint set exists which is parallel to the tunnel face and dipping against the driving direction [2, 3].

In order to limit the computational effort but at the same time being able to reach large values of rock mass fracturing in the vicinity of the excavation face (and therefore small joint spacing and small intact rock blocks), a coupled discontinuum-continuum model (CDCM) has been set-up. More in detail, in the central part of the model the rock mass has been modelled as a discontinuum medium, e.g. as an assemblage of deformable rock blocks and discontinuities, while in the outer part of the model, away from the excavation face, the rock mass is modelled as an equivalent-continuum medium, e.g. the material properties are equivalent to the rock mass properties. A sketch of the CDCM model is reported in Figure 1, while an example of blocky faces is shown in Figures 2a and 2b:
A Coupled Discontinuum-continuum Model for the Analysis of Face Instabilities in Blocky Rocks

Figure 1: Geometry of the CDCM model

Figure 2: a) Large breakout at the tunnel face in granite; b) Sliding/shearing of rock blocks along steep discontinuities in gneiss (after [4])

2 MODEL SET-UP

Three different models have been run. The joint spacing and the depth of the tunnel have been varied in order to study the effects of both the rock mass fracturing and the in-situ state of stress on the stability conditions of the excavation face.
2.1 Inner Discontinuum Part

The discontinuum medium is modelled in 3DEC as an assemblage of discrete blocks and the discontinuities are treated as boundary conditions between blocks. In the proposed models, a rock mass with three orthogonal joint sets, defined in a deterministic way, is simulated. The first set (K1), is steeply inclined (dip angle = 70˚) and dipping against the driving direction. The second set (K2) has also a dip angle of 70˚ but dips with the driving direction. Finally, the third set (K3) is sub-vertical and parallel to the tunnel axis. The joint spacing is set to 0.8 m and 1.2 m in models 1 and 2, respectively. In model 3, a joint spacing of 1.2 m is adopted, but the depth of the tunnel is increased from 1000 m to 1500 m. Discontinuity planes of very low strength and deformability properties have been considered (friction angle $\phi = 12^\circ$; cohesion = 0 MPa; normal stiffness $J_{kn} = 1.6$ GPa and shear stiffness $J_{ks} = 0.3$ GPa). Regarding the rock block parameters, the intact rock material properties are assigned. The rock properties, summarized in Table 1, are representative of a Granite with fair mechanical properties. An elastic perfectly plastic material following the Hoek-Brown failure criterion is assumed.

Table 1: Material parameters for the rock blocks in the discontinuum zone

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Uniaxial compressive strength UCS [MPa]</td>
<td>160.0</td>
</tr>
<tr>
<td>Density $\rho$ [MN/m$^3$]</td>
<td>25</td>
</tr>
<tr>
<td>$J_{kn}$ [$\text{GPa}$]</td>
<td>0.0268</td>
</tr>
<tr>
<td>$J_{ks}$ [$\text{GPa}$]</td>
<td>30000</td>
</tr>
<tr>
<td>Poisson’s ratio $\nu$ [-]</td>
<td>0.23</td>
</tr>
<tr>
<td>$s$ [-]</td>
<td>1.00</td>
</tr>
<tr>
<td>$a$ [-]</td>
<td>0.50</td>
</tr>
</tbody>
</table>

2.2 Outer Continuum Part

The rock mass in the continuum part, away from the excavation face, is modelled as an equivalent continuum material. Although a discontinuum approach would seem more appropriate, it would largely increase the computational effort, which is basically controlled by the number of blocks included in the model. Therefore, the use of a mixed Discontinuum-Continuum model may be advisable in the analysis of such a large scale problem. The continuum medium is assumed to be elastic perfectly plastic following the Hoek-Brown failure criterion. The rock material properties have been scaled according to Hoek and Diederichs [5] by assigning a GSI value of 45 for model 1 and a GSI value of 50 for models 2 and 3. The corresponding rock mass properties are presented in Table 2.
A Coupled Discontinuum-continuum Model for the Analysis of Face Instabilities in Blocky Rocks

Table 2: Rock mass properties in the continuum part

<table>
<thead>
<tr>
<th></th>
<th>GSI = 45</th>
<th>GSI = 50</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rock Mass Young`s modulus E (_{rm}) [GPa]</td>
<td>6709.5</td>
<td>9215.6</td>
</tr>
<tr>
<td>Poisson`s ration (\nu) [-]</td>
<td>0.23</td>
<td>0.23</td>
</tr>
<tr>
<td>m(_b) [-]</td>
<td>3.51</td>
<td>4.19</td>
</tr>
<tr>
<td>s [-]</td>
<td>0.0022</td>
<td>0.0039</td>
</tr>
<tr>
<td>a [-]</td>
<td>0.508</td>
<td>0.502</td>
</tr>
<tr>
<td>Rock mass UCS [MPa]</td>
<td>7.17</td>
<td>9.6</td>
</tr>
</tbody>
</table>

The interface between discontinuum and continuum part is modelled as a “fictitious joint” of well defined mechanical properties. The strength parameters are set to very high values to avoid any displacement (both normal and shear) along the interface. The normal stiffness and the shear stiffness have been computed as \(k_n = E/\Delta z\) and \(k_s = G/\Delta z\) where \(E\) and \(G\) are the Young’s modulus and the Shear modulus and \(\Delta z\) is the mesh size at the vicinity of the interface. This is equivalent to a layer of continuum material of thickness \(\Delta z\) between the discontinuum and the continuum part. Since the continuum part is only needed to minimize boundary effects, this assumption for the normal and shear stiffness does not influence the result in the vicinity of the tunnel face. Instead, it is also large enough that prevents block overlapping thereby avoids any inconsistency in the 3DEC solver algorithm.

3 NUMERICAL RESULTS

The numerical simulations performed allow to analyse the development of a failure zone in the vicinity of the tunnel face, i.e. the zone where the rock blocks are actually falling into the tunnel, which is one of the direct concerns regarding tunnel stability [6].

3.1 Results

The results obtained for the three different models are reported in Figures 3a to 3c. In all the three models, the face is unstable, with block displacements larger than 40 cm. However, with respect to the extent of the failure zone at the tunnel face, model 1 (joint spacing 0.8 m) exhibits a more critical condition and a wider failure zone. This means that the breakout volume at the tunnel face is the largest in this case. On the basis of this first result, it can be concluded that the rock mass fracturing, expressed
by the average joint spacing, has a paramount importance in determining the extent of the face instability in case of blocky rock condition. However, one should be aware of the fact that large degrees of fracturing do not necessarily correspond to face instabilities in blocky rocks. It is generally recognized that the mechanisms leading to instability may occur at the face if at least one joint set exists which is sub-parallel to the face and dipping at steep-very steep angles against the drive direction [2]. The structurally controlled instabilities are also controlled by the joint surface quality. It has been frequently reported that larger instabilities can occur if the joint planes are covered by soft minerals (such as chlorite). The very low joint strength and deformability properties used in the numerical simulations have aimed at reproducing this condition. Regarding the influence of the in-situ state of stress, from the comparison of the results of model 2 and 3 (overburden equal to 1000 m and 1500 m respectively), it can be noticed that the magnitude of the face instability is comparable for the two models. However, in model 3, a larger block longitudinal displacement is observed (Figures 4a and 4b). The numerical simulations also show that failure occurs at the tunnel crown. This is an expected circumstance which is confirmed by field observations too.

Figure 3: Results of the numerical simulations representing the extent of the failure zone at the tunnel face: a) model n.1; b) model n. 2 and c) model n. 3.
Figure 4: a) Block longitudinal displacement for model 2); b) Block Longitudinal displacement for model 3).

4 CONCLUSIONS

Face instabilities have proved to be one of the most important issues that may arise when driving a tunnel in blocky rock masses. In order to evaluate the influence of the rock mass fracturing, and the in-situ state of stress, a numerical study has been performed with the commercial code 3DEC (Itasca®). In order to completely catch the discontinuous nature of the medium near the excavation face, a coupled discontinuum-continuum model has been set-up. The model aims to reproduce a fractured rock mass at medium stress levels where at least one joint set exists sub-parallel to the tunnel face and dipping against the driving direction. The results of the analyses show that the magnitude of the face instability is largely controlled by the rock mass fracturing degree. Decreasing joint spacing leads to larger face instabilities in terms of extent of the failure zone, and to larger block displacements. Further studies can help better assess the influence of the joint surface quality and orientation (i.e., varying strength and deformability properties) on failure zone development at the tunnel face. They can also aid to better identify the main potential failure mechanisms controlling the phenomenon.
REFERENCES


Numerical Modeling in Mechanized Tunneling
Enhanced Monitoring and Simulation Assisted Tunnelling (EMSAT)

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²GEODATA, Leoben, Austria

Abstract

The contribution proposes an approach to combine site-acquired monitoring and machine data in mechanised tunnelling with numerical simulation models to enhance the quality of numerical predictions that has been developed in the European project Enhanced Monitoring and Simulation Assisted Tunnelling (EMSAT). The system uses a web-based data infrastructure that has been designed to store both monitoring data and simulation results as well as all required project specifications in a unified data structure that can be accessed via internet. By the proposed concept, site measurements and machine data can be used to continuously update and improve a numerical simulation model and to increase its predictive capabilities. Numerical simulations are carried out using a 3D finite element model that is equipped with an automatic pre-processing tool and directly linked to the EMSAT system using web services. To demonstrate the capabilities of the proposed system, a validation example is given that compares simulation results to site measurements from a tunnelling project in southern Europe.

Keywords: Monitoring, automatic modelling, web services, finite element, mechanised tunnelling
1 INTRODUCTION

Numerical simulations are a powerful tool to investigate and predict the structural behaviour of tunnels and their surroundings with respect to the excavation and construction process. However, their complexity usually limits their application to the design phase of a tunnel project or for the backanalysis of hazards. In particular the limited knowledge of the actual ground conditions may furthermore lead to rather imprecise predictions. On the other hand, a huge amount of monitoring and machine data is acquired by means of (partly automatic) measurement systems, that could vastly improve the numerical simulation models. If combined, monitoring data can be used to update the numerical simulation models that can in turn be used for reconnaissance ahead of the tunnel face, the evaluation of possible steering scenarios or the detection of deviations from the design. This contribution presents one step towards the big challenge of automated steering in mechanised tunnelling.

2 THE EMSAT CONCEPT

Enhanced Monitoring and Simulation Assisted Tunnelling (EMSAT) is a European research project under the Eureka-Eurostars programme that is being carried out by GEODATA, the Institute for Subsurface Engineering of Montanuniversität Leoben and the Institute for Structural Mechanics of Ruhr University Bochum. In the project the Institute for Structural Mechanics and Geodata have developed a unique approach that combines the acquisition of monitoring data with the web-based generation and execution of numerical simulations that are based on these monitoring data. For this purpose, a data infrastructure has been established that stores all specifications of a tunnelling project along with real-time monitoring data such as inclinometer and levelling measurements. Simulation models are generated automatically by means of an automatic modelling tool [4] that is invoked through a web service. The input parameters for the simulation model are retrieved from the EMSAT-inf data base, that contains both general project specifications and up-to-date monitoring data. Simulation results are then written back to the EMSAT data base to allow for a comparison of simulation-based predictions with the measured values. Engineers can use this comparison to detect deviations from the design or potential hazards and to evaluate appropriate counter measures. Figure 1 depicts the communication scheme within the EMSAT project: at the construction site, a local database server holding an instance of the KRONOS application, a commercially available tunnel
information system produced by Geodata, is installed [1, 2, 3]. Monitoring data and project specifications are stored here and can be retrieved via internet to clients that can be located anywhere. One of these clients is the EMSAT-sim server that is a high performance computer dedicated to numerical simulations. Both the simulation input data and the simulation results are transferred via web services between the EMSAT components.

![EMSAT Communication Overview](image)

**Figure 1:** Topology of the EMSAT project – communication scheme

### 3 WEB-BASED SIMULATION MODEL

The EMSAT-sim component comprises the numerical simulation model ekate, which has been developed in the framework of European Research Project TUNCON-STRUCT for the simulation of shield driven tunnels. Based on a model previously
developed [5], this model has now been supplemented in EMSAT with an automatic modeller [4] using a preprocessing procedure with a high degree of automation and parallelisation techniques to enhance the efficiency of 3D simulations. The simulation model accounts for all relevant components of the mechanised tunnelling process such as stepwise excavation, hydraulic thrust jacks, heading face support by both mechanical and fluid boundary conditions, frictional contact between the shield skin and the surrounding ground and the ringwise installation of lining and tail void grouting. Details on the simulation model can be found in [6]. With these components of the model as well as the interactions between them, an appropriate numerical model simulating the shield tunnelling process is established that has been verified with measured data from a real tunnel project in Section 4.

The model has been designed such that the input parameters retrieved from EMSAT-inf are automatically translated into geometrical properties, information on material parameters and process-related data such as support and grouting pressures. On invocation through the EMSAT-sim web service (see Figure 2a), the required model parameters (Figure 2b) are read from the EMSAT-inf database and the simulation model is generated (Figure 2c). Errors that may occur due to erroneous input data, communication faults or problems in the numerical simulation are handled by means of status indicators such that users have full control of the simulation workflow at any time. The simulation results are written back to the database where they can be found by means of a unique simulation ID.

4 VERIFICATION AND VALIDATION

The verification and validation of EMSAT-sim and the EMSAT workflow has been conducted by means of a tunnelling project in southern Europe that has been equipped with a KRONOS installation on site. A set of two geotechnical sections has been selected that characterise the longitudinal section of the tunnel. Hence, simulations have been carried out for these two sections (Case A and Case B) in order to evaluate the predictive capabilities of EMSAT. In the present study, the focus has been put on the vertical displacements of the ground surface directly above the tunnel (see marked point in Figure 3).

The ground model consists of five soil layers with different properties as shown in Figure 3. The depth of each soil layer is assumed to be constant along the Y-axis and varies in X-Z plane. Each layer is defined as a polygon shape that can be automatically processed by the model generator. Based on the geological survey, the
groundwater table has been assumed to be 7m below the ground surface. The water pressure above this level is fixed at the value 0 to represent the situation of soils without groundwater. An elastoplastic model using Drucker-Prager yield criterion with a linear isotropic hardening has been applied as constitutive model for the soil. The behaviour of the shield is modelled as linear elastic with a modulus of elasticity \( E = 210 \text{ GPa} \) and a Poisson’s ratio of \( \nu = 0.3 \). The shield is divided into 3 parts: front, center and tail with a total length of 9.75m. Each part has its own length, diameter and thickness. The conicity of the shield is given by the outer diameters of the shield skin in each segment that varies from 6.19m to 6.15m from shield front to shield tail respectively. The tunnel lining and the tail void grouting are considered as linear elastic. The former is simulated as a 0.3m concrete layer and a 0.145m mortar layer is representing the latter. The young modulus for tunnel lining is assumed as 35 GPa, corresponding to concrete grade C40/50. The grouting mortar has been modelled with a modulus of elasticity of \( E = 3 \text{ MPa} \). Both support and grouting pressures are defined by their mean value along the centerline of the tunnel and a linear gradient.

Figure 2: Invocation scheme of the EMSAT-sim web service: a) web service interface; b) input parameters; c) simulation model
accounting for the increase of pressures with depth. The tunnel construction has been modelled via a step-by-step simulation procedure consisting of phases: excavation, application of support pressure, moving the shield, application of grouting pressure and lining installation. The tunnel model consists of 32 excavation steps of 1.5m length.

Cases A and B are distinguished by the location of the soil layers. While in Case A, soil layers 4 and 5 are assumed at the level of the tunnel face, in Case B the tunnel crosses layers 3 and 4. It should be noted that most of parameters are kept constant for all cross sections, while some parameters such as grouting and support pressures and the depth of soil layers have been chosen directly from recordings of the machine data on site. The length of the computational domain (X-axis direction) is corresponding to the number of excavation steps. The width and height of the model (Y-axis and Z-axis directions) are chosen far enough from tunnel axis to avoid boundary effects (approximately 5 diameters). Only half of the tunnel model needs to be analysed due to the assumption of symmetry in geometry, material properties
and other conditions assigned to the model. Specifically, the model dimensions are 48m long, 62m wide and 50m deep in the X,Y,Z directions respectively. The tunnel is characterised by a diameter D of 6.19m and an overburden of about 2.6 D (16m). The chosen point, Point 1, for comparing to measured data (the marked point in Figure 3) is located on the top surface above the tunnel axis with a distance of 13.5m ahead of the tunnel crown. The model has been discretised using 8200 27-node hexahedral elements.

Figure 4 and Figure 5 show a comparison of calculated and measured settlements of the monitoring point in Case A and B respectively. In Case A, the tunnel is excavated in stiff soil. Here, the predictions match well the measured settlements. In Case B, however, the tunnel is located in weaker soils. Here, the shape of the settlements is also in good accordance with the monitoring data. Yet, the slope of the predicted evolution of settlements increases significantly compared to the measurements (see Figure 5). The shape of the settlement trough has also been investigated by means of a second monitoring point (Point 2 in Figure 5) that is located 12m aside the tunnel axis. The comparison of this second point shows the same trend as Point 1, however
with a much smaller discrepancy of only 0.4mm.

A possible reason for the observed deviations are insufficiently accurate material parameters for the soil model. Furthermore, the distribution of the support pressure at the heading face, that is modelled by means of a linear distribution does not accurately map the real distribution that is biased by the revolution of the cutting wheel and an inhomogeneous filling of the excavation chamber in EPB shields. The study shows that the model is capable of reproducing the observed deformations in an appropriate way. To obtain an even better accordance, however, the model needs to be adapted by means of the simulation scheme that EMSAT provides. Here, the fully automatic model instantiation serves as a valuable tool in order to continuously improve the model parameters in course of the excavation.

5 CONCLUDING REMARKS

In this paper, the concept of the project Enhanced Monitoring and Simulation Assisted Tunnelling (EMSAT) has been presented. Its main purpose is integrate moni-
toring data and numerical simulation models automatically through a web-based data structure. The automatic web-based instantiation and execution of numerical simulations for mechanised tunnelling projects is based on recorded monitoring and machine data that are stored in a specific database (EMSAT-inf) and processed by means of a web service for numerical simulations (EMSAT-sim). The proposed methodology shows a large potential for the use of numerical simulations along with the actual tunnel construction. In particular, the simulation parameters can be continuously updated as more data from excavation process are available. The validation example shown here exhibits a good qualitative accordance of simulation results with site measurements. With the EMSAT system, the quantitative accuracy can be improved by incorporating site data to correct inappropriate assumptions from the design phase. The presented system is regarded a powerful tool extending the capabilities of Kronos, an approved, commercial tunnel information system used worldwide. By help of the new and innovative automatic simulation web service Kronos will contribute to further improve safety and efficiency of TBM tunnelling.

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Finite Element Simulation of Ground Behaviour due to Modular Approached Tunnel Work

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²Graduate Student, Chiba Institute of Technology, Japan
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Abstract

A modular approached tunnel method is a new mechanized tunnelling method which has been developed to construct large scale tunnels undercrossing the existing traffics in urban areas. During the box jacking operation, a box-module is driven forward by applying mechanical forces and excavating the soil in front of the box-module with boring machine. The step-by-step insertion of the box-module forms a lining frame of the tunnel in the ground and after completion of the lining frame, the tunnel is complete by excavation of soil enclosed in the lining frame.

In this study, the step-by-step advancement and excavation processes of the box-module are modelled using the finite element remeshing technique. Three dimensional FEM analyses are conducted to simulate the construction process of a modular approached tunnel work in soft cohesive soil in Tokyo.

Keywords: Modular approached tunnel method, Finite element method, Construction process, Displacement
1 INTRODUCTION

A modular approached tunnelling method [1] is a new mechanized tunnelling method which has been developed to construct large scale tunnels under-crossing the existing railroad tracks or other existing main artery traffics in urban areas.

In the method, a lining frame is first formed in a soil ground by with step-by-step excavation using a small tunnel boring machine and insertion of a box-module as shown in Figure 1. After completion of the lining frame, the framed-in soil of the lining frame is excavated.

![Overview of the modular approached tunnelling](image)

**Figure 1:** Overview of the modular approached tunnelling

As the excavation by the tunnel boring machine is of small scale and guided by the existing adjacent box-module, it is possible to perform safe construction even where the overburden is small. Therefore, as the modular approached tunnelling method has a variable for a large cross section with extremely shallow overburden earth coverage, the method has been used extensively for the construction of road tunnels under the existing rail track.

Since many advances such as the development of new excavation machines have been made in order to optimise the modular approached tunnelling method, the magnitude of soil deformation has become small. However even with recent advancements of the method, the tunnelling in soft clayey ground, where the SPT-N value is close to zero, is still a major technical challenge to engineers.
In this paper, the advancement and excavation process of the tunnel boring machine and the box-module are modelled using the finite element method in order to investigate the effect of the step-by-step construction process on the ground response. The proposed modelling techniques are applied to simulate a box-jacking tunnelling work in Japan and the numerical results are compared with the field measurements.

2 FINITE ELEMENT MODELLING OF EXCAVATION AND ADVANCEMENT OF THE BOX-MODULES

Figure 2 shows the excavation and insertion processes of box-jacking operation in the modular approached tunnelling method. The tunnel boring machine and the box-module are driven forward by applying mechanical pull jack forces and excavating the soil in front of the tunnel boring machine with its cutting face. The magnitude and distribution of the ground deformations are largely controlled by the construction processes of each box-jacking. Therefore, when estimating the ground deformation caused by the modular approached tunnel construction, care should be taken of how to model the characteristics of the machine and the construction processes.

Figure 2: Construction process of box-module

Because of the complex boundary conditions of a box-jacking tunnelling problem, the use of the finite element method is one of the useful methods to investigate the ground deformation behaviour. In reality, the stress-strain state of the soil changes continuously as the tunnel boring machine and the box-module advances. Then, in order to fully understand the ground deformation mechanism associated with box-jacking, the deformation caused by excavation and insertion processes of the box-module needs to be investigated.

In this study, the excavation process is modelled by introducing excavating finite elements in front of the tunnel boring machine [2]. The advancement of the tunnel boring machine is modelled by (i) remeshing the finite elements at each time step, (ii)
introducing the excavating finite elements of a fixed size in front of the tunnel boring machine, and (iii) applying external forces for the advancement of the machine and the box-module. Sequential illustrations of the modelling of the excavation at the cutting face of the tunnel boring machine (TBM) and the advancement of the machine and the box-module are shown in Figure 3.

Figure 3: Modelling of advance of the box-module

Figure 3(a) shows the status of the tunnel boring machine and the box-module at reference time $t_0$. In order to model the external pull forces applied to the tunnel boring machine, forces are applied at the nodes of the tunnel boring machine. During
the time interval of $t_0$ to $t_0 + dt$, the excavating elements and the soil elements adjacent to the tunnel boring machine elements will deform by the external force (Figure 3(b)). The tunnel boring machine will act as rigid bodies since a large value of stiffness is used for the elements representing the tunnel boring machine. After obtaining a solution for $t = t_0 + dt$, the finite elements are remeshed as shown in Figure 3(c). The new mesh will have the same mesh geometry relative to the tunnel boring machine as $t = t_0$, but the location of the tunnel boring machine and the box-module has shifted. Again, the excavating elements will be placed in front of the cutting face before applying external forces given for the next time step. By doing so, the construction processes of the box-module and the associated stress-strain changes of the ground are numerically simulated in a continuous manner.

In the analysis, Goodman type joint elements [3] were placed at the interface (A) between the soil and the box-module and (B) between the existing box-module and the advancing box-module, in order to investigate interface friction effects on ground deformation as shown in Figure 4.

![Figure 4: Arrangement of joint elements](image)

### 3 FINITE ELEMENT SIMULATION OF THE MODULAR APPROACHED TUNNELLING WORK

Three-dimensional finite element analyses were conducted to simulate the construction process of a lining frame of a modular approached tunnel work in Japan. The finite element code used in these analyses was developed by the first author. The sixty rectangular box-modules of 0.85 m wide, 0.85 m high and 30.0 m long were inserted in order to build the lining frame. These box-modules were integrated finally to the lining frame which is approximately 23.10 m wide and 8.14 m high with earth covering of only 1.20 m underneath major rail tracks as shown in Figure 5. The site stratigraphy determined from borehole logs is also shown in Figure 5.
Figure 5: The formation of the box-modules (lining frame) and the site stratigraphy on the cross section.

The order of the construction processes

Figure 6: The order of the construction processes of the box-module
Figure 6 shows the order of the box-module insertions. The box-modules (B, C, D, E) at the top part of the lining frame were first constructed, and then the box-modules (G, F, H, I, J, K) at the vertical wall of the lining frame were constructed, after which the box-modules (L, M, N, O) at the invert were constructed.

3.1 Finite Element Simulation of The Settlement Behaviour of The Lining Frame During Advancement of Box-modules in The Invert Section

In the construction site, the contractor measured vertical displacements of the existing top part of the lining frame during advancement of the box-modules of the invert part of the lining frame, at (already integrated) B10, B5, A, C5 and C9 (see Figure 7) until the box-module O2 was completely advanced. The proposed modelling techniques of modular-approached tunnelling construction are applied to simulate the advancing of box-modules in the invert section of the lining frame and the numerical result is compared with the field measurements.

In this study, due to uncertainty of the ground stress histories, the isotropic elastic model was used to model the stress-strain behaviour of the soil, the tunnel boring machine and the box-modules. Most of the input parameters were determined from the results provided by laboratory tests on samples obtained at various depths in the construction site. Other input soil parameters, which were not able to be determined from these tests, were assessed by the results of the in-situ geotechnical tests. The applied pull jacking forces were obtained from the actual driving record of the machine. Summary descriptions of the soil divisions and input parameters based on the examination of site samples were given in Table 1. Since the box-module was filled with mortar after completion of advancement, the properties were different between the existing box-modules (lining frame) and the advancing box-module. The material properties of the excavating elements depend on various factors such as the method of excavation, machine characteristics, the size of the elements, etc. They need to be obtained by trial and error to match the volume change of the elements to the actual advancement of the tunnel boring machine. Therefore for the excavating elements, a Young’s modulus of $E=300$ kPa, Poisson's ratio of $\nu=0.1$, density of $\rho=1,786$ g/cm$^3$ and the thickness of 1 m were selected by matching the computed advancement of the shield machine at a given time step to the measured field movement data. The stiffness of the joint elements are listed in Table 2. Figure 8 shows three-dimensional finite element model using the analyses.
Figure 7: Location of the measurement points

Figure 8: Three dimensional finite element model

Table 1: Input parameters

<table>
<thead>
<tr>
<th></th>
<th>Young’s modulus</th>
<th>Poisson’s ratio</th>
<th>Density</th>
</tr>
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<tbody>
<tr>
<td>Bank</td>
<td>5600(kPa)</td>
<td>0.333</td>
<td>1.735 (g/cm³)</td>
</tr>
<tr>
<td>Fine sand</td>
<td>5600(kPa)</td>
<td>0.333</td>
<td>1.786 (g/cm³)</td>
</tr>
<tr>
<td>Silt</td>
<td>1000(kPa)</td>
<td>0.444</td>
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<tr>
<td>TBM and advanced box-module</td>
<td>470000(kPa)</td>
<td>0.300</td>
<td>1.786 (g/cm³)</td>
</tr>
<tr>
<td>Existing box-modules</td>
<td>47000000(kPa)</td>
<td>0.290</td>
<td>2.300 (g/cm³)</td>
</tr>
</tbody>
</table>
The order of the advancing of box-module in the invert section was (N2 and L1) → N3 → (M2 and L2) → (M3 and N4) → (M4 and N5) → M5 → (M6 and N6) → M7 → O2. Braces ( ) indicates that two box-modules were advanced simultaneously.

In the actual construction, after completion of the advancement of the box-module, backfill injection into the overexcavated cavity between the box-module and soil was conducted when a large cavity was encountered. However the backfill injections did not modelled in the calculation because there is no record for the backfill injections.

<table>
<thead>
<tr>
<th>Table 2: Stiffness of joint elements</th>
</tr>
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<tbody>
<tr>
<td>Type A (between the soil and the box-module)</td>
</tr>
<tr>
<td>Type B (between the box-modules)</td>
</tr>
<tr>
<td>Type B (between the box-modules)</td>
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Figures 9 and 10 show the measured and the calculated vertical displacement trough on the top part of the existing lining frame at the measurement points 1 during the advancement of the invert box-modules (L, M, N, O) respectively. Since the backfill injection did not modelled in the calculation, the calculated displacements were larger than the measured results.

In the box jacking work, the soil in front of the cutting face of a tunnel boring machine is extremely disturbed by its cutting operation. The strength of soil in the invert part of the lining frame is decreased due to the advancements of the box-module and the lining frame sinks under its own weight. Therefore both the calculated and measured vertical displacements are increased as each box-module is excavated. Since a larger vertical displacement occurred above each advancing box-module, the final transverse displacement became large at the centre of the lining frame as shown in Figures 9 and 10. The shape of the computed displacement trough at the top of the lining frame was similar to the measured values.

The measured and computed vertical displacement at the point A is plotted against the order of the box-module constructions in Figure 11. Although the calculated value demonstrated an increase in vertical displacement during initial advancement of box-
Figure 9: Measured vertical displacement trough on the top part of the existing lining frame

Figure 10: Calculated vertical displacement trough on the top part of the existing lining frame
Figure 11: Vertical displacement of point A against the order of the box-module constructions

3.2 Finite Element Simulation of The Settlement Behaviour of The Lining Frame Due To The Module approached Tunnel Work

Modules L1, L2 and N3, both the calculated and measured vertical displacements were almost identical after the insertion of the N3.

Modular-approached tunnel method in the field involves a series of construction stages (e.g. excavating and installation of box-modules at the top part, at the walls and at the invert section of the lining frame). Obviously these procedures are very complicated and it is very difficult to model all the stages in great detail. However, the proposed finite element techniques described above highlighted in particular the effect of the excavations and insertions of box-module in the invert section of the lining frame and both the calculated and measured vertical displacements of the lining frame were almost identical. An attempt was made here to assess the ability of the finite element method to simulate the excavations and the insertions of all box-modules in the construction site in order to investigate the effect of the step-by-step construction processes on the ground responses.

The finite element simulation was conducted in the same manner of the above-mentioned simulation of the construction of the invert section of the lining frame. The input parameters were given in Table 1 and Table 2.
The order of the advancing of box-module was \{Top\} A → B1 → C1 → B2 → C2 → B3 → C3 → C4 → B4 → B5 → C5 → C6 → B6 → B7 → C7 → C8 → B8 → B9 → C9 → E → B10 → D → \{Wall\} G1 → G2 → G3 → G4 → H1 → F1 → G5 → G6 → F2 → H2 → F3 → H3 → F4 → H4 → H4 → F5 → H5 → F6 → H6 → \{Invert\} → (N2 and L1) → N3 → (M2 and L2) → (M3 and N4) → (M4 and N5) → M5 → (M6 and N6) → M7 → O2. Braces ( ) indicates that two box-modules were advanced simultaneously.

Figure 12 shows the calculated vertical displacement trough on the top part of the existing lining frame and the ground at the measurement points 2 during the advancement of all box-modules. Since the backfill injection did not modelled in the calculation, the calculated vertical displacements were much larger than the measured results.

![Figure 12: Calculated vertical displacement trough during advancement of all box-modules](image-url)
Figure 13 shows the computed pre-unit vertical displacement for four construction sections, i.e. the top(A,B,C,D,E), the wall(G), the wall(F,H) and the invert(L,M,N,O). The per-unit vertical displacement is the mean vertical displacement in the construction section that is equal to the total vertical displacement divided by the number of inserted box-modules in the section.

The vertical displacement due to the advancement of box-module became very small after the completion of the construction of the wall (G). As the completion of the top and the wall (G) sections, a sort of T-shape roof- column structure is formed in the ground and it acts as a support to reduce the vertical displacement of construction processes afterward. The completion of the wall (F and H) increases such support efficiency and the vertical displacement of the invert section is the smallest of all construction sections.

**Figure 13:** The per-unit vertical displacement for each construction section
4 CONCLUSION

In this paper, the advancement and excavation process of the new box-jacking tunnel method were modelled using the finite element method in order to investigate the effect of these construction processes on the ground response. The excavated finite elements were introduced in front of the cutting face of the tunnel boring machine, and the operation of box-module advancement and soil excavation were simulated using the finite element remeshing technique at each time step of the analysis. The proposed modelling techniques were applied to simulate a box-jacking tunnelling work in soft soil ground in Japan and the results were compared with the field measurements.

The vertical displacement profiles of the lining frame were obtained from the three-dimensional finite element simulation using the proposed modelling technique for constructions of sixty box-modules. The shape of the computed settlement trough at the top of the lining frame was similar to the measured results. Since the backfill injection did not modelled in the calculation, the calculated vertical displacements were larger than the measured results. However both the calculated and measured vertical displacements of the lining frame were almost identical for the construction of the invert section of the lining frame.

As the completion of the top and the wall sections, a sort of roof-column structure is formed and it acts as a support to reduce the vertical displacement of construction processes afterward.

REFERENCES


Geotechnical Calculation and Process Controlling Approach for Shield Tunnelling Settlement Minimisation in Urban Areas

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Abstract

The paper focuses on the approach for the calculation, control and minimisation of settlements in urban area in soils very close to foundations of buildings and bridges showing a case study at the shield tunnelling project of Caracas Metro Line 5.

The geotechnical characterisation of a tunnel alignment with characteristic values for the geotechnical parameters is generally associated to a degree of uncertainty which is covered by the safety factors in the respective norms. This has a significant impact on the settlement calculations that will yield unrealistic results if they solely rely on characteristic geotechnical values. The experiences gained show how parametric studies performed with Mohr-Coulomb model and efficient 3D finite element simulations that picture the key shield-ground interaction parameters in combination with sophisticated real-time Process Controlling of the shield drive parameters, provides the key for a safe shield tunnelling in complex underground conditions.

Keywords: Shield tunnelling, urban area, settlement calculation, process controlling, TBM-ground interaction
1 INTRODUCTION AND PROJECT OVERVIEW

The tunnels of Caracas Metro Line 5 consist of twin tubes excavated with 5,88 m diameter EPB shields by the Contractor Construtora Norberto Odebrecht. The case study presented here refers to the shield drives below and between the foundations of an existing bridge and viaduct at the shore of River Guairé.

2 DESCRIPTION OF THE CASE STUDY

2.1 Approach at the Case Study

The approach is based on the combination of the finite element simulations to determine the target values for the excavation and the data management and Process Controlling during excavation. The aim of the approach is to analyse the system behaviour - in situ and in real-time - of all interactions between soil and shield excavation and compare the design and real-time values.

The German DIN 1054 [2] describes the concept of system behavior as building-material-environment interaction. For complex geotechnical structures and interactions the observation method is recommended. The goal is to verify the design values during construction using different measurement systems. Forecasts should be reviewed and the method of calculation shall be adjusted when the behaviour of buildings and constructions is not as expected [6]. This approach can also be transferred on the excavation analysis of the interaction soil-excavation.

Table 1: Geotechnical parameters for 3D FEM simulation, Layer 2-4 = area tunnel face

<table>
<thead>
<tr>
<th>No.</th>
<th>Layer</th>
<th>Thick. [m]</th>
<th>γ [kN/m$^3$]</th>
<th>C [kPa]</th>
<th>φ [°]</th>
<th>E [MPa]</th>
<th>ν [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Fill</td>
<td>6</td>
<td>21</td>
<td>10</td>
<td>30</td>
<td>40</td>
<td>0.3</td>
</tr>
<tr>
<td>2</td>
<td>Clayey sands</td>
<td>8</td>
<td>17</td>
<td>20</td>
<td>34</td>
<td>50</td>
<td>0.3</td>
</tr>
<tr>
<td>3</td>
<td>Coarse sands</td>
<td>4</td>
<td>21</td>
<td>10</td>
<td>34</td>
<td>55</td>
<td>0.3</td>
</tr>
<tr>
<td>4</td>
<td>Coarse gravels</td>
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<td>22</td>
<td>10</td>
<td>37</td>
<td>100</td>
<td>0.3</td>
</tr>
<tr>
<td>5</td>
<td>Coarse sands</td>
<td>4</td>
<td>21</td>
<td>10</td>
<td>34</td>
<td>55</td>
<td>0.3</td>
</tr>
<tr>
<td>6</td>
<td>Coarse-med. gravels</td>
<td>3</td>
<td>22</td>
<td>10</td>
<td>40</td>
<td>100</td>
<td>0.3</td>
</tr>
<tr>
<td>7</td>
<td>Fine silty sands</td>
<td>5</td>
<td>20</td>
<td>10</td>
<td>35</td>
<td>50</td>
<td>0.3</td>
</tr>
</tbody>
</table>
2.2 Geotechnical Conditions

The area is characterized by interlayered silts and permeable loose sands and gravels with relatively poor geotechnical parameters due to the previous infrastructure construction works. On the safe side the worst geotechnical parameters were selected for the soils at the river bed and river shore (Table 1).

2.3 Interference with Existing Infrastructures

In the influence area of the shield drives there are several bridges and viaducts (Figure 1 left). The 3D numerical model comprises firstly the shield drive below river Guairé and then the drive under the caisson foundation of the Lincoln bridge (indicated as Box 2) with a minimum soil overburden between the tunnel crown and the foundation base of barely 90 cm. Immediately after crossing the bridge foundations, the piled foundations of the pillars of a highway viaduct (indicated as column A and column B) are driven by at both sides of the tunnel with a horizontal distance of 1,25 m.

![Figure 1: Overview of the area of case study (left) and simplification of the existing foundations (right)](image)

For simplification, the simulation modeled the position of the different foundations perpendicular and parallel to the tunnels, slightly displaced from their actual position, but considering the shortest horizontal distances to the tunnel, see Figure 1 right.
3 SETTLEMENT CALCULATION MODEL

3.1 Model for the Shield Tunnelling Process

The 3D FEM simulation of the shield drive is performed with the ‘pressure model’ [4], [5]. Settlements are controlled in practice mostly by the shield operational pressures. The shield acts as a pressurized system comprised by the pressures that govern the stress state variations in the ground as described next.

Face pressure $P_1$ actively controlled by the foam conditioning and the pressure of the muck in the excavation chamber. This pressure and its distribution on the face govern the pre-relaxation of the ground prior to the shield arrival.

Face pressure $P_3$ governed by the grout injection of the ring gap, defines the final stress state variation in the ground and the subsequent consolidation process. The grouting pressure $P_3$ is actively adjusted with the control of grout injection volume and injection pressure into the ring gap.

Pressure $P_2$ is the pressure acting in the shield gap, between the cutterhead and the shield’s tailskin. This pressure generates from the communication between face and tailskin through the shield gap and is therefore indirectly controlled by $P_1$ and $P_3$.

The stress state variations and consequently the settlements are controlled in great manner by the shield operation parameters. For the given geotechnical conditions of excavation, the shield operation can actively be adjusted to govern the stress state variations in the ground. However, there are other variables which also contribute to the final amount of settlements, such as shield advance speed, thrust contact forces transferred to the face, soil conditioning as well as local variable geotechnical conditions and consolidation. The consideration of such variables in a numerical model requires a high effort and it is still associated to a degree of uncertainty.

Figure 2: 3D model of a shield machine applying the pressure model: Pressures $P_1$, $P_2$ and $P_3$
3.2 Finite Element Method Model for the Case Study

The FEM model performed with Plaxis 3D Tunnel is shown in Figures 3 and 4 with the relative position of foundations and the two tunnel tubes. The caisson (Box 2) and the piled foundations (Columns A and B) were simulated as blocks with volume elements with a weighed stiffness between concrete and soil according to the foundation configuration and geometry. The calculation was performed using a conventional Mohr-Coulomb constitutive model for the soils with the parameters given in Table 1. The model was calculated for different shield pressures P1, P2 and P3 in order to capture the sensitivity of the ground to a variation of the shield parameters aiming to finally provide the Contractor with the detailed target pressures at each excavation cycle. The calculated settlements are presented in Chapter 5.

![Figure 3: Cross-section of the FEM model with the relative position of tunnel and foundations.](image)

4 PROCESS CONTROLLING OF THE SHIELD DRIVE

4.1 Process Controlling Approach

The Contractor equipped the project with the Process Controlling system from Maidl Tunnelconsultants (MTC). The Process Controlling approach consists in verifying the target parameters defined during design stage with actual shield excavation data on the basis of a real-time analysis of shield-soil interaction and adjustment of parameters according to the encountered conditions ([1], [3], [7]). In order to capture the actual shield-soil interaction and compare it with the one assumed during design stage, Process Controlling is implemented by the use of software Maidl-PROCON.
4.2 Monitored Shield Drive Parameters and Over Surface Points

State-of-the-art TBMs such as the ones used in Caracas, are equipped with sensors that register all operational parameters in frequencies of 10 seconds. Operational parameters from operating pressures and temperatures, injection flows, advance forces to jack elongations and speeds are registered. The monitoring of the operational parameters allows to detect the deviation of parameters from their target reference values.

For the purpose of the present paper, we focus on the control of the primary parameters that control the settlement development at the surface. These are, as described in the pressure model in Chapter 3.1, the face pressure and face pressure distribution, the grouting pressure and grouting volumes.

The area to be crossed was intensively monitored at the surface. The paper focusses on the measurement points PL3 and PL4 on Box 2 (Figure 2).

5 COMPARISON OF ESTIMATED AND MONITORED BEHAVIOUR

Figures 5 and 6 show the calculated and measured settlements in time at points PL4 and PL3 of Box 2 after the first and second shield drive respectively.

For the first drive it becomes clear how the primary settlements were simulated quite precisely in both points. However, there was a divergence of results in the secondary
settlements measured at the point closest to the first drive (PL4) that resulted in a total settlement of -7 mm against the calculated -5 mm, hence 40% higher than calculated. On the other hand, after the second drive (Figure 6), the settlement at the point closest to the drive (PL3), merely increased –1 mm, against the calculated –5 mm. The real settlement was 80% lower than the predicted one.

Figure 5: Settlements development in time as calculated with Plaxis and as measured after the first shield drive

Figure 6: Settlements development in time as calculated with Plaxis and as measured after the second shield drive
This can be explained by the analysis of the operational pressures and volumes via the Process Controlling system. The face pressures $P_1$ in both shield drives were kept very precisely on the target values, but the grouting pressures $P_3$ did differ. In the first drive, the grout injection procedure was acceptable, but not as perfect as possible for the requirements of this very critical shield drive.

This can be observed in Figure 7 where the injected grout volumes are represented during two representative excavation cycles under the bridge foundations of first and second shield drives respectively. At the first drive (Figure 7 left) the real grout volume always lies slightly below the target volume curve. The observations performed in the first shield led to improve the grouting procedure in the second shield as illustrated in Figure 7 right where mostly the real volume curve lies over or overlapping the target curve.

**Figure 7:** Target and real grout injection volume curves at representative excavation cycles below the bridge foundations for the first shield drive (left) and the second shield drive (right)

In conclusion, with the optimum grout injection operation in the second drive full control of the pressurised system was attained (Figure 2), the stress state variations in the ground were governed by the shield operation and consequently minimum settlements were achieved.

6 **SUMMARY AND CONCLUSIONS**

From the results presented in this paper we point out that whilst a greater effort in the numerical simulation and material laws could have improved settlement prediction in 40% (Point PL4 first drive), the improvement of the Process Controlling during execution allowed to reduce settlements in 80% compared to the predicted ones (point
PL3 second drive) and to have 85% less settlements in the second drive compared to the first one.
The need to consider worst case scenarios and safety factors together with the effort required for numerical methods to capture all influence variables does not completely fulfill the target of attaining realistic predictions and providing accurate commands to the shield drive during the works.
The value of numerical methods is undeniable. Without the numerical calculations, most probably the shield drive would have never been planned in these extreme conditions. However, a higher effort invested in the execution real-time Process Controlling in combination with a numerical simulation of the key shield operational parameters for settlement control has proven successful in this project. It allowed to maximise the use of the EPB shield technology potential and reduce additional costs to the project whilst proving and assuring safety during the design and construction stages.

REFERENCES


Shield Tunneling Advancement Simulation using 3D FEM considering Distance Factor and its Validation

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Abstract

A new approach of shield tunnel advancement for soil deformation calculation is presented using 3D finite element method. Applying step by step tunneling loads in this method, total soil deformation is calculated by summation of induced displacement from the first loading step to the current step deformation. In this approach, shield machine face distance from monitoring locations is taken as a factor which affects way of calculating soil displacement. If TBM machine face distance from the monitoring location is assumed to be less than a specific distance say “D”, drained condition is used to calculate soil deformation; on the other hand, if the machine face distance from monitoring location is more than “D”, undrained condition is used for soil deformation calculation. Additionally field data of an EPB shield tunneling site were gathered and used to investigate the validity of proposed approach.

Keywords: EPB tunneling, 3D FEM, advancement simulation, monitoring location
1 INTRODUCTION

Both field study and numerical investigation have shown that tunneling is a three dimensional procedure. Shield machine advancement and tunneling construction sequences using 3D analysis have been investigated by previous researchers (see e.g. [3], [4], and [5]).

In order to simulate earth pressure balance tunneling, authors already proposed a way of 3D analysis of tunnel advancement and loading sequences [1]. In that work, total displacement was obtained based on the sum of the all previous loading step’s displacement plus displacement of the current step. Drained or undrained behavior of soil during tunneling usually was decided by the soil type and the boundary condition. Later on, it was found that using FEM and summation of loading step displacements required to takes into account another factor. In this way, effect of distance (length of drainage path) is also considered which impacts on the drainage condition of ground for soil displacement calculation. Meanwhile, field data of twin EPB shield tunneling were gathered and used here to validate FEA results. Additionally, high permeability of a specific soil type which was observed in field was applied to analysis procedure, when making comparison between field results and FEA outputs.

2 ADVANCEMENT APPROACH OF SHIELD TUNNELING BASED ON DISTANCE OF MACHINE FACE FROM MONITORING LOCATIONS

2.1 General

In this work, FEA is performed using a program that was already developed by Komiya et al. [3]. Tunnel advancement procedure using 3D mesh is carried out by applying step by step tunneling loads. In each step, face pressure at the front of tunnel is calculated by increasing linearly from top to bottom of shield machine face. Tail void grouting is also applied perpendicular to tunnel perimeter through the entire length of one ring exactly at the back of shield machine. Initial vertical stress applied in 3D models is obtained by considering overburden load and ground water table. Then, difference between face pressure and initial earth pressure is applied to elements in front of the tunnel face; similarly difference between grouting pressure and initial earth pressure is also applied at the back of shield machine as an acting force in each loading step.
Applying these forces, nodes of elements are displaced, and stress and strain are developed throughout the mesh. Developed stresses in elements at each step are used as an initial stress for the next loading step. In each step, undeformed mesh is used for applying forces and obtaining stress, strain, and deformation. Then final displacement of any node at any step (any advancement step of tunnels) is obtained by adding of all deformation of previous steps plus developed deformation of current step. Above mentioned procedure has been already described by these authors of [3]; due to necessity, here, its summary is presented:

In front of shield TBM machine, difference between face pressure and earth pressure is applied as follows:

\[
\text{Loading step 1: } P_1 = (F_P - E_P) \\
\text{Loading step 2: } P_2 = F_P - (E_P + S_{t1}) \\
\text{Loading step } n: P_n = F_P - (E_P + S_{n-1}) \tag{1}
\]

At the back of TBM machine, difference between grouting pressure and earth pressure is applied as follows:

\[
\text{Step 1: } G_1 = (G_P - E_P) \\
\text{Step 2: } G_2 = G_P - (E_P + S_{t1}) \\
\text{Step } n: G_n = G_P - (E_P + S_{n-1}) \tag{2}
\]

in which \( P_n \) and \( G_n \) are applied stress as an input loading to the elements in front and back of shield machine in loading step “n”, respectively; \( F_P \) is the face pressure values at loading step “n” obtained from field data; \( G_P \) is the grouting pressure values in loading step “n” obtained from field data; \( S_{t1} \) is induced stress in elements at loading step “n-1”; \( E_P \) is the earth pressure values at loading step “n” obtains using \( E_P = \frac{1}{3}(\sigma_v + 2\sigma_h) \) in which \( \sigma_v \) and \( \sigma_h \) are vertical and horizontal stresses of soil.

2.2 Procedures of Presented Approach

In the previous paper done by these authors, drained or undrained behaviour of soil was determined mainly based on soil type [1]. Using summation of displacement in all of the loading steps in 3D analyses in this approach requires to takes into account another factor. In this paper, effect of distance (length of drainage path) is also taken into consideration. If shield machine face distance from monitoring location is
assumed to be less than a specific distance say “D” same as shown in Figure 1, drained condition is used to calculate soil deformation; on the other hand, if shield machine face distance from monitoring location is more than “D”, undrained condition is used. Considering relationship between drainage path and degree of consolidation, this relationship can be presented using the following equation:

\[ T_v = \frac{c_v \cdot t}{x^2} \]  \hspace{1cm} (3)

in which \( T_v \) is time factor, \( c_v \) is consolidation coefficient, \( t \) is construction time, and \( x \) is drainage path. By approaching of tunnel face to a monitoring location, when tunnel face distance from monitoring locations is decreasing, say less than D, and by assuming relatively small amount of construction time, \( t \), and constant consolidation coefficient, time factor, \( T_v \), is increasing, which leads to the higher degree of consolidation. For high degree of consolidation, drained analysis is preferable as it also suggested by others like Vermeer and Meier for deep excavation [6]. On the other hand, if the distance from monitoring locations is more than D value, undrained analysis is used. In the next part, D value is defined for a specific case study. It should be mentioned that during drained condition, earth pressure is calculated using effective stress of soil, while during undrained condition total stress of soil is used for earth pressure calculation.

**Figure 1:** Drained or undrained condition for soil displacement calculation based on shield machine face distance from monitoring location, “D”.

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*Alireza Afshani, Hiroshi Dobashi, Shinji Konishi, Kazuhiro Komiy, Hirokazu Akagi and Kaho Orihara*
3 SHILED TUNNELING CASE STUDY

3.1 General Description

This site is located in Yokohama, Japan, and intended to be a motorway of total length about 8.2 km. About 5.9 km of this route is a side by side twin tunnel with diameter of about 12.5 m.

Table 1: Soil layers description and their property.

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Description</th>
<th>γ (kN/m^3)</th>
<th>c (kN/m^3)</th>
<th>φ (degree)</th>
<th>E (Mpa)</th>
<th>ν</th>
<th>K₀</th>
</tr>
</thead>
<tbody>
<tr>
<td>B</td>
<td>Fill material</td>
<td>14.0</td>
<td>30</td>
<td>0</td>
<td>1.2</td>
<td>0.45</td>
<td>0.80¹</td>
</tr>
<tr>
<td>Ac</td>
<td>Cohesive soil</td>
<td>15.5</td>
<td>35</td>
<td>3</td>
<td>3.3</td>
<td>0.45</td>
<td>0.80¹</td>
</tr>
<tr>
<td>Ks</td>
<td>Sand and sandstone</td>
<td>19.5</td>
<td>60</td>
<td>42</td>
<td>289</td>
<td>0.3</td>
<td>0.33²</td>
</tr>
<tr>
<td>Kms</td>
<td>Sandy mudstone</td>
<td>19.0</td>
<td>1840</td>
<td>10</td>
<td>492</td>
<td>0.35</td>
<td>0.16³</td>
</tr>
<tr>
<td>Km</td>
<td>Mudstone</td>
<td>18.5</td>
<td>2020</td>
<td>7</td>
<td>430</td>
<td>0.35</td>
<td>0.16³</td>
</tr>
</tbody>
</table>

¹Based on Standard Specifications for Tunneling, Shield tunnel, Japan Society of Civil Engineers, 2006
²Based on Jacky’s formula
³A value of experience obtained during operation of shield machine

Main part of this route is excavated using Earth Pressure Balanced Shield tunneling method. Excavation construction in both lines are being done separately in a way that outbound is 30 meter ahead of inbound line. Length of each lining is 2 meter, in which this distance has been also taken as each advancement length. Soil layers and their material parameters have been shown in Table 1.

3.2 Measured Vertical Displacement at Monitoring Locations

In the Yokohama site, two lines are named as Outbound, and Inbound, each of which has two monitoring locations (MLs). MLs at each line were located of about after 15 (ring No. 8) and 50 m (ring No. 25) from the launching shaft at starting point. In each of these MLs, vertical displacement of the soil is measured at various depths before, during, and after passing of shield machines. Number of measurement devises at each ML has been shown in Table 2. Orientation of measurement devices schematically depicted in Figure 1.

Table 2: Number of measurement devices at each monitoring location.

<table>
<thead>
<tr>
<th>Line</th>
<th>Monitoring location 1 (ML1)</th>
<th>Monitoring location 2 (ML2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Outbound</td>
<td>5</td>
<td>6</td>
</tr>
<tr>
<td>Inbound</td>
<td>6</td>
<td>7</td>
</tr>
</tbody>
</table>
3.3 Simulation of EPB Tunnel Advancement using Presented Approach

3.3.1 Mesh generation

Using longitudinal profile of the Yokohama site, a 3D mesh of the twin lines for about 120 m long was prepared. Diameter of each tunneling (outbound and inbound) is 12.3 meter; length, width, and height of the whole mesh are 120, 127.5, and 54 meter, respectively. Figure 2 shows the whole 3D FE mesh, twin tunnels, and launching shafts, as well as monitoring locations. All of the elements in mesh are 8 nodded cubic. Due to the large scale of 3D mesh, and in order to reduce the time of calculation, soil constitutive model is assumed to be linear elastic.

![Figure 2: Generated 3D mesh.](image)

3.3.2 Estimation of “D” distance for soil deformation calculation based on drained or undrained condition

During tunnel advancement, shield machine face mainly passes through sandstone, mudstone, and sandy mudstone. Mudstone is usually taken as a low permeable soil formation. Field survey in this site shows that after applying of machine face pressure, generated excess pore water pressure is dropping to its initial value after of about 1 or 2 days. Furthermore, taken pictures of the mudstone samples from similar project at Yokohama area manifest that mudstone at this area has some horizontal sand lenses as well as numerous horizontal and long vertical fissures which act as a drainage path. Figure 3 presents an example of mudstone in this area. So, knowing high permeability characteristic of mudstone at this site, it was decided to use machine face distance from monitoring locations, shown “D” distance in figure 1, as a measure to determine drained or undrained condition for calculation of soil deformation.
According to Figure 1, estimation of “D” distance depends on the shield machine advancement rate and soil type. In this site, average daily advancement of machine is around 3 rings per day, as the length of each ring is 2 meter, so, “D” is assumed to be 6 meter in this case.

**Figure 3:** An example of mudstone at Yokohama area. A: Horizontal cracks without any bond between them; B: Long vertical fissures as well as horizontal cracks.

### 4 COMPARISON OF FIELD DATA WITH PREDICTED RESULTS

Presented approach validation is presented in this part by comparing of FEA results with measurement data of Yokohama site. Figures 4 to 7 show vertical displacement of soil at the location of measurement devices based on the distance from monitoring locations (MLs). TBM machine face distance from the monitoring location determines drained or undrained condition for soil deformation calculation. As it was mentioned before, during drained condition, earth pressure is calculated using effective stress of soil while during undrained condition, total stress of the soil is used. Vertical displacements calculated by FEA in the graphs are consistent with measured field data. In both of field data and calculated results by FEA, graphs show a slight jump in two cases, one when machine face is approaching the ML, and the other one is the time when machine tail is passing monitoring section and applying tail void grouting. As the difference between face pressure and earth pressure at the front and difference between earth pressure and tail void grouting pressure at the back of machine is the reason which causes heave or settlement of soil, using of drained or undrained condition considerably affect the earth pressure value.
Alireza Afshani, Hiroshi Dobashi, Shinji Konishi, Kazuhiro Komiya, Hirokazu Akagi and Kaho Orihara

**Figure 4:** Comparison of vertical displacement of soil at monitoring location 1, outbound measured in field with output of Finite Element Analysis.

According to the results of FEA in graphs of 4 to 7, first part of these graphs show a slight settlement. This happens because during the time when shield machine face distance from monitoring location is more than “D” value, undrained condition is used for soil deformation calculation. As in undrained condition, earth pressure is calculated by supposing total stress of soil; therefore, earth pressure value is becomes equal or slightly more than face pressure, and consequently difference between earth pressure and face pressure becomes negative which leads to soil settlement. However, final predicted heaves are in good harmony with measurement results.

**Figure 5:** Comparison of vertical displacement of soil at monitoring location 2, outbound measured in field with output of Finite Element Analysis.
Figure 6: Comparison of vertical displacement of soil at monitoring location 1, inbound measured in field with output of Finite Element Analysis.

Figure 7: Comparison of vertical displacement of soil at monitoring location 2, inbound measured in field with output of Finite Element Analysis.

5 CONCLUSION

New approach of 3D shield tunneling advancement was presented for soil displacement calculation. In this approach, shield machine face distance from monitoring locations is taken as a factor which affects way of calculating soil displacement. If shield machine face distance from monitoring location is become less than a specific distance say “D”, drained condition is used to calculate soil deformation; on the other hand, if shield machine face distance from monitoring location becomes more than “D”, undrained condition is used. In the case of undrained condition, total stress and in the case of drained condition, effective stress is used to calculate earth pressure. Comparing of FEA results using presented approach showed
good harmony with field measurement. Because of the presented method’s simplicity and its good accuracy, it is a useful tool for 3D soil deformation prediction. Additionally using this method, soil deformation due to the neighbour tunneling construction is also well-predicted.

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Computational Framework for 3D Adaptive Simulation of Excavation and Advancement Processes in Mechanized Tunneling

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Abstract

The simulation of the advancing process for arbitrary alignments during shield tunneling requires a continuous adaption of the finite element mesh in the vicinity of the tunnel face in conjunction with a steering algorithm for the Tunnel Boring Machine (TBM) advance. Moreover, the finite element mesh should match the actual motion path of the shield machine resulting from the FE-analysis in each excavation step. For this purpose, a steering algorithm that provides the numerical model with the required information to keep the TBM on the track is introduced. This algorithm necessarily requires automatization of mesh generation in the vicinity of the tunnel face within the advancing process. The steering algorithm and the 3D automatic mesh generation are continuously coupled during the numerical simulations of shield driven tunneling processes. The applicability of the proposed approach for capturing the projected excavation path and to perform numerical simulation of TBM drives along curved alignments is demonstrated.

Keywords: Steering algorithm, hybrid re-meshing, mechanized tunneling
1  INTRODUCTION

In numerical simulations of shield driven tunneling processes, the realistic modeling of both the excavation process and the advancement of the Tunnel Boring Machine (TBM) is a challenge. For a better understanding of these processes during tunnel construction, the interactions between the shield machine and the surrounding soil need to be investigated, yet this excavation process is difficult to model with existing finite element models. In addition, the simulation of the advancement of the machine as an independent body, that interacts with all relevant component of the model, requires a realistic kinematics model of the shield machine which is often not included [14]. A prototype for a process-oriented three-dimensional finite element model for simulations of shield-driven tunnels in soft, water-saturated soil has been developed and successfully used for systematic numerical studies of interactions in mechanized tunneling [7]. This model has been re-formulated and extended to partially saturated soils and more advanced constitutive models for soils in the context of a integrated design support system for mechanized tunneling (see, e.g. [11, 10]). Furthermore, several finite element models have been proposed, addressing the difficulties inherent in the simulation of the excavation process. Many of these models account for excavation by removing finite elements from the excavated volume in front of the machine, and then by applying the nodal forces necessary to preserve equilibrium [1, 3]. A more realistic representation of the excavation process, based on mesh adaptation, using so-called "excavating elements" in front of the machine has been proposed by [8]. In this paper, a steering algorithm is presented in the context of 3D modeling TBM advancement processes. The algorithm serves as an virtual guidance system which automatically determines the exact position and the driving direction of the TBM in three dimensional space. Hereby, the model provides the vertical and horizontal deviations of the machine, its shield orientation, and provides direct input for the jacking cylinders. In addition, a problem specific re-meshing procedure is introduced for the simulation of the excavation process. In this process, a hybrid mesh generation procedure adapts the spatial discretization in the vicinity of the tunnel face, according to the actual position of the TBM, in order to better capture the excavated geometry.

2  COMPUTATIONAL MODEL

The numerical 3D finite element method for shield tunneling used here as a basic framework for the new TBM advancement model has been presented in a number
of publications (see, e.g. [11]). It has been implemented in the object-oriented finite element framework KRATOS [4] and is denoted as ekate (Enhanced Kratos for Advanced Tunneling Engineering). This finite element model takes into account all relevant components involved in shield tunneling such as the tunnel boring machine (TBM), the hydraulic jacks, the lining structure, the frictional contact between the shield skin and the soil and the supporting measures at the face and the tail gap, respectively, and their interactions.

![Figure 1](image_url)

**Figure 1:** a) Main components involved in mechanized tunneling: (1) soil, (2) tail gap, (3) pressurized support medium, (4) cutting wheel, (5) shield skin, (6) hydraulic jacks and (7) segmented lining. (b) Modeling of interactions between soil and TBM in the simulation model ekate: (1) heading face support, (2) frictional contact between shield skin and soil and (3) grouting of the tail gap) and components of the simulation model: TBM, hydraulic jacks, lining and grouting mortar

Figure 1 shows the main components involved in mechanized tunneling (left) and their representation in the finite element model (right). These components are considered as independent sub-models interacting with each other. The interaction between the surrounding soil and the shield machine is accounted for by means of a surface-to-surface frictional contact formulation in the framework of geometrically nonlinear analysis [9]. The hydraulic jacks are represented as CRISFIELD truss elements connected to the surface of tunnel lining element from one side and to the pressure wall of the TBM by tying utility. The simulation of the tunnel advances is performed in a step wise procedure. The soil and the grout are formulated within the framework of the theory of porous media (TPM) as two-phase materials.
3 FINITE ELEMENT MODELING OF THE ADVANCING PROCESS

3.1 The kinematic model of the shield machine
The realistic modeling of the advancing process and the interaction between the TBM and the surrounding environment requires a realistic kinematic model of the shield machine [5]. Therefore, a nonlinear kinematic analysis of the shield, based on the action forces imposed on the shield and on the inertial forces due to the shield, is performed. The action forces result from hydraulic jacks pushing against the machine, earth/slurry pressure at the cutting face, friction with surrounding soils, and the fluid flow of processes of the support fluid and grouting mortar, whereas the inertial forces are due to the self weight of the shield and of the equipment. Furthermore, the taper and the thickness of the shield skin are accounted for in the geometrical representation of the TBM, guaranteeing a realistic distribution of the ground reaction forces in both circumferential and longitudinal directions. A Lagrangian finite element analysis of large deformations that satisfies both the displacements and forces boundary conditions imposed by shield machine operation provides the actual TBM kinematics. Within this approach the shield machine is modelled as a deformable body using total Lagrangian finite element, as using the total lagrangian elements avoids nonphysical distortion of the shield geometry.

3.2 Steering correction algorithm
The TBM is advanced by hydraulic jacks that are attached to the machine which push against the previously installed lining ring. The pressure exerted by these jacks must overcome the resistance generated by the surrounding soil. Moreover, since the machine is heavier at the head, the jack forces are highest in the invert and conversely the lowest in the crown. Driving the shield along curves requires additional steering forces along the sides to ensure the machine follows the intended three dimensional curve. Furthermore, the resistance of the ground and the self weight of the machine could cause a deviation from the described path and this would require steering forces to correct the TBM trajectory. When the steering or the so-called deviation correction is needed, the pressure at individual hydraulic jacks is adjusted to produce deflection torques in the horizontal and vertical direction. In the computational model, the shield machine is pushed forward by extending hydraulic jacks represented by CRISFIELD truss elements. These are connected to both the surface of the lining and the shield. The jack elongations are accomplished by providing the initial strains which describe the desired elongation. Respective values for the jack
pressures may be obtained as a simulation result [12]. In accordance with tunneling practice, a reliable steering algorithm that provides the numerical model with the required information to keep the TBM on the track is developed. This TBM advancement algorithm serves as an artificial guidance system which automatically determines the exact position and the driving direction of the TBM in three dimensional space providing the vertical and horizontal deviation, shield orientation and direct input for the jacking cylinders. -

Figure 2: Jack thrusts exerted by hydraulic jacks on the shield wall and lining

In the aforementioned steering algorithm, the jack forces exerted by hydraulic jacks on lining segment in advancing direction are obtained as the result of the simulation. Figure 2 shows the result of the computed jack forces during the advancement along straight path. As expected the jack forces are highest in the invert and conversely the lowest in the crown.

4 AUTOMATIC MODELING OF THE EXCAVATION PROCESS

The simulation of the advancing process for arbitrary alignments by means of the proposed steering algorithm requires a continuous adaption of the finite element mesh in the vicinity of the tunnel face. Furthermore, the finite element mesh should match the actual motion path of the shield machine resulting from the FE-analysis in each excavation step. For this purpose, a re-meshing algorithm is developed in order to automate the process of mesh generation in a domain in the vicinity of the tunnel face within the advancing process.
4.1 The meshing algorithm

The main aim of this meshing algorithm is to describe the new geometry by generating a new mesh automatically. By using the so-called piecewise linear system (PLS), not only is the boundary of excavation path described but also the external boundary of the re-meshing domain. Therefore, the internal facet of the PLS should be automatically updated to describe the new internal boundary, where the external facets are still fixed. This can be realized by sweeping the new facets representing the new excavated part and adding it to the PLS after each advancing step. The kinematic analysis of the shield within the steering procedure provides the exact position and geometry of the new facets as well as the center of the cutting wheel and some reference points. To mesh the domain, a 3D Delaunay meshing algorithm TetGen [6] is used to generate unstructured mesh consist of tetrahedral elements. Delaunay-based algorithms are capable of producing quality meshes and provide control over mesh sizing throughout the domain.

4.2 Region of interest

The region of interest is the region of the mesh that is continuously generated during TBM advance in the vicinity of the tunnel face. It must include at least the excavation geometry. It can be as large as the whole simulation domain or be limited to a small region around the heading face. In any case, the engineer will make the decision based on the available data and his engineering experience. To enable a dynamic and efficient simulation of arbitrary TBM advancement paths, information of the target excavation path is incorporated in the definition of the region of interest. For this purpose, a hybrid mesh approach, attempting to combine the advantages of both structured and unstructured mesh layouts is introduced. The resulting mesh of this hybrid approach will automatically match both the external boundary in terms of connectivity to an existing boundary mesh and, internally, the projected motion path of the shield and the heading face as shown in Figure 3.

4.3 Modeling of excavation

The re-meshing algorithm works in conjunction with the steering algorithm. The steering algorithm simulates the advancing process in a step-by-step procedure. After each advancing step the re-meshing algorithm is invoked and generates a new computational mesh describing the new excavation geometry. The re-meshing al-
Figure 3: Hybrid mesh representation of excavation geometry; (a) Mesh components (b) Compatible mesh using hexahedral and tetrahedral elements.

gorithm uses the results from the steering algorithm as input for generating the new mesh. The exact geometry and position of the TBM after each advancing step will be extracted and used to generate the new mesh preserving the deformed configuration of the previous excavated geometry. By doing so, the excavation and the advancement of the shield machine are numerically simulated in a continuous manner. After obtaining the new mesh, several mesh operations and optimization techniques are required as follows:

- Optimization algorithm to project all central nodes of the higher order tetrahedral elements to their correct position in order to represent the exact curve and the circular shape of the shield.

- Generation of a new surface mesh to represent the excavation boundary, tunnel face and contact surfaces.

- Variable transfer algorithm: after the re-meshing, the values of the stresses and the internal variables on the new mesh need to be calculated from those obtained in the original deformed mesh. This is necessary because the equilibrium condition needs to be satisfied before conducting the next advancing step. An appropriate algorithm based on Superconvergent Patch Recovery (SPR) [2] for the transfer of these internal variables is adopted.
5 NUMERICAL APPLICATION TO CURVED TUNNEL ADVANCE

Figure 4: L9 Tunnel "Mas Blau"; (a) representation of the simulation domain (b) tunnel alignment and the position of three cross section

Table 1: Material parameters used in the finite element model

<table>
<thead>
<tr>
<th>model part</th>
<th>$\gamma$ [kN/m$^3$]</th>
<th>$\varphi$ [°]</th>
<th>$c$ [MPa]</th>
<th>$E$ [MPa]</th>
<th>$\nu$ [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lining</td>
<td>25.0</td>
<td>-</td>
<td>-</td>
<td>30000</td>
<td>0.2</td>
</tr>
<tr>
<td>Shield</td>
<td>76.2</td>
<td>-</td>
<td>-</td>
<td>210000</td>
<td>0.27</td>
</tr>
<tr>
<td>Soil</td>
<td>28.5</td>
<td>29</td>
<td>0.11</td>
<td>2100</td>
<td>0.28</td>
</tr>
</tbody>
</table>

The advancing process and the application of the new excavation technique are demonstrated by considering the southwestern part of the L9 tunnel "Mas Blau". It consists of a single twin-track tunnel that runs along a strongly curved path. A review of the geological situation around the "Mas Blau" site has been provided by [13]. The tunnel is characterized by a diameter (D) of 9.4 m and a cover depth of 14.0 m. The tunnel alignment and the simulation domain are shown in Figure 4. In this study, the elastic model for the lining and the shield machine with basic material properties summarized in Table 1. For the modeling of the soil, a DRUCKER-PRAGER plasticity model is used.

The shield advancement and the excavation process were simulated using the described steering and re-meshing algorithm, respectively. The simulated and the designed tunnel course in vertical and horizontal planes are shown in Figure 5. As can be seen in these figures, the TBM does not move steadily along its course, instead it wriggles along the tunnel path due to the alternating jack forces.
Figure 5: Simulated and designed tunnel path: a) X-Y plane, and b) X-Z plane

Figure 6 shows the jack forces in three different positions during the simulation of the advancing process. The nonuniform jack thrust distributions illustrate the power of the steering algorithm and the efficiency of the algorithm that controls the TBM position and keeps it on the intended alignment. The diagram in Figure 6 shows the change of the steering forces during the steering process in the horizontal and vertical directions. Thus, the finite element results are consistent with the actual shield advancement procedure and guidance system, respectively, that is used to provide the operator with the required information to keep the machine on course. The results of the re-meshing algorithm are introduced in Figure 7 for different advancing steps. The results obtained from this simulation demonstrate the efficiency and the high applicability of the re-meshing algorithm to capture the exact excavation path. In addition, it is possible to advance the TBM at different speed, the meshing algorithm will account for the size of the mesh automatically.

6 CONCLUSIONS

A computational framework for simulating the advancement and the excavation process of shield machines during TBM tunneling using the finite element method was proposed. A kinematic description of the movement of the TBM was used to simulate the advancement process along arbitrary (curved) tunnel alignments. The simulation model is able to account for drift off phenomena observed in TBM tunneling and always keep the TBM on the desired course. The hydraulic jack forces are computed from the simulation of the advancement process. For the modeling of the excavation
process a problem specific re-meshing strategy has been introduced. The advancement process of the TBM and the soil excavation was simulated step wise using a coupled steering and re-meshing algorithm to represent the exact ground movement around the shield. Thus, the settlements of the excavated surface are predicted depending on the actual TBM position and TBM-ground interactions. It was shown, that the proposed model is able to simulate the motion of the TBM along arbitrary paths with minimum effort required by the user in the preprocessing stage. Further development of the model will focus on incorporating the influence of the discretization error in the re-meshing strategy during the excavation process.
Figure 7: FE mesh representing the excavation geometry for different advancing steps (Section-1, Section-2, Section-3 in Figure 4)

7 ACKNOWLEDGEMENTS

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REFERENCES


Numerical Simulation of Ground Movements Due To EPB Tunnelling in Clay

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Abstract

This paper compares the results of two different 3D finite element models for simulating EPB mechanized tunnelling in clay. One model is created in Plaxis 3D™ and the other using the Kratos-Ekate software. The Plaxis model represents the construction process as a sequence of discrete steps, where soil elements are deactivated, and the conical shield is modelled by imposing appropriate boundary conditions at each step. The segmental concrete lining is brought into contact with the surrounding soil by activating an annulus of grout with time-dependent mechanical properties. In contrast, the Kratos model provides a process-oriented simulation that represents the TBM as a distinct deformable structure in frictional contact with the soil. The shield is advanced using an array of hydraulic jacks that react against the lining system and can be used to control the shield orientation. The Kratos model offers advantages in simulating the TBM trajectory, but involves much greater computational complexity. In this paper we compare results from the two FE models, using a linearly elastic-perfectly plastic (MC) soil model and similar grout properties, to establish how details of the tunnel construction simulations affect the predicted ground movements.

Keywords: Tunnelling, EPB, finite element model, ground deformations, lining forces
1 INTRODUCTION

Mechanized tunnelling is an established and flexible technology for the construction of tunnels in urban areas. The tunnel construction process in soft soils causes short and long term ground deformations resulting from a disturbance of the in situ stress state of the soil and pore pressures due to the heading face support, the shield skin friction and the gap grouting. The procedure for controlling the advance of the EPB machine can affect significantly the development of far-field soil deformations, as well as the structural forces within the lining system. Therefore, analyses of the ground response require accurate modelling of the constitutive material behaviour and construction process. Finite element (FE) methods have been used to simulate tunnel construction since the early 1980s, but there are still relatively few studies involving three-dimensional modelling of mechanized tunnelling [e.g., 1 - 4] compared to conventional tunnelling (i.e., open-face, sequential excavation and support). A process-oriented three-dimensional (3D) finite element model for simulating mechanized tunnelling in soft ground conditions has been proposed in [5] and demonstrated in simulations of slurry shield tunnelling in clay [6]. More recently this model has been extended for tunnels in partially saturated soft soils [7]. However, increasing the complexity of the numerical model generates very large computational costs, while each component of the model is still subject to certain assumptions.

In order to answer the question of an optimal balance between accuracy of the solution and complexity of the model, two different modelling approaches are presented in this paper. A comparative study investigates the effect of the mechanized tunnel excavation on ground settlements and structural forces in the lining by means of Plaxis 3D™ and Kratos-Ekate [8] model. In order to capture the influence of specific modelling assumptions, the two models are calibrated using a simplified reference excavation procedure (Appendix A). For this reference situation, the model parameters are calibrated such that their response matches the analytical solution presented in [9]. The comparison is focused on the surface deformations, and axial forces and moments in the lining. Further, numerical issues relating to element types and numerical integration are also addressed briefly in this paper.
2 MODELING OF EPB TUNNELLING

Two different approaches are used to simulate the shield tunnelling process. One numerical model is created using Plaxis 3D™ FE program and the second model is created using Kratos, an object-oriented FE framework for multi-physics simulations. Both models are based on the geometry of the Herrenknecht EPB machine currently being used to bore tunnels for the Crossrail project in London, with a 7.1 m diameter cutting wheel and 12 m tapered steel shield. The analyses consider ground conditions associated with a "base" tunnel design section with a cover depth of 12.45 m (16 m to springline) below ground surface, where the tunnel is excavated within the London Clay. The clay extends to a total depth of approximately 60 m and overlies the Lambeth Group (approximately 12 m thick) and the Thanet sands (lower aquifer). The analyses consider an idealized 100m long straight horizontal trajectory for the EPB machine within a uniform 60 m deep clay layer. The FE model assumes a lateral boundary located 300 m from the tunnel centerline (to ensure accurate representation of far field ground movements), with symmetry in the longitudinal plane such that only a half-section of the tunnel (and EPB machine) is represented. Both models are using same geometry (300 × 100 × 60 m), and the tunnel construction is performed within 66 excavation rounds (each 1.5 m long). The models assume undrained shear conditions within the clay (i.e., the model assumes there is no migration of pore water within the clay mass over the time frame of the tunnel construction), and represent a profile where undrained shear strength varies linearly with depth (Table 1). The analyses assume that the groundwater table is coincident with the ground surface and pore pressures are hydrostatic. Calculations have been performed for two different in situ stress conditions, $K_0 = 1.0$ and 1.5.

2.1 Plaxis Model

This section describes the development of the 3-D FE model using the general purpose geotechnical software Plaxis 3D™. The model uses 10-node solid tetrahedral elements with quadratic interpolation of displacements (and pore pressures) to represent the soil mass, while the tunnel lining is simulated using 6-noded plate elements. The model assumes zero lateral displacements along the exterior vertical boundaries of the mesh. The clay is modelled as an elastic-perfectly plastic material with a Mohr-Coulomb yield criterion [MC] that is subject to undrained shearing. Table 1 lists the input parameters used to represent the London Clay profile using the MC model.
Figure 1: Modelling approaches for EPB mechanized tunnelling: a) Plaxis 3D™ b) Kratos-Ekate

Initial horizontal stresses are specified by an assumed $K_0$ condition. Figure 1a shows a schematic figure of the boundary conditions used to represent the EPB tunnel-boring machine in the Plaxis 3D FE model. The 12 m long shield is represented by a set of 8x 1.5 m long segments with uniform radial displacement-defined boundaries that approximate the conical surface of the shield. The face conditions are represented through a uniform face pressure (the current base case assumes 150 kPa). Conditions in the tail void and initial ring assembly are represented by a uniform grout pressure (with grout pressure of 100 kPa), which extends over one tunnel segment. The lining is then activated with an initial external diameter, 6.8 m and a ring of hardening grout is activated around the lining. Table 1 summarizes the elastic properties of the lining and grout. In order to account for the effects of grout set-up properties, a time hardening model [6] is used to represent the time dependent stiffness of the activated...
grout. Since the EPB machine is assumed to advance forward in steps, the time parameter is introduced by assuming a reasonable excavation rate of 1 m/h [11]. As a result, each excavation step corresponds to a 1.5 h time step.

Table 1: Material properties used in FE models

<table>
<thead>
<tr>
<th>Material</th>
<th>Model</th>
<th>ρ [kg/m³]</th>
<th>E [MPa]</th>
<th>v</th>
<th>Su [KPa]</th>
<th>E1/E28</th>
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<td>P</td>
<td>Mohr Coulomb</td>
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<td>15+1.1·z</td>
<td>0.25</td>
<td>7.5+ 0.55·z</td>
</tr>
<tr>
<td></td>
<td>K</td>
<td>Drucker Prager</td>
<td></td>
<td></td>
<td>0.5</td>
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<tr>
<td>Grout</td>
<td>P</td>
<td>Kasper 2005</td>
<td>1,500</td>
<td>[0.5-100]·10¹</td>
<td>[0.18-0.4]</td>
<td>-</td>
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<tr>
<td></td>
<td>K</td>
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<td>50</td>
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<tr>
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<td>7,600</td>
<td>210,000</td>
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</table>

2.2 KRATOS Model

For this comparative study, a second model has been created using the object-oriented FE framework KRATOS [11]. It has been designed to match the PLAXIS model by means of deactivation of soil elements and installation of tunnel lining and grouting elements. It is a simplified version of the “ekate” model for process-oriented simulation of mechanized tunneling (see [8]) and accounts for the shield as a deformable body moving through the soil and interacting with the ground through frictional surface-to-surface contact. The volume loss due to the excavation process follows naturally the real, tapered geometry and the overcutting of the shield machine. The soil is modeled as a two-phase fully saturated material [5], accounting for solid and water as distinct phases according to the theory of mixtures, where the plastic behavior of the solid phase is described by a Drucker-Prager plasticity model. Grout is modeled as a fully saturated two-phase material with a hydrating matrix phase, considering for time-dependent stiffness and permeability of the matrix material of the cementitious grout [12]. Therefore, the grouting pressures are applied at the front face of the grout elements instead of directly at the soil (see Fig. 1b). The excavation of one slice of soil (1.5 m) is performed in three successive steps, which provides
continuous temporal development of excavation-induced settlements. All material properties have been chosen similar to the Plaxis model and have been calibrated by means of a reference excavation model (see Table 1 and Appendix A). The FE mesh of the model uses 27-node hexahedron elements for all components (soil, lining, grouting, shield) and has a total number of 453600 DOFs. For future comparisons, the full-featured version of the “ekate” model will be used, adding simulation of the steering process by means of hydraulic jack elements to study the influence of free movements of the shield and fluctuations in jack forces on the loading of the lining and the behavior of the surrounding ground.

3 RESULTS AND COMPARISONS

3.1 Comparison of Surface Settlements

The two modelling approaches are compared by examining the differences between the ground surface displacements and the structural forces in the lining. Figure 2a-d compares the vertical and horizontal components of surface deformations along the transverse mid-plane of the FE model ($y' = 0 \text{ m}$) for different $K_0$ values (1.0-1.5) at three reference locations of the EPB machine, $y'/D = 7.1, 0.7$ and -6.8 (where $y'$ is the longitudinal distance from the center of the FE model and $D$, the nominal lining diameter). At the first location represents conditions the EPB shield becomes fully embedded in the FE model, at the second the face approaches the central-plane of the model and at the third the face of the machine approaches the rear face of the model.

It can be seen that both vertical and horizontal displacements of Plaxis and Kratos show good agreement for $K_0=1.0$, for all the three positions of the machine. This can be explained by the fact that for $K_0 = 1.0$, the main deformation mode of the excavation boundary is uniform convergence. For this deformation mode Plaxis and Kratos models produce similar deformation shapes at the tunnel cavity i.e., the imposed uniform boundary displacements in Plaxis produce a similar shape for the excavation boundary as the approach used in Kratos, where the excavation boundary freely deforms by means of relaxation of stresses.

In general, the analyses with $K_0 = 1.5$ produce larger far field settlements ($x > 15-20 \text{ m}$) and larger lateral deformations than the analyses with $K_0 = 1.0$. While the horizontal displacements are similar for the two FE models, the vertical displacements differ substantially for the $K_0 = 1.5$ case (for $x \leq 20 \text{ - 30m}$). This is explained by the fact that deformations of the excavation boundary for $K_0 = 1.5$ ovalization as well as uniform convergence modes [12]. The imposed boundary conditions used to simulate


the machine in Plaxis do not capture the ovalization but predict a wider settlement trough (than for $K_0 = 1.0$). In contrast Kratos predicts very small centreline settlements due to passage of the EPB shield ($y'/D \leq 0.7$ to -6.8) with maximum surface settlements occurring at an offset location, $x = 10 - 15m$. While these results are readily explained from the numerical models, they have yet to be resolved with respect to real field data.

Figure 2: Comparison of surface settlements trough for Plaxis and Kratos FE models for different positions of machine with respect to middle cross section

Figure 3 shows the development of surface settlements as a function of time for the longitudinal mid-plane. For $K_0 = 1.0$ the surface settlements troughs of the two models are similar. Plaxis predicts larger settlements ahead of the EPB machine since the imposed boundary displacements are larger than the free deformations of the
tunnel cavity (allowing the formation of the gap) in Kratos. However final settlements of the two models are comparable at \( y'/D = -6.8 \). There is a significant difference in surface settlements for the \( K_0 = 1.5 \) case seen also in Figure 3. It becomes clear that the ovalization mode is represented in Plaxis, only after the passage of the EPB machine (after 50m advance), where the surface settlement trough starts to diverge from the \( K_0 = 1.0 \) case. In contrast differences between the two \( K_0 \) cases become apparent at the excavation face, and much smaller settlements are predicted for the \( K_0 = 1.5 \) case.

![Figure 3: Comparison of surface settlements trough for Plaxis (P) and KRATOS (K) model for different positions of machine with respect to middle cross section](image)

3.1 **Comparison of Structural Forces**

Figure 4 compares the computed structural forces in the lining for the two models after tunnel construction. The structural forces were plotted for a reference lining ring located at the mid-plane. The plotted values correspond to values averaged first over the ring elements and then over the ring width.

It can be seen that the differences in structural forces are negligible for \( K_0 = 1.0 \). As expected, the bending moment is almost zero for \( K_0 = 1.0 \) (uniform convergence), whereas for \( K_0 = 1.5 \) it is positive for the springline and negative for crown and
Figure 4: Structural forces in linings for two different modelling approaches and $K_0$ values, for the lining ring in the middle after construction process is completed invert (ovalization). For the bending moment there is a good match between the two models for both $K_0 = 1.0$ and $K_0 = 1.5$. For the axial force, the two models give similar results for $K_0 = 1.0$, while for $K_0 = 1.5$ there is a big difference at the springline due to the significant effect of the ovalization mode. The observed mismatch is smaller than expected, due to the small initial stiffness of the grout that allows for significant deformations, absorbing the imposed displacements by the surrounding soil. As a result, only a part of the displacements affects the lining.
4 CONCLUSIONS

This paper compares results of two three-dimensional FE models for simulating the mechanized tunnel construction. The Plaxis model uses simplified boundary conditions to represent the EPB tunnelling process, while Kratos represents the TBM as a distinct deformable structure in contact with the soil. The analyses simulate the Crossrail EPB machine and assume undrained conditions with stiffness and strength parameters typical of London Clay. Results are compared for cases with $K_0 = 1.0$ and $1.5$. The two FE models produce quite similar predictions of surface deformations and lining forces for the $K_0 = 1.0$ case. However, there are large differences in the predicted behaviour at $K_0 = 1.5$ that can be directly attributed to the assumed boundary conditions of the two models. The more comprehensive Kratos model predicts an ovalization of the tunnel cavity resulting in smaller surface settlements above the tunnel. Lining forces are quite comparable for both $K_0$ cases, although Kratos tends to predict smaller axial thrusts and larger bending moments at the springline for $K_0 = 1.5$ than Plaxis. While comprehensive process oriented FE models such as Kratos are clearly superior for representing (and controlling) the advance of a tunnel boring machine, the predictions of the simpler Plaxis model appear to provide very reasonable predictions of far-field ground deformations and tunnel lining forces.

5 ACKNOWLEDGEMENTS

Research on tunnel modelling at MIT has been supported by Ferrovial-Agroman, the second Author (JN) has been supported by the Ruhr-University Research School funded by Germany’s Excellence Initiative (DFG GSC 98/1). Additional Support was provided by the German Science Foundation (DFG) in the framework of the Collaborative Research Center SFB 837. This support is gratefully acknowledged.

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APPENDIX - VALIDATION OF SOIL MODELS

Plaxis (Mohr-Coulomb) and KRATOS (Drucker-Prager) soil models are calibrated using a simple excavation test. A simple model with the same dimensions (300 × 100 × 60 m), boundary conditions and soil characteristics as the tunnelling model is created in both, Plaxis and Kratos. Further, a simple excavation test is performed, where the elements representing the excavation sequence are removed, and the soil freely deforms at the excavation boundary by relaxation of in situ stresses. This test is performed for both software for $K_0 = 1.0$ and $K_0 = 1.5$. In Figure A1 the horizontal and vertical displacements at the soil surface for both models are presented. As it can be seen, there is a near-perfect match for the results of both models.

![Figure A1: Structural forces in linings for two different modelling approaches and $K_0$ values, for the lining ring in the middle after construction process is completed](image)

Figure A1: Structural forces in linings for two different modelling approaches and $K_0$ values, for the lining ring in the middle after construction process is completed
On the Effects of the TBM-shield Body Articulation on Tunnelling in Soft Soil

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Abstract

When a Tunnel Boring Machine (TBM) is driven in soft soil, the TBM-shield constantly interacts with the surrounding soil profile excavated by the cutting wheel. The interaction pattern of shield-soil interface displacements determines compression and extension sectors in the surrounding soil. Soil compression is generated when the shield displaces the excavated profile in outward direction; soil extension happens when the shield fits inside that profile. This aspect of TBM behaviour, referred to as shield-soil kinematical interaction, has been demonstrated in a recent study investigating the monitoring data from the Hubertus tunnel in The Hague. The TBM used at the Hubertus tunnel was not equipped with a shield-body articulation. The articulation, designed to limit the undesired shield-soil interactions of the kind described, was present in the TBMs used at the North-South metro line in Amsterdam. This study aims to quantify the consequences of using a shield articulation in terms of shield-soil kinematical interactions. The study, comparing the results from the Hubertus and the North-South line tunnels, revealed remarkable differences, although other discriminating aspects have to be accounted for. The fundamental understanding of the kinematical interactions is crucial to building reliable numerical models for TBM driving in soft soil.

Keywords: TBM, kinematics, shield-articulation, interaction
1 INTRODUCTION

During the last decade advanced numerical models were developed describing the staged construction process of mechanised shield tunnelling [3]. Those models, usually accounting for detailed aspects such as TBM features, operational choices, and process fluids’ handling, can potentially lead to tailored predictions on the effects of tunnelling in soft soil [2]. However, the current understanding of the interaction between the slurry-shield TBMs and their surroundings does not appear sufficiently detailed yet to be fully captured by those models [1]. It is clear, however, that the shield-soil interaction, and especially the ground displacement around the shield periphery, gives a significant contribution to the overall soil deformation [4]. Other aspects such as soil excavation, face support strategy, tail-void grouting, grout consolidation, and tunnel lining deformation contribute as well to the final deformations, but are not investigated here.

This paper focuses on the physical interaction between a TBM-shield driving in soft soil and its surroundings. A model capturing several aspects of the kinematic behaviour of a TBM will be presented. This has been verified by comparing the TBM monitoring data obtained during the construction of the Hubertus Tunnel, a double-tube road tunnel in The Hague, and of the North-South line metro tunnel in Amsterdam. The study revealed differences in terms of amplitude and spatial distribution of the ground displacement around the shield periphery as they occurred in practice. However, other discriminating aspects such as shield diameter and geometry and alignment’s curvature have to be accounted for. The results suggest that the fundamental understanding of the kinematical interactions between a TBM and its surroundings is crucial to building reliable models for TBM driving in soft soil.

2 REFERENCE PROJECTS

2.1 The Hubertus Tunnel – The Hague

The Hubertus Tunnel was constructed between 2006 and 2007, and consists of two parallel tubes, north and south, 1666.70 m and 1653.48 m long, respectively. Each of the two 9,400 mm diameter tunnels contains two car lanes. Situated in a residential area, the tunnel passes close to the foundations of some houses and in part underpasses several low buildings on a military barracks. The non-articulated Hydroshield-type TBM used was 10,235 mm long, with a front diameter of 10,510 mm, and a rear one of 10,490 mm. A standard radial overcutting of 10 mm was also
used. The cutting wheel was supported by a longitudinally displaceable spherical bearing. The tail-void grouting system consisted of six injection openings, of which only the upper four were actually used. The final concrete lining was constructed with 2 m long rings with an external diameter of 10,200 mm. The theoretical tail void thickness was 165 mm. As in most bored-tunnel projects, overcut, shield length and tapering had been optimized according to the alignment’s sharpest curve. The smallest curvature radius was in the south alignment and amounted to 542.300 m. The tunnel tubes were bored from east towards west and the sharpest curve was in leftward direction. The deepest point of the tunnel axis was located 27.73 m below surface. The tunnel was mainly driven through dune sand consisting of well packed silty sands and sandy silts with some clay. The tip resistance $q_c$ of the cone penetration tests ranged between 10÷40 MPa in the layer crossed by the tunnel [5].

2.1 The North-South Line Tunnel – Amsterdam

The eight single-track bored tunnels serving the new North-South metro line were constructed between April 2010 and December 2012 with four twins slurry-shield TBMs. The analysis will focus on only one of the tunnels, namely the eastern one of the two connecting Amsterdam Central Station with the upcoming Rokin Metro Station. The tunnel, with a length of 723.90 m, was bored from north towards south, and its sharpest curve with a curvature radius of 240 m was bored in leftward direction. The tunnel crosses the very heart of Amsterdam, and although the alignment is entirely located underneath public roads. At several locations, the excavation occurs as close as 3 m from the foundations of adjacent historical buildings founded on wooden piles. The Hydroshield-type TBM used was 7,920 mm long, equipped with a mid-length articulation feature able to provide an extension of additional 200 mm, equivalent to an articulation angle between front and rear sectors of about 1.82°. The shield front part had a diameter decreasing from 6,880 mm to 6,875 mm, and the rear part an even diameter of 6870 mm. The cutting wheel, also supported by a longitudinally displaceable spherical bearing could be completely retracted within the TBM-shield, but also shifted in front of it, with the capability to produce a radial overcutting of 18.6 mm. The tail-void grouting system consisted of six injection openings distributed along the tail circumference and constantly used during drive. The final concrete lining was constructed with 1.5 m long rings with an external diameter of 6,500 mm. The theoretical tail void thickness ranged from 208.6 mm to 190 mm. The deepest point of the tunnel axis was located at -23.946 m N.A.P.
(Dutch Reference System), and approximately 25 m below surface. The tunnel was mainly driven through the so-called second sand layer, consisting of relatively densely packed, moderately coarse sand. The tip resistance $q_c$ in that sand layer ranged between 20–30 MPa.

### 2.2 TBM Guidance System

The combination of measuring devices and reference points located either inside the shield and along the lining in place provides position and spatial orientation of the TBM. The driving system is based on two virtual points of the shield which are supposed to follow the planned tunnel alignment. Both lay along the longitudinal axis of the shield, the first one (front), in the plane of the shield face, and the second one (rear) around the mid-length of the shield (RPF and RPR in Figure 1, respectively).

![Figure 1: Shield positioning system: reference points and measuring devices (courtesy of VMT GmbH)](image)

Every few seconds the monitoring system provides to the TBM operator the horizontal and vertical deviations of the reference points from the planned alignment. Positive values are arbitrarily given to rightward and upward deviations. Other derived values were also provided (e.g. tendencies, pitch, roll, yaw).
Machine operators constantly aim to follow the planned alignment with both target points. However, this is not always possible as it sometimes requires high steering forces, therefore increasing the risk of damaging the concrete lining already in place. In those cases it may be preferable to advance with a skewed orientation of the machine when this involves smaller driving forces. The skewing required for a smooth driving may differ in direction and amount along the alignment, and its quantification is expected to provide a useful insight into the interaction of the TBM-shield with its surroundings.

3 KINEMATIC MODEL FOR A TBM

3.1 Theoretical Model

As seen, the motion of a TBM-shield can be fully described by the consecutive positions occupied by two of its points (if roll is disregarded). Additionally, the motion of an undeformable rectangle (i.e. a simplified cross section of a TBM) driven along a circular path with constant curvature is described by a centre of rotation, a curvature, and the angle between the curvature radius and the rectangle. Different combinations of these elements lead to different motion paths. When the centre of rotation is connected to first quarter (front-half) of the bottom side, at the top side a phase of relaxation is followed by the recompression of the pre-relaxed surrounding soil. After that, the outward drifting of the rear half of the rectangle displaces the surrounding soil beyond the range disturbed by the passage of the first half. On the bottom side an opposite behaviour is observed.

This configuration most closely models the theoretical TBM steering system, in which RPF and RPR are meant to follow the design alignment, and the observed TBM behaviour. Although the exactness of this description is discussed later, the general trend holds that the method of steering strongly influences the interaction with the surrounding soil and compression-relaxation (or unloading-reloading) cycles of the soil around the shield may occur particularly in curves. Furthermore, a certain degree of drifting of the machine tail may be expected given the “advanced” position of the rear reference point (RPR).
3.1 Logged-data Based Model

A similar study of the TBM’s kinematic behaviour with respect to the surrounding soil was conducted based on the observed positioning data. The horizontal and vertical deviations from the planned alignment of the front and rear reference points were processed such as to obtain the shield’s position and orientation at each tunnel advance. At each advance the actual position of the shield could be compared with the excavated soil profile, and this comparison allowed one to quantify the displacements induced by the advancing shield. The excavation profile has in turn been obtained as the record of the positions incrementally occupied by the cutter head as the TBM advanced. For simplicity, the shield has been assumed non-deformable. Given the soil conditions and the shield features, this condition appeared reasonable for the front part of the shield, but is indeed less perfect for its tail. The numerical model implemented in MATLAB showed to be able to quantify the amount and distribution of the displacements induced by the advancing shield on the surrounding soil.

3.2 Calculation of the Interaction Displacements

In section 3.1 it was shown that the theoretical drive of a TBM leads to sectors of the shield where the surrounding soil is compressed, and others where it is extended. Such behaviour was also expected during the drive of the actual TBMs. To demonstrate that it proved useful to compare the relative position of the shield-skin at each advance stage with respect to the excavation profile, that is the combined tracks of the cutting wheel and of the cutting edge. Comparing the relative position required to calculate at each advance step of the TBM the relative distance between the

Figure 2: Centre of rotation connected to the mid-point of the first half of the bottom side
On the Effects of the TBM-shield Body Articulation on Tunnelling in Soft Soil

discretized grids of the shield and of the excavation profile. Additionally, specifying which of the two was external (or internal) allowed to distinguish between soil compression and extension conditions. These relative displacements were referred to as shield interface displacement.

4 RESULTS

Two useful values in shield-tunnelling are the horizontal and vertical tendencies. Those are obtained as the difference per unit length of the horizontal and vertical deviations of the shield reference points from the theoretical alignment. The tendencies are an indication of the relative positioning of the shield as compared to its theoretical one at each advance stage. As higher the tendency, as more accentuated the yawing/pitching behaviour of the shield. For instance, the horizontal tendency

\[ T_h = \frac{(d_{h,RPF} - d_{h,RPR})}{d_{RP}} \]

where \(d_{h,RPF}\) is the horizontal deviation of RPF, \(d_{h,RPR}\) the horizontal deviation of RPR, and \(d_{RP}\) is the distance between RPF and RPR.

It is often observed that the tendencies vary a lot even between parallel tubes closely spaced, even when bored with the same TBM. This suggested the working hypothesis, adopted hereafter, for the tendencies to be at the same time a picture of the actual driving behaviour of the shield, but also the combination of constructive, measuring, and geological uncertainties not easy to spot and isolate. It is assumed for each TBM to have its intrinsic tendencies, and those were established during the driving of straight sectors. Those are then assumed as reference values. The deviations from the reference values were studied as representative of the actual behaviour of the shield.

Figure 3 reports the shield-soil horizontal kinematical interactions in the Hubertus tunnel Southern tube at the tunnel spring. The sectors -1660.000 ÷ -1160.200 and -1072.200 ÷ -580.490 are straights, the sector -1160.000 ÷ -1072.200 is in a leftward curve with a radius of 1000 m, and the sector -580.490 until the end is in a leftward curve with a radius of 550 m. Positive values of shield-soil interaction represent extension of the soil at the passage of the shield tail, negative values represent compression. The sectors where the soil is compressed are limited. Also limited is the compression rate, always below 20 mm. In the 550 m curve there is a trend for the
shield right side to adhere to the excavated profile with modest compressions. Conversely, in the second straight sector the shield left side appears to adhere well to the excavated profile. The reasons for this behaviour are to be investigated further. In the first and second sectors the shield appears well positioned in the middle of the steering gap, with the interaction mostly fluctuating within the bandwidth $+10 \div +30$ mm. The positioning of the shield right in the middle of the steering gap would result in an even interaction rate of $+20$ mm.

Figure 3: Shield-soil calculated horizontal kinematical interactions; Hubertus tunnel – Southern tube

Figure 4 reports the shield-soil horizontal kinematical interactions in the eastern alignment of the North-South line tunnel at the level of the tunnel spring. The study covers the sector from Central Station to the Rokin new metro station. The drive until advance $+220$ follows first a rightward then a leftward curve both with a curvature radius of 325 m. The alignment proceeds then straight until advance $+446$, where a leftward curve with a radius of 240 m begins. In the first sector, in the rightward curve ($0 \div +85$) overcutting is used and the interactions stay almost always positive (negative peaks are not physically possible due to the presence of frozen soil and are to be further investigated). In the leftward curve ($+85 \div +220$) compressions up to 40 mm are observed. In the second sector the shield drills very well balanced within the steering gap, keeping mostly evenly distributed positive values of interaction. In the third sector the presence of the sharp curve is reflected in the soil compression
observed at the right-hand side. Also in the same curve it is possible to identify where overcutting has been used, especially in the sector +515 ÷ +530. When that is the case, both right and left interactions become positive.

![Kinematical interaction with the excavation profile (combination of CW4CE profiles)](image)

**Figure 4:** Shield-soil calculated horizontal kinematical interactions; North-South line tunnel – Running tunnel 1/East

## 5 CONCLUSIONS

Solid numerical analyses of shield-tunnelling in soft soil have to be based, among others, on an improved understanding of the physical processes occurring at the shield-soil interface. The kinematical analysis presented offers an analytical tool in that direction. Comparing the behaviour of an articulated and a non-articulated shield showed that the articulation offers higher stability, with smaller fluctuations on behalf of the interaction displacements. The articulation also enhances to shield’s capability to follow the excavated gap, limiting the induced soil displacements. Direct measurements for the validation of the interface displacements as above determined is currently not available. However, two possible alternatives seem viable. First, the subsurface displacements around the tunnel can be monitored during construction and compared with the shield-induced calculated displacements. Second, the calculated interface displacements can be converted into a stress distribution on the shield skin. The integral of the pressures on the entire shield has to balance the TBM driving forces. This second validation option is to be pursued in further research.
REFERENCES


3D Numerical Prediction for TBM-EPB Excavations under Railways Bridges in Milan (Italy)

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Abstract

The paper deals with the prediction of settlements induced by the use of TBM-EPB technology for tunnelling in urban areas with specific reference to tunnelling under railway bridges. The construction of the underground line M5 has been selected as real case for evaluating the capability of sophisticate 3D FEM analyses to be an useful tool for prediction and mitigation of the impact of TBM-excavated tunnel in urban area in a real time process during the excavation works. Two different commercial codes have been employed following these steps: i) calibration of the constitutive model of the soil based on the experimental data available from in situ and laboratory tests; ii) definition of the most important geometrical and physical aspects of the 3D TBM modelling; iii) calibration of the 3D models on the monitored data in the initial free field sections of the TBM-EPB excavations; iv) prediction of induced settlements in the most critical branches (i.e. excavation under railways bridges in Zara street, Milan); v) eventual design improvement; vi) comparison with monitored data. The paper shows an interesting case history where 3D advanced numerical analyses have played a crucial role in verifying the design and operative works.

Keywords: 3D numerical prediction, TBM-EPB, settlement predictions in urban area
1 CASE HISTORIES

The case history here exposed is represented by the TBM underpassing, between Ca’ Granda and Istria M5 stations, of two railway bridges, part of a railway ring line around Milan.

These reinforced concrete structures were built in the 1920s and consist of six spans resting on two end piers and five columns on direct foundations. The spans have a maximum length of 11 m and the height of the intrados is on average 4.5 m from street level.

The route of TBM passes beneath the central column of the two bridges. The passage of trains was not interrupted during the works because of the importance of the rail connection.

Figure 1: Plan of the passage
2 3D MODELLING WITH DIFFERENT 3D FEM CODES

2.1 Midas GTS

2.1.1 Excavation model

The finite elements model of the TBM EPB tunnel excavation tries to reproduce, as faithfully as possible, all the different characteristics that this process implies in reality. The excavation progress is modelled through a multi-step analysis which describes the passage of the TBM in a representative domain of ground.

![Example of a calculation step](image)

Two concentric semi-circles (as the model is completely symmetric, only half domain is considered) represent the part of excavated ground. The sizes of the two diameters are respectively the EPB shield diameter ($D_E$) and the diameter of the installed pre-casted concrete lining ($D_I$). In the longitudinal direction of excavation, the model is divided in 1.5 meters slices, which is the size of a pre-casted lining element. In each step the TBM advances by this distance, activating and deactivating various elements as follows. First both solid elements groups located inside the external EPB diameter $D_E$ are deactivated, and at the same time the steel 2-D shell elements modelling the
EPB shield (with thickness of 6 cm) are activated. A face pressure is applied on the excavation face to simulate the chamber pressure acting against the excavation face. Then, as the TBM moves forward, a set of 2D shell elements is activated at the internal diameter $D_I$ (made of concrete with 40 cm thickness) 6 meters behind the excavation face. To model the liquid grout injected between the installed lining ring and the surrounding ground, a radial pressure is introduced. Once the grout hardens, the solid elements located between $D_E$ and $D_I$ are re-activated with a change in material properties from ground to hardened grout. The movement of the shield takes place thanks to hydraulic jacks with push on the last settled lining ring and is modelled through a line load on the lining edge.

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**Figure 3:** Chamber pressure acting against the excavation face. Hydraulic jacks pressure acting on the lining edges.

Another element is introduced to give further accuracy to the model. The shield is characterized by a light conicity ($R_{max} = 4700$ mm; $R_{min} = 4688$ mm). This particular shape of the shield affects the ground displacement. A radial pressure as shown in Fig. 5 is applied to the shield to induce the desired conical shape.
Figure 4: Detail of grout pressure between lining shells and ground

Figure 5: Applied radial pressure and induced deformed shield shape
2.1.2 Free-field calibration of the model
The analysis started with a free-field calibration of the model. A back analysis was possible thanks to monitoring data in a subway section excavated before reaching the railway bridges. In this phase it was necessary to assess the influence of parameters such as the conicity of the shield, the constitutive model, and the increase rate of the ground stiffness with depth.
The conicity of the TBM proved itself to be essential for the modelling, since the results regarding the vertical settlements of the ground change dramatically with or without this aspect, as shown in Table 1.

Table 1: Influence of the TBM-shield conicity

<table>
<thead>
<tr>
<th>Vertical settlements [mm]</th>
<th>without conicity</th>
<th>with conicity</th>
</tr>
</thead>
<tbody>
<tr>
<td>10,0</td>
<td>16,0</td>
<td></td>
</tr>
</tbody>
</table>

2.1.3 Bridge section: influence of the grout arch stiffness
Due to the complexity of the geometry and considered the availability of computing capacity, the two bridges were represented with equivalent loads. Furthermore, the two bridges were analysed one at the time, in a domain 30 m wide and 30 m deep in section, and 45 meters long in the direction of the tunnel axis.
The analysis without ground improvement (Table 2) forecasted a maximum vertical settlement of 20,4 mm for the first bridge and 23,8 mm or the second bridge. These values suggested the need of further preventative measures, such as the concrete injections around the tunnel already mentioned before. The grout arch is represented in Fig. 6, and was given a stiffness E = 900 MPa. With these characteristics, the settlements were lowered to 11,0 mm and 12,3 mm for the two bridges.

Figure 6: Ground improvement : grout injections arch under the two railway bridges
Table 2: Maximum vertical settlement under the bridges

<table>
<thead>
<tr>
<th></th>
<th>Bridge n°1</th>
<th>Bridge n°2</th>
</tr>
</thead>
<tbody>
<tr>
<td>without ground improvement</td>
<td>20.4 mm</td>
<td>23.8 mm</td>
</tr>
<tr>
<td>with ground improvement</td>
<td>11.0 mm</td>
<td>12.3 mm</td>
</tr>
</tbody>
</table>

The assessment of the influence of the grout arch stiffness gave some interesting results, as we can see in Table 2 and in Fig. 6. The graph shows a strong decrease of the vertical settlements as increasing the stiffness E. This proves that such a constructive solution is very effective in preventing excessive ground level deformations.

![Figure 7](image)

Figure 7: Vertical displacements of the ground level above the tunnel axis, under Bridge n°1, as a function of the grout arch stiffness

Table 3: Vertical displacements of the ground level above the tunnel axis, under Bridge n°1, as a function of the grout arch stiffness

<table>
<thead>
<tr>
<th>E [MPa]</th>
<th>130</th>
<th>300</th>
<th>600</th>
<th>900</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical settlements [mm]</td>
<td>17.8</td>
<td>14.6</td>
<td>11.9</td>
<td>11.0</td>
</tr>
</tbody>
</table>
2.2 PLAXIS 3D Tunnel

2.2.1 Excavation model

The analysis using the FEM-code Plaxis follows the same principles and ideas explained in the previous paragraph. Some improvements in the model are however introduced regarding some technical details of the excavation process, such as the grease coat placed between the TBM-shield and the lining shell before the grout injections nozzles. In addition the liquid grout was modelled as a liquid substance under pressure, and not only as a radial pressure.

![Plaxis Excavation Model](image)

Figure 8: Plaxis Excavation Model

2.2.2 Bridge section displacement prediction

Compared to GTS model, the railway bridges area are represented by a single domain with the two bridges and surrounding buildings entirely represented, instead of two separated domains with bridges equivalent loads as in the Midas analysis.
The main target (as visible in Fig. 10) is to understand whether the ground improvement under the bridges is necessary or not, as the situation hardly changes from the free field excavation area. To avoid the use of the ground improvement, chamber and grout pressure have been amplified to balance the higher stress state which characterizes the ground in the bridge area. Among the various analyses that were carried out, the most significant results are shown in Table 4.
**Figure 10:** Vertical settlements along longitudinal direction

**Table 4:** Higher Pressure and Ground Improvement Maximum Displacements.

<table>
<thead>
<tr>
<th></th>
<th>Bridge n°1</th>
<th>Bridge n°2</th>
</tr>
</thead>
<tbody>
<tr>
<td>w/o ground improvement</td>
<td>12.0 mm</td>
<td>11.7 mm</td>
</tr>
<tr>
<td>with ground improvement</td>
<td>5.0 mm</td>
<td>5.4 mm</td>
</tr>
</tbody>
</table>

In Figure 11 the induced settlement on the bridges and in the building is shown, The magnitude is less than 5 mm considering a stiffness for the grouting of 9000 MPa.
COMPARISON WITH MONITORED DATA

The final and most interesting part of this model is the comparison with the monitored data after the TBM-EPB excavation under the bridges in Milan. At the end, preventive ground improvement of those zones was realized, as the guarantees provided by the control of deformation and mortar injections behind the rings of concrete segments were not considered sufficient to proceed in the absence of extra protection. The following Table 5 summarizes the results from the two finite element models and the monitored data.

As both of the codes have demonstrated, the ground improvement has been necessary. In fact, to allow the excavation to be done without ground improvement, chamber and grout pressure should have been set to out of common range values with dangerous and unknown potential consequences.
Table 5: Finite Elements and Monitored Data Maximum Displacements.

<table>
<thead>
<tr>
<th></th>
<th>Displacement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Monitored Data</td>
<td>3.0 mm</td>
</tr>
<tr>
<td>Midas GTS</td>
<td>11.0 mm</td>
</tr>
<tr>
<td>Plaxis</td>
<td>5.4 mm</td>
</tr>
</tbody>
</table>

The difference between the two codes results can be related to the different values assigned to the improved ground arch: E=900MPa in Midas versus E=9000 MPa in Plaxis.

From an operational point of view, ground improvement provided an important action which clearly reduced the “volume lost” and the magnitude of the surface movements, its direct consequence. The development of subsidence was an immediate reaction to the passage of the face as was predictable from the rheology of the geotechnical formations, consisting of sands and gravels under the water table.

REFERENCES


Structural Design of Segmental Tunnel Linings

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Abstract

Segmental tunnel linings are designed as final liners in modern TBM tunnels to withstand all different kinds of loadings without excessive deformation and significant settlement on the ground surface. The design loads for segments are categorized in three classes of “primary loads”, “secondary loads” and “special loads”. In this paper, effect of vertical and horizontal earth loads, water pressure, dead weight, surcharge and soil reaction are analyzed as primary loads, while effect of construction loads such as TBM thrust jack forces and grouting pressure are studied as secondary loads. Available standards and methods to design segmental linings are presented and latest developments in design of reinforced concrete segments including reinforcement rebars, joints, and connections are revealed. A hybrid reinforcement system including limited amount of conventional bars together with fiber reinforced concrete (FRC) is studied as a viable alternative to RC segments. Proposed design method is applied to several tunnel cases and the results show that wider, thinner and more durable segments with lighter reinforcement and less damage of segments during construction can be adopted to save the construction cost in tunneling industry.

Keywords: Concrete, design, fiber, segmental lining, tunnel
1 INTRODUCTION

Precast concrete segmental lining are installed in the TBM-bored tunnels to support the surrounding ground, control groundwater inflow and maintain required operational cross-sections during their design life. Segmental linings are designed with force and moment capacities higher than developed forces and moments due to primary and temporary loads to ensure safety during the service life of the tunnel as well as the construction period.

A procedure is presented to design concrete lining against embedment loads, jack thrust forces, forces due to cross section change in longitudinal joints, and other secondary loads. These secondary loading cases include, but not limited to, storage load, lifting load, handling load, grouting pressure, and longitudinal moments. Design of concrete lining for a large diameter tunnel using proposed approach is discussed. In addition to traditional reinforcement with steel bars, a new reinforcement system using fibers are introduced.

2 DESIGN OF SEGMENTAL TUNNEL LINING

2D analysis of tunnel cross-sections is performed to calculate primary loads applied to the tunnel linings. Discontinuum analyses are used to study the axial and shear forces, and bending moments in the lining for a portion of the alignment in the rock. The most critical sections for such analyses include shallow cover sections, sections in the weak rock and in the close proximity of buildings. Lining forces due to the primary loads in the soil are obtained from the results of finite element analyses, modeling the soil as a continuum media around the tunnel lining. Results of these methods are compared with elastic equation method proposed by International Tunnel Association guidelines [1] and segmented double ring beam model with joints and soil interaction represented by springs according to JSCE [2]. Maximum bending moment, axial, and shear forces are used for the design of main transverse reinforcement.

On the other hand, tensile stresses developed due to TBM jack thrust force on segments during construction phase is used to design reinforcement around circumferential joints. After assembly of a complete ring, the TBM moves forward by pushing its thrust jacks on the bearing pads of the newest assembled ring. The high thrust jacking forces that are introduced in the lining results in high compression stresses under the thrust jack shoes as well as tensile spalling and bursting stresses between the jack pads and deep within the material. The magnitude of the thrust jack force when TBM passing through the rock, can be estimated from the sum of forces required for boring into the rock, friction resistance between the outer surface of shield machine and the ground and hauling
resistance of trailing gears. Fukui method [3] can be used which empirically relates ground characteristics to cutting forces using parameters such as tunnel diameter, cutter diameter, and cutter distance. In another method [4], total TBM thrust are calculated based on the rock strength, cutter radius, cutter tip width, cutter spacing, cutter penetration and the number of cutter disks. Total thrust force of the machine in the soil is determined by the sum of all the resisting forces against machine’s excavation operation. According to Japanese Specifications for Tunneling [2], the resisting forces include resistance caused by friction between the outer surface of shield machine and the ground, pressure acting on the cutting face, frictional resistance due to friction between the segments in a tail section and the tail seals, and hauling resistance of trailing gears. The magnitude of the average thrust force for each jack pair is estimated by dividing total thrust force over the number of jack pairs. Since on the sharp curves, the machine thrust is higher on the convex side of the curve and lower on the concave side, a factor of safety of 3 is applied to estimate maximum thrust jack force for each jack pair. Using an analytical approach, compressive stresses under the jack pads are obtained dividing the jack pair force by the contact zone area, and compared with the factored compressive strength. Diagram of Iyengar [5] can be used to calculate bursting tensile stresses. On the other hand, a simple approach to analyze the bursting action is recommended by ACI 318 [6] section 18.13 for calculation of bursting forces. In a numerical approach, a 3D Finite Element (FE) analysis is performed simulating a regular segment in its actual geometry and jack thrust forces are applied on the contact zones between rams and circumferential side of the segment. Solid elements are used for this analysis. The translational degrees of freedom at the end of the second segment as part of the previously erected ring are therefore fixed in all three directions. A distributed gasket force of 40 kN/m is distributed over the gasket groove area and maximum thrust force per pad is applied. As results of all different approaches, compressive and tensile stresses are obtained and compared with concrete strength. When tensile stresses are greater than the tensile strength of concrete, special joint bars, often referred to as ladder bars, are designed to deal with these stresses. Moreover, in the longitudinal joints, axial forces are transmitted over contact surfaces using the same action; as such, longitudinal ladder bars are designed similarly. The segments are checked for various secondary loads. To check against lifting loads, segments are assumed to be lifted using the grout hole, where the lift force generates a moment in the segment due to its own self weight which is checked against the moment capacity of the segment section. The segments are also checked against storage loads. The action effects during storage can be calculated according to the simply supported moment formulas and checked with moment capacity of the section.
Tunnel lining reinforcement are checked for annular grouting and contact grouting as filling the annular space with semi-liquid grouts is required in order to control and restrict settlement at the surface and to securely lock the lining ring in position. Grout pressure is limited to a minimum value slightly higher than the water pressure, and a maximum about the overburden pressure. According to a model by Zhong et al. [7], grout pressure on the tunnel crown is calculated. The vertical gradient of radial grout pressure is determined by taking the equilibrium between the upward component of the total grout pressure and the downward components by the tunnel dead weight and the tangential component of the grout’s shear stresses [8]. A linearly varying radial pressure distribution is applied from the minimum grout pressure at the crown increasing by as much as calculated vertical gradient at the tunnel invert, in order to simulate annulus grouting load condition. Analysis shows that annulus grouting pressure results in large axial forces, to be checked with the compressive capacity of the segments. Radial backfilling also known as contact grouting is performed by radial injection through holes provided in the concrete lining. Radial backfilling is required once it is verified that an annular gap still exists between the lining extrados and excavation profile. The forces applied to segments in this case is similar to longitudinal back grouting when only one of the grouting pipes is pumping the grout into the annular space around the segments. Segment capacity is also checked against maximum bending moments and axial forces due to contact grouting load condition. On the other hand, a minimum reinforcement is designed in the longitudinal direction for shrinkage and temperature control. According to ACI 318, minimum ratio of reinforcement area to gross concrete area using Grade 60 steel bars is 0.0018.

3 APPLICATION OF DESIGN PROCEDURE TO A SPECIFIC CASE

Presented structural design approach is applied to design of reinforced concrete segmental lining for a large TBM-bored tunnel with 10.9 m internal diameter in a mixed phase geological condition. Designed lining thickness and compressive strength of concrete are 400 mm and 50 MPa, respectively. Segment geometry is a flat section with full wall thickness that contributes to the strength of the ring. The length of the segmental rings is 1.7 m, which takes into consideration the tunnel diameter, alignment, gasket sealing length limitations, optimization of mucking operations, as well as the size and the weight of the rings. The number of segments selected to comprise a ring is 6 plus 1 key in order to limit the number of contact joints and the potential for a defect to affect water-tightness of the tunnel without unnecessary handling, storage, and ring erection issues due to the weight of the segments. Design of segment and reinforcing steel is performed.
considering stresses due to different loading conditions. The design loads for segments consist of primary and secondary loads. Using results of UDEC (Universal Distinct Element Code) analyses for the portion of alignment in fractured rock, maximum axial forces and bending moments in the lining due to embedment loads in rock are 2,036 kN/m and 33.8 kN.m/m, respectively. Permanent loads applied on tunnel lining in the soil were obtained through different types of analysis. In a numerical approach, finite element analyses were performed using PLAXIS, assuming a ground volume loss of 0.5%. As shown in Figure 2, PLAXIS results yield maximum axial forces and bending moments of 1,780 kN/m and 60.6 kN.m/m, respectively. In another approach, following International Tunnel Association guidelines [1], maximum axial force and bending moment per linear meter of the segment are 2,129 kN/m, and 127 kN.m/m, respectively. In addition, a segmented double ring beam model with soil interaction represented by springs and loads on the tunnel is performed. Maximum bending moment for a segment using this method is 156 kN.m and maximum axial force is 1,666 kN. In a relatively conservative approach, results of the elastic equations method are used for the design of main reinforcement. The maximum factored bending moment due to stacking load case in the middle of segment is 95.2 kN.m. This is compared with the stripping bending moment capacity of the segment obtained as 175 kN.m, assuming stripping compressive strength of concrete ($f'_c$) as 15 MPa.

Cutting forces required for boring is calculated as 14,100 kN according to Fukui method for intact rock strength of 77 MPa, assuming number of disk cutters as 80. Cutting forces estimated by CSM method is 10,500 kN. Taking 14,100 kN as the operational thrust force required for boring into the rock, and assuming this value is about 60% of the maximum required cutting force, the maximum required thrust force of TBM for boring is estimated as 23,500 kN. Results of analysis due to thrust jack force loading case are shown in Figure 3. Results indicate that maximum transverse bursting tensile stress developed under the jack pad over the width of the segment is 1.34 MPa, which is less than tensile strength of the concrete (3.9 MPa). However, transverse spalling tensile stresses developed in areas between the jack pads, and jack pads and longitudinal joints are significant. The maximum transverse spalling tensile stress is 6.57 MPa, which is more than splitting tensile strength of concrete (3.9 MPa). Therefore, steel bars are designed to take the shortfall between the tensile strength of the concrete and the developed stresses.
Figure 1: A typical section of the segmental lining ring

Figure 2: Axial forces and bending moments obtained from PLAXIS analysis due to permanent loads acting on the lining in the soil

Also, segment capacity is checked against longitudinal back grouting pressure of 225 KPa at the crown [7], linearly increasing to 265kPa at the invert of the tunnel. This loading case results in developing large axial forces (1,277-1,573 kN/m) and bending moments in the range of 0-114 kN.m/m.

4 FIBER REINFORCED CONCRETE LININGS

Fiber reinforced concrete (FRC) can be used in the manufacturing of segmental linings to improve mechanical behaviour of concrete. A moment-axial force interaction diagram is shown in the Figure 4, comparing FRC segments with 28 steel rebars with cross sectional area of approximately 100 mm². Results show that application of FRC may lead to the total or a partial removal of rebars and improving the production efficiency with regards to the traditional solution.
**Figure 3:** Bursting and spalling tensile stresses developed in segments due to TBM jack thrust force

**Figure 4:** Comparing FRC lining with steel bars
5 CONCLUSION

Presented procedure for structural design of segmental tunnel lining includes design of lining for primary loads, design of joint reinforcement for jack thrust forces and cross section change, and check the design against secondary loads such as storage, lifting, handling, grouting and longitudinal bending. Application of proposed design approach to a case of large diameter tunnel and utilization of shear recovery systems in penetration areas indicate that wider, thinner and more durable segments with lighter reinforcement and less damage of segments during construction can be adopted to save the construction cost in tunneling industry.

REFERENCES


Numerical Analysis on Measured Pipe Behavior during Pipe Jacking

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Abstract

In pipe jacking method, the main factor influencing on jack force is the friction resistance between pipes and ground, and the over-cutting area between pipes and ground is a significant factor giving influence to the friction resistance. Therefore, the excavation radius by TBM is a little bit larger than the outer radius of pipes to reduce the friction resistance. So far, most of the models estimating jack force do not consider the over-cutting area, and the pipe behavior is not still clarified well. To overcome the above problems, the new analysis model called as “stack pipe model” has been developed, which can take over-cutting into consideration. This study aims to validate this analysis model by site measurement data. As a result, it was confirmed that the developed stack pipe model could represents the measured pipe behavior during pipe jacking reasonably.

Keywords: Pipe jacking method, numerical analysis, site data
1 INTRODUCTION

In pipe jacking method, the main factor influencing on the jack force is friction resistance between pipes and ground. The interaction between pipes and soil during jacking behaves in a complex manner due to the effect of many factors during construction stage. Among them, over-cutting area between pipes and ground is a significant factor giving influence on the friction resistance. Therefore, the excavation radius by TBM is a little larger than the outer radius of pipes to reduce the friction resistance. So far, most of the models estimating jack force do not considered the over-cutting area, and then the pipe behavior is not still clarified well. For example, Barla et al (2003) [1] introduced a two dimensional analysis of the undrained and drained stages to simulate the pipe-soil stresses during jack process. With regard to three-dimensional analysis, Shou and Chen (2006) [2] studied the pipe-soil interaction behavior and the ground improvement effect of curved pipe jacking. To overcome the above problems, a new analysis model called as “stack pipe model” has been developed (Sugimoto and Asanprakit 2010 [3]), which can take over-cutting size into consideration. This study aims to validate this analysis model through site measurement data. In addition, behavior of pipes during jacking process is also investigated.

2 GROUND-PIPE INTERACTION MODEL

1.1 Ground-pipe Interaction

Ground spring in radial direction is applied to represent the normal earth pressure acting on the pipe periphery. The ground reaction curve shown in Figure 1 was adopted as the interaction model between the ground and the pipe. In the figure, $u_n$ is the length of the perpendicular line from the initial excavation surface to the lining surface (+: outward), and $\sigma_n$ is the normal earth pressure acting on the lining. $\sigma_n$ is composed of the constant initial normal earth pressure due to overburden load $\sigma_{n0}$, and the earth pressure change $\Delta \sigma_n$, which depends on $u_n$. In analysis, $\sigma_{n0}$ is introduced by a prestress force in the ground spring before analysis. $\Delta \sigma_n$ is generated as a result of the analysis. Furthermore, due to over-cutting, the earth pressure acting on the pipe is usually less than the earth pressure at rest. Therefore, it is necessary to use the earth pressure with over-cutting effect. To represent the over-cutting effect, the enforced displacement, which is the over-cutting length $L_o$, must be applied at the outer end of ground springs in analysis.
The relationship among the excavation surface, overcutting and pipe surface is shown in Figure 2, which is explained as follows:

1) When the pipe surface is outside the original excavation surface, the earth pressure is in passive state since the pipe pushes the ground.

2) When the pipe surface is between the original excavation surface and outside the excavation surface after deformation, the earth pressure is in active state.

3) When the pipe surface is inside of the excavation surface after deformation, the original excavation surface moves freely until its deformation stops and there is a gap between the excavation surface and the pipe surface. In this case, there is no earth pressure acting on the pipe surface. This phenomenon is due to self-stabilization of the ground especially in the case of stiff ground.
1.2 Ground Creation Curve

The nonlinear ground reaction curve developed in the kinematic shield model (Sugimoto and Sramoon 2002 [4]), as shown in Figure 3, was adopted to represent the ground reaction curve in Figure 1(a). The relationship of the coefficient of earth pressure in the vertical and horizontal directions, \( K_v \) and \( K_h \), and the distance from the initial excavation surface to the pipe surface, \( u_n \) (+: outward), in Figure 3 can be represented by

\[
K_i(u_n) = \begin{cases} 
(K_{i0} - K_{i\text{min}}) \tanh \left( \frac{a_i u_n}{K_{i0} - K_{i\text{min}}} \right) + K_{i0} & (u_n \leq 0) \\
(K_{i0} - K_{i\text{max}}) \tanh \left( \frac{a_i u_n}{K_{i0} - K_{i\text{max}}} \right) + K_{i0} & (u_n \geq 0) 
\end{cases} 
\tag{1}
\]

where \( K_{h0} \) = coefficient of earth pressure at rest; \( K_{v0} \) = coefficient of initial vertical earth pressure normally equal to 1; subscripts max and min indicate the upper and lower limits of the coefficient of earth pressure, respectively; \( a_h \) and \( a_v \) = gradient of function \( K_h \) and \( K_v \) at \( u_n = 0 \), respectively. Moreover, the coefficient of earth pressure in any direction, \( K_{\theta} \), can be interpolated as

\[
K_{\theta}(u_n, \theta) = K_v(u_n) \cos^2 \theta + K_h(u_n) \sin^2 \theta 
\tag{2}
\]

where \( \theta \) = angle measured from downward vertical direction to \( u_n \).
Finally the earth pressure, the initial earth pressure and the change of earth pressure along a radial direction can be obtained as follows:

\[
\sigma_n = \sigma_{n0} + \Delta \sigma_n \\
\sigma_{n0} = K_n (0, \theta) \sigma_{v0} \\
\Delta \sigma_n = (K_n (u_n, \theta) - K_n (0, \theta)) \sigma_{v0}
\]  

\[ (3) \]

3 APPLICATION

3.1 Site Description

A pipe jacking project using concrete pipe with dimension of 1.5m in inner diameter, 2.43m in length, and 0.14m thick was carried out as shown in Figure 4. The tunnel length is 141.19m including a curvature of radius 200m. The tunnel is at 2.09m ~ 3.19m deep, in a sandy layer. The groundwater level is from -1.0m ~ -5.0m. The measurement was carried out at pipes No. 18 and No.22 along the tunnel alignment. Analysis condition is shown in Table 1.

**Figure 4:** Site plan

**Table 1:** Analysis conditions

<table>
<thead>
<tr>
<th>Component</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density $\gamma$ (kN/m³)</td>
<td>18</td>
</tr>
<tr>
<td>Coef. of earth p. $K_{h\text{min}}, K_{h0}, K_{h\text{max}}$</td>
<td>0.3, 0.426, 5.0</td>
</tr>
<tr>
<td>$K_{v\text{min}}, K_{v0}, K_{v\text{max}}$</td>
<td>0.3, 1.0, 5.0</td>
</tr>
<tr>
<td>Coef. of subgrade reaction $k_{h}$ (MN/m³)</td>
<td>10765</td>
</tr>
<tr>
<td>Coef. of subgrade reaction $k_{v}$ (MN/m³)</td>
<td>10765</td>
</tr>
</tbody>
</table>
3.2 Measurement Result

The measurement of the pipe No. 18 was carried out along the tunnel alignment. Figures 5(a) and 5(b) show the measured deviation of the pipe from the trace of TBM in horizontal and vertical directions, respectively. From these figures, the following were found:

1) In horizontal direction, when the pipe is at the distance of 45m where is before the beginning curve point (BC), the pipe moves to the convex side of the curve about 38mm. When the pipe just passes the BC point, it moves to the concave side about 25mm. At the distance 60m, it moves back to the convex side about 20mm. A deviation about 17mm to concave side occurs at distance from 62m to 65m. At distance from 70m to 80m, the pipe moves to the convex side about 20mm. At the ending curve point (EC), the pipe moves back again to the concave side about 5mm. After the pipe passes the EC, its deviation is back to the convex side about 25mm.

2) In vertical direction, before the BC the pipe moves upwards about 17mm. When the pipe was at distance of 58m, its deviation changes to bottom about 10mm. After that, it consistently moves upwards about 10mm

These can come from the displacement of the pipe within the over-cutting area and movement of excavation machine entering into the curvature.
Numerical Analysis on Measured Pipe Behavior during Pipe Jacking

3.3 Numerical Analysis

The stack pipe model (Sugimoto and Asanprakit 2010 [3]) using the ground reaction curve was applied to analyze the site project. The analysis results are shown in Figure 6. From this figure, the following were found:

1) In horizontal direction, the pipe is on the tunnel center until the distance of 35m. Then it moves to the convex side when it goes to the BC. At the distance of 5m before the BC, the deviation is about 7mm. Near the BC, it moves to the concave side 15mm. It moves back to the convex side 7mm constantly when it is within the trajectory between the BC and EC. At the EC, the pipe moves to the concave side and the deviation is 12mm. When the pipe passes the EC about 5m, the deviation is 7mm to the convex side. After that, the pipe tenses towards the tunnel center. These can be considered from the flexibility behavior of the joints between pipes while transferring force.

2) In vertical direction, at the distance from 0m to 40m, the pipe is upwards 0.2mm. Then the pipe is almost on the tunnel center until the BC. It starts to move upwards from the distance of 60m, and reaches 4mm at the distance of 90m. These can be considered as following. Before the distance of 60m the groundwater level is lower than the pipe, and the pipe self-weight is less than excavated soil, so the tunnel is slightly upwards. In addition, the tunnel is strictly constraint around the BC therefore the pipe is kept on the tunnel center. The groundwater begins to touch the tunnel from distance of 60m, and tunnel is totally submerged near the distance of 90m, therefore the tunnel is upwards due to buoyancy.

Figure 6: Numerical results of deviation of pipe in horizontal and vertical directions
4 CONCLUSION

In comparing the results between the measurement and numerical analysis, the following conclusion is made:
The stack pipe analysis model gives a reasonable behavior of the pipe as compared with the site data during jacking.

ACKNOWLEDGEMENT

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Towards Topology and Shape Optimised Concrete Linings for Shear Load Transfer in Ring Joints

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Abstract

Mainly, due to general computational demands and apart from already well-established fields of application in serial production, optimisation methods are – in contrast to mechanical engineering – still seldom utilised to solve standard civil engineering design problems. However, mechanical tunnelling requires a high number of pre-fabricated lining segments, usually made of reinforced concrete, and thus meets that prerequisite of optimisation-based design analyses ideally. Exemplified by cam and pot connections for shear load transfer, the value of two promising optimisation procedures is investigated. First, topology optimisation has been applied to engineering problems of rising complexity accompanied by convergence studies. Results identify stiffness requiring regions analogue to notional strut and tie models that clearly indicate concrete areas where reinforcement or fibre add-on is necessary. Second, shape optimisation is utilised by means of movable nodes to minimise the total area of the same cam structure. Hereby, restrictions are limited deflections and maximal stresses allowed. It turns out that the outcome of topological and shape optimisation is mutually dependent and hence a combination of both in subsequent application is expected to deliver optimal results.

Keywords: Optimisation, Topology, Shape, Mechanical tunnelling, Concrete lining
1 INTRODUCTION

In Germany the design of concrete lining segments is traditionally carried out having interlocking surfaces in the joints of succeeding rings – also known as a cam and pot connection. While a tunnel is under construction, relative displacements and associated shear forces between adjacent rings do occur and have to be carried without damage [6]. Else the watertightness is at risk, which would cause immense costs and efforts in case of reconstruction. For simplicity and moreover because in neighbouring countries it has already been carried out successfully, current recommendations tend to favour plane over interlocking surfaces. With this approach the challenge to provide an effective load transfer by a special construction detail is simply avoided. But it has to be analysed very carefully if an abandonment of any kind of interlocking is indeed the best possible solution. Especially being aware of different construction stages and known insufficient bedding conditions, relative deformations between concrete lining segments can easily exceed given tolerance limits. To clarify which approach, either interlocking or plane surfaces, leads to the best solution, structural optimisation helps.

2 OPTIMISATION TYPES – AN OVERVIEW

Three criteria are commonly valid for every kind of optimisation [3]. Central to any task is the objective function, for example the minimisation of costs for a tunnel project or the number of tendons used for bridge design. Second, various constraints limit the task. For instance, the minimal lining thickness allowed for a tunnel shell or safety requirements to be met by the design of pre-stressed concrete structures. Finally, a number of design variables varies the design and enables optimisation, e. g. the concrete strength class or the applied lining material in the tunnel. Two promising approaches are set out in the reminder.

2.1 Shape Optimisation

Basically, shape optimisation is understood as the claim for an optimal geometry with respect to given load and bearing conditions. E. g.: A rectangular cross-sectioned cantilever beam’s maximal vertical displacement has to be kept at minimum varying its height and width with respect to the contour-length (Figure 1).
Figure 1: Analytical optimised cantilever beam

Here, the deflection of an elastic cantilever beam \(v_0\) subjected to a distributed load \(q\) is described in dependency of the beam’s dimensions \((b, h)\), both optimisation variables, acc. to Eq. (1):

\[
v_0 = \frac{W_i}{F_i} = \int_0^l M_1 \frac{M}{EI} dx = \frac{q l^4}{8EI} = \frac{3ql^4}{2Ebh^3}
\]  

(1)

To solve the optimisation problem in Eq. (2) the Lagrangian-function can be utilised. Analytically, one gets the optimal result by means of width and height ratio: \(h/b = 3\).

\[
L(b,h,\lambda_1) = v_0(b,h) + \lambda_1 (2b+2h-U) = \frac{3ql^4}{2Ebh^3} + \lambda_1 (2b+2h-U)
\]  

(2)

Only very fundamental problems like this can be optimised analytically. For more complex ones analytical analyses are no longer applicable and numerical models in combination with mathematical search algorithms need to be utilised to identify local as well as global minima. Factors like numerical model discretisation or the choice of applied optimisation procedures have a strong influence on the result’s quality [9].

2.2 Topological Optimisation

Topological or layout optimisation can be described as a special type of shape optimisation [1] having the task to find the most suitable distribution of material and stiffness within a pre-defined region. Certainly, objective criteria (e.g.: gain maximum stiffness with minimal volume) and given constraints (load and bearing conditions for instance) have to be met. Here, the only available design variables \(\eta_i\) are pseudo material-densities, applied to each finite element of the structure. These densities are allowed to have values between \(0 \leq \eta_i \leq 1\), although the final aim is to avoid intermediate values completely. To achieve this, suitable interpolation schemes such as SIMP (Solid Isotropic Material with Penalization) are applied (ref. to [8], [2]). Finite elements having a density equal to 0 are not necessary at all while in return those with a density \(\eta_i = 1\) need to remain part of the structure. Therefore, a clear distinction between elements that are required and those that are not is crucial [5].
interpretation of the result is similar to the concept of notional strut and tie models in truss structures as discussed e.g. in [7]. In addition, its acuteness is also regulated by a pre-defined volume reduction factor, which is often set to 50% by default. But more accurate outcome w.r.t. a clear indication of the truss system instead of a detailed layout as required in this study is received by a value set to 75%. Figure 2 illustrates the outcome of a topological optimisation process regarding a typical reinforced concrete structure: A single-span beam with concentrated point load at midspan is, starting from an initial geometry, iteratively optimised until a converged optimal solution is reached.

![Figure 2: Principle of the topological optimisation process [2]](image)

Application of topological optimisation to solve realistic and even more complex engineering problems can be found in literature [5], [2]. To compare results obtained by analyses of a reference structure shown in Figure 3 a), a benchmark study is performed. Final results from the literature are presented next to own ones employing ANSYS® simulation. All three solutions have been carried out assuming the same general specifications as qualitatively illustrated in a) and defined next to the figure.

![Figure 3: Benchmarking of topology-optimisation](image)

The solution in c) shows the result of an optimisation procedure, where an enhanced SIMP approach is utilised to receive a very clearly defined truss system [5]. Similar results, illustrated in d), can be achieved using a program that is provided by a
research group from Denmark [2]. Again, a very precise distinction between elements having a density equal to 0 or 1 is obtained. The remaining part b) of the figure shows the result of our topological optimisation carried out with ANSYS®. The general shape of the truss-system comparing all cases is equal and regions of high stiffness can easily be identified from the analyses. Transferred to the material usually used in tunnel linings, at these locations the concrete must be strengthened by conventional reinforcement and/or fibres. Compared to the other results, the ANSYS® solution seems not satisfying at first sight. But although the truss system is not as clear, obtained results are still fair enough for the actual optimisation purpose. More precise results can also be gained by use of finer discretised models as will be shown consecutively.

3 OPTIMISATION OF CAM AND POT

3.1 Topological Optimisation of Cam and Pot

Since in a topological optimisation the only design variables are the densities of each individual finite element, the quality of the optimisation result is strongly related to the number of elements used for idealisation of the analysed structure. While few elements lead to a decrease of required computation time, a large number improves the solutions accuracy. Therefore, the lower bound of required elements is crucial for the desired optimisation result and can be assessed in convergence studies. Exemplarily, the results for the optimisation of a cam-structure are presented in Figure 4. As can be seen, a very coarse discretisation does not give a satisfactory solution, whereas a very fine one leads to over-detailed results. Since lesser computation times are generally intended and, moreover, because too precise indicated truss systems cannot be reproduced by bars or fibres in real live anyway, here a calculation with a fine mesh gains the aspired quality.

![Number of elements and computation time](image)

**Figure 4:** Convergence study of a cam structure
Next, as an example, the start geometry has been chosen acc. to formwork drawings from an actual on-going construction project. The optimisation is performed following the concept of “maximum static stiffness” [1] a.k.a. the standard formulation: Maximise global structural stiffness and minimise total volume. Naturally, the design of either cam or pot cannot be carried out exactly like an optimisation suggests. However, just a precise location of stiffness requiring areas is of interest. Hence, the percentage value of volume reduction in neighbouring elements is set to 75 %. Analysing the results in Figure 5, typical regions can be identified that need further reinforcement in order to provide the structure with more stiffness. It turns out that the optimisation result is strongly dependent on dimension “a” (compare e.g. additional central strut in Figures 4 and 5) and further on load application and bearing conditions. These remain to be analysed in more detail in future investigations.

Two potential alternative design concepts are illustrated in Figure 5. Both are understood as an addition to the already existing standard rebar-reinforcement in concrete linings segments. Concept a) is based on a combination of conventional reinforcement with locally placed steel-fibres or steel-fibre-cocktails. In concept b) rebar-reinforcement, anchored by yielded (stainless) steel plates, is used.

![Figure 5: Topological optimisation results and derived alternative design concepts](image-url)
3.2 Applied Shape Optimisation

In contrast to the previously discussed topological-optimisation, shape optimisation allows for higher numbers of variables. Further, these are no longer artificial densities that claim for physical interpretation but common engineering quantities like material parameters or structural dimensions. Again, along with the total number of variables, the computational demand of the solution process rises and specific mathematical methods are required to accomplish the optimisation efficiently. In a last scenario the “First-Order-Optimisation” [1] has been chosen due to its derivative-based highly efficient approach. The objective is to adopt the most suitable geometry by variable dimensions $H_1$, $H_2$ and $B_2$ w.r.t. given a given load “F” and boundary conditions. Optimal is a minimal total area “A” without exceeding restrictions of maximal vertical deflection ($v_{\text{max}}$) and stresses ($\sigma_{\text{max}}$) allowed. Initially the design variables $H_1$, $H_2$ and $B_2$ are again taken from a real reference project. The distributed load is a representative value of a shear force in ring joints. Restrictions are set to artificial values to gain a converged solution. Configuration and results are contained in Figure 6. Movements of the marked nodes in part b) are restricted horizontally and vertically acc. to the legend. Part c) shows the initial and optimal geometry obtained, whereas d) displays the change of variables during the solution. Both dimensions $H_1$ and $H_2$ are equal in the end which leads to equally distributed nodes at the prolonging edge. $B_2$ reaches its lower restriction bound equivalent to a cam being nearly disappeared. This minimises the stress in the scenario. Acc. to the example investigated here, plane surfaces seem to be optimal. This is indeed not accepted as the final truth because of self-set limits for only three design variables and artificially chosen restriction bounds. Hence, optimisation of more sophisticated configurations is intended for future analyses.

**Figure 6:** Example of shape optimised geometry and corresponding results
4 CONCLUSIONS AND OUTLOOK

It has been shown that both optimisation approaches, either topological or shape, lead to an optimal solution on its own. Topological optimisation gives a sufficiently clear distinction between required and non-required densities, which can be interpreted like strut and tie models in reinforced concrete design. The outer shape of investigated structures is flexible and covers simple to complex geometries. Shape optimisation modifies any outer shape to the best suited layout. Reflecting the current goal of research – gain optimal-shaped and layouted reinforced concrete connections of subsequent linings segments – both approaches separately applied seem not to be enough. Rather a mutual dependency has to be considered, since each change of geometry or of structural stiffness also affects the other respectively. Their combination should be consecutive. Here the challenge remains that topological optimisation requires a fixed element mesh whereas reconfiguration by shape optimisation changes the discretisation constantly.

ACKNOWLEDGEMENT

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Experimental and Numerical Investigation on Shallow Tunnelling in Unsaturated Soils

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Abstract

Excavation of shallow tunnels with the New Austrian Tunnelling Method (NATM) requires proper assessing of the tunnel face stability, to enable an open-face excavation, and the estimation of the correspondent surface settlements. Soils in a partially saturated condition exhibit a higher cohesion than in a fully saturated state, which can be taken into account when assessing the stability of the tunnel face. For the assessment of the face support pressure, different methods are used in engineering practice, varying from simple empirical and analytical formulations to advanced finite element analysis. Such procedures can be modified to account for the unsaturated state of soils. In the present paper two possibilities were explored. Tunnel face stability was firstly analysed numerically, then the results were compared with a simple analytical formulation. Unsaturated triaxial tests were performed, varying both consolidation stress and matric suction, to calibrate the numerical models. The constitutive model utilized in the numerical analysis was the Extended Mohr-Coulomb Model. Both the closed-form solution and the numerical analysis showed that tunnel face stability can greatly benefit from the enhanced cohesion generated in partially saturated soils.

Keywords: NATM, face support pressure, unsaturated soil, numerical analysis
1 INTRODUCTION

Several studies have been conducted on the face stability of shallow tunnels in sand and clay ([6]) and on associated settlements ([3]), but no or little attempt has been made to extend existing tunnel models in saturated or dry conditions to partially saturated conditions.

The present numerical study assesses the face stability of shallow tunnel in unsaturated soil considering the variation of cohesion [2] and of Young’s modulus [4] with saturation. The water table lies below the tunnel in a soil exhibiting a certain capillary rise, so that the tunnel is driven in a partially saturated layer. A method is developed in FLAC$^{3D}$ to account for the benefits of partial saturation to face stability and reduction of settlements. The linear elastic model with Mohr-Coulomb failure criterion, extended to partially saturated states [2], was calibrated with triaxial tests on unsaturated soil and used in the numerical computation. During consolidation the tunnel face is fixed in all directions and afterwards is released to evaluate its stability. The results show how the combined action of partial saturation on face stability and settlements can be easily described via numerical modelling.

2 MODELLING TECHNIQUE

Mohr-Coulomb failure criterion has the expression

$$\tau = c' + \sigma' \tan \phi'$$

(1)

According to Bishop [1] the effective stress of unsaturated soils can be expressed with

$$\sigma' = (\sigma - u_a) - \chi(u_a - u_w)$$

(2)

where $u_a$ is the pore-air pressure in the voids, $\chi$ is a fitting parameter that varies nonlinearly with the degree of saturation and is equal to unity for saturated soils, and $u_w$ is the pore-water pressure. According to Fredlund et al. [2] $\chi$ is a function degree of saturation:

$$\chi = S^k$$

(3)

where the fitting parameter $k$ depends on the soil type. According to the previous formulation, the unsaturated condition increases the total cohesion of the material:

$$c_{\text{unsat}} = c' + S^k(u_a - u_w)\tan\phi'$$

(4)

In the present study suction is assumed to develop linearly from the water table:
\[ u_a - u_w = \gamma z \]  \hspace{1cm} (5)

where \( z \) is the vertical coordinate, equal to zero at the water table and increasing towards the ground surface.

The degree of saturation is calculated from the Soil Water Characteristic Curves (SWCC) obtained experimentally. According to van Genuchten [5] the SWCC has the expression

\[ S = S_r + \frac{1 - S_r}{[1 + (\alpha s)^n]^m} \]  \hspace{1cm} (6)

In Eq. (6) \( S_r \) is the residual degree of saturation, \( s = (u_a - u_w) \) is the suction and \( \alpha, n, m \) are fitting parameters.

The density of the soil changes alongside with the degree of saturation with the formula

\[ \rho = \rho_d + n S \rho_w \]  \hspace{1cm} (7)

Rahardjo et al. [4] proposed to relate the Young’s modulus of unsaturated and saturated soils to the net mean stress and the suction:

\[ E_{\text{unsat}} = a(\sigma_n - u_a)^m + b(u_a - u_w)^n + c(\sigma_n - u_a)(u_a - u_w) \rho \]  \hspace{1cm} (8)

3  PARAMETERS OF THE NUMERICAL SIMULATION

To calibrate the parameters a series of triaxial tests at different suction values was carried out. A SWCC, to directly relate the degree of saturation and the suction, would be beneficial, but not necessary, as its parameters can be back-calculated from Eq. (3).

3.1  Soil Water Characteristic Curve

The SWCC is generally dependent on grain distribution and dry density. For a dry density of 1.86 g/cm\(^3\) the parameters of the SWCC were back-calculated from Eq. (3):

<table>
<thead>
<tr>
<th>( S_r )</th>
<th>( \alpha ) (kPa(^{-1}))</th>
<th>( n )</th>
<th>( m )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.00669</td>
<td>2.834</td>
<td>0.647</td>
</tr>
</tbody>
</table>

3.2  Unsaturated Triaxial Tests

A loessy soil with a silt content of 64.4%, a sand content of 24.7% and a plasticity index of 4% was chosen for the tests.
A series of direct shear tests for the saturated soil and two series of unsaturated CD triaxial test were performed to calibrate parameter \( k \) in Eq. (4). As shown in Figure 1, the value \( k = 12.759 \) best-fits the data \((c_{sat} = 5.9 \text{ kPa}, \phi' = 32.3^\circ)\).

Soil density decreases vertically from a saturated density \( \rho_{sat} \) of 2.18 g/cm\(^3\) according to Eq. (6), considering a dry density of \( \rho_d = 1.86 \text{ g/cm}^3 \) and a porosity \( n \) of 0.321. The numerical simulation needs the specification of two elastic parameters, namely the bulk modulus \( K \) (responsible for the volumetric strain) and the shear modulus \( G \) (responsible for the deviatoric strain), which can be related to the elastic modulus \( E \) and the Poisson’s ratio \( \nu \) obtained from the triaxial tests.

The parameters \( a, b, c, n, m \) and \( p \) of Eq. (8) were obtained fitting the Young’s moduli obtained from the triaxial tests (Figure 2) and their values are shown in Table 2.

<table>
<thead>
<tr>
<th>a</th>
<th>b</th>
<th>c</th>
<th>m</th>
<th>n</th>
<th>p</th>
</tr>
</thead>
<tbody>
<tr>
<td>817.123</td>
<td>29.335</td>
<td>-326.429</td>
<td>1.416</td>
<td>1.167</td>
<td>0.627</td>
</tr>
</tbody>
</table>
Triaxial tests provided a mean value of $v$ equal to 0.35 and thus $K$ and $G$ can be calculated.

4 NUMERICAL MODEL

A numerical model of a shallow tunnel, having a diameter $D$ of 5m, a soil cover of 2.5m, is generated in FLAC$^3$D with an elastic liner supporting the lateral surface and a free tunnel face after consolidation. The water table is located 0.5m below the bottom of the tunnel. The effective stress in the model has been calculated with Eq. (2) and Eq. (3) which corresponds to assuming an enhanced cohesion in the unsaturated soil as in Eq. (4). Also, the density decreases with increasing suction and decreasing degree of saturation as in Eq (7). Similarly to Figure 2, the Young’s modulus depends on the suction value and the mean net stress. Therefore, initial stresses in the model are used to update the Young’s modulus with Eq. (8).

![Figure 3: Contour of the displacement magnitude](image)

The maximum settlement is observed 1.35m ahead of the tunnel face and has a magnitude of only 1.6 mm. Therefore the tunnel face is regarded as stable.

4.1 Comparison without a Closed-form Formula

Based on parametric studies, Vermeer & Ruse [6] proposed the closed-form formula of Eq. (9) to calculate the limit support pressure of the tunnel face, suggesting that, when the calculated pressure is negative, open-face tunnelling could be performed.

$$p_f = \gamma D \left( \frac{1}{9\tan\varphi'} - 0.05 \right) - \frac{c}{\tan\varphi'}$$ (9)
Considering the values of $\gamma$ and $c$ at the bottom and at the top of the tunnel, the results listed in Table 3 are obtained.

**Table 3:** Limit pressures calculated with formula of Vermeer & Ruse

<table>
<thead>
<tr>
<th>$c$ (kPa)</th>
<th>$\gamma$ (kN/m³)</th>
<th>$p_f$ (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>9.1 ÷ 27.6</td>
<td>21.3 ÷ 21.4</td>
<td>-0.9 ÷ -30.3</td>
</tr>
</tbody>
</table>

A negative value is predicted both at the tunnel top and at the tunnel bottom, confirming that there is no need to provide a support pressure to the tunnel face, which corresponds to the outcome of the numerical analysis.

5 CONCLUSION

A method has been developed to numerically analyse the face stability of shallow tunnels in unsaturated soils considering the spatial variation of soil strength, density and Young's modulus. An existing theoretical closed-form formula for the prediction of the limit support pressure of the tunnel face has been extended to the unsaturated case and compared to the numerical results. The results obtained with both methods predict stability of the tunnel face.

REFERENCES


Periphery Trench for Reducing the Impact of Surface Subsidence on Structures

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Abstract

Tunnel excavations, underground mines and the collapse of abandoned shallow natural cavities induce a surface subsidence. The existing structures and infrastructures can be damaged by the strain and settlement of the soil in the subsided zone. Different strategies can be used to deal with the subsidence hazard before, during and after the subsidence occurrence. Some mitigation methods exist for reducing the impact of the subsidence on the existing structures. One of them is the periphery trench. The method consists in cutting the soil around the existing building then filling the trench with a compressible material. The effect of horizontal strain will be absorbed by the partial closure of the trench. Numerical methods (2D and 3D, finite element method and distinct element methods) are used to evaluate the optimal dimensions and the position of the periphery trench according to the magnitude of subsidence, the relative position of the settlement trough and the structure and the nature of the structure itself (mainly the position of the foundation). In addition, the numerical methods are used to determine the characteristics of the filling materials that can be recommended. A numerical parametric study has been carried out focussing mainly on the Young modulus of filling material. The numerical modelling results show clearly that the trench is efficient when the building is located on the compression zone of the subsidence. Moreover the distance between the trench and the foundation should be greater than one meter. The filling material must have a Young modulus ranging from 1 to 10 % of the Young modulus of soil.

Keywords: Tunnels, mines, subsidence, damage, mitigation, trench
1 SUBSIDENCE DESCRIPTION AND CONSEQUENCES

1.1 Subsidence Mechanism and Components

Subsidence is a vertical displacement of the ground surface over areas where mineral ores were removed. Subsidence causes ground surface deformation resulting in a range of problems from deep holes with vertical sides that pose physical threats to people, to more subtle forms of subsidence characterized by sagging of the ground surface producing more damage, over larger areas, affecting nearly all manmade structures. The subsidence breaks up classically into a vertical movement of the ground, called subsidence, and a horizontal displacement (Peck, 1964, Standing, 2008, Al Heib, 2008). All forms of slope (tilt and strain) are calculated as the difference in subsidence and horizontal displacements between two points close together divided by their distance apart. Figure 1 presents the theoretical curves of vertical displacement, horizontal displacement, tilt, horizontal strain and curvature for an underground mine. Traditionally, only the vertical displacements are obtained by direct survey measurements, the others parameters are estimated using empirical and analytical approaches (Lack et al., 1992, Deck et al., 2003). The subsidence characteristics depend on the characteristics of underground cavities (depth, surface, etc.). The influence angle $\gamma$ determines the limit impact of subsidence on structures and infrastructures. The influence angle corresponds to the vertical direction and the line that connects negligible subsidence point to the edge of the underground voids form this angle. The maximum damages observed on structures are located in the zone of maximum horizontal extension strain defined by the angle $\theta$ (Figure 1).
Periphery Trench for Reducing the of Surface Subsidence on Structures

1.2 Damages of Structures

The influence of subsidence on buildings and infrastructures has become an important and costly environmental issue during mining and after the closure of mines (Edjossan-Sossou et al., 2012). The figure 2 shows the effect of compression strain on an existing structure. The trench plays a role in this case of situation to reduce the damage level.

The figure 3 idealizes the different movements that can affect the structure due to surface subsidence. The vertical component of ground movements (subsidence) causes changes in ground gradient, which can adversely affect, for example, drainage, tall buildings and machinery in factories. The tilt, horizontal strains (extension and compression) and curvature are the causes of the most commonly observed type of subsidence damage. Extension is characterized by the pulled open joints in masonry. The compression strain results in the: squeezing-in of voids such as doors and windows and the horizontal movements of masonry blocks.

Figure 1: Subsidence Parameters (O: layer open, Am: maximal subsidence, γ and θ: influence angle and maximum strain angle, D: depth, Wc: critical width)
1.3 Mitigation Methods of Subsidence Consequences

The mitigation methods are very important to protect structures from the subsidence effects. The table 1 summarizes the existing methods and can be used up today. The mains methods of the risk management are classified on three categories: i) The reduction of the hazard intensity by reducing the probability of the cavity collapse and the propagation to the surface; ii) The strengthen of the structure by increasing the capacity of the structure to resist against the hazard; iii) The reinforcement of the soil and the foundation to stop the propagation of the collapse to the surface.
Periphery Trench for Reducing the of Surface Subsidence on Structures

Figure 4: Presentation of peripheral trench technique

Table 1: Mitigation methods to protect buildings against subsidence

<table>
<thead>
<tr>
<th>MITIGATION METHOD</th>
<th>NEW</th>
<th>OLD</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>SOIL</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Filling</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td>Periphery trench</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td>Reinforcement by injection</td>
<td>x</td>
<td>x</td>
</tr>
<tr>
<td>Reinforcement by geotextiles</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>FOUNDATION</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Improvement the type of foundation</td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>Horizontal sliding Interfaces between soil and structure</td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>Adaptation a foundation</td>
<td></td>
<td>x</td>
</tr>
<tr>
<td><strong>STRUCTURE</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Structure type and function</td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>Implantation</td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>Architecture</td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>Dimensions and conception of the structures</td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>Using materials</td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>Vertical interfaces and joints</td>
<td>x</td>
<td></td>
</tr>
<tr>
<td>Reinforcement of the structure</td>
<td>x</td>
<td>x</td>
</tr>
</tbody>
</table>
We focus the paper on the periphery trench; this method concerns the soil and the foundation types. The raison of this choice is low cost and the application of the method for individual houses. The Periphery trench is a vertical slot realized in the existing soil at a horizontal distance from the structure. The trench is filled with a compressible natural or artificial material. The method is useful for existing and new structures (Peng et al., 1996).

2 STATE OF KNOWLEDGE

Few back analyses exists to demonstrate the efficiency of the method and to give executable recommendations. Many authors (Whittaker and Reddish, 1989; Kratzsch, 1983; Luo et al., 1992; Peng et al., 1996; Al Heib, 2008), propose this mitigation method. The following remarks can be listed:

- Different filling material can be used (gravel, coke, peat, etc.), with few data concerning the mechanical proprieties.
- The width of the trench varies between 40 and 60 cm, the distance between the structure and the trench limit is about 2 m and the depth of the trench varies between 60 cm and 1 m.
- The trench is placed under the level of the foundation.

2.1 In-situ Observations

Different research actions were done to evaluate the trench performance. The analyses were based on the in-situ observations and numerical methods (Luo & al. (1992) and Al Heib (2008). Peng et al. (1996) follow the behavior of 12 individual houses to determine the performance of the trench. The houses were under the influence of subsidence induced by a longwall of deep coalmine, the horizontal strain varies between 5 mm/m and 15 mm/m. The trench was around three house borders; the filling material was coke. The trench reduced the tension strain about 35% and 65% for the compression strain. From this experience they recommended a trench with 60 cm width and 60 cm depth.
Periphery Trench for Reducing the of Surface Subsidence on Structures

2.2 Numerical Modeling

Luo et al. (1992) used 2D finite element model to investigate the dimension of the trench. The boundary conditions correspond to a curvature of 5/10-5 l/m and a compression horizontal strain of 0.30 mm/m. The comparison was done between the two configurations with and without the trench allowing determining the contribution of the trench by reducing the horizontal strain. Figure 6 shows the ratio of the reduction of stresses for different parameters of width and depth of the trench. The structure stress decreases with the increasing of both depth and the width of the trench. The ratio becomes less important when the width is equal to 1.2 m and the depth is greater than 0.6 m under the foundation level. Luo & al. conclude that the trench is sufficient with the above values with good efficiency around 60%.

Figure 5: The effect on the trench on the strain from in-situ measurement in USA (Peng et al. 1996)

Figure 6: Effect of the trench on stress distribution on the structure function of the width and the depth of the trench Luo & al., 1992)
2.3 Physical Modeling

Hor et al. (2010) used a physical model to study the impact of the trench in subsidence reduction. The building model was positioned in maximum slope zone and in tensile zone. Two distances of 1.4 m and 0.4 m between the trench and the building located in tensile zone were tested, whereas only a distance of 1.4 m was assigned for the building in maximum slope zone. The physical model allows estimating the efficiency of the peripheral trench by either comparing the horizontal strain in the building or the strain of the ground surrounding it under conditions with and without the presence of the trench.

Table 2: Average horizontal strains of building and its surrounding ground

<table>
<thead>
<tr>
<th>Building’s position</th>
<th>Distance between trench &amp; building (m)</th>
<th>Horizontal ground strain (mm/m)</th>
<th>Horizontal building strain (mm/m)</th>
<th>Strain reduction (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Without trench</td>
<td>With trench</td>
<td>Ground</td>
</tr>
<tr>
<td>Maximum slope zone</td>
<td>1.4</td>
<td>-2.88</td>
<td>-0.93</td>
<td>-0.20</td>
</tr>
<tr>
<td>Tensile zone</td>
<td>1.4</td>
<td>-2.26</td>
<td>-1.79</td>
<td>-0.16</td>
</tr>
<tr>
<td></td>
<td>0.4</td>
<td>-</td>
<td>-</td>
<td>-0.16</td>
</tr>
</tbody>
</table>

For structure located in maximum slope zone, the average compressive strains in Table 2 shows a reduction of almost 70% of the horizontal strain of the ground surrounding the building in the case with trench compared to the case without trench. On the other hand, the reduction is equal to 35% if building strains are considered. For structure in tensile zone, the trench diminishes about 21% of the surrounding ground strain. Around 56% and 38% of strain reduction are found for buildings with trenches respectively at 1.4 m and 0.4 m distance. From the obtained results, we can provide the following judgments:
• The efficiency of the trench depends on the position of the building. In terms of ground strain, the trench is much more effective for the maximum slope position; while it is less effective, regarding to the building strain.

• The closer the distance between the building and the trench, the less effective is the peripheral trench.

3 CHARACTERIZATION OF FILLING MATERIALS

The contribution of this study is to confirm the role of the trench for different ground strain, in the presence of a masonry structure. We developed a soil and structure model with the presence of the periphery trench. The considered structure is a masonry wall in 2D. The distinct element method allows taking into account the interface between soil and structure, and the interface between the different structure elements. One soil layer is modeled 50 m wide and 15 m high. The boundary conditions correspond to a horizontal strain compression (Figure 7). The soil corresponds to isotropic and homogenous material and is characterized by linear elastic-perfectl plastic behavior with Mohr-Coulomb criterion. The masonry wall behavior is elastic, an interface was simulated between the structure and the soil, the interface facilitates the tangential displacement when the soil is loaded by the subsidence and compression strain. The wall is stiffer than the soil. The trench width is 50 cm and the depth is 1 m. It is placed 1 m from the structure limit and the trench is first considered as empty. The boundary conditions are no vertical displacements at the bottom and the two vertical boundaries are subjected to horizontal displacements. Figure 7-b presents two case studies corresponding to 6 mm/m and 12 mm/m. The trench contributes to the decreasing of the soil deformation. The following remarks can be done: the deformation of soil and the structure are smaller than the applied deformation on the boundaries, the reason is the presence of a rigid masonry structure, and the trench reduces the soil strain by about 45%. Whatever, the trench increases slightly the strain of the base of the structure.
Figure 7: 2D Numerical model (a) to study the effect of trench on soil and structure and Horizontal strain for different configurations (b)

The second numerical model concerns a 3D numerical model using finite elements code. We studied different values of Young Modulus of filling material for the trench. The geometry of the model is presented on the Figure 8. The model corresponds to a thickness of the soil layer equal to 20 m. The structure corresponding to a slab is located in the maximum tilt zone and the distance between the structure and the trench is equal to 1.4 m. The dimensions of the trench are 0.6 m in width and 1.2 m in depth.
The effect of the ratio on $\frac{E_{tp}}{E_s}$ is studied by increasing the value of the Young modulus of the trench $E_{tp}$. The ratio $E_{tp}/E_s$ varies between 1 and 30%. These configurations were compared to the configuration without trench. An example of the influence of the trench is illustrated in Figure 9 for $E_{tp}/E_s=10\%$. The influence of the trench on the horizontal strain is obvious. One can observe that trench modifies and form a barrier against the propagation of the horizontal strain.
Table 3 and figure 10 present the maximal horizontal strains of the soil, the trench and the structure for different values of $E_{tp}/E_s$. The relation between the reduction of the deformation and the ratio $E_{tp}/E_s$ is non linear. For ratios $E_{tp}/E_s$ between 1% and 10%, a positive impact on the reduction of soil and structure strains is observed. Whatever, when the ratio is more important, the impact is negligible and the trench is not very efficient. When the ratio $E_{tp}/E_s=1\%$, which correspond to very compressible material, the reduction of the horizontal strain is 80%.

![Figure 10: Effect of the trench on horizontal strain function of the ratio of soil and trench Young modulus](image)

**Table 3:** Strain reduction (%) at function of $E_{tp}/E_s$ ratio

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Trench efficient indicator</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$E_{tp}/E_s=30%$</td>
</tr>
<tr>
<td>$\varepsilon_{h_{\text{max}}} \text{ (soil)}$</td>
<td>-</td>
</tr>
<tr>
<td>$\varepsilon_{h_{\text{max}}} \text{ (structure)}$</td>
<td>-8%</td>
</tr>
</tbody>
</table>
4 CONCLUSION

The physical and numerical modeling approaches were carried out to study the behaviour of a mitigation technique – i.e. peripheral trench - used to reduce the damage to buildings due to ground movements. The trench described in this paper has proved very effective in reducing the compressive strain in the ground and also in the building. Its efficiency varies depending on the building location relatively to the subsidence trough, and on the distance from the building. The obtained results are interesting since they are comparable with those from observations and numerical modeling. The trench is efficient when the filling material is very compressible, its Young modulus must be less than 10% of that of the soil. The reduction of the horizontal strain of the soil may decrease the damage of structures located on the subsidence influence zone.

REFERENCES


Seismic Prediction on Tunnel Boring Machines – Comparison of Active Seismic Sources and Drilling Noise Measurements

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Abstract

At the GFZ German Research Centre for Geosciences the "Integrated Seismic Imaging System" (ISIS) has been developed. The most important components of this system are pneumatic hammers acting as seismic sources, geophones as receivers integrated into rock anchors and a processing and interpretation system. The latter is based on the concept of tunnel surface waves that are converted to shear waves at the tunnel face. This system is patented and now in commercial application by the Herrenknecht AG. In the framework of the GEOTECHNOLOGIEN project “Seismic Observations for Underground Development” (SOUND) the acquisition equipment of ISIS has been applied to realize an acquisition campaign on a tunnel construction site in Spain. In contrast to the conventional approach of ISIS, here the cutter head producing noise during tunnel excavation was used as a seismic source and only seismic receivers needed to be deployed. This paper compares active and passive seismic measurements on tunnel boring machines with respect to technical aspects, the measuring process and performance parameters like structure resolution and exploration range.

Keywords: Seismic prediction, comparison of seismic sources, TBM noise measurements
1 INTRODUCTION

The major task for tunnel seismic measurement systems is to predict the rock condition ahead of the tunnel face and in particular to warn against mechanically weak rock zones, e.g. fault zones. In recent years a couple of seismic systems for the application on tunnel boring machines (TBM) have been developed. These systems generate seismic waves either actively by controlled seismic sources such as actuators, impact hammers and blasting [3], [5], [6] or passively by vibrations produced by the cutter head of the TBM [2], [7]. The emitted seismic waves are reflected at rock inhomogeneities and the back-travelling waves are received by geophones or accelerometers mounted at the tunnel surface or within boreholes. The tunnel seismic measurement geometry is comparable to that of borehole seismic measurements. Hence, seismic data processing techniques developed for Vertical Seismic Profiling (VSP) methods are also used for tunnel seismic measurements. The paper compares active and passive seismic measurements on TBMs using the Integrated Seismic Imaging System (ISIS) developed by the GFZ.

2 INTEGRATED SEISMIC IMAGING SYSTEM (ISIS)

2.1 Seismic Data Acquisition

The ISIS receiver (Figure 1) is a screwable glass-fibre reinforced polymer resin rock anchor equipped with three orthogonal mounted geophones [4] building a 3-component (3C) geophone. This geophone anchor can be installed in drill holes in two ways. For permanent installation the receivers are glued into the boreholes with a two-component epoxy resin. Alternatively the geophone anchor is screwed into a closed tube equipped with an internal screw thread which is cemented in the drill hole. After finishing the measurements the receiver can be removed. Only the tube remains in the rock and is lost. For TBM seismic measurements usually four geophone anchors, two on each tunnel side wall, are installed in 1 m or 2 m deep boreholes. The positions and horizontal intervals of the receivers depend on the machine type and the accessibility of the tunnel wall for anchor installation and removing. An addition to the ISIS receiver system is the tubbing geophone (Figure 1). This 3C geophone is also self-constructed by the GFZ for use on TBMs with segment lining. It consists of three geophones with 28 Hz eigenfrequency like the ISIS geophone anchor. For seismic measurements the tubbing geophone is screwed into the grout hole casing of a tubbing segment.
Figure 1: The ISIS receiver system (left) comprises a removable anchor equipped with a 3C geophone at its tip and a tube which is cemented in the borehole. For seismic measurements the geophone anchor is screwed into the tube. The tubbing geophone (right) which is screwed into a grout hole casing of a tubbing segment is an addition to the ISIS receiver system.

For active measurements a pneumatically driven impact hammer is used as seismic source (Figure 2). The seismic waves are generated by the impact of an accelerated 5 kg moving mass. Prior to the impact, the hammer is pre-stressed toward the tunnel surface with a mass equivalent of 200 kg to achieve a good coupling. In combination with a TBM the hammer can be used in all directions. In recent applications the hammers have been installed at the TBM grippers. The small trigger error of less than 0.1 ms and the reliable repeatability of the emitted signals allow a significant improvement of signal-to-noise ratio by vertical stacking. Several ten thousands of shots with the impulse hammers have been carried out during TBM tunnelling as well as drilling and blasting tunnelling in Switzerland, France and Scotland.

Figure 2: Main components of the pneumatic impact hammer source.
2.2 Seismic Data Processing and Imaging

ISIS uses tunnel surface waves for exploration ahead the tunnel [1]. These waves are mainly converted into shear waves at the tunnel head face. The shear waves are reflected at geological structures with different seismic impedance (the product of seismic velocity and density). The backward travelling waves are reconverted into surface waves at the tunnel head face and recorded by the ISIS receivers. In the seismogram section the backward travelling surface waves are characterized by signals with linearly declining travel times towards larger source and receiver offsets. The data processing is quickly done and comprises methods to suppress the direct wave field as well as coherent noise signals by filtering techniques. For imaging a 3C-Kirchhoff-Migration is used in the ISIS software [8] taking advantage of the wave propagation model. During test measurements reflectors up to a distance of 200 m ahead the tunnel had been detected. The resolution of objects is limited to a range of 5 to 10 m depending on the frequencies of the tunnel surface waves which usually lie in the range of 200 to 400 Hz for a crystalline rock mass environment.

3 TBM NOISE MEASUREMENTS

A passive seismic experiment on a TBM in Spain was carried out in autumn 2012. The seismic wave field, excited primarily by the cutter head, was recorded with four portable tubbing geophones. At the beginning of each recording day two tubbing geophones were screwed into tubbing segments at approximately 20 m behind the cutter head, one on the right and one on the left hand tunnel side wall, and remained in these rings till the end of the recording day. Two further tubbing geophones were deployed 8 to 10 rings distant from the first ones depending on the actual conditions in this area, again on both tunnel side walls. One ring is 1.3 m wide. These four geophone positions – permanent only during one recording day – together with a triaxial accelerometer in approximately 18 m distance behind the cutter head constitute the main part of the acquisition setup. For the whole survey the accelerometer was permanently installed on a non-rotating part of the main TBM-bearing to record the pilot signal. Additionally the seismic wave field was recorded by a tubbing geophone and an ISIS geophone anchor, both stationarily deployed one ring apart at a distance of up to approximately 210 m behind the cutter head. A sub set of the data was chosen for a first analysis of the noise measurements on the TBM. The analysed 37 minutes recording window contains active excavation (e.g. records 10 to
18 in Figure 3b) as well as inactive TBM times (e.g. records 0 to 9 in Figure 3b). The raw data were correlated with the accelerometer component perpendicular to the tunnel axis. A bandpass filtered (100-150-550-650 Hz) receiver gather for the tubbing geophone next to the cutter head on the left tunnel side is shown in Figure 3b. The frequency analysis of the radial component of this geophone and the geophone on the opposite tunnel side (see Figure 3c) show that the signal for seismic exploration basically ranges from approximately 200 to 600 Hz. Other TBM noise data acquisition setups result in a lower frequency range (e.g. approximately 40 to 100 Hz, [7]). The high spectral amplitudes below 100 Hz are probably not connected with the noise from the cutter head and contain no coherent noise. It is assumed that most of the signal in this frequency range is caused by electrical and mechanical interference due to power lines, the conveyor belt or hydraulic pumps.

Figure 3: a) Stationary receivers: permanent installed tubbing geophone and ISIS geophone anchor, b) correlated and filtered data from a tubbing geophone and c) power spectra of the two nearest tubbing geophones to the cutter head for record 21 (during active excavation).

4 CONCLUSIONS

A passive seismic experiment on a TBM in Spain was carried out for a first test of using parts of ISIS for TBM noise measurements. First results of this experiment show that with the ISIS acquisition setup in the neighbourhood of the cutter head a higher frequency range of approximately 200 to 600 Hz can be obtained for the seismic exploration signal compared to some other passive data acquisition configurations. Furthermore, it can be shown that these TBM noise measurements have a comparable frequency range like measurements using an active source with ISIS.
REFERENCES


Structural Behaviour of Segmented Tunnels taking into Account the Interaction between Rings

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Abstract

The aim of this paper is to study the effect of the interaction between rings for segmented tunnels by using different axial loads and different orientations of the segment joints in the structural behaviour of the lining. In recent years, one of the methods used in the construction of tunnels in soft soils is the shield or Tunnel Boring Machine (TBM) method. Due to this constructive method, an axial load acts in the tunnel, producing an interaction between rings by means of the ring joints. This study was performed by using three-dimensional non-linear analyses with the Finite Element Method (FEM). The joints were modelled by using contact elements. It is concluded that the interaction between rings affects the structural behaviour of the tunnel, increasing its structural capacity. As well as, the failure mechanism is affected by this interaction.

Keywords: Segmented tunnels, interaction between rings, axial load and structural capacity
1 INTRODUCTION

In general, the tunnels built in soft soil by the Tunnel Boring Machine (TBM) method present a primary lining, which consists on precast segments that they work as temporary or permanent support. Figure 1a shows the TBM method [1], while the basic parts of a segmented tunnel are depicted in Figure 1b [2]. There is a joint between segments (longitudinal joint), as well as between rings (lateral joint). For this reason the segmented tunnels cannot be considered as a continuous ring. Therefore, these joints must be considered in the analyses [3].

The segments are of reinforced concrete and are set by the TBM at the same time that makes the excavation. After the TBM collocates the segments, forming a ring, this moves by means of thrust cylinders also called jacks, inducing an axial load in the tunnel (Figure 1a). This axial load remains acting in the lining even after the TBM has finished the tunnel. Therefore in this study, the effect of this axial load in the structural behaviour of the lining is evaluated. This study was performed by means of non-linear analyses of a typical tunnel built in Mexico.

2 NUMERICAL MODELS

Three-dimensional non-linear models of coupled rings were performed. The geometry of a typical tunnel built in Mexico was considered. Each ring has 6 segments plus a keystone. The internal diameter is of 770 cm, while the thickness and

Figure 1: Segmented tunnel: a) TBM method [1], b) Basic parts [2]
the width of the segments are 35 and 150 cm, respectively (Figure 2). Likewise, different axial loads and different orientations of the segment joints were considered to evaluate the structural behaviour of the lining.

![Figure 2](image): Geometry of a typical tunnel built in Mexico: a) front view; b) isometric view

### 2.1 Loads in the Lining

In order to simulate the effect of soil pressures on the tunnel, a radial load was applied [4]. This load was divided in an uniform and an ovalisation load (Figure 3).

![Figure 3](image): Soil pressures at the tunnel [4]: a) uniform radial load; b) ovalisation load

The uniform radial load is mainly determined by the depth at which the tunnel is located, whereas the ovalisation load is determined by the type of soil. This ovalisation load is obtained by means of equation 1 [4]. The load applied to the numerical model was increased until to reach the failure of the ring. Two criteria were taken into account in order to consider the failure: excessive cracking in the segments and excessive deformation of the rings.

\[ p = q - \Delta q \cos(2\theta) \]  

(1)
Where $p$ = final radial load (uniform and ovalisation load)
$q$ = uniform radial load
$\Delta q$ = ovalisation radial load
$\theta$ = circumferential angle

2.2 Description of Numerical Models

The segments were modelled by using an 8-node solid element and an adequate material model was selected in order to simulate the concrete behaviour: the cracking in tension and the crushing in compression; while the reinforcing steel was modelled as smeared material (volume ratio).

The segment and the ring joints were modelled by using contact elements, which allow to create discontinuous finite element models. For the segment joints, the contact between the surfaces was considered, as perfectly rough. This case corresponds to an infinite friction coefficient, where only the rotation of the segment joints is considered, as the typical behaviour obtained by experimental tests [5]. Likewise, for the ring joints the slip between the surfaces was considered, according to a Coulomb type law.

The numerical models are depicted in Figure 4. The main difference among the models was the orientation of the joints. For the first model, one weak plane was modelled due to the concurrent joints in the contiguous rings (Figure 4a). Likewise for the second and third model, two and four wake planes were considered, respectively (Figure 4b, c).

The William and Warnke concrete model was used [6]. This model considers the tensile and crushing failure. The shear behaviour was considered by means of the Drucker-Prager model. In order to reduce the computational effort, the compressive damage was neglected, because the damage in the segments is mainly by tensile stresses as reported in [5]. The mechanical properties of the materials are shown in Table 1. Where $f_t$ is the uniaxial tensile strengh, $\nu$ is the Poisson coefficient and $E$ is the elasticity modulus. On the other hand, $\beta_T$ y $\beta_c$ are the shear transfer coefficients of an open and close crack, respectively. Likewise, $f_y$ and $f'c$ are the yield stress of the steel and the uniaxial compressive strength of the concrete, respectively. Finally, the cohesion ($c$), angle of internal friction ($\phi$) and dilatancy angle ($\psi$) were defined.
Figure 4: Numerical models: a) model 1; b) model 2; c) model 3

Table 1: Mechanical properties of the materials used in the models

<table>
<thead>
<tr>
<th>Material</th>
<th>$f_t$ MPa</th>
<th>$v$</th>
<th>$E$ MPa</th>
<th>$\beta_c$</th>
<th>$\beta_T$</th>
<th>$f_y$ MPa</th>
<th>$f'_c$ MPa</th>
<th>$C$ MPa</th>
<th>$\phi$ rad</th>
<th>$\psi$ rad</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete</td>
<td>2.75</td>
<td>0.2</td>
<td>25685</td>
<td>1</td>
<td>0.01</td>
<td>---</td>
<td>elastic</td>
<td>14.0</td>
<td>0.2</td>
<td>0.2</td>
</tr>
<tr>
<td>Steel</td>
<td>---</td>
<td>0.2</td>
<td>205940</td>
<td>---</td>
<td>---</td>
<td>412</td>
<td>---</td>
<td>---</td>
<td>---</td>
<td>---</td>
</tr>
</tbody>
</table>
3 NUMERICAL RESULTS

Figure 5 shows the Load-Deformation curves by considering different values of axial loads. The structural behaviour of the lining is modified by the interaction between rings given by the axial load. The structural capacity increases as this interaction is higher.

Figure 5: Load-Deformation curves by considering different axial loads and orientations of joints: a) model 1; b) model 2; c) model 3
The structural capacity decreases as the weak planes increase. Likewise, the weak planes provide more flexibility in the rings, in order to produce that the effect of the interaction between rings decreases in the structural capacity. Since reductions of capacity of up 16% for the first model, 14% for the second model and 5.3% for the third model are obtained.

Figure 6 shows the obtained damage pattern. The damage increases as the axial load is higher, due to a strong interaction between rings. With this interaction, the deformations in the ring will be reduced, thus the failure mode is due to the internal loads which produce excessive cracking in the segments.

Likewise, if the axial load is low then the interaction between rings decreases. This produces that the rings have a similar behaviour of an isolated ring. The failure mode is due to excessive deformation of the rings. This type of failure depends mainly on the mechanical behaviour of the joints than the strength of the segments.

Figure 6: Damage in the rings: a) axial load = 11200 kN; b) axial load = 112 kN
4 CONCLUSIONS

The interaction between rings affects the structural behaviour of tunnel. Higher interaction will produce a higher structural capacity. Likewise, continuous joints cause weak planes, which affect the interaction between rings, producing a reduction of the structural capacity.

On the other hand, the type of failure is also affected by the interaction between rings. Rings will fail by excessive deformations with a low interaction, since the ring behaviour is similar as isolated rings. With higher interaction, the rings will work together, thus the deformations will be reduced. In this way, the failure is mainly due to the internal loads.

REFERENCES


In-situ Measurement and Numerical Analysis on Tunnel Lining and Ground Behaviour due to Shield Excavation

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Abstract

In urban tunnelling, often construction is done under some constrained conditions. In this paper, the ground movement due to shield excavation is observed and some measurements of the first tunnel lining pressure and stress development due to the excavation of second shield tunnel close to the first tunnel are carried out. The influence of the construction loads due to shield tunnelling on the surrounding ground and nearby structures is discussed. Numerical analysis is also carried out with elasto-plastic constitutive model of soil in which the mechanical characteristic of ground material and the construction process are appropriately taken into consideration. Comparing the results of the analyses with the monitoring data, the behaviour of surrounding ground and existing structure due to shield excavation is investigated focusing on the influence of the construction loads. The finite element analyses are carried out with FEM tij-2D software using an elasto-plastic constitutive model of soils, named subloading $t_{ij}$ model. It is revealed that during the construction of the following tunnel loads acted on the lining of the preceding tunnel. The elasto-plastic FE analysis is able to explain these actual behaviours adequately that cannot be expressed by the analysis using the conventional stress release ratio.

Keywords: Shield tunnel, monitoring, numerical analysis, subloading $t_{ij}$ model, construction load
INTRODUCTION

In urban shield tunnelling, often construction is done at about 1m distance under some constrained conditions such as the limitation of the road width and the existence of the underground structures. In these neighbouring constructions, it is necessary to evaluate the influence and assess the safety of surrounding ground and existing structure due to shield excavation adequately. The calculation method with stress released ratio is usually used in Japan to predict the ground behaviour due to shield excavation. However, because ground movements are depended on some construction conditions, for example cutter face pressure, backfill grouting pressure, control position of shield machine and so on, ground deformation behaviours are different each construction steps.

In this paper, the ground movement due to shield excavation is observed and some measurements of the first tunnel lining pressure and stress development due to the excavation of second shield tunnel close to the first tunnel are carried out. The influence of the construction loads due to shield tunnelling on the surrounding ground and nearby structures is discussed. Numerical analysis is also carried out with elasto-plastic constitutive model of soil in which the mechanical characteristic of ground material and the construction process are appropriately taken into consideration. Comparing the results of the analyses with the monitoring data, the behaviour of surrounding ground and existing structure due to shield excavation is investigated focusing on the influence of the construction loads. The finite element analyses are carried out with FEM tij-2D software using an elasto-plastic constitutive model of soils, named subloading $t_{ij}$ model [2]. This constitutive model has already applied to some braced excavation problems [1] and tunnel excavation problems [3], and calculation results had good response to model test results.

SOILS CHARACTERISTICS AND CONSTRUCTION CONDITIONS

Figure 1 show the soil profile and installation position of monitoring apparatus in the target construction site of shield excavation. The specifications of tunnel lining and excavation conditions when the shield passes through the monitoring section are shown in Table 1. The monitoring section was set at the area which is very close the second tunnel, where the second shield tunnel is obliquely upward of the first tunnel with a distance of about 1.1m.
Holocene layer and Pleistocene layer exist from the ground surface to the place near this monitoring site. Holocene layer is composed of a fine sandy layer (Aus), soft clay layers (Amc1 and Amc2), a sand gravel layer (Amg) and a sandy layer with medium density (As). On the other hand, Pleistocene layer exist in the lower part of Holocene layer, and is composed of a stiff clay layers (Oc2), a dense gravel layer (Tg) and a dense sandy layer (Os2). Table 2 shows some characteristics of each layer.

The upper ground of the first tunnel is formed with Amc2 and Ams soft layers. Meanwhile, hard layer Oc2 and Tg exist in the lower side ground of the first tunnel. In contrast, dense sand gravel layer Amg exist in the upper ground of the second tunnel, and soft clay layer Amc2 exists at the lower ground of the second tunnel.

The vertical displacement above the first tunnel and the horizontal displacement along with the vertical distance beside the first tunnel due to shield excavation are measured. Moreover, to evaluate the influence of the construction loads on the first tunnel lining due to second shield excavation, first tunnel lining pressure and cross section force are monitored.

**Figure 1:** Installation position of measurement equipment
Table 1: Specification of segments and shield excavation condition

<table>
<thead>
<tr>
<th></th>
<th>1st tunnel</th>
<th>2nd tunnel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shield excavate type</td>
<td>Slurry type</td>
<td></td>
</tr>
<tr>
<td>Outer diameter of shield / of tunnel lining (m)</td>
<td>5.44 / 5.30</td>
<td></td>
</tr>
<tr>
<td>Tunnel lining type</td>
<td>Cast-iron segment (Width : 0.9 m/ring)</td>
<td></td>
</tr>
<tr>
<td>Number of pieces</td>
<td>6 (A-type:3, B-type:2, Key:1)</td>
<td></td>
</tr>
<tr>
<td>Young modulus E (kN/m²)</td>
<td>1,7×10⁷</td>
<td></td>
</tr>
<tr>
<td>Ares A (m²)</td>
<td>0.0246</td>
<td></td>
</tr>
<tr>
<td>Area moment of inertia (m⁴)</td>
<td>0.00013</td>
<td></td>
</tr>
<tr>
<td>Overburden pressure (kN/m²)</td>
<td></td>
<td>340 240</td>
</tr>
<tr>
<td>Cutter face pressure (kN/m²)</td>
<td>Planning value</td>
<td>245 185</td>
</tr>
<tr>
<td></td>
<td>Monitoring value</td>
<td>190 – 280 170 – 190</td>
</tr>
<tr>
<td>Backfill grouting pressure (kN/m²)</td>
<td>Planning value</td>
<td>360 280</td>
</tr>
<tr>
<td></td>
<td>Monitoring value</td>
<td>Driving 100 – 360 100 – 300</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Stopping 280 – 310 –</td>
</tr>
<tr>
<td>Pitching (%)</td>
<td>Area -40.0 – -35.0 -8.0 – -6.8</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Average -35.5 -7.5</td>
<td></td>
</tr>
<tr>
<td>Excavation line shape (vertical) (%)</td>
<td>Over cut (mm)</td>
<td>-33.0 -10.0 15</td>
</tr>
</tbody>
</table>

3 CALCULATION CONDITION

3.1 Constitutive Model of Soil

The finite element analyses have been carried out with FEM tij-2D software using an elasto-plastic constitutive model of soils, named subloading tij model. The constitutive model requires a few soil parameters which can easily be obtained from conventional laboratory tests data. This model can consider influence of intermediate principal stress on the deformation and strength of soils, dependence of the direction of plastic flow on the stress paths, influence of density and/or confining pressure on the deformation and strength of soils.

3.2 Load Model considering Construction Loads

The considerable loading factor and the point of view of each construction stage are denoted as follows, and the calculation model for each step is shown in Figure 2.

3.2.1 Load model at the arrival of cutter face

When the cutter face pressure is larger than the lateral pressure, thrust acts on the ground consequently heaving occurs above the tunnel. In the reverse situation of the pressure balance, opposite phenomenon happens. Briefly, the ground deformation
behaviour depends on the balance between the cutter face pressure and the lateral pressure. In this paper, this influenced ratio is 15% of the differential pressure between both pressures according to reference [4] is applied.

3.2.2 Load model at the first half of shield machine
The ground deformation in this stage significantly depends on the slurry pressure of the cutter face in the additional void between the ground and the shield machine. When the slurry pressure is smaller than the overburden pressure, the settlement occurs just above the tunnel. However, because the slurry pressure is larger than the lateral pressure in the lateral direction, horizontal expansion occurs. Therefore, pressure at the cross section equals to the differential pressure between the cutters face pressure and the ground pressure around tunnel. Moreover, the self-weight of the first half of shield machine without buoyancy force adds pressure on the cross section.

3.2.3 Load model at the second half of shield machine
It has been thought that the influence of backfill grouting pressure is dominant in the ground deformation around tunnel in this stage. But in these monitoring results, the backfill grouting pressure is smaller than the overburden pressure, and it is impossible to heave the ground above the tunnel only by the backfill grouting pressure. Investigating the shield excavation conditions in detail, it is revealed that the thrust to the ground resulting from the difference between the excavation direction and shield machine position is considerably large. Therefore, the cross section pressure equals to the differential pressure between the backfill grouting pressure and the ground pressure around tunnel, the thrust to the ground resulting from the difference between the excavation direction and shield machine position, and the self-weight of the second half of shield machine without buoyancy force.

3.2.4 Load model at the tail passing
Because the backfill grouting pressure is smaller than the overburden pressure in these monitoring results, the settlement occurred again after the tail passed till the cutter face passed. It is proved that the backfill grouting pressure is controlled to the initial balanced condition with ground pressure around tunnel adequately. Therefore, the cross section pressure equals to the differential pressure between the backfill grouting pressure and the ground pressure around tunnel.
<table>
<thead>
<tr>
<th>Step</th>
<th>Calculating formula</th>
<th>Load model</th>
</tr>
</thead>
</table>
| 1 step | \( (arrival \ of \ cutter \ face) \)  
\( \Delta p_1 = (p_{ch} - p_h) \times \alpha \)  
\( p_{ch} \): cutter face pressure \( (\text{center}) \)  
\( p_h \): lateral pressure  
\( \alpha \): influence ratio balance between the cut face pressure and the lateral pressure \( (\alpha = 0.15) \) | \( \Delta p_1 = (p_{ch} - p_h) \times \alpha \) \( (\alpha = 0.15) \) |
| 2 step | \( (\text{first half of shield machine}) \)  
\( \Delta p_2 = p_G + (p_s - p_i) \times \beta_1 \)  
\( p_G \): self-weight of the first half of shield machine without buoyancy force  
\( p_s \): slurry pressure  
\( p_i \): lateral pressure after 1 step  
\( \beta_1 \): corrected value from 3D to 2D \( (\beta_1 = 0.75) \) | |}
| 3 step | \( (\text{second half of shield machine}) \)  
\( \Delta p_3 = p_G + (p_s - p_i') \times \beta_2 + p_p \)  
\( p_G \): self-weight of the first half of shield machine without buoyancy force  
\( p_s \): back fill grouting pressure  
\( p_i' \): lateral pressure after 2 step  
\( p_p \): pressing load based on the excavation direction and shield machine position  
\( \beta_2 \): corrected value from 3D to 2D \( (\beta_1 = 0.25) \) | |}
| 4 step | \( (\text{tail passing}) \)  
\( \Delta p_4 = (p_s - p_i') \times \beta_3 \)  
\( p_s \): back fill grouting pressure  
\( p_i' \): lateral pressure after 3 step  
\( \beta_3 \): correction factor from 3D to 2D \( (\beta_3 = 0.5) \) | |}

Figure 2: Illustration of calculation model for ground deformation due to shield excavation

3.2.5 Correction factor for transforming 2D condition to 3D condition

This load model considering the construction loads with three dimensional problems is different in each excavation stage. Because the 3D ground deformation is replaced to the 2D problem in this study, correction factors for transforming 2D condition to 3D condition are presumed based on the reference simulating results as Figure 2.
3.3 Outline of FE Analysis

Figure 3 shows the finite element mesh. The lateral boundary is extended to the 45-degree influence line from the bottom of the first tunnel, and the lower boundary is configured twice the shield diameter from the bottom of the first tunnel taking into account the dense sandy layer and stiff clayey layer under the first tunnel. The first tunnel lining is installed immediately after the completion of the first shield excavation and before the excavation of the second shield tunnel.

Soil parameters for elasto-plastic FE analysis are shown in Table 2. These parameters are decided based on the reference [1], because the same elasto-plastic FE program was employed for the braced excavation at the similar ground condition site. Also, the initial stress in the ground is set to become a prescribed over consolidated ground by self-weight consolidation and uniform loading/unloading simulation from the bottom of the ground. Since the excavation time of the shield tunnel is short, about half an hour, the elasto-plastic FE analysis is carried out considering undrained condition.

![Finite element mesh](image)

**Figure 3:** Finite element mesh

**Table 2:** Soil parameters for elasto-plastic FE analysis

<table>
<thead>
<tr>
<th>Layer</th>
<th>Lower Depth (GL-m)</th>
<th>SPT N-value (times)</th>
<th>Unit Weight (kN/m³)</th>
<th>Deformation Modulus E (kN/m²)</th>
<th>Poisson Ratio</th>
<th>Soil Parameters for Elasto-plastic FE Analysis (FEM tij-2D)</th>
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</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>λ</td>
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<tr>
<td>Holo-cene</td>
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<tr>
<td>Pleisto-cene</td>
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<td>90</td>
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<td>252000</td>
<td>0.30</td>
</tr>
</tbody>
</table>
4 RESULTS AND DISCUSSION

Figure 4 shows the comparison between observed data and calculation results for vertical displacement above the first tunnel and horizontal displacement at a distance of 1m from the first tunnel in each excavation steps. In these figures, FE calculation results with elasto-plastic model and elastic model are illustrated.

At the arrival of cutter face, settlement above the first shield tunnel and expanded horizontal deformation are to some extent in the observed data. On the other hand, the ground movement around first tunnel is insignificant in FE calculation results as the cutter face pressure is balanced with the lateral pressure. This tendency is confirmed in both elasto-plastic and elastic calculation results.

At the first half of shield machine, 4.7mm settlement above tunnel and about 3mm expanding horizontal deformation occur in monitoring data. In elasto-plastic calculation results, about 10mm settlement above tunnel and about 3mm expanding horizontal deformation at spring line of first tunnel are seen. Because the cutter face pressure is smaller than the lateral pressure above the first tunnel, this loading model tends to the stress released condition. Therefore observation and calculation have a good correspondence with this settlement tendency qualitatively. Since the Amc2 layer in FE analysis, which is deposited above the first tunnel, is considered softer than the actual ground condition, calculation settlement is twice of the observational settlement. As a result, the horizontal ground at the right shoulder part of first tunnel mobilizes inside tunnel, and it shows different tendency with the observed data. The settlement trough above the first tunnel of the elasto-plastic calculation indicates local settlement. On the other hand, in the elastic calculation the settlement trough is rather gentle in shape, in short, both settlement troughs show different tendency.

At the second half of shield machine, due to the influence of the thrust to the ground, about 7mm heave and maximum 5mm expanding horizontal deformation occurred in the monitoring ground. The elasto-plastic analysis shows about 4mm heave and maximum 5mm expanding horizontal deformation at the spring line position of the first tunnel corresponding position of the shield machine in the field. The magnitude of the settlement above the shield tunnel is different between the observation and elasto-plastic calculation. But the increment of the settlement between the first half and second half of shield machine in the elasto-plastic FE analysis has quantitatively very good agreement to that of the observed data.
In situ Measurement and Numerical Analysis on Tunnel Lining and Ground Behaviour due to Shield Excavation

Figure 4: Comparison of vertical and horizontal ground deformation between observed data and calculation results for each calculation step
This same tendency is also captured in the distribution of lateral displacement at 1m distance from the tunnel. On the other hand, the elastic calculation results can explain the tendency of the heave above the tunnel and the expanding phenomenon for considering the thrust model based on the shield machine position. However, for instance, the influence of heave extends to the far ground surface, the elastic calculation results are obviously different in comparison to the monitoring results and the elasto-plastic calculation results. Moreover, the heave above the tunnel significantly larger compare to the expanding horizontal deformation, this tendency differs vastly from the observation and the elasto-plastic calculation.

The deformation behaviour at the tail passing is similar to its behaviour at the second half of shield machine. Nevertheless, as the backfill grouting pressure balances with the lateral pressure, the local heave is restricted appropriately and elasto-plastic calculation results have a good correspond with monitoring data in comparison with elastic calculation results.

Figure 5 shows the distribution of the first tunnel lining pressure increment and stress development due to the second shield excavation based on the comparison between observation and elasto-plastic calculation.

At the close part of the first tunnel lining to the second shield excavation, the thrust and the cross section force fluctuate. But there is little change at the opposite part. The elasto-plastic FE analysis perfectly capture the influence of the construction load the same way as observed in the monitoring data. The elasto-plastic FE analysis is able to explain these actual behaviours adequately that cannot be expressed by the analysis using the conventional stress release ratio.

5 CONCLUSIONS

In this research, some results can be concluded as follows;

1) The shield excavation is divided four construction process arrival of the cutter face, the first half of shield machine, the second half of shield machine, and the tail passing. The ground movement around shield tunnel are different in each construction steps. Owing to predict the ground deformation behaviour due to shield excavation appropriately with numerical analysis, it is necessary to take into account of the construction load in each construction stage.
2) It can be thought that some factors for ground deformation are - i) the balance between the cutter face pressure and the lateral pressure at the cutter face, ii) the balance between the cutter face pressure and the lateral pressure at the first half of shield machine, iii) the differential pressure between the backfill grouting pressure and the ground pressure around tunnel at the second half of shield machine and at the tail passing, and iv) the self-weight of shield machine. Sometime, the thrust associated with the difference between the excavation direction and shield machine positions has an impact to ground deformation.

3) The FE analysis with elasto-plastic constitutive model of soil and the load model considering construction loads is employed to simulate the ground deformation behaviour around tunnel and the nearby tunnel lining pressure and stress development. It is revealed that the elasto-plastic FE analysis is able to explain actual behaviours adequately that cannot be expressed by the analysis using the conventional stress release ratio. Moreover, the elasto-plastic calculation results have a good agreement with the monitoring data in comparison with the elastic calculation results.

Figure 5: Comparison of distribution of earth pressure increment and cross section force of 1st tunnel lining between observed data and calculation results for each calculation step.
REFERENCES


Constitutive Models for Support Materials
An Analytical Model of a System of the Concentric Rings to State the Stresses in the Material Heterogeneous Cross Section of a Steel Shotcrete Tunnel Lining

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Abstract

A composite lining cross section from shotcrete and steel elements is the most common design to support the ground shortly after work excavation. The shotcrete incorporates steel elements into the ground. The steel elements provide an immediate response to the ground yielding after the steel arch is put on. As shotcrete becomes stiff with time the cross section becomes rigid and the lining response increases. Another layer of shotcrete later enhances the rigidity of cross section further. The composite cross section displays a time developing modulus of elasticity. This cross section feature allows a controlled release of primary ground stress around the excavation on the lowest response and it brings the ground to bear a part of ground load itself. The article presents underlying assumptions of the cooperating ring method involved in the analytical model that features the heterogeneous cross section. This method is developed according to the analytical solution for calculating the stress-strain state in circular multi-layer ring compound that was formulated by prof. Bulytchev. In addition the article presents as well a computer program HOMO that performs all calculations and displays an example.

Keywords: Primary lining, stress state, cross section, steel, shotcrete
1 INTRODUCTION

Design of a steel shotcrete lining structure (SSLS) that supports the ground massive round an excavation and bearing load capacity assessment are commonly carried out by methods that have been developed for reinforced concrete structures. However, these methods do not cover a number of features that are inherent for the SSLS only and the general terms of SSLS interaction with the rock mass. First of all it is necessary to appoint an existential causal bilateral dependency of both SSLS and ground massive. Secondly, there is changeable stiffness of SSLS lining structure given due construction stages of shotcrete layers and shotcrete’s setting. Each layer has a different value of modulus of elasticity in the initial SSLS lining operational period. The values of the moduli increase in both with time passing. The lining structure is continuously loaded by ground massive since a steel truss arch has been positioned and the first shotcrete layer sprayed on ground wall of excavation. The load is not constant on the SSLS. It develops with the construction stages as additional layers of shotcrete are sprayed and till the shotcrete setting is definitely terminated. From the first layer shotcrete spraying the SSLS yields and strains arise in the cross sections of SSLS there. A successful stabilization of ground massive by SSLS completes the yielding process of both as it stops once and for all. The present article brings to attention a method dealing the state of stress in materially heterogeneous cross sections that complies with the SSLS work conditions.

2 METHOD OF COOPERATING RINGS

A method of cooperating rings is an analytical method based on functions of a complex variable of potentials and Kolosov-Muschelischvili's functions (Bulytchev, 1982) figures up a stress-strain state in heterogeneous cross section of a circular lining made out of steel and shotcrete.

The method assumes a dismantlement of the heterogeneous lining cross section on a compound from several concentric circular rings of two types according the material homogeneity (Fig. 1). One is homogenous made from shotcrete and the other is heterogeneous made from shotcrete and steel elements regularly embedded in shotcrete.
A ground load is dispensed on particular rings by radial stress coefficients \((K_i)\). For each ring it is set a value of the radial stress coefficients that satisfy the condition of a continual radial displacement between adjacent ring walls and the boundary stress condition on inner and outer walls of ring compound (Fig. 2). A stress on the outer boundary equals to the radial ground load on lining and the stress on inner boundary equals zero.

**Figure 1:** Dismantlement of heterogeneous lining cross section in ring compound

**Figure 2:** Mathematical ring model
The radial stress coefficients are functions of ring wall thickness and ring wall radius, deformation characteristics of ring material (Poisson's ratio, modulus of elasticity) and in a heterogeneous ring wall also of a layout of embedded steel pieces. Rings cooperation follows from the continuity condition of wall radial displacements of adjacent rings. These displacements are expressed by recurrent formula that enumerates radial stress coefficients. The value of the radial stress coefficient in the ring is enumerated from the values of radial stress coefficients of the previous rings. Enumeration of the radial stress coefficients by recurrent formula starts with the first inner ring, continues on the next rings and terminates at the last outer ring.

\[ p_k = p_0(k) + p_2(k) \cos 2\theta \]
\[ q_k = q_2(k) \sin 2\theta \]
\[ p_0(k) = \left( \prod_{i=k+1}^{n} K_0(i) \right) p_0 \]
\[ \begin{pmatrix} p_2(k) \\ q_2(k) \end{pmatrix} = \left[ \prod_{i=k+1}^{n} \begin{pmatrix} K_{pp}(i) & K_{pq}(i) \\ K_{qp}(i) & K_{qq}(i) \end{pmatrix} \right] \begin{pmatrix} p_2 \\ q_2 \end{pmatrix} \]

\( K_0(i), K_{pp}(i), K_{pq}(i), K_{qp}(i), K_{qq}(i), i=1,\ldots,n \) radial stress coefficients in the \( i \)th ring.

Condition for simplified solution:

\( p_n = p_0, q=0, K_{0(1)}=K_{pp(1)}=K_{pq(1)}=K_{qp(1)}=K_{qq(1)}=0 \)

Each heterogeneous ring is homogenized first. The heterogeneous ring homogenization provides an alternative modulus of elasticity that substitutes the moduli of steel and concrete in the ring. The alternative modulus \((G^*)\) of heterogeneous ring is set from the shear elastic moduli of shotcrete \((G_a)\) and steel \((G_b)\) as their weighted average with regard to the width of the concrete segment and steel bar respectively in ring section Fig.3. Poisson's ratio is considered in both materials identical.

\[ G^* = \frac{G_a \cdot a + G_b \cdot b}{(a + b)} \]

**Figure 3:** Homogenization of heterogeneous ring
For each heterogeneous ring are set redistribution coefficients of tangential stress (a_in, a_out). These redistribution coefficients convert the state stress with the alternative modulus to the state of stress in the steel and concrete at the heterogeneous ring. The redistribution coefficients of tangential stress are determined from the radial displacements of both concrete segment and steel pieces that are equal for both (Fig.4).

![Redistribution coefficients of tangential stress](image1)

The homogenization second procedure joints two rings of different stiffness (moduli) into a provisional ring with a provisional elastic modulus and thickness that equals the sum of both ring predecessors. The ring homogenization starts with two inner compound rings. Then the homogenisation goes on further and the next compound ring in order is jointed to the provisional ring created in step before until there one ring is left. The two ring homogenization condition is that the two rings set radial displacement on the set outer wall equals the radial displacement of the single ring with new elastic modulus (Fig.5).

![Homogenization of ring compound](image2)
The complete analysis of cooperating rings method disclose an article in TUNNEL (Vojtasik, Hrubesova, Mohyla, Stankova, 2010). A way of method employment at structure analysis states a diagram on Fig.6.

![Diagram of cooperation rings method in structural analysis](image)

**Figure 6:** Cooperation rings method in structural analysis

### 3 EXAMPLE

The results of the stress state in one of the transient construction stages with two different moduli of shotcrete display the table 1 that summarizes the input data of geometry and deformation parameters lining cross section (Fig.1). The thicknesses of rings, the elastic moduli of shotcrete and the calculated values homogenized cross section (\(E_{\text{HOMO}}\)) and the redistribution coefficients of tangential stress (\(a_{\text{in}}, a_{\text{out}}\)). The diagrams in Fig. 7 show for comparison purposes the state of stress for both homogenized cross section and in cross section with actual materials in shotcrete and steel.

**Table 1:** Ring input data and results of homogenization calculation

<table>
<thead>
<tr>
<th>Construction stage 2 (Time = 1.0 day)</th>
<th>Shotcrete</th>
<th>Steel</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(a_{\text{in}})</td>
<td>(a_{\text{out}})</td>
</tr>
<tr>
<td>1</td>
<td>0.871</td>
<td>0.870</td>
</tr>
<tr>
<td>2</td>
<td>0.870</td>
<td>0.869</td>
</tr>
<tr>
<td>3</td>
<td>0.870</td>
<td>0.869</td>
</tr>
<tr>
<td>4</td>
<td>0.931</td>
<td>0.932</td>
</tr>
<tr>
<td>5</td>
<td>0.932</td>
<td>0.932</td>
</tr>
<tr>
<td>6</td>
<td>0.933</td>
<td>0.935</td>
</tr>
</tbody>
</table>
4 CONCLUSION

The method of cooperating rings is suitable to state the transient deformation parameter of the heterogeneous steel concrete cross-section during construction periods until the development of this parameter terminates with lining construction completion and shotcrete setting. This method should be considered as a simplified solution that gives readily deformation parameters and stress state of a complex structure that all are got more exactly but with much more effort time.

REFERENCES


Masonry Weathering of the Metro de Paris Gallery: Modeling via a Continuum Approach

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Abstract

The causes of the degradation of an underground structure, generally observed during the operation, can be related to several weathering processes, mainly the degradation of mechanical properties of the ground, and the creep of the lining constitutive materials. These phenomena can accelerate deformation and deflection which induce crack appearance and propagation and may reduce the mechanical strength of tunnels. In this study, an elasto-viscoplastic constitutive model with strain softening is used to reproduce the underground structures behavior during the degradation and/or ageing. An application to the Paris metro galleries which are affected mainly by meteoric and waste water infiltrations is conducted. Most of these structures (tunnels, stations and access corridors) are arched and made of a mixture of masonry, millstone and concrete. Despite the heterogeneity of the material constituting the tunnel lining, an equivalent homogeneous model is chosen and a strain softening approach is used to reproduce localization of the deformation associated to crack appearance. The numerical results show that we can reproduce, qualitatively, the underground structure behavior during degradation and assess the state of different structure components.

This study is as a part of a project devoted to diagnostic methodologies for tunnels and underground structures in operation (MéDiTOSS) funded by the French National Agency for Research (ANR).

Keywords: Tunnel, Degradation, Softening, Finite Element Method
1 INTRODUCTION

Ancient masonry, composed by stones, concrete and/or mortars, has an excellent record as a durable and attractive material, but deterioration processes is a weak point knowing that it can reduce its effectiveness. Deterioration of masonry results from several mechanisms, including crystallization, freezing and thawing, chemical attack by water and/or other substances. The deterioration rate is a function of composition, pore structure, manufacturing procedure and structural designed [4, 9]. In addition, the strength properties of rock materials decrease with an increase in their weathering degree and water content. An investigation has been done to determine the relationship between uniaxial compressive strength and degradation for selected rock types (limestone, sandstones), by using regression analyses to determine whether degradation was a useful predictor of compressive strength [6]. According to other authors [8], the chemical degradation induces the diminution of elastic modulus and material cohesion.

These different materials were used to build almost supports of Paris metro galleries and tunnels. During the construction of these galleries, a lost timbering was set up to support the excavation of the access corridors. Tunnel lining and timbering are currently damaged under weathering conditions mainly around abutments. The issue discussed in this paper is the numerical modeling of the whole structure including the contact zone between soil and tunnel, their behavior and their degradation over time. The followed approach allows us to analyze the response of the system and to identify the most damaged areas. The numerical model will serve as a basis support for the patrimony management.

2 DESCRIPTION OF THE STRUCTURE CONDITIONS

The galleries of Paris metro station here considered are at a depth of 3.2 meters. The tunnel linings are different between stations and are constructed by non-homogeneous materials: vaults were built with millstone and limestone blocks assembled in a gray fine concrete with various aggregates (limestone, flint), abutments are constructed with coarse concrete with different aggregates, mortar, fin concrete, limestone and / or millstone, and inverts, which are detached from the tunnel liners, are built with concrete. Figure 1 show carrots realized in the tunnel lining material for different stations, the samples appears very fragmented and disaggregated specifically at the extrados.
Non-destructive tests (impedance, ground penetrating radar), allow to gauge the state of degradation and fracture of the masonry lining. These tests have confirmed that there are voids in the base and in the extrados of abutments [2], which are due to the disappearance of the woodwork layer or disaggregating of concrete coarse. The void size may exceed 15 cm in certain cases.

Figure 1: Photos of carrots carried out in abutment (SP’I) or at the bottom of abutment (SRP’I) in different stations of Paris metro [7]

3 NUMERICAL MODEL OF GALLERY

Figure 2 shows the numerical model, of the considered gallery, based on finite element method (Abaqus software). Different parts and layers represents the ground, the limestone vault, the wholesome abutment part and the degraded abutment part, the concrete raft and the interface layer which is representing rotten wood and most degraded area. The heterogeneity of the surrounding layers (ground) is not taking into account. The quadratic reduced-integration elements (CPE8R) are used and the contact between the ground and the lining or between the lining and the interface is defined as tie contact. Despite the heterogeneity of the material constituting the tunnel lining, an equivalent homogeneous model is chosen to represent each zone of different parts of gallery. For this model, the mortar and the stone are not explicitly distinguished, average mechanical characteristics assuming continuous material, are estimated. In this approach the problem lies mainly in the choice of representative mechanical properties for the equivalent medium. Consequently, local deformation, stress concentration and cracks, at the level of the interface between a masonry bloc and the associated joint cannot be cached accurately, but the model can reproduce the damages obtained finally at a macro scale level which is represented by a few masonry blocs and their interface.
The meshed domain considers the geometrical and load symmetries. In addition to the geostatic load, an overload of 220 kPa is taken into account over a length of 6 m. Time is taken as an explicit which is around 240 years of operating ages.

**Figure 2:** Geometry of numerical model gallery (FEM)

### 4 STRESS STRAIN LAW AND MATERIALS BEHAVIOR

The failure surface evolves with plastic strain and is also affected by weathering, thereby degrading the strength and reducing the size of the elastic domain [5]. According to the degradation results found by some authors [6], dry or saturated limestone may lose more than 50% of its compressive strength, dry sandstone can lose more than 45%, but in its saturated state, the strength loss may reach 65%. This analysis was carried out by taking into account the different sizes of limestone sandstone and rocks. In this paper, strength reduction is reproduced by softening curve of cohesion (d) as a function of inelastic strain ($\varepsilon^{in}$). The softening curves of interface and degraded part of abutment part are considered almost linear (Figure 3).

The numerical model takes into account the time dependent behavior of gallery. The behavior of each zone of gallery is considered elasto-viscoplastic with Drucker-Prager criterion [1, 10], but the interface and the degraded part of abutment behaviors’ are related to the softening laws of cohesion (d) of both of them. The creep strain is modeled as a simple power law equation relating the strain rate ($\dot{\varepsilon}^{in}$) to both time and appropriate stress ($\sigma^{cr}$).

$$\dot{\varepsilon}^{in} = \Lambda(\sigma^{cr})^n t^m$$  \hspace{1cm} (1)
The soil and lining behaviors are considered unassociated with zero dilatancy angles ($\psi = 0$). An analysis to determine the correct confinement loss rate ($\lambda$) of these galleries has been done [10]. Table 1 shows the properties of the soil, interface and tunnel lining obtained from characterization tests.

### Table 1: Mechanical properties of all parts of the modeled gallery

<table>
<thead>
<tr>
<th>Material</th>
<th>$\rho$ [kg/m$^3$]</th>
<th>$E$ [GPa]</th>
<th>$\nu$</th>
<th>$d_0$ [kPa]</th>
<th>$\beta$ [%]</th>
<th>$A$ [Pa$^{-1}$]</th>
<th>$n$</th>
<th>$m$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground</td>
<td>2000</td>
<td>0.25</td>
<td>0.30</td>
<td>37.5</td>
<td>40.9</td>
<td>3$^{\circ}$-14</td>
<td>1</td>
<td>-0.3</td>
</tr>
<tr>
<td>Interface</td>
<td>560</td>
<td>11.0</td>
<td>0.23</td>
<td>1500</td>
<td>20</td>
<td>5$^{\circ}$-19</td>
<td>2</td>
<td>-0.4</td>
</tr>
<tr>
<td>Vault</td>
<td>2400</td>
<td>16.0</td>
<td>0.3</td>
<td>3000</td>
<td>44.8</td>
<td>2.8$^{\circ}$-10</td>
<td>1</td>
<td>-0.9</td>
</tr>
<tr>
<td>Wholesome abutment</td>
<td>6.40</td>
<td>2000</td>
<td>0.3</td>
<td>5850</td>
<td>5$^{\circ}$-18</td>
<td>2</td>
<td>-0.5</td>
<td></td>
</tr>
<tr>
<td>Degraded abutment</td>
<td>2000</td>
<td>3.20</td>
<td></td>
<td>2500</td>
<td>40.9</td>
<td>5$^{\circ}$-19</td>
<td>2</td>
<td>-0.5</td>
</tr>
<tr>
<td>Raft</td>
<td>2200</td>
<td>8.00</td>
<td></td>
<td>2160</td>
<td></td>
<td>2.26$^{\circ}$-11</td>
<td>1</td>
<td>-0.8</td>
</tr>
</tbody>
</table>

$\beta$: friction angle; $A$, $n$ and $m$ represent viscoplastic parameters [10].

5 RESULTS

Inelastic strain as a function of tunnel lining thickness ($e$) is analysed as shown in figure 4. These curves plotted at different times show that the extrados zone ($e \geq 0.6$ m) is the most damaged area. This numerical result is well supported by the probability of occurrence of cracks and damaged areas (figure 5), which is plotted by using results geo-endoscopic test and visual observations of some carrots carried out in the tunnel lining.
Figure 4: Vertical inelastic strain $\varepsilon_{v}^{in}$ as a function of tunnel lining and interface thickness ($e$) in 50, 100 and 200 years.

Figure 5: Probability of occurrence of cracks and most degraded areas across the masonry thickness [3].

Figure 6: a: Evolution of $\varepsilon_{v}^{in}$ and $\sigma_v$ in the abutment of tunnel lining (7.5 m of depth); b: Vertical inelastic strain in the tunnel lining in 100 years.
Figure 7: Evolution of $\varepsilon_v^{in}$ and $\sigma_v$ in the intrados of tunnel lining vault

Figure 6 shows the evolution of $\varepsilon_v^{in}$ in fixed point of the extrados of abutment that increases over time and the evolution of vertical stress in the same point that decreases. If the tunnel lining deforms more over its capacity of recovering, loads decreases, which leads to increase the instability of galleries. The inelastic strain, as shown in figure 7, tends to traction at the intrados of vault beyond a certain strain value.

6 CONCLUSION

The developed numerical model is able to reproduce the tunnel lining degradation over time. The obtained results are in adequacy with the visual observations and the non-destructive analysis results: the most damaged zones are the extrados and the base of abutments and extensions are localized in the intrados of the tunnel roof. Numerical analysis shows that a losing of strength alters the distribution of stresses in the gallery during its life. Therefore, this model captures the redistribution of stresses associated to the damaged areas and the required elements to estimate the evolution of damage. This numerical procedure will be then further used to assess the aging of these structures and to define the condition for sustainable regeneration.
REFERENCES


A Multiscale Oriented Concept for the Finite Element Analyses of Fiber Reinforced Concrete Tunnel Linings

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Abstract

A multiscale oriented modeling concept is proposed for the finite element analyses of segmental tunnel linings made of steel fiber reinforced concrete. At the micro level, the pullout behavior of a steel fiber embedded in concrete matrix, either straight or with hooked end, with or without inclination with respect to the loading direction, is investigated by means of analytical or numerical models. These models take into account interfacial slip, the inelastic deformation of fibers and the localized damage of concrete. A representative volume element (RVE) containing a number of fibers is used to describe the composite behavior at the meso-scale. For a certain opening crack, the bridging effect is obtained by the integration of the individual pullout response of all fibers crossing the crack; the mechanical properties of the RVE is analyzed using a hybrid fracture-micromechanics model. At the structural level, the Finite Element Method is used, applying the Embedded Crack approach, for which the cohesive behavior along macro-cracks and the material properties of the continuous parts are obtained from the homogenization of fiber-crack interactions and the corresponding RVE, respectively.

Keywords: Fiber Reinforced Concrete, Multiscale Model, Fiber Pullout, Composite property, Structural behavior
1 INTRODUCTION

1.1 Background

For the design of tunnel linings, in addition to guarantee safety against structural failure, criteria such as ductility and durability play an important role. While the need for sufficient ductility is triggered by various construction induced loading, which may lead locally to damage, the requirement for durability in the sense of resistance against corrosion is motivated by the anticipated life time of tunnels, which usually is 100 years and more. In classical reinforced concrete linings the ductility of the material is accomplished by adding steel bars as reinforcement in the regions of concrete structures subjected to tension. However, opening cracks may considerably limit the durability and therefore the applicability of traditional reinforced concrete (RC) structures. Using life cycle oriented design concepts for tunnel linings, given the above mentioned requirements for ductility and durability, fiber reinforced concrete (FRC) becomes particularly attractive due to its high ductility, or even strain-hardening behavior, accompanied with distributed cracking at small crack widths not affecting the structural durability.

The development of fiber reinforced concrete (FRC) can be traced back to the 1960’s, when the significance of adding steel fibers to enhance the ductility of plain concrete was recognized. The expansion of the research has brought us FRC composites featuring various types of fiber reinforcement using different materials, sizes, shapes, surfaces, contents, etc., recently leading to high performance materials such as engineered cementitious composites (ECC) [12]. In addition to the experimental investigation, large efforts have been devoted to the modeling of FRC on the material as well as on the structural level. Typically, a phenomenological approach is adopted, where the enhanced ductility of FRC is taken into account by means of a modification of the softening law in plain concrete models (e.g. [7]). More recently, advanced Finite Element technologies are applied, for example the Partition of Unity approach, by representing the slip behavior of individual fibers at a macroscopic level [18].

1.2 Multiscale Oriented Modeling Concept

For the numerical analyses of FRC composite materials and structures, a multiscale oriented approach enables to formulate the behavior of every constituent (fiber and matrix in our case) and their interactions at different length scales [8]. Aiming at the development of a simulation and optimization platform for segmental tunnel linings made of FRC or, using a hybrid approach, of FRC composite in conjunction with conventional reinforced concrete, a multiscale modeling concept is proposed in the present work (Figure 1):
Multiscale Modeling of FRC Materials and Structures

Figure 1: Multiscale oriented approach to the analyses of tunnel linings made of steel fiber reinforced concrete (in conjunction with conventional reinforced concrete).

- At the structural (macroscopic) level, the Finite Element Method is used, making use of a model which is suitable to capture propagating cracks. The interface behavior along the macro-cracks and the material properties of the intact parts are obtained from the analysis of the corresponding representative volume elements (RVE).

- An RVE containing a number of distributed fibers is used to describe the composite material behavior at the meso-scale. For a certain growing crack within the RVE, the bridging effect is obtained from the summation of pullout responses of all the fibers intercepting the crack. The effective mechanical properties of the RVE is generated by means of homogenization methods based on continuum micromechanics.

- At the micro-level, analytical and numerical models are developed in order to describe the pullout behavior of single fiber embedded in the matrix with arbitrary inclination w.r.t. the crack plane.
In the following sections, we first describe the models for single fiber pullout behavior which are validated for different cases. Next, making use of the pullout models and by up-scaling the information from the micro level, the fiber bridging effect on an opening crack and the constitutive response of an RVE are analyzed at the meso-scale. In the third part, the Embedded Crack model is introduced and applied for the preliminary numerical simulation of an FRC structure.

2 MICRO-LEVEL: SINGLE FIBER PULLOUT MODELS

2.1 Straight Fiber Pullout with or without Inclination

As the basic case, straight fiber pullout without inclination with respect to the loading direction has been well investigated by means of laboratory tests, numerical simulations as well as analytical models. It is generally accepted that the pullout procedure can be divided into three stages, i.e. bonded state, debonding stage and pulling-out phase. In the present work, an interfacial friction law is proposed. The load-displacement relations at the free end are obtained for the whole pullout process.

Compared to the situation described above, modeling the pullout behavior of an inclined fiber involves additional complexities correlated with the additional frictional stress, plastic deformation of the fiber and change of the geometrical state, caused by the lateral pressure on the interface, yielding of the steel and partial damage of the matrix, respectively (see e.g. the experimental observation in [11] and the analytical model in [5]). The situation of an inclined fiber embedded in the matrix is illustrated.

![Figure 2: Illustration of the inclined straight fiber pullout problem: (a) crack initiation (θ - inclination angle, O - initial intersection of fiber and crack, C - embedded end of fiber); (b) geometrical state of the fiber during pullout (Δ - crack opening, S - concrete spalling).](image-url)
in Figure 2a. For a certain pullout state, the additional fiber-concrete interactions is analyzed via two sub-models referred to as the cantilever-AB and the beam on elastic foundation-BC (Figure 2b). In both submodels, the force equilibrium on the fiber sections is analyzed; the free end pullout force corresponding to the current pullout state is calculated.

The model developed in the present work is validated by means of the experimental results reported in [11]; the pullout responses are well described (see [24]).

2.2 Pullout of Hooked End Fiber

In comparison to straight fibers, steel fibers with deformed geometry usually exhibit higher ductility during the pullout process. One of the most widely applied types of deformed steel fibers is the hooked end fiber, characterized by the hook on each end. During the pullout procedure, the resistance of the hooked end to straightening often contributes, as an anchorage effect, to the main portion of the total pullout force, in comparison to the case of a straight fiber, where the interfacial behavior plays the main role [19].

![Numerical simulation of the hooked end fiber pullout: Load-displacement diagram (upper), contour plots of the von Mises stress (lower-left) and the compressive damage (lower-right).](image)

To support the analytical formulation, numerical models for the single fiber pullout behavior are developed, using the Finite Element Analysis software Abaqus (Figure 3). The numerical simulation provides not only the pullout load-displacement relation at the free end, but also an insight into the problem and a reference for the formulation of the analytical model.

Regarding the analytical model proposed in the present work, the anchorage effect of the hooked end is represented by a multi-linear load-displacement relation, capturing a sequence of key states during the pullout progress. For every key state, the force equilibrium on the segments of the hooked end is analyzed and the resulting an-
chorage force is calculated, taking into account the yielding of steel and the damage of concrete. This submodel is then combined with the straight fiber pullout model described in the previous section, in order to predict the load-displacement relation during the hooked end fiber pullout with an arbitrary inclination w.r.t. the loading direction.

Figure 4: Inclined hooked end fiber pullout model validation: Comparison between the results of analytical model and experiments.

The model is validated with the experimental results reported in [21]. From Figure 4 we can see that the analytical model captures successfully the major feature observed during the pullout of a hooked end steel fiber in concrete matrix for different values of the strength of concrete and steel fiber and the inclination angle.

3 MESO-LEVEL: FRC COMPOSITE PROPERTIES

With the pullout behavior of single steel fiber embedded in concrete matrix investigated, the scope is now to analyze the behavior of fiber reinforced concrete at the meso level, considering an RVE of the composite material under tensile loading. From the micromechanical point of view, the increasing tension on the RVE will lead to the initiation of microcracks at the inherent flaws within the RVE [4]. As the load increases, those microcracks tend to open and propagate, however, unlike in the situation of plain concrete, their opening is constrained by the bridging effect of the fibers intersecting the cracks. To this end, it is essential to investigate first the bridging stress vs. crack opening relationship.
3.1 Crack Bridging Effect

In [13], for a homogeneous (in terms of position) and isotropic (w.r.t. orientation) distribution of fibers in the composite, the integration of the pullout forces of all fibers crossing a crack divided by the cross sectional area of the RVE provides the bridging law. While in [13], explicit analytical formulations of pullout force-displacement relation are used, the pullout response in the present work is obtained numerically.

3.2 Mechanical Properties of the RVE

The composite behavior under further tensile loading after the initiation of micro-cracks, i.e. whether those cracks grow quickly, leading to the softening and failure of the material, or the RVE shows pseudo strain-hardening response, which is typically observed in high performance fiber reinforced cement composites materials (HPFRCC [15]), depends on the contribution of the crack bridging effect due to fibers.

In the present work, we present a prototype of a novel meso-scale model, combining concepts of fracture mechanics at the level of individual micro cracks bridged by fibers and continuum micromechanics. This model is inspired by a conceptual model proposed for unreinforced brittle materials with microcracks in [17]. Before adopting this model for FRC, however, it was modified by allowing the evolution of the number of micro-cracks such that realistic concrete constitutive behavior can be replicated. Then, in the present model for FRC, the stress intensity factor, on which the Griffith fracture criterion \( K_I = K_{Ic} \) is based, is modified by defining \( K_I^f = K_I + K_f \) (see e.g. [23]), in order to take into account the influence of fiber bridging mechanisms on the level of micro-cracks. Here the modification term \( K_f \) depends on the crack size, the fiber content and bonding properties. The full model is currently in progress. As a first attempt, a prototype is tested. The constitutive behavior of the composite material, obtained from using \( K_I^f \) in the upscaling procedure proposed in [17], is illustrated in Figure 5. The transition from a brittle to ductile behavior is well represented.

The meso-scale model is able to represent the overall mechanical behavior of the composite material generated in the form of a continuum constitutive law at the level of the RVE including an ascending branch. Transition from distributed cracks to localized fracture is detected on the basis of the localization tensor related to the concrete-fiber composite material [20]. As soon as the loss of ellipticity is signalled, the behavior is governed by the opening of a macro-crack which is represented by using the Embedded Crack model.
Constitutive behavior of the FRC composite

![Graph showing constitutive behavior of the FRC composite](image)

Figure 5: Preliminary results for a hybrid fracture and micro mechanics based meso-scale model for FRC: Different constitutive behavior of the composite material with different extent of fiber enhancement (represented by the parameter $\alpha$).

4 MACRO-LEVEL: EMBEDDED CRACK MODEL

The localization of deformation into macro-cracks is represented on the level of Finite Elements by adopting the Embedded Crack approach [6, 16]. In this model, the scale transition is accomplished by an additive decomposition of the displacement field $u$ into a large scale (continuous) portion $\bar{u}$ and a discontinuous portion $\hat{u}$, representing the local displacement jump: $u(x) = \bar{u}(x) + \hat{u}(x)$. Since the element enrichment used for $\hat{u}$ is restricted to the element domain, the additional parameters connected with the displacement jump are resolved by static condensation without introducing global degrees of freedom [2, 6].

By adopting the Embedded Crack formulation proposed in [6] and defining appropriate traction-separation laws, the behavior of structures made of plain concrete can be simulated. Furthermore, by replacing the cohesive law on the crack interface with that obtained according to the traction-separation law from the meso-scale analysis of the composite material, the behavior of structures made of fiber reinforced concrete can be simulated (see Figure 6).
Figure 6: Numerical simulation of the L-shape experiment [3]: (a) The computed load-displacement relation compared with the experimental results and the plot of crack pattern; (b) comparison of the results with and without fiber reinforcement.

5 CONCLUSIONS

In this paper, the essential components of a multi-scale oriented modeling framework for the finite element analyses of steel fiber reinforced concrete materials and structures have been presented. At the lowest level, the pullout behavior in various cases of single steel fiber embedded in concrete matrix is described by analytical or numerical models. The models have been successfully validated by means of representative experimental results, capturing the major mechanisms involved in the pullout of single fibers. Then, the crack bridging law at the level of an RVE of the composite material is obtained by means of integration over the response of individual fibers distributed across the crack. A prototype of a hybrid fracture-micromechanics model for the constitutive response of the RVE at the meso-level is proposed; a first testing example reveals the potential of this model. The bridging law for a propagating crack is used as the basis for a macroscopic representation of cracks on the level of Finite Elements using the Embedded Crack approach. A preliminary structural simulation shows the performance of the multiscale model.

The work presented so far is not yet the full picture of the multiscale oriented scheme for the modeling, simulation and optimization of FRC and hybrid RC-FRC materials to be used for large structures, e.g. tunnel linings. The steps in progress and future work include...
the completion of the modified fracture-micromechanics model for the FRC composite behavior on the meso-scale, particularly the analysis of the strain-hardening phenomenon and the transition from strain hardening to strain softening (localization) behavior in connection with the Embedded Crack model,

- a fully 3D implementation of the Embedded Crack model in conjunction with a homogenization based interface law for FRC

- and the application of the complete multiscale framework to the analyses of tunnel lining segments at the structural scale.

ACKNOWLEDGEMENT

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REFERENCES


The Sacrificial Gallery in Tunnelling

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Abstract

The tunnelling process, without considering the excavation method (NATM, TBM or similar), includes the use of linings or supports (e.g. concrete, shotcrete, metal ribs) bearing the stresses transmitted by the soil and the external loads around the tunnel. For the design of these elements, the crown is the most important point for analysing its structural behaviour, from a stress and/or displacement point of view. The use of a new constructive procedure, named Sacrificial Gallery method has been verified with Finite Elements Methods, providing a considerable decrease in the displacement field at the crown of the excavation, which implies a significant reduction of the support section and the increase of the work safety.

Based in the Convergence-Confinement Method, this new method basically consists on the previous excavation of an additional small diameter gallery initially unsupported over the main tunnel. This new previous driving would benefit the main tunnel, since the displacement and stress field affecting the main tunnel specifically in the crown during implementation and operation would be partially assumed by the Sacrificial Gallery, reducing the stress-strain field in the crown and the top heading area.

Keywords: Sacrificial Gallery, Convergence-Confinement Method, tunnel, displacement, crown
1 INTRODUCTION

During the design and execution of underground works, one of the main problems is the sizing of the lining elements. Usually, the support will be light for high quality soils or shallow depths and will be heavy for soft soils and weak rocks ro deep depths with large deformations respectively. Even many times the works requires the use of an over-excavation to assume the deformations occurring during the construction [3]. For these cases, the different types of soil or rocks have led to use support systems (metal ribs, girders, rock-bolts, etc) which allow to carry out the tunnel execution with special attention to displacement values in the crown.

Considering the previous mentioned issues, the excavation of a new complementary small diameter gallery (named Sacrificial Gallery), based on the concept of the Convergence-Confinement Method and according to a stage excavation process above the main tunnel, will assume a portion of the stress-strain field through its own deformation. The analysed models based in the Finite Elements Method are presented.

2 THE CONVERGENCE-CONFINEMENT METHOD

The Convergence-Confinement Method (CCM) was fully developed by Panet and Guenot [8] [9] [10] and based on the previous works of Terzaghi [11, 12], Fenner [4], Pacher [7] and Lombardi [5] [1] [6], and has been a point of reference for tunnels implementation by the New Austrian Tunnelling Method (NATM) technique and frequently used with Tunnelling Boring Machine (TBM).

The CCM is looking for a balanced relationship between the deformations that occur in an initially circular tunnel, excavated in a single phase and subjected to a hydrostatic stress-field, assuming that all deformations occur in a perpendicular plane to the tunnel axis and considering a two-dimensional plane strain problem; at this moment, the tunnelling driving is executing correctly and the lining -temporary or permanent- is loading, according to the distance to the tunnel face and therefore to the tunnel area unsupported. Then, a lining section placed near the face does not bear all the existing loads, since some of them are absorbed by the face (face effect) in a process of redistribution of efforts and therefore stresses. However, when the face has moved well away from the plane support, the face effect decreases to the total radial deformation of the excavation or, otherwise, the support carries the full design load.

According to this, the three basic components of the well-known Convergence-Confinement Method are the curves: Longitudinal Deformation Profile (LDP), Ground Reaction Curve (GRC) and Support Characteristic Curve (SCC) (Figure 1):
3 METHOD PROCEDURE

The Ground Reaction Curve (Figure 1) is defined by the OEM curve which starts at a point O - where the internal pressure $p_i$ is equal to the initial stress $\sigma_0$ - while at the point M, the internal pressure is null and the radial displacement ($u^M_r$) is maximum. Therefore, the action of an external force (radial force) acting on the Sacrificial Gallery (initially unsupported) will cause an inward radial displacement on the affected soil, and providing the collapse is not reached, the main tunnel (below the Sacrificial Gallery) would be benefited from the possibilities of movements inside this one. So, the displacement and stress field affecting the mail tunnel - specifically in the crown - during the implementation and operation tasks would be assumed, in part, by the Sacrificial Gallery, executed previously to the main tunnel excavation, reducing the stress-strain field in the crown and top heading area (Figure 2).
The models were designed for a driving stage method (both for the main tunnel and the Sacrificial Gallery) based on a similar material found in the Madrid’s Underground (Metro) extension (miga sand) \cite{2} (Figure 3). The considered design parameters and model were:

- Soil material model: Mohr-Coulomb
- Main Tunnel: semicircular with an internal radius of 5m
- Depth to the crown of the main tunnel: 20m
- Sacrificial Gallery: circular with a radius of 0.5m
- Lining of the Sacrificial Gallery (steel): 0.8cm (100A), 1.6cm (100B)
- Distance from the crown of the main tunnel to centre of Sacrificial Gallery: 1m and 2m
- Checkpoints located on the crown of the main tunnel, at checkplanes away from the tunnel entrance (Figure 3): F (18m) and I (19m)

The obtained results would be compared taken into account the models without Sacrificial Gallery (initial values of vertical displacement at the checkpoints) and the models adding the Sacrificial Gallery, analysing the values obtained as difference of curves, as it can be noted in Figure 4. Table 1 includes a summary with the ratio of vertical displacements obtained for Checkpoints F and I, comparing the results for the Main Tunnel without Sacrificial Gallery (Final\textsubscript{0}) and with Sacrificial Gallery (Final\textsubscript{1}), attending to the distance from the crown to the centre of the Sacrificial Gallery and the thickness of lining of the Sacrificial Gallery.
Figure 3: Main tunnel and Sacrificial Gallery driving phases with indication of checkpoints F and I location.

Figure 4: Compared results of vertical displacements for Checkpoints F and I (upper curves for main tunnel with Sacrificial Gallery and different thickness; lower curve for main tunnel without Sacrificial Gallery). Note that X- and Y-axis correspond to step number and vertical displacement respectively.
Table 1: Ratio of vertical displacements at Checkpoints F and I. Δ is the percentage of displacement reduction.

<table>
<thead>
<tr>
<th>CheckPoint</th>
<th>Distance to Crown</th>
<th>Curve</th>
<th>Final₀/Finalₙ</th>
<th>Δ</th>
</tr>
</thead>
<tbody>
<tr>
<td>F</td>
<td>100cm</td>
<td>Initial-100A</td>
<td>1,1352</td>
<td>13,52%</td>
</tr>
<tr>
<td></td>
<td>100cm</td>
<td>Initial-100B</td>
<td>1,1280</td>
<td>12,80%</td>
</tr>
<tr>
<td></td>
<td>200cm</td>
<td>Initial-100A</td>
<td>1,1000</td>
<td>10,00%</td>
</tr>
<tr>
<td></td>
<td>200cm</td>
<td>Initial-100B</td>
<td>1,1000</td>
<td>10,00%</td>
</tr>
<tr>
<td>I</td>
<td>100cm</td>
<td>Initial-100A</td>
<td>1,4146</td>
<td>41,46%</td>
</tr>
<tr>
<td></td>
<td>100cm</td>
<td>Initial-100B</td>
<td>1,3888</td>
<td>38,88%</td>
</tr>
<tr>
<td></td>
<td>200cm</td>
<td>Initial-100A</td>
<td>1,2008</td>
<td>20,08%</td>
</tr>
<tr>
<td></td>
<td>200cm</td>
<td>Initial-100B</td>
<td>1,2028</td>
<td>20,28%</td>
</tr>
</tbody>
</table>

Figure 5: Reduction of vertical displacement at checkpoint I for values without (lower curve) and with Sacrificial Gallery (upper curves with different lining thickness 100A and 100B).
5 CONCLUSIONS

The results obtained for 100cm and 200cm distances from the centre of the Sacrificial Gallery to the main tunnel crown and for a depth of 20m, clearly show how the first value (100cm) for both considered checkpoints F and I provides better results in the reduction of vertical displacements in the crown of the main tunnel (in some cases around 40%). These best values are given by the distance decreasing between the Sacrificial Gallery (understood like a stress diverter) and the main tunnel, reducing the weight of the soil on the principal driving. This fact is very important in order to reduce the support of the main tunnel and the time of execution and, finally, the safety of the works, since it could be used for squeezing or large deformation soils protecting the main tunnel. This procedure is under International Patent.

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Experimental Investigation on Static Behaviour of Tunnel Lining Strengthened by Textile-reinforced Concrete

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Abstract

In order to tackle the applicability of TRC in strengthening tunnel lining, the eccentric compression experiment of short columns strengthened by TRC on tensile surface are conducted based on simplified experiment under mainly different strengthening parameters, specifically in eccentricity, preloading level and the ratio of textile in TRC. The experiment results show that the ultimate bearing capacity of tunnel lining strengthened by TRC is improved more significantly when the tunnel lining section is under large eccentric compression; Preloading level is detrimental on improving strengthening effect; It is useful to improve strengthening effect by increasing the ratio of textile in TRC to certain extent, but it shows decreasing trend.

Keywords: Textile-reinforced concrete; tunnel lining; strengthening; simplified experiment, short columns
1 INTRODUCTION

With the increase of investment in transportation infrastructure, the number of tunnels has been increasing in China significantly. It is reported that [1], by the end of 2011, there are 16284 tunnels mainly including road tunnels and railway tunnels in operation in China, with an overall total length of 11123km. New tunnels are continuously being built, specifically more than 2500 tunnels, 4600km in total length. Those tunnels shown up after 1990s are commonly constructed by the New Austrian Tunnelling Method which has been proved to be a very economic and flexible mode of construction. As a result, almost all of those are lined tunnels. However, with the increase of number of tunnels and operation time, in recent years, more and more deterioration of lining has been found in many tunnels. Such deterioration often results in loss of bearing capacity, significant reduction in stiffness, excessive cracking, and other problems. Consequently, lining retrofitting becomes increasingly important to make the lining structure serviceably and durably, that is to say, to guarantee the normal use.

Textile reinforced concrete (TRC) is a new developing composite material where multi-axial fabrics are used in combination with fine grained concrete. It has gained increasing popularity due to the favourable properties, namely corrosion resistance, anti-high temperature, better ability in anti-crack, ease and speed of application and minimal change in geometry. In the past years, research about reinforced concrete structures strengthened by TRC had been conducted extensively, in summary, mainly focused on the following three aspects: improvement bonding effect between fibre and fine grained concrete [2-4], accurate calculation model of TRC [5-8] and flexural and shear strengthening of reinforced concrete beams and plates [9-14]. Besides, there are some scholars beginning to explorer the long-term performance of TRC [15-18]. The aim of this article is to investigate the applicability of TRC in strengthening tunnel lining, and the related research hasn’t been reported. As we know, there are obvious differences between tunnel lining and reinforced concrete beams or plates, both in appearance and force condition. Consequently, it is necessary to conduct experimental research considering the tunnel lining load conditions.
2 EXPERIMENTAL PROGRAM

2.1 Simplified Experiment Design

Full-scale experiment is undeniably the best method in studying the static behaviour of tunnel lining strengthened by Textile-reinforced concrete. However such experiment, in reality, is far too costly and difficult to execute. Therefore, it needs to make simplification.

Tunnel lining is commonly simplified as a short reinforced concrete column under axial load and torque [19]. The research about tunnel lining strengthened by CFRP based on the short reinforced concrete column has been reported recently [20-21]. The short column is defined as [22]:

\[ \frac{L_0}{h} \leq 5 \]  

(1)

Where \( L_0 \), \( h \) are the heights of short column and cross-section respectively.

Figure 1: Reinforcement configuration of a tunnel in Hangzhou, China

The prototype structure of experiment is based on the typical tunnel lining in Hangzhou, China, whose reinforcement configuration is presented in Figure 1. It is strictly requirement for the load equipment if the experiment is conducted based on prototype short column (45cm×36cm×225cm). Consequently, considering various factors, mainly the limitation of load equipment, it decides to carry out experiment on reduced scale specimen.
In order to properly simulate the interfaces between different materials, such as concrete and strengthen layer, the steel and concrete, prototype materials are chose for reduced scale specimens. So, similarity scale of gravity is 1 which is not satisfied with \( C_p = 1/ C_L \) deduced according to similar theory, where \( C_p \) , \( C_L \) are gravity and geometric similarity scale respectively. However, for the general small span structure, the effect of structural weight on structure stress and deformation can be ignored [23]. Therefore, it is feasible to carry out experiment of tunnel lining strengthened by textile-reinforced concrete by using prototype materials.

2.2 Test Specimens

Due to the limitation of equipment, the size similarity scale is 3 and \( L_0 / h = 4 \). Consequently, the size of the short column is decided according to similar theory, as shown in Figure 2. In order to apply the eccentric load conveniently, the columns are fabricated with considering corbel shape.

![Figure 2: Geometry of column specimens](image)

**Figure 2:** Geometry of column specimens
Table 1: Parameters detail of specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Preload</th>
<th>Number of layer of textile</th>
<th>Eccentricity</th>
</tr>
</thead>
<tbody>
<tr>
<td>S-0-0-75</td>
<td>0</td>
<td>-</td>
<td>75mm</td>
</tr>
<tr>
<td>S-0-1-75</td>
<td>0</td>
<td>1</td>
<td>75mm</td>
</tr>
<tr>
<td>S-0-0-100</td>
<td>0</td>
<td>1</td>
<td>100mm</td>
</tr>
<tr>
<td>S-0-1-100</td>
<td>0</td>
<td>1</td>
<td>100mm</td>
</tr>
<tr>
<td>S-0-2-100</td>
<td>0</td>
<td>2</td>
<td>100mm</td>
</tr>
<tr>
<td>S-20-1-100</td>
<td>20%</td>
<td>1</td>
<td>100mm</td>
</tr>
<tr>
<td>S-40-1-100</td>
<td>40%</td>
<td>1</td>
<td>100mm</td>
</tr>
</tbody>
</table>

The experimental work includes a total of 7 square section columns, and can be divided into three sets with different eccentricity, preloading history and ratio of textile in TRC. The test program and specimen properties are summarized in Table 1. In the specimen notation (e.g. S-20-1-100); “S” is abbreviation of ‘Sample’. “20” refers to preloading level of the specimen which means 20% of ultimate bearing capacity from control specimen. “1” represents the number of textile layer. “100” indicates the eccentricity.

2.3 Material Properties

All specimens are cast from one batch of concrete with a 28-day compressive strength of $f_c = 31.3 MPa$ and reinforced with 4 $\phi 12\text{mm}$ longitudinal ribbed bars ($f_y = 341 MPa$) symmetrically placed. Transverse reinforcement is provided with rectangular ties $\phi 6@125\text{mm}$ made of smooth bars ($f_y = 219 MPa$). The clear concrete cover for the ties is 15 mm. The reinforcement details for the specimens are shown in Figure 2. The heavily reinforced brackets are designed at both ends of the column specimens to prevent any local failures of the end zones.

The TRC consists of a textile reinforcement inserted into a fine-grained concrete matrix. The heavy-tow-yarns of the fabric are composed with carbon fibre in 90° and glass fibre in 0° and aligned with an interval of 10mm. The material properties of different directions are presented in Table 2.
The fine-grained concrete is a convenience blend obtained from [24] with a maximum grain-size of 1.2mm. The average compressive strength of the fine-grained concrete can be determined at 75MPa and the average bending tensile strength at 4.3MPa when testing prisms with size in 160mm×40mm×40mm.

2.4 Specimen Preparation

The concrete is cast in a special steel formwork. The specimens are compacted and vibrated using hand-held mechanical vibrator. After curing in a humid condition for 28 days, the specimens is prepared for strengthening. The strengthening layer is installed as follows: Fist, the strengthen surface of column is sandblasted to remove the loose cement layer, then cleaned by water and dried out by air dryer; Second, the fine grained concrete prepared previously is then troweled onto the sandblasted surface with a 3~5 mm thick layer; Third, the textile layer is firmly hand pressed into the wet mortar to ensure its adequate embedding; Finally, another fine grained concrete layer covers the textile layer completely with the same thickness. If there are two or more textile layers being need to apply, the operation repeats until all textile layers are applied and covered by mortar.

Strengthening layer is applied on the full length of the specimens. Preload is applied if there is load history designed for the short column. All short columns including the strengthening layer are constructed using almost the same experimental conditions in order to compare their experimental results.
2.5 Test Setup and Loading

Figure 3: Test setup and instrumentation

A hydraulic actuator is used to apply the axial load to the column. The top of the specimen is attached to the actuator, while the bottom is supported on the steel reaction frame. Both supports are designed as hinged connections with predefined eccentricity.

A total of five linear variable displacement transducers (LVDTs) and 17 strain gauges are used for every specimen (including 8 steel strain gauges and 9 concrete strain gauges). Figure 3 shows the layout of LVDTs and strain gauges. The specimens are loaded using a 500kN capacity compression actuator under load control at a rate of 0.5kN/s in the initial stage of the test. However, when the load is approaching to limit bearing capacity, the loading method turns to displacement control at a rate of 0.2mm/s for safe. The load, longitudinal displacement and strains are monitored and filed using the data acquisition system automatically at a fixed time interval while the lateral deflections are measured manually at each load increment. The tests are performed up to failure when the concrete crushes on the compression region.
3 EXPERIMENTAL RESULTS AND DISCUSSION

3.1 Failure Mode

All of column specimens initially behave in a nearly similar manner, including strengthened and control specimens. First cracks generally appear at almost close to mid-height on the tension region of columns. As the load increases, it is seen that the existing cracks propagate and new flexural cracks appear before reaching the ultimate load. These tensile cracks occur along the column length. Cover spalling starts at the compression region of the section, and then a sudden loss of strength is measured. Photographs of all strengthened columns tested in this study are shown in Figure 4. The modes of failure exhibit that most of the tested columns crush near the mid-height of the specimen indicating a typical failure of the column in compression. During the experiment, there is no interface failure being found between strengthen layer and concrete. That is different from the research result reported previously. With the increase of textile ration in TRC, the number of main cracks is increasing, however, the length, width and intervals of main cracks for strengthened columns increasingly decrease compared with the control column at the same load level.

3.2 Load-deflection Behaviour

3.2.1 Eccentricity influence on strengthening effect

The experimental load-deflection diagrams strengthened with one textile layer for different eccentricities are presented in Figure 5. It shows that no significant lateral deformations are measured for both eccentricities at 100mm and 75mm. But the
lateral displacements are larger for short column of 100mm eccentricity comparatively due to the eccentricity effect. The diagram indicates that short columns strengthened by textile-reinforced concrete in longitudinal side surface far away eccentricity can improve the bearing capacity and control deformability significantly. However, significant distinction of strengthening effect has been obtained for different eccentricities, specifically the ultimate bearing capacity increases 20.39% for short column with 100mm eccentricity while it just is 9.04% for 75mm eccentricity. It is concluded that eccentricity significantly affects the short column ultimate load capacity and deformation strengthened by TRC in longitudinal side surface far away eccentricity, and the TRC is more suitably used to strengthen tunnel lining when the section is under large eccentric compression.

Figure 5: Load-deformation curve under different eccentricity

3.2.2 Textile layers influence on strengthening effect

The experimental load-deflection diagrams with 100mm eccentricity strengthened with different textile ratios in TRC are shown in Figure 6. The diagram indicates that the ultimate bearing capacity improves and ductility reduces with the increase of textile ratio, and both show a decrease trend. In this study, for short columns strengthened by one textile layer and two textile layers, the ultimate bearing capacity increase 20.39% and 29.21% respectively.
3.2.3 Preload level influence on strengthening effect

The typical load-deformation curves of different preloading level are illustrated in Figure 7. The ultimate load capacity of column specimens is significantly influenced by preloading level. Columns show higher ultimate bearing capacity under lower preload. For columns with 100mm eccentricity strengthened by one textile layer under different preloading level, with 0%, 20% and 40% of limit capacity of control specimen, the ultimate bearing capacity increases 20.39%, 16.14% and 13.23% respectively.
3.3 Lateral Deformation

The classical lateral displacements of short columns strengthened by TRC, such as “S-0-1-100” and “S-0-2-100”, measured by the five lateral LVDTS are presented in Figure 8. The lateral deformation is induced under a combination of initial moment and second order moment. For design calculations of ordinary pinned RC columns under second order deformation, a sine-shaped deformation is often assumed [25]:

\[ y = u_m \sin \left( \frac{\pi z}{L} \right) \]

Where \( u_m \) maximum lateral displacement at the midheight section; \( z \) refers to longitudinal coordinate variable; \( l \) represents length of the column; and \( y \) represents lateral displacement at the \( z \) position. The Figure demonstrates that this model is a good prediction for a different level of load from initial stage to the failure stage, as presented in Figure 8. Thus the sine-shaped model can be used for columns strengthened by Textile-reinforced concrete in longitudinal side face.
Figure 8: Classical Deformation curve of strengthened columns
3.4 Strain Distribution of Concrete Section

![Graph showing strain distribution of concrete section](image)

Figure 9: Classical strain distribution of concrete section at midheight of strengthened parameters

The stain distribution at mid-height of the short columns strengthened by TRC shows good agreement with linear relation, as demonstrated in Figure 9. Consequently, the plane section assumption can be used to analyse the whole loading process of tunnel lining strengthened by textile-reinforce concrete in the next step.
4 CONCLUSIONS

(1) Eccentricity has significant influence on the strengthening effect. Furthermore, TRC is more suitably used to strengthen tunnel lining when the section is under large eccentric compression.

(2) For the large eccentric columns, ultimate bearing capacity increases while the ductility reduces with the increase of textile ratio in TRC, but the strengthening efficiency shows decreasing trend.

(3) Preloading level has significantly detrimental effect on the strengthening effect, the larger of preload, and the worse of strengthening effect.

REFERENCES


Modeling of Tunneling in Squeezing Ground Conditions
A Closed-Form Solution for the Ground Response Curve of Circular Tunnels Considering Large Deformations

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Abstract

The analysis of the ground response to tunnel excavation is described usually in terms of the characteristic line of the ground, which relates the support pressure with the cavity wall displacement. Squeezing conditions may lead to large convergences, sometimes greater than 10-20% of the tunnel radius, while the majority of the existing formulations for the Ground Response Curve (GRC) are based on the theory of infinitesimal deformations. This paper presents a large strain analytical solution for the GRC considering a linearly elastic-perfectly plastic material that obeys the Mohr-Coulomb failure criterion with a non-associated flow rule. The case of out-of-plane plastic flow is included, taking place when the longitudinal stress is no longer the intermediate principal one. Comparisons with the classic small strain solution as well as with an approximate large strain solution which neglects elastic deformations in the plastic zone are presented, demonstrating moreover the influence of plastic dilatancy. The derived relations are found in perfect agreement with finite element results; hence, apart from their usefulness in convergence assessments when extreme squeezing conditions are expected, they can provide also valuable benchmark for numerical procedures which take into account large deformations.

Keywords:  Closed-form solution, dilatancy, ground response curve, large strain, squeezing
1 INTRODUCTION

Large convergences are encountered often in underground projects, which combine high overburden with poor ground properties. Various reports can be found in the literature dealing with tunnel cases under heavily squeezing conditions, e.g. the Gotthard base tunnel in Switzerland \[3\]. A widely used method for the estimation of the ground behaviour during tunnel excavation considers the characteristic line of the ground, which relates the support pressure with the wall radial displacement. A circular tunnel cross-section under axisymmetric plane strain conditions is usually used as the static system, corresponding reasonably well to the situation that prevails in deep tunnels far behind the face.

The majority of the existing analytical solutions concerning the GRC are based on the small deformation theory taking into consideration several elastoplastic constitutive models with different post-failure behaviour. The most common of them for rock-like materials are the elastic-perfectly plastic, the elastic-brittle plastic and the strain softening one. Furthermore, some viscous models have been developed in order to account for time-dependent effects, which may be significant in case of squeezing rocks with considerable rheological behaviour, e.g. creep.

On the contrary, few attempts have been made for the derivation of the GRC accounting for finite deformations. Papanastasiou and Durban \[4\] studied both problems of expanding and contracting cylindrical cavities in an infinite isotropic medium using the Mohr-Coulomb (MC) as well as the Drucker-Prager hardening solid, resulting in differential equations which must be solved numerically. Their results found to be in good agreement with experimental ones. Later, Yu and Rowe \[7\] presented an analytical solution to the problem of cavity wall unloading (for cylindrical or spherical cavities) using the linearly elastic-perfectly plastic MC model. However, in order to express the cavity contraction curve in closed form, they ignored elastic deformations within the plastic zone. Vrakas and Anagnostou \[6\] extended recently this study, presenting an accurate solution to the problem without neglecting elastic deformations in the plastic region, considering the more general elastic-brittle plastic case as well as examining the influence of the out-of-plane stress (for the two-dimensional case). It is shown that when this stress is no longer the intermediate principal one, an inner second plastic ring is formed, where an edge flow takes place, similarly to the small strain case presented by Reed \[5\].
The presented paper is based upon the extended report [6]. It outlines the derivation of the large strain GRC, summarizes the equations for the GRC, shows the validity limit of the small strain assumption and compares the closed-form solution with numerical results. It shall be noted here, that this study focuses on fully drained cases without seepage flow or dry grounds, where the influence of the water is negligible.

2 LARGE STRAIN CLOSED-FORM SOLUTION FOR THE GRC

The classic problem of a cylindrical cavity with radius \( a_0 \), unloaded from the in situ state of stresses in an infinite medium is examined (Fig. 1). An isotropic initial stress field expressed by the stress \( \sigma_0 \) is considered, while the gravitational forces are neglected; therefore, the problem becomes one-dimensional with respect to the radial direction. During unloading, the radial, \( \sigma_r \), and the tangential stress, \( \sigma_t \), constitute the minor and the major principal stress, respectively, due to the convention of positive compression that is used. It shall be noted, that the considered stresses correspond to the Cauchy ones (i.e., force per current unit area), while the appropriate logarithmic definition is adopted for the strains.

As the internal support pressure, \( \sigma_a \), is reduced successively, the ground behaviour around the opening is initially purely elastic. The radial stresses decrease while the tangential ones increase until the MC criterion is satisfied at the tunnel wall. This takes place when the critical value \( \sigma_{\rho_1} \) is reached. Then, the material starts to become plastified forming a plastic zone of radius \( \rho_1 \) around the cavity (Fig. 1). The higher order terms of the Hencky strains during elastic response are neglected, otherwise a closed-form analytical solution could not be obtained, except for an incompressible material. This approximation leads to the classic small strain relations for the elastic domain with respect to the Eulerian (or spatial) radial coordinate \( r \).

Accounting for the equilibrium equation on the deformed configuration, the MC failure criterion, the boundary condition at the cavity wall and the continuity of stresses at the elastoplastic interface, the stresses inside the plastic ring can be expressed in terms of the ratio \( r/a \), where \( a \) denotes the current tunnel radius. In contrast to the small strain formulation [5], the determination of the displacement field is a prerequisite here for the estimation of the stress field. Considering moreover the plastic flow rule and the continuity of radial displacements at the elastoplastic boundary, the cavity contraction curve can be calculated after some mathematical treatment. The axial stress is obtained by the corresponding elastic constitutive relation \( (\varepsilon_z = 0) \).
As the support pressure $\sigma_a$ is further reduced to the value $\sigma_{\rho_2}$ (ensuring that $\sigma_{\rho_2}$ is positive, which is true under certain conditions concerning the problem parameters, see Eq. 3), a second inner plastic ring of radius $\rho_2$ begins to form, where the longitudinal stress, $\sigma_z$, remains equal to the tangential one. Inside this region, an out-of-plane plastic flow takes place. The radial as well as the tangential stresses are still given by the same relationships in both zones as in the previous case, whereas the continuity of displacements at radii $\rho_1$ and $\rho_2$ in combination with the appropriate flow rule in each zone must be considered for the estimation of the displacement field. The necessary relations for the construction of the GRC accounting for large strains are given below. The general expressions as well as the complete mathematical process for their derivation can be found in [6]. The corresponding small strain solution of the problem is presented in [5]. Hence, regarding that the elastic material properties are expressed by the Young’s modulus, $E$, and the Poisson’s ratio, $\nu$, while the plastic ones are given by the cohesion, $c$, the friction angle, $\varphi$, and the dilation angle, $\psi$, the tunnel wall displacement (positive inwards), $u_o$, can be calculated:

$$
\frac{u_a}{a_o} = \begin{cases}
\left[1 + \frac{E}{(1+\nu)(\sigma_a - \sigma_a)}\right]^{-1} & , \sigma_a \geq \sigma_{\rho_1} \\
1 - \left[T_{\rho_i} + (\delta/\Omega_{\rho_i}) \times f_i (1, R_i)\right]^{-1/\kappa+1} & , \begin{cases}
\sigma_{\rho_2} \leq \sigma_a < \sigma_{\rho_1} (i = 1) \\
\sigma_a < \sigma_{\rho_2} (i = 2)
\end{cases}
\end{cases}
$$

where $m = \frac{1 + \sin \varphi}{1 - \sin \varphi}$, $\sigma_D = \frac{2c \cos \varphi}{1 - \sin \varphi}$, $\kappa = \frac{1 + \sin \psi}{1 - \sin \psi}$, $\delta = \frac{\kappa + 1}{m-1}$, $\sigma_{\rho_1} = \frac{2\sigma_o - \sigma_D}{m+1}$, $\sigma_{\rho_2} = \frac{(1-2\nu)\sigma_o - (1-\nu)\sigma_D}{m-\nu(m+1)}$, $\bar{x} = x + \frac{\sigma_D}{m-1}$ (abbr.), $R_i = \frac{\bar{x}_{\rho_i}}{\bar{x}_a}$, $\omega_{11} = \frac{1+\nu}{E} \left[1 - \nu(\kappa + 1)\right]$, $\omega_{12} = \frac{1}{E} \left(1 - 2\nu \kappa\right)$, $\omega_{21} = \frac{1+\nu}{E} \left[\kappa(1-\nu) - \nu\right]$, $\omega_{22} = \frac{2}{E} \left[\kappa(1-\nu) - \nu\right]$, $\Omega_{\rho_i} = \exp\left[\left(\omega_{1i} + \omega_{2i}\right)\bar{\sigma}_o\right]$, $\Omega_i = \left(\omega_{1i} + m\omega_{2i}\right)\bar{\sigma}_a$.
\[ T_{o1} = \left[ 1 + \frac{\sigma_o - \sigma_{p1}}{E/(1 + v)} \right]^{x+1} R_1^\delta, \quad T_{o2} = T_{o1} + \frac{\delta}{\Omega_{o1}} f_1 \left( R_2, R_1 \right), \quad (8) \]

\[ f_i \left( x, y \right) = \sum_{n=0}^{\infty} \frac{\Omega_i^n}{n!(n+\delta)} \left( x^{n+\delta} - y^{n+\delta} \right). \quad (9) \]

By neglecting elastic deformations in the plastic zone(s), the solution is simplified to a large degree, given by a single relation during elastoplastic response [7]:

\[ u_a = \begin{cases} \left[ 1 + \frac{E/(1 + v)}{\sigma_o - \sigma_a} \right]^{-1}, & \sigma_a \geq \sigma_{p1} \\ 1 - \left[ 1 - R_i^\delta + T_{o1} \right]^{\frac{1}{x+1}}, & \sigma_a < \sigma_{p1} \end{cases} \quad (10) \]

Figure 1: Computational model for a deep circular tunnel, with the developed plastic zones.

### 3 ANALYTICAL AND NUMERICAL RESULTS

An application of the aforementioned relations is presented here, in conjunction with some numerical results obtained with the Abaqus software [1]. The considered finite element model is the widely used axisymmetric strip with the proper vertical displacement restraints. The computational domain is discretized by using

[263]
quadrilaterals elements (CAX4), while one infinite element (CINAX4) is incorporated in order to simulate the unbounded far field (Fig. 2). The classic MC model describes the material behaviour, having a pyramidal failure surface and plastic potential in the principal stress space as well as satisfying the Koiter’s rule [2] in case of a singularity stress state, i.e. the incremental plastic strain is given by the sum of the components of the two flow rules.

The computational examples consider the rock properties proposed by Kovári et al. [3] for the Sedrun section of the Gotthard base tunnel in Switzerland, based on experimental investigations of the ETH Zurich: $E = 2000$ MPa, $c = 0.25$ MPa and $\phi = 23^\circ$. The Poisson’s ratio $\nu$ is taken equal to 0.25. The overburden depth is approximately 900 m, which corresponds to an initial stress $\sigma_o = 22.5$ MPa.

Figure 2: Axisymmetric finite element model for the calculation of the GRC.

Figure 3 presents the GRCs for a non-dilatant ($\psi = 0^\circ$) and a dilatant material ($\psi = 10^\circ$ and $\psi = 20^\circ$, respectively). The finite element results fit almost perfectly the analytical ones for both formulations, while the approximate large strain solution that ignores elastic deformations in the plastic zone underestimates the tunnel wall displacements. It can be seen in these graphs that the error of neglecting the elastic deformations in the plastic zone is bigger at the lower dilation angle range. The error of the classic small strain solution is small up to convergence ratios $u/a_o$ of 10%, a limit value which in the present example still corresponds to significant support pressures. One recognizes, furthermore, that the small strain solution may lead to irrational results, providing convergences greater than the initial tunnel radius (Fig. 3b-c). Finally, it can be confirmed, that the dilatancy favours the developed displacements to a great extent due to the unrestrained volumetric increase of the assumed constitutive model.
Figure 3: Ground response curves for a dilation angle: (a) $\psi = 0^\circ$, (b) $\psi = 10^\circ$ and (c) $\psi = 20^\circ$. 
4 CONCLUSION

A closed-form solution for the GRC was presented considering finite strains. The proposed analytical expressions can be used for convergence assessments in tunnelling under extreme squeezing conditions, while they constitute also trustworthy benchmark for numerical procedures which account for geometric nonlinearities.

REFERENCES


Evaluation of a Modified Hardening Model for Squeezing Rocks in Tunneling

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Abstract

The relationship between the displacement of the excavation boundary and the rock pressure is important for tunnel design particularly under so-called squeezing conditions. In the case of rotational symmetry, it can be expressed by the “Ground Response Curve” (GRC). The GRC is usually determined by assuming that the ground behaves as a linearly elastic, perfectly plastic material obeying the Mohr-Coulomb yield criterion (“MC” model). This model cannot map the non-linear stress-strain behavior and the radial stress dependency of the stiffness modulus observed in triaxial tests on kakiritic rocks (a typical squeezing rock from the Alps). In order to investigate the influence of those factors on the GRC, we adopt an alternative model, which is based on the well-known Hardening Soil model but, due to some modifications, needs only common triaxial tests to identify its parameters. This modified Hardening Soil model (“MHS” model) maps the results of triaxial tests (particularly, the observed dependency of the behavior on the stress level) better than the MC model. Comparative ground response analyses show that the MHS model leads to a steeper GRC than the MC model. As there is no need to make a more or less arbitrary assumption concerning the Young’s modulus, the predictions of the MHS model exhibit a smaller scatter than the MC model.

Keywords: Modified hardening soil model, ground response curve, squeezing ground
1 INTRODUCTION

The relationship between rock pressure and rock deformation is important for tunnel design particularly under so-called squeezing conditions [1]. The quantification of this relationship presents uncertainties which are also associated with the constitutive behavior of the ground. The usually adopted constitutive model for squeezing ground is the linearly elastic and perfectly plastic material obeying the Mohr-Coulomb yield criterion. However, this model (hereafter referred to as “MC model”) cannot map some aspects of the behavior observed in triaxial tests [2]. More specifically, kakiritic rocks (a typical squeezing rock from the Alps) exhibit a non-linear stress-strain behavior and a stress dependent modulus [3]. An alternative model, the so-called “Deviatoric Hardening” or “DH” model, has been considered recently [4, 5] in order to take into account the non-linear stress-strain relationship. This model leads to similar results as the MC model provided that the Young’s modulus of the latter is taken equal to the secant modulus [4, 5]. However, the dependency of the stiffness on the radial stress was not taken into account by the DH model. Applying the modulus determined at a specific stress level to other stress levels may lead to inaccurate predictions.

The well-known Hardening Soil model (“HS model”) of the PLAXIS finite element code [6] can map the non-linear stress-strain behavior under triaxial conditions as well as the observed stress dependency of the stiffness. By performing some slight modifications of the HS model, which do not disable its basic features, it is possible to take into account the effect of the intermediate stress, to simplify the numerical implementation and to determine its parameters by using common triaxial tests only. The capability of this modified Hardening Soil model (“MHS model”) under triaxial drained shear conditions and its advantages relatively to MC model is discussed in [3]. Here, we investigate whether the MHS model, which maps better the behavior in triaxial tests, leads to significantly different results than the MC model. Section 2 outlines briefly the MHS model, while Section 3 presents a comparative analysis.

2 MODIFIED HARDENING SOIL MODEL

The HS model is formulated within the framework of elasto-plasticity, i.e. the axial strain $\varepsilon_1$ is divided into an elastic part and a plastic part. The overall response of the HS model during primary loading in drained triaxial tests fulfills Duncan and Chang’s hyperbolic relationship [7]. The elastic part of the axial strain depends linearly on the deviatoric stress $q$. The elastic unloading-reloading modulus $E_{ur}$ depends in general
on the minimum principal stress $\sigma_3$, according to a power law [3]. An additional model parameter, the modulus $E_{50}$, denotes the modulus in primary loading when $q$ is half of the deviatoric stress at failure $q_f$. The modulus $E_{50}$ depends on stress according to the same power law as $E_{ur}$.

The MHS model is different from the HS model with respect to, (i), the dependency of the yield surface on the Lode angle (the yielding function is based on Benz’s formulation using Matsuoka-Nakai criterion [8]); (ii), the dilatancy law (instead of Rowe’s dilatancy law [9], which greatly overestimates the contractant behavior at low deviatoric stresses [8], the dilatancy law proposed by Soreide [10] is adopted); and, (iii), to the cap hardening part, which is not taken into account in the present model. The detailed description of the MHS model can be found in [3].

Table 1: Parameter values

<table>
<thead>
<tr>
<th>Parameter</th>
<th>MC model Set 1</th>
<th>MC model Set 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E$ [MPa]</td>
<td>1800</td>
<td>1000</td>
</tr>
<tr>
<td>Other parameters: $\nu = 0.3$, $c = 0.569$ MPa, $\varphi = 30^\circ$, $\psi = 6.4^\circ$</td>
<td></td>
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<table>
<thead>
<tr>
<th>Parameter</th>
<th>MHS model Set 1</th>
<th>MHS model Set 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>$m$</td>
<td>0.91</td>
<td>1.88</td>
</tr>
<tr>
<td>Other parameters: $E_{ur, ref} = 1800$ MPa, $E_{50, ref} = 1152$ MPa, $p_{ref} = 5$ MPa, $\nu = 0.3$, $c_f = 0.569$ MPa, $\varphi_f = 30^\circ$, $\psi_f = 6.4^\circ$, $R_f = 0.9$</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The MHS model has a total of nine parameters. Similar to the MC model, every parameter has a clear physical meaning and can be determined via conventional triaxial tests. In the present case, we determined the model parameters by considering the example of a typical kakirite sample taken from the Sedrun section of Gotthard Base Tunnel [2]. The sample was subjected to a multistage consolidated drained test (CD test) under radial pressures of 2, 5 and 9 MPa. The moduli were determined under the radial pressure of 5 MPa. The detailed process to determine the model parameters for the MC and MHS model can be found in [3].

Table 1 summarizes the parameter values of the MC model and of the MHS model. The two parameter sets of the MC model are different with respect to the Young’s modulus. Parameter set 1 is based on the unloading-reloading modulus, while set 2 considers the secant modulus.
The two parameter sets considered for the MHS model are different only with respect to the parameter $m$ of the power law expressing the stress dependency of the moduli, which is equal to about 0.91 or 1.88 depending on the considered modulus ($E_{ur}$ or $E_{50}$, see [3]).

3 GROUND RESPONSE CURVE

One important feature of the MHS model is that it accounts for the dependency of the deformation moduli on the radial stress. It better maps the behavior observed in triaxial tests with varying stress levels than the MC model [3]. The effect of this stress dependency on the ground response to tunnel excavation will be investigated in this section by means of a computational example.

Under the simplifying assumptions of plane strain conditions and rotational symmetry, which are true for a deep cylindrical tunnel crossing homogeneous and isotropic ground with a uniform and a hydrostatic initial stress field, the relationship between the rock deformation and the rock pressure can be expressed by a single curve, the “Ground Response Curve” (GRC) [11].

Fig. 1 shows the GRCs for the two constitutive models and the parameter sets of Table 1. The initial stress was taken equal to 7.5 MPa, which corresponds to an overburden of about 300 m. The GRCs of the MC model were calculated using known closed-form solutions [12]. The equations for the MHS model were solved numerically by the finite element code ABAQUS, in which we implemented a UMAT subroutine with the MHS model. The far field boundary of the numerical solution domain was taken equal to a distance of 20 tunnel radiuses from the tunnel center.

The MC parameter set 1, which is based on the unloading-reloading modulus, lead to considerably smaller convergences than set 2, which considers the secant modulus. The reason is that the unloading-reloading modulus of set 1 underestimates the strain before failure [3]. For the MHS model, the convergences lie between the upper and lower values of the MC model. The prediction range is narrower than the MC model (Fig. 1a), which presents the problem of selecting an adequate Young’s modulus.

As mentioned in Section 2, the moduli for both models were determined at the radial stress $\sigma_3 = 5$ MPa. The moduli of the MHS model decrease with decreasing minimum principal stress, i.e. in the problem under consideration with decreasing radial stress.
The stress reduction is significant especially in the vicinity of the tunnel wall, where $\sigma_a$ decreases down to zero. This is why the GRC of the MHS model becomes steeper than the one of the MC model in the low stress range (where $\sigma_a \leq 5 \text{ MPa}$).

**Figure 1:** (a) GRC for MC and MHS models. (b) GRC for MC and MHS models in the high stress range (parameters according to Table 1)
Figure 2: Distribution, (a), of the tangential stress along the radial direction and, (b), of the tangential strain (normalized by the tangential strain at the tunnel wall) (parameters according to Table 1)

In the high stress range (where $\sigma_a \geq 5$ MPa, see Fig. 1b), the opposite happens. The MHS model predicts smaller convergences than the MC model because its moduli are higher than the constant modulus of the MC model. The higher the $m$ value, the more pronounced the stress dependency and the smaller the convergences will be.

One fundamental feature of the MHS model is that plastic deformations develop right from the start of shearing and the stress field fulfills the yield condition everywhere around the tunnel immediately after unloading. A plastic zone does not exist in the
sense of the MC model. In the MC model, plastic deformations occur only after the stress state reaches the yield condition and this happens within a zone of limited extent around the tunnel (Fig. 2a). Within this zone, the tangential stress $\sigma_t$ decreases towards the tunnel. The latter happens also with the MHS model, but in a larger zone around the tunnel (Fig 2a). In addition, the peak of the tangential stress $\sigma_t$ is less pronounced than in the MC model (Fig 2a). In spite of this difference, both models predict similar strain distributions along the radial direction. More specifically, the major portion of the strain occurs in the MHS model within a zone coinciding with the plastic zone of the MC model (Fig. 2b).

4 CONCLUSIONS

The dependency of the stiffness on the minimum principal stress influences the ground response to tunnel excavation particularly in the case of deep tunnels, where the minimum principal stress considerably decreases in the vicinity of the opening. The MHS model leads to smaller convergences in the high stress range (i.e. the elastic zone of the MC model). In the vicinity of the tunnel wall, the convergences increase faster than the ones of the MC model due to the decreasing moduli. Furthermore, the MHS model eliminates the need for arbitrary assumptions concerning the Young’s modulus.

ACKNOWLEDGEMENTS

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Time Dependent Numerical Analysis for Investigation of Entrapment Risks in DS-TBM Tunnelling in Squeezing Grounds

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Abstract

Jamming of shielded TBMs can occur in squeezing ground due to excessive convergence of the ground in weaker rocks under high in-situ stress. This typically coincides with machine downtimes includes weekends, stoppages for machine repair or maintenance, when the ground has sufficient time to reach its maximum displacement. In this study, to evaluate of impact of time factor on the possibility of machine seizure, a comprehensive 3D time dependent finite difference simulation of a double shield TBM (DS-TBM) in squeezing ground was performed. The modelling allowed for observation of the impact of tunnel advance rate on the possibility of machine jamming in the squeezing grounds. For this purpose, time-dependence 3D model included material properties of rock mass related to creep in severe squeezing conditions. This paper offers overviews of the results of the modelling include longitudinal displacement and sectional stresses on shields and on segmental lining versus time in the different advancement rates.

Keywords: Time dependent behaviour, Double shield TBM, Squeezing ground, 3D numerical simulation, TBM entrapment
1. INTRODUCTION

The tunnel excavation by a shielded TBM is a continuous process, relative to the gradual movement of the ground, unless a major delay in the operation is experienced. Entrapment of shielded TBMs can occur in the squeezing ground due to excessive convergence of the walls during extended machine downtimes, including weekends, stoppages for machine repair or maintenance, or other operational issues. This shows that the "time" factor plays an important role should be considered when evaluating the stability of the underground opening and designing its support system because considerable amount of deformation and contact pressure may develop with time[6].

There are several cases where shielded TBM was entrapped when there was a slowdown in operation or standstill in the TBM drive. For example, the Nuovo Canale Val Viola (Italy, double shielded TBM, D=3.60 m)[5], the Ghomroud Tunnel (Iran, double shielded TBM, D = 4.50 m) [3] and the Yindaruqin Irrigation Project (China, double shielded TBM, D=5.54 m)[5] are some cases that the TBM became trapped because of squeezing ground during a one-week holiday stop or during a maintenance stop. This suggests that maintaining a high daily advance rate and reducing downtimes may have a positive effect in avoiding entrapment.

As stated by the International Society of Rock Mechanics (ISRM), squeezing rock is the time dependent large deformation related to the progressive yielding, which occurs around the tunnel and is essentially associated with creep, caused by exceeding a threshold shear stress. Deformation may terminate during construction or continue over a long period of time [2].

In this study, to evaluate of impact of time factor on possibility of DS-TBM seizure in long deep tunnels in potentially squeezing ground, a comprehensive 3D modelling was developed. Time dependent constitutive model including a Burger-creep visco-plastic model (CVISC) was applied in the numerical models for describing the time dependent response of the tunnel walls associated with severely squeezing conditions. The model estimates tunnel convergence during excavation and predicts the magnitude of loads on the shield and segmental lining in squeezing conditions in the different advancement rates, allowing estimating the frictional forces between the rock and shield versus time and thus the required machine thrust to move the machine forward.
2. THREE DIMENSIONAL NUMERICAL MODELLING

2.1. Assumptions and Considerations for 3D Modelling

To analyse the stress-strain behaviour of rock mass in a tunnel excavation with a double shield TBM, a comprehensive 3D model has been developed in FLAC$^{3D}$ so that all properties of the main DS-TBM components can be used as variables at each step of analyses. The 3D block model and relevant dimensions were selected and implemented, as shown in Figure 1a. The inner diameter of tunnel is specified to be 11 m refer to studies from the Lyon-Turin Base Tunnel [1]. In this study, the presence of water pressure and consolidation problems is not taken into account in the numerical modelling. Moreover, the in situ state of stress is assumed to vary linearly with depth and it represents the conditions along the Base Tunnel at depth of nearly 600 m. The ratio between the horizontal and vertical stress components ($\sigma_h/\sigma_v$) in the rock mass is assumed to be $K=1$. The rock mass parameters are shown in Table 1. The rock mass is assumed to follow a linear elastic and perfectly plastic behaviour according to Mohr-Coulomb failure criterion. Figure 1b shows the schematic modified view of the assumed DS-TBM arrangement in the case of squeezing rock. Additionally, discretization of numerical model of DS-TBM is shown in the discrete numerical model in Figure 2. Table 1 shows the main features of the DS-TBM to be considered in the modelling. The shield, segmental lining, and annular gap backfill were considered to behave as linear elastic material, with pertinent properties listed in Table 2. Excavation stages were exactly simulated with respect to a tunnel excavation by a DS-TBM. In modelling of the shield skin, the total weight of the TBM was applied by normal stress to the 45° invert area of shield which was in contact with tunnel invert. Thrust force of cutter on the cutter-head is applied as normal stresses to the excavation face.

![Figure 1: a) Geometric dimensions of the numerical model of a DS-TBM b) Schematic modified DS-TBM arrangements in squeezing rock from [4]](image_url)
Figure 2: Numerical model of tunnelling with DS-TBM a) complete model b) discretization of model

Table 1: Rock mass parameters from Lyon–Turin Base Tunnel [1] and Geometric dimensions for DS-TBM components [4]

<table>
<thead>
<tr>
<th>DS-TBM components</th>
<th>Rock mass parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cutterhead length [m]</td>
<td>0.75</td>
</tr>
<tr>
<td>Elastic modulus, E [GPa]</td>
<td>1.40</td>
</tr>
<tr>
<td>Front shield length [m]</td>
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<tr>
<td>Poisson’s ratio, ν</td>
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<td>Rear shield length [m]</td>
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<tr>
<td>Cohesion, c [MPa]</td>
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<tr>
<td>Shield thickness [cm]</td>
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</tr>
<tr>
<td>Friction angle, φ</td>
<td>* 28</td>
</tr>
<tr>
<td>Lining segment width [m]</td>
<td>2</td>
</tr>
<tr>
<td>Lining segment thickness [cm]</td>
<td>45</td>
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Table 2: Mechanical properties of DS-TBM components [4]

<table>
<thead>
<tr>
<th>Material properties</th>
<th>Unit</th>
<th>Shield</th>
<th>Segmental lining</th>
<th>Soft backfilling</th>
<th>Hard backfilling</th>
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<tbody>
<tr>
<td>Elastic modulus [GPa]</td>
<td>200</td>
<td>36</td>
<td>36</td>
<td>0.5</td>
<td>1.0</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>-</td>
<td>0.3</td>
<td>0.2</td>
<td>0.3</td>
<td>0.3</td>
</tr>
<tr>
<td>Unit weight [kN/m³]</td>
<td>76</td>
<td>30</td>
<td>24</td>
<td>24</td>
<td></td>
</tr>
</tbody>
</table>

For preventing of the penetration of rock mass into shield elements due to large displacements, beside the interface elements, a code was developed in FLAC3D that controls all displacements with respect to non-uniform overcut. In addition, increasing of gap due to conical shape of the shield is considered in the code.

2.2. Creep Model of the Analysis

Table 3 summarized the creep parameters that were used in the analysis.

Table 3: Creep Constitutive parameters, CVISC model [1]

<table>
<thead>
<tr>
<th>Maxwell shear modulus, G^M [MPa]</th>
<th>566</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maxwell viscosity, η^M [MPa,year]</td>
<td>27.98</td>
</tr>
<tr>
<td>Tensile strength, σ_t [MPa]</td>
<td>8.5e-3</td>
</tr>
<tr>
<td>Kelvin viscosity, η [MPa,year]</td>
<td>4.26</td>
</tr>
<tr>
<td>Kelvin shear modulus, G^K [MPa]</td>
<td>498</td>
</tr>
</tbody>
</table>
3 RESULTS OF NUMERICAL ANALYSIS

The result of analysis includes the deformation and contact stresses versus advance rates. All of the monitoring points in the tunnel and on the cutterhead and both of shields relating to the interaction between the rock mass and machine components are monitored in each time step of the analyses.

3.1. DS-TBM Time Dependent Excavation Results

Figure 3 depicts the longitudinal displacement profile (LDP) and longitudinal maximum principle stress profile (LSP) at the lower shoulder of tunnel (point WF) in the contact and non-contact points between rock mass and TBM when machine advance rate is 24 m/day. As displayed in the Figure, the closure of gap between cutterhead and the ground occurs at end of the cutterhead, but contact stress on the cutterhead is nearly zero. This means that the contact between cutterhead and ground started in last solving time step, and minimal stresses from rock is applied to the cutterhead. However, a slowdown or standstill in TBM advance may cause extended area of contact between rock and cutterhead in the wall, and hence higher stresses.

Figure 3: LDP at tunnel circumference and LSP on TBM components along tunnel wall

Also the closure of gap between the front shield and the ground occurs at first 2.5 m of the front shield, and rock mass starts to load on the front shield up to 10.1 MPa. Due to conical shape of the shields, the contact stress between ground and rear shield is initially reduced to 1.95 MPa. But after a few time steps that equals 2 hour, contact between ground and rear shield occurs and ground starts to load on the rear shield with up to 4.4 MPa of pressure. When installing of segmental rings and backfilling operation starts, load is transferred uniformly from ground to lining by filling material and gradually arises versus time, then remains constant at about 5 MPa.
3.2. Effect of Advancement Rate

Figure 4 shows the impact of different advance rates on deformation and applied stress on TBM components.

![Figure 4: LDP and LSP for different advancement rates](image)

As well, the front shield, when the advance rate is 6 m/day, is loaded 1.2 MPa more than when advance rate equal to 48 m/day. This value is 1.7 MPa for rear shield. Also for advance rate of 48 m/day the segment experiences 2.5 MPa load less than the same segment when the advancement rate is 6 m/day during the course of excavation. This shows that the entrapment of shielded TBMs can occur in the squeezing ground during extended machine downtimes or in the lower advance rates.

3.3. Evaluation of TBM Entrapment Risks

Figure 5 shows the maximum sectional principal stresses on machine components for various advance rates. In these cases, the maximum principal stress is observed in the invert. This is due to non-uniform overcut with smallest amount in the invert can cause closure of gap so the cutterhead and shields starts to support the excavation walls. Also the weight of the machine provides additional confinement to the excavation surface near the tunnel face. On the other hand, as shown in Figure 5, the impact of advance rate on applied stress from rock mass to cutterhead and front shield is not registered, but the rear shield experiences lower magnitudes of load in high speed TBM excavation.

3.4. Effect of Advance Rate on Loading of Segmental Lining

As illustrated in Figure 6a, after installation of segmental lining, ground pressure is transferred uniformly to the ring by backfill. This shows the importance of backfill.
Moreover, the role of advance rate on the loading of lining is significant. In this case, for advance rate of 48 m/day the segmental ring experiences about 2.5 MPa less than when the advancement rate is 6 m/day.

Figure 5: Maximum sectional principal stresses on TBM components for different advancement rates

Figure 6b depicts the diagram that is used for prediction of maximum principal stress on the segmental ring at the variable advancement rates. According to this diagram, average stress on the lining ring is mainly dependent on advance rate. By developing such diagrams for a specified tunnel, one can calculate the magnitude of stress on the lining for a given advance rate and apply this value for designing of the segmental lining.

Figure 6: a) Principal stress on the segmental ring b) stress-advancement rate diagram
4. CONCLUSIONS

A comprehensive 3D simulation of ground behavior for excavation of a tunnel by a DS-TBM is used to evaluate the possibility of machine entrapment in potentially squeezing ground. The impact of different advance rates on deformation and applied stress on TBM components and segmental lining is investigated by using the results of modeling. Effect of machine advance rate is implemented by performing of time dependent creep analysis of the ground.

The results show that the front shield is subjected to the lower rate of loading behind the face, but effect of advance rate on loading of main TBM components is more visible in the rear shield. Also the average stress on the lining ring depends on advance rate. The loading diagrams for a given tunnel project can allow for calculation of loading on the lining at specified advance rates. The results can be used to evaluate the potential for entrapment of shielded TBMs in the squeezing ground during extended machine downtimes or for lower advance rates.

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Tunnel Behavior Subjected to Repeated Shear Deformation – model Test and Finite Element Analysis

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Abstract

In this research, a new tunnel device has been developed to investigate fundamental dynamic action on the tunnel in the case of earthquake applying repeated shear deformation to the ground. The device consists of a model tunnel which is free to move along with the soils of the ground, and a frame around the ground to allow the ground movement in the horizontal direction. In the tests, repeated shear deformation is applied through the application of horizontal inertial force after the completion of tunnel excavation. Changing the value of the horizontal seismic coefficient, effect of the shear deformation around the tunnel with different intensities has been investigated. The corresponding non-linear finite element analyses are also conducted using FEMtij-2D program where elastoplastic subloading $t_{ij}$ model is used as a constitutive model of soil. In the model tests, shear strain of the ground and earth pressure around the tunnel are measured corresponding to the inertial force. The earth pressure acting on the tunnel is compared between the model tests and numerical simulations and discussed in this paper. It is revealed that shear deformation due to ground acceleration resulting from the earthquake has significant effect on the earth pressure around the tunnel.

Keywords: Finite element analysis, repeated shear deformation, tunnel excavation
1 INTRODUCTION

Earthquakes are happening all the time all over the world. Numbers of tunnels are increasing day by day to meet the demand of modernization. During earthquake soil is experienced with shear deformation due to ground acceleration. Therefore, it is necessary to understand the tunnel behavior during and after earthquake for a safe and secured tunnel construction. It is said that there is generally little influence of seismic motion on tunnel structures. The superstructures such as bridge, building etc. significantly vibrate as the resonance of the superstructures itself dominate which occurs along with the inertial force during seismic motion. However, the resonance of the tunnel is less significant as the inertial force acting on the tunnel is smaller than that of the surrounding ground and the tunnel vibrates following the displacement and deformation of the ground. Furthermore, since the shield tunnel is built to some extent at the subterranean deeper place which is experienced with lesser deformation during earthquake, and as a circular tunnel section is strong against deformation, it is thought that a tunnel structure is not almost experienced the effect of earthquake. However, it is revealed in an investigation, reported by Japan Society of Civil Engineers [2], crack occurred to secondary lining concrete in a shield tunnel during the Great Hanshin-Awaji Earthquake. Therefore, it requires sufficient investigation in the design stage as non-uniform sectional force occurs at that places where there are sudden changes in ground conditions and at the conjugation of soil-structure. In this research, a new tunnel device has been developed to investigate fundamental dynamic action on the tunnel in the case of earthquake applying repeated shear deformation. The corresponding finite element analyses are conducted with FEMtij-2D program where elastoplastic subloading $t_y$ model [4] is used as a constitutive model of soil. This model can describe typical stress deformation and strength characteristics of soils such as the influence of intermediate principal stress, the influence of stress path dependency of plastic flow and the influence of density and/or confining pressure.

2 DESCRIPTIONS OF MODEL TESTS AND NUMERICAL SIMULATIONS

2.1 Layout of Model Tests

Figure 1 shows a tunnel apparatus after the modification of the previous tunnel device [7]. In the device, the excavation part can be moved freely upward and downward, and left and right without friction by a bearing and a horizontal slider attached in the device. The weight of the entire model tunnel is balanced with a counter weight which is acted
Tunnel behavior subjected to repeated shear deformation

through a fixed pulley set at the top of the device. As a result, the tunnel excavation can be simulated by leaving it to an equilibrium condition of the vertical and lateral earth pressures controlling the amount of shrinkage of the tunnel diameter. The total diameter of the tunnel is 10cm and the device consists of a shim at the center of the tunnel surrounded with 12 segments having load cells to measure earth pressure acting on the tunnel. The length of the apparatus is 120cm, and the tunnel devices are set up somewhat middle of the apparatus to avoid boundary effect to the results of the experiments. The apparatus consists an aluminum block which can be moved in the vertical direction. The dimension of the apparatus is chosen with a scale of 1:100 between the model and prototype scales. Therefore, the diameter of 10cm of the tunnel corresponds the tunnel of 10m in the real ground condition. Mass of aluminum rods, having diameters of 1.6 and 3.0 mm mixed in a ratio of 3:2 in weight, is used as ground material. The unit weight of the aluminum rods mass is 20.4kN/m³, and the length is 50 mm. The properties of the ground mass consisting of aluminium rods are similar to the properties of dense sand [7]. The initial ground is made in such a way that the earth pressure around the tunnel becomes similar to the earth pressure at rest adjusting the block of aluminum set at the bottom of the apparatus. The tunnel excavation is simulated by controlling the shrinkage of the tunnel device. In the previous apparatus the frames at both sides of the ground were fixed to restrict the edge movement in the horizontal direction. In the new apparatus, the frames of the both sides are set with hinges at the bottom end to allow the ground movement in the horizontal direction. The upper parts of the frames of the both sides are connected with an aluminum bar to apply the same shear deformation at both left and right sides of the ground. The dotted lines in Figure 1 show the direction of shear strain in a particular direction which can be achieved with the new apparatus. A load cell is attached to the left side frame which is connected with a handle to apply inertial force in the ground. The horizontal displacement of the ground is measured with a dial gauge which is attached to the left side frame from which the magnitude of the average shear strain (γ) is obtained. The initial ground is taken (γ=0) after the completion of the tunnel excavation. In the experiment, at first tunnel excavation is simulated by shrinking the tunnel device with 4mm, the shrinkage of the tunnel is represented with dᵣ. Then the shear strains are applied in the ground up to 4 cycles. Here, two values of inertial forces are applied – in Case 1 the horizontal seismic coefficient (kₕ) is 0.2, and in Case 2 kₕ is equal to 0.4. The tests have been conducted for a fixed overburden ratio, D/B=2.0, where D is the depth from the ground surface to the top of the tunnel and B (=10cm) is the width of the tunnel.
2.2 Layout of Numerical Analyses

The numerical analyses are performed with the same scale of the model tests. Figure 2 shows a typical mesh used in the finite element analyses. Isoparametric 4-noded elements are used in the mesh. Both vertical sides of the mesh are free in the vertical direction, and the bottom face is kept fixed. To simulate the tunnel excavation, negative volumetric strain in the tunnel elements is applied which corresponds the amount of radial shrinkage of the tunnel. This is an important simulation technique to consider the free movements of the tunnel. Two-dimensional finite element analyses are carried out with FEMtij-2D using the subloading $t_{ij}$ model [4]. Model parameters for the aluminum rod mass are shown in Table 1. The parameters are fundamentally the same as those of the Cam clay model except the parameter $a$, which is responsible for the influence of density and confining pressure. The parameter $\beta$ represents the shape of yield surface. The parameters can easily be obtained from traditional laboratory tests. Figure 3 shows the results of the biaxial tests for the mass of aluminum rods used in the model tests. From the stress-strain behavior of the element tests simulated with subloading $t_{ij}$ model, it is noticed that this model can express the dependency of stiffness, strength and dilatancy on the density as well as on the confining pressure. It is clear that the strength and deformation behavior are very similar to those of dense sand. The dotted lines represent the numerical results for a
Tunnel Behavior Subjected to Repeated Shear Deformation

Figure 2: Finite element mesh

Table 1: Parameters of soil materials

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value or Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\lambda$</td>
<td>0.008</td>
</tr>
<tr>
<td>$\kappa$</td>
<td>0.004</td>
</tr>
<tr>
<td>$e_{NC}$</td>
<td>0.3</td>
</tr>
<tr>
<td>$R_{cs}$</td>
<td>1.8</td>
</tr>
<tr>
<td>$\nu_c$</td>
<td>0.2</td>
</tr>
<tr>
<td>$\beta$</td>
<td>1.2</td>
</tr>
<tr>
<td>$a$</td>
<td>1300</td>
</tr>
</tbody>
</table>

Same parameters as Cam clay model

shape of yield surface (same as original Cam clay at $\beta = 1$)

$R_{cs} = (s_1/s_3)_{cs(\text{comp.})}$

Poisson's ratio

1.2

0.2

1.8

0.3

0.004

0.008

1300

kPa

Figure 3: Stress-strain-dilatancy relation of aluminum rods mass

confining pressure of 1/100 times the confining pressure in the experiments. The initial stress levels of the ground are calculated by applying the body forces due to the self-weight ($\gamma = 20.4\, \text{kN/m}^3$), starting from a negligible confining pressure ($p_c = 9.8 \times 10^6$ kPa) and an initial void ratio of $e = 0.35$. After self-weight consolidation, the void ratio of the ground is in between 0.28 to 0.30. The value of $K_0$, derived from the simulation of the self-weight consolidation is in between 0.70 and 0.73.
3 RESULTS AND DISCUSSIONS

Figure 4 represents observed and computed relation of the horizontal seismic coefficient with shear strain of the tunnel ground for cyclic loading. It is seen that the after few cycles of loading the stiffness of the ground material becomes constant. The numerical analyses perfectly capture the behavior of the model ground for both Case 1 and Case 2.

![Figure 4: Relation of horizontal seismic coefficient with shear strain](image)

Figure 5 and 6 shows the observed and computed earth pressure distributions for Case 1 and Case 2, respectively. The plots are drawn in the 12 axes corresponding to the radial direction of the 12 load cells towards the center of the model tunnel. The figures represent the value of earth pressure corresponding to the horizontal seismic coefficient ($k_h$). The figure also shows the earth pressure at initial ground and after the amount of shrinkage $d_l=4$mm. It is seen that for the excavation of the tunnel earth pressure decreases around this tunnel due to the arching effect, the same as the results of the references [3], [6], [7], and [8]. It is seen in the results of the 1st cycle, due to the application of shear strain the earth pressure increases significantly at the right shoulder and its opposite side when $k_h=0.2$ (Case 1) and 0.4 (Case 2). When $k_h=-0.2$ (Case 1) and -0.4 (Case 2), i.e. ground is moved to the left direction, earth pressure decreases at the right shoulder and its opposite side, and at the same time earth pressure increases at the left shoulder and its opposite side. The results of the other cycles show the repetition of increase and decrease of earth pressure at the right and left shoulders and their opposite sides the same way as the 1st cycle. For the horizontal seismic coefficient $k_h=0.2$ (Case 1), the increase of earth pressure around the tunnel is lesser than that of $k_h=0.4$ (Case 2) as expected. Though the numerical simulation produces larger value of the earth pressure compare to the model test, the mode and the tendency of the increase and reduction processes of it are similar to that of the model test.
Figures 7 and 8 illustrate the history of the change in earth pressure where the surrounding earth pressure of the tunnel is divided into six domains. The results represent the change of earth pressure of each domain for the shear deformation of 4 cycles. The results confirm the repetition of increase and decrease of earth pressure of each domain during the application of repeated shear to the right and left directions. With the repeated loading the stiffness increases as a mechanical characteristic of the ground material, hence the earth pressure around the tunnel increases to some extent. Interface element between the ground materials and the tunnel are not considered in the analyses which might be the cause of the over prediction of the results.

**Figure 5:** Distribution of earth pressure around tunnel : model tests
Figure 6: Distribution of earth pressure around tunnel: numerical simulations

Figure 7: History of earth pressure around tunnel: model tests
4 CONCLUSIONS

From the model tests and numerical simulations it is found that the earth pressure around the tunnel increases and decreases significantly according to the direction of the repeated shear. This is because the direction at which the arching effect is developed changes with the loading direction. Therefore, it is necessary to consider this important factor in a tunnel design for withstanding from the seismic motion during earthquake. The numerical analyses capture the distributions of earth pressure of the tunnel in different cases.
REFERENCES


Numerical Simulation of Construction Processes in Mechanized Tunneling
Interaction Modelling for Coupling Simulations using SysML

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¹Ruhr Universität Bochum

Abstract

Mechanized tunnelling is a complex engineering technology involving various processes like excavation, grouting and ring building. Many different numerical simulations are performed to analyze their effects on ground behaviour and risks on the built environment. Usually, these simulations are carried out independently without considering possible effects between each other. However, since these effects are very important to obtain more accurate analysis results, there is a need to explicitly model the coupling between the simulations. Hence, an interaction model framework for mechanized tunnelling is being developed by formally describing the structural and behavioural relations between simulations. Exchanging necessary data between coupled numerical simulations can improve the overall system performance. In this paper, the Systems Modelling Language (SysML) is used to describe the simulation models and their coupling within the interaction framework. In particular, structure diagrams model relations and behaviour diagrams are used to show the interactions. A case study involving the driving simulation and the grouting simulation is discussed in detail.

Keywords: SysML, coupling simulations, interaction modelling
1 INTRODUCTION

A tunnel boring machine (TBM) is prone to have effects on the surrounding ground and especially on the built environment above the ground surface. It is important to study and simulate the effects of the excavation process beforehand to prevent excessive settlements and thereby avoid damages to the built environment. Existing simulations in mechanized tunnelling (MT) include the driving process, the grouting infiltration process, advance exploration and others. These simulations are often performed separately without considering the interdependencies between them. Therefore, these interactions have to be considered to realistically simulate the complete mechanized tunnelling process and to reproduce the effects of MT more accurately. Since the numerical simulations are performed by multiple teams using different software, the loose coupling method must be used [4], where communication can be established at the data level by coupling the outputs of one simulation with corresponding inputs of another simulation. The OpenMI (Open Modelling Interface) is a software component interface definition that facilitates loose coupling of interacting environmental processes at the implementation level [5]. While dealing with complex coupling problems, a formal description of simulations and their interaction is necessary to implement loose coupling.

Systems Modelling Language (SysML [1]) is a graphical modelling tool that enables the application of systems engineering concepts to model complex systems like the simulation system considered in this paper. It describes the structure and behaviour of the entire system at various levels of abstraction necessary to realize coupling. It enables the teams involved in interaction to visually see when and how parameters are exchanged. Visualizing the interaction model reduce mistakes early in the design phase, allows one to identify complexity, aids understanding, improves communication between teams and helps to couple simulations with utmost ease. Considering the importance of modelling the interaction, this paper will discuss the application of SysML in creating an interaction model.

2 CONCEPT

Systems engineering is an interdisciplinary approach which helps to look into a complex problem in its entirety. Figure 1 shows a complex simulation system composed of numerical simulations like driving (A)[2], grouting (B)[3] and advance exploration (C), where interaction is denoted by the connection lines between simulations. A connection line only denotes that there is some type of interaction between the simulations. When and where the data is exchanged remains unclear. Interaction
Interaction Modelling for Coupling Simulations using SysML

Figure 1: Interaction modelling - Concept

modelling helps us to describe interactions in detail at a level of abstraction required to couple the simulations. An interaction model is an abstraction of the complex simulation system which graphically describes its components (simulations) and the interactions (connections) between them. A general-purpose graphical modelling language like the SysML [1] which can be used to develop an interaction model. SysML diagrams are used to build an interaction model for specifying, analyzing, designing and verifying complex simulation systems. It allows engineers to model system requirements, system structure and system behaviour based on nine types of diagrams described by the OMG SysML standards. These nine diagrams can be grouped into structure diagrams, behaviour diagrams and requirement diagrams. Structure diagrams describe what simulations are connected and what parameters are exchanged in the simulation system. Behaviour diagrams describe how the simulations interact with each other and how parameters are exchanged. Requirement diagrams, as the name suggests, formally define the requirements for a system to operate as expected. The behaviour of the interaction model can also be animated with SysML tools like IBM Rhapsody automatically generates code that can be used to implement loose coupling. Since the discussion of SysML is beyond the scope of this paper, only four diagrams will be introduced informally.
The development of an interaction model occurs in two stages as shown in Figure 1. In stage 1, the components of the simulation system (i.e. simulations and connections) are identified to describe the structure of the complex simulation system. Simulations are represented by blocks in SysML block definition diagram (bld) and the connections are defined in the internal block diagram (ibd). The input and output parameters are assigned to the simulation blocks. The ibd is used to show the connection between the input and output parameters of the simulations. In stage 2, the behaviour of the components of the simulation system is defined using the SysML behaviour diagrams namely, state machine diagram (stm) and sequence diagram (sd). The stm captures the definite state of individual simulation blocks during their simulations. Figure 1 (stage 2, top) shows two definite states of simulation A (idle and running). The sd describes the order of events, operations and messages required to implement an interaction between simulations. Figure 1 (stage 2, bottom) shows the sequence of events between the simulations when they are active. In this figure, the number 1 is the message sent from A to B and the number 2 is sent from B to A.

3 CASE STUDY

The interaction modelling concept is demonstrated using a simple case study which discusses an exemplary interaction scenario between the driving simulation (DS) and the grouting infiltration simulation (GS). The DS is an FE simulation that computes the settlements due to the excavation of the soil by the TBM. The GS computes the infiltration of the grout material into the surrounding soil. The simulations are considered as black boxes with known sets of input and output parameters and modelled at an abstract level. The interaction between the simulations is illustrated in Figure 2. The DS needs permeability calculated by the GS. The GS requires the pore pressure value from the DS to calculate the permeability value. The GS is invoked in three cases by the DS. First, the GS is invoked after the DS completes its first step in order to verify the assumed permeability that was used to compute the first time step. Second, when there is a change in ground layer at some point of time during the DS, the GS is invoked. Third, the GS is also invoked periodically to verify whether the current permeability value is still within acceptable limits. The central point of discussion is on how interaction can be modelled to accomplish coupling at the data level using SysML. The reader should keep in mind that the spotlight is not on the simulations but on the concept used to demonstrate how interactions can be modelled to aid loose coupling of simulations at the data level using SysML.
Interaction Modelling for Coupling Simulations using SysML

The interaction model is developed in two stages as described in the concept. In the first stage the components of the simulation system under consideration (simulations and connections) are identified and their structure is described. The DS and the GS are the two simulations considered for this case study. Both these simulations consist of one input and one output that can be described in a bdd. The input of the DS is the output of the GS and vice versa as described by the ibd which describes the connection between simulation blocks (Figure 3). Pore pressure which is the output of the DS becomes the input of the GS by flowing from the \textit{porePressureOut} port to the \textit{porePressureIn} port. Similarly, the permeability which is the output of the GS forms the input of the DS. It flows from the \textit{permeabilityOut} port in the GS to the \textit{permeabilityIn} port in the DS. The structure diagrams do not tell us how the parameters are exchanged. They only show what components are connected and what parameters are exchanged.

In the second stage, the behaviour of the simulation components is described to show how and when the parameters are exchanged. The behaviour of the simulation blocks and the connections are defined using the stm and the sd respectively. Figure 4 shows the stm of the GS (left) and the DS (right) respectively. The stm gives an abstract...
description of the behaviour of the simulations by defining their finite states and transitions between the states during the simulation. Inactive and running states are defined for the GS. Inactive is its default state. Similarly, inactive and active states are defined for the DS. The active state is a complex state which in turn consists of two sub states, running and idle. The interaction between the simulations is coordinated by the Interaction Platform (IP). The start event from IP triggers a transition from the inactive state to the active state. During this transition, the DS is initialized. The default sub-state within active state is the running state which starts execution of the DS. A layer change during the DS starts the GS and triggers the state transition from running to idle. This also triggers a transition from inactive to running in the stm of the GS. During this transition, pore pressure is received and the GS is initialized. When the execution of the GS is complete, the permeability values are sent to the IP and the simulation terminates. The IP now triggers a transition from idle to running state in the stm of the DS and the next time step starts. This process continues until the DS is complete. The stop event is called when the simulation is complete.

The communication between the simulations is defined using the sd (Figure 5). It describes how, when and in what order these simulations execute and exchange data between them. The sd in Figure 5 describes the interaction between the DS and the
GS via IP. The IP starts the DS by triggering *start()* event. The DS prepares for com-
putation by reading the inputs during the *initialize()* operation. Execution starts after
the initialization. All the operations inside the outer conditional box 'loop' repeats
itself for n time steps. If there is a layer change or at specified time steps, the DS
suspends temporarily and sends the pore pressure (*sendPorePressure()*)) to IP. This
triggers IP to start the GS by sending the pore pressure (*start(porePressure)*)) received
from the DS. Once the GS is complete, it sends back the permeability (*sendPerme-
ability()*)) to IP and the DS is resumed (*resume()*)) from where it is suspended and
continues execution. All the events and operations inside the inner box ‘opt’ occur
only if there is a layer change or at specified time steps. Modelling the interaction
with SysML helps visualize, analyze and verify the interaction model and helps to
decide what kind of coupling solution can be used to enable the interaction. SysML
tools such as IBM Rational Rhapsody simplify the modelling process and provide
animation functionalities for the behaviour diagrams that is useful to verify the in-
teraction model developed.
4 CONCLUSION

The systems engineering approach can be used to build complex simulations system and model the interaction between them. The Systems Modelling Language (SysML) enables the application of the systems engineering approach to complex interaction problems and supports high level collaboration between teams without disturbing the focus of their research. The case study discussed demonstrates how this approach can be used to model interactions. The same technique will be used to extend the complex simulation system to include all the sub processes of MT and couple them at the data level.

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Probabilistic Assessment of Tunnel Construction Time Using Dynamic Bayesian Network

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Abstract

Construction time estimates are important parameters for decision-making in transport infrastructure projects. At present, the construction time is usually assessed deterministically by experts; probabilistic models are not commonly employed in practice. A main reason is that the existing models often do not provide a realistic estimate of the overall uncertainty. In this contribution, we present a Dynamic Bayesian Network (DBN) model, which aims to more realistically represent the uncertainties in tunnel construction time estimates and which provides an understandable graphical representation of the model assumptions. The model considers the geotechnical uncertainties as well as uncertainties associated with human and other external factors. It includes the common variability of the construction performance and the occurrence of extraordinary events (failures) such as tunnel collapses. Analyses of construction performance data from tunnels constructed in the past provide a basis for estimation of failure rate and for determination of unit time distribution, which are the essential inputs of the probabilistic model. A case study demonstrates the applicability of the DBN model and the possibility of updating predictions with new information obtained during the construction process.

Keywords: Construction time, risk analysis, Dynamic Bayesian Networks, construction performance, failure rate, probabilistic model
1 INTRODUCTION

Estimates of tunnel construction time and costs are highly uncertain. This uncertainty originates in the unknown geotechnical conditions, in natural variability of the construction performance and in unpredictable influence of common factors such as quality of planning, quality of construction management and external political and economical factors. Today, the construction cost and time are typically assessed deterministically. The deterministic estimates are, however, often underestimated, as shown for example in Flyvbjerg et al. (2002). This systematic underestimation of project cost and time causes severe problems to the construction industry and to society in general, because it does not provide a realistic information basis for decision-making (Hägler, 2012). The uncertainty of the construction time and cost estimates should be quantified and communicated with the stakeholders and with the public. The need of probabilistic prediction of construction time has been recognized in the tunnelling community in recent years (Lombardi, 2001; Reilly, 2005; Grasso et al., 2006; Edgerton, 2008).

Different approaches exist for analysing the risks and for assessing the uncertainty in the construction time and cost estimates. The existing models and approaches are summarized in Špačková (2012), where the advantages and limitations of the models are also discussed. In this paper we briefly describe a model using Dynamic Bayesian Networks (DBNs) for probabilistic modelling of the tunnel construction time, which was originally proposed in Špačková and Straub (2013) and Špačková et al. (2013). The first paper focuses on the algorithm for evaluation of the DBN, the later paper discusses a methodology for statistical analysis of data from past projects, which should be used for learning the parameters of the model. The model is suitable for utilization in all phases of the project: in early planning phases it provides a rough estimate based on limited amount of information and in later phases the estimate can be continually updated with new information, also during the construction itself.

2 DBN MODEL OF TUNNEL CONSTRUCTION TIME

DBN model of tunnel construction process is displayed in Figure 1. Each slice of the DBN represents a tunnel segment of length $\Delta l$ (here $\Delta l = 5m$). The $i$th slice thus represents a tunnel segment between position $(i-1)\Delta l$ and $i\Delta l$. All variables are modeled as constant within a segment, i.e. the model implies that the geotechnical conditions and construction performance do not change within a segment. The individual variables of the DBN are summarized in Table 1 and described below.
Figure 1: DBN model for tunnel construction process. The variables of the model are summarized in Table 1.

The geotechnical conditions are modelled by variables zone and ground class. Zone $Z_i$ represents the locations of the quasi-homogenous geotechnical zones along the tunnel axis. Ground class $G_i$ describes the geotechnical conditions in the $i$th segment, it correspond to the commonly used geotechnical classification systems (RMR, $Q$-system) or to a project specific classification. In this paper, ground class is defined deterministically for given zone.

Construction performance is modelled by variables human factor, geometry, construction method and unit time. Human factor $H_i$, which represents the common factors influencing the construction performance, is in one of the three states “unfavourable”, “neutral” or “favourable” throughout the entire tunnel construction, i.e. the $H_i$s are fully dependent from one slice to the next. Geometry $E_i$ models different cross-sections along the tunnel (a typical cross-section vs. extended cross-section for emergency parking places EPP), the special conditions at the beginning and end of the tunnel and at the location where the tunnel passes an existing chemical plant. Construction method $M_i$ describes the excavation type and the related support pattern applied in the $i$th segment and it is defined conditional on the ground class $G_i$. 
and tunnel geometry $E_i$. For every construction method $M_i$ and human factor $H_i$, the unit time $T_i$ (i.e. time needed at excavation of the $i$th segment, an inverse of commonly used advance rate) is defined by a conditional probability distribution, $f(t_i|m_i, h_i)$, which should ideally be determined from analysis of data in past tunnels. To facilitate the application of the exact inference algorithm suggested in Špačková and Straub (2013), the variable $T_i$ is discretized.

**Table 1:** Summary of variables of the DBN model.

<table>
<thead>
<tr>
<th>Id.</th>
<th>Variable</th>
<th>Type</th>
<th>States of the variable</th>
</tr>
</thead>
<tbody>
<tr>
<td>$Z$</td>
<td>Zone</td>
<td>Random/Discrete</td>
<td>1,2,...,7</td>
</tr>
<tr>
<td>$G$</td>
<td>Ground class</td>
<td>Random/Discrete</td>
<td>3,4,5</td>
</tr>
<tr>
<td>$H$</td>
<td>Human factor</td>
<td>Random/Discrete</td>
<td>Favourable, neutral, unfavourable</td>
</tr>
<tr>
<td>$E$</td>
<td>Geometry</td>
<td>Determin./Discrete</td>
<td>$37 \text{ m}^2, 43 \text{ m}^2, 46 \text{ m}^2$</td>
</tr>
<tr>
<td>$M$</td>
<td>Construction method</td>
<td>Random/Discrete</td>
<td>3-37, 3-43, 3-46, 4-37, 4-43, 4-46, 5-37, 5-43, 5-46</td>
</tr>
<tr>
<td>$T$</td>
<td>Unit time</td>
<td>Random/Discretized</td>
<td>$0, t_{\text{int}}, 2t_{\text{int}}, ..., 14.5\text{ [days]}$ *</td>
</tr>
<tr>
<td>$F$</td>
<td>Failure mode</td>
<td>Random/Discrete</td>
<td>Failure, No failure</td>
</tr>
<tr>
<td>$N_F$</td>
<td>Number of failures</td>
<td>Random/Discrete</td>
<td>0,1,2,3,&gt;4</td>
</tr>
<tr>
<td>$T_{\text{cum}}$</td>
<td>Cumulative time</td>
<td>Random/Discretized</td>
<td>$0, t_{\text{int}}, 2t_{\text{int}}, ..., 1392^{**}$ [days]</td>
</tr>
<tr>
<td>$T_{\text{extra}}$</td>
<td>Delays caused by failures</td>
<td>Random/Discretized</td>
<td>$15, t_{\text{int}}, 2t_{\text{int}}, ..., t_{\text{extra},99.9} \text{ [days]}^{***}$</td>
</tr>
<tr>
<td>$T_{\text{tot}}$</td>
<td>Total time</td>
<td>Random/Discretized</td>
<td>$0, t_{\text{int}}, 2t_{\text{int}}, ..., (1392 + t_{\text{extra},99.9})\text{ [days]}$</td>
</tr>
</tbody>
</table>

* $t_{\text{int}}$ is the discretization interval of time variables, $t_{\text{int}} = 0.5 \text{ day}$.

** upper bound of cumulative time = 96 x 14.5 = (number of segments) x (upper bound of unit time)

*** $t_{\text{extra},99.9}$ is the 99.9 percentile of $T_{\text{extra}}$
The model also takes into account the occurrence of extraordinary events (failures of the construction process), i.e. events that stop the excavation for 15 or more days. These are modelled by variables failure mode and number of failures. **Failure mode** $F_i$ represents the possible occurrence of an extraordinary event in segment $i$, it is defined conditionally on $H_i$ and $G_i$. The probability of failure occurrence is determined based on failure rates observed in the past tunnels. **Number of failures** $N_{F,i}$ represents the total number of failures from the beginning of the tunnel up to the segment $i$.

The main output of the model is the **total construction time**, $T_{tot}$. In the DBN, it is computed as the sum of construction time excluding extraordinary events, $T_{cum}$, and the time delay caused by extraordinary events, $T_{extra}$. **Cumulative time** $T_{cum,i}$ is the time needed for the excavation of the tunnel up to the location $i\Delta l$. It is defined as the sum of $T_{cum,i-1}$ and the unit time in segment $i$, $T_i$: $T_{cum,i} = T_{cum,i-1} + T_i$. $T_{extra,i}$ is the time **delay due to occurrences of failures** (extraordinary events) in the tunnel construction up to the segment $i$. Assessment of the total construction time, $T_{tot}$, is in most cases of interest for the tunnel as a whole or for a section of the tunnel. Therefore, it is computed only at the end of the tunnel, in slice $I$, as illustrated in **Figure 1**.

Definition of variables ground class, construction method, geometry, unit time and cumulative time follows the procedure originally proposed for the Decision Aids of Tunnelling (DAT) model (see e.g. Einstein, 1996). Similar modelling of failure occurrences was originally used in Sousa and Einstein (2012).

### 3 NUMERICAL EXAMPLE

DBN model is applied to a 480 m long Czech tunnel, which was built as a part of an underground extension project. The tunnel has only one tube. The first section of the tunnel serves as an access tunnel and it will not be utilized after the completion of the project. The remaining section of the tunnel will be used as a ventilation plant and as a dead-end rail track. The New Austrian tunnelling method (NATM) was used for construction of the tunnel. The tunnel is constructed in homogeneous conditions of sandstones and clay stones. Based on the geotechnical survey, the tunnel is divided into seven quasi-homogeneous zones, the geology is categorized into three ground classes.
The resulting estimates of construction time are presented below. First the prior estimate from a planning phase is shown in Figure 2, second the updated estimate with performance observed during the construction of the first 150 m of the tunnel is shown in Figure 3.

**Figure 2:** Prior prediction of the tunnel construction time: (a) Uncertainty in the estimated excavation progress (b) Probability density function (PDF) of total construction time for the whole tunnel.

**Figure 3:** Updated prediction of the tunnel construction time: (a) Uncertainty in the estimated excavation progress (b) Probability density function (PDF) of total construction time for the whole tunnel.
In the Bayesian updating process, the conditional probability distribution of unit time $f(t_i|m_i,h_i)$ is also updated. An example of the prior and updated probability mass function (PMF) for construction method 3-37 and $H_i = "neutral"$ is shown in **Figure 4**.

![Figure 4: Prior and updated PMF of unit time per 5 m for construction method 3-37, $H_i = "neutral"$.](image)

### DISCUSSION AND CONCLUSION

A Dynamic Bayesian Network (DBN) model for probabilistic estimation of tunnel construction time originally proposed in Špačková and Straub (2013) and Špačková et al. (2013) is briefly described. Its utilization is illustrated on an application example of a Czech tunnel. For the prior prediction during the design phase, the parameters of the DBN model are determined by expert assessments informed by the statistical analysis of data from past tunnels. During the construction, the prior prediction is updated based on the observed performance. Also the model parameters, such as unit time, are updated with this new information. The results show that the proposed DBN model allows one to more realistically assess the uncertainty in the construction process.

### REFERENCES


Process Simulation of Microtunnelling Operations for Productivity Assessment Depending on Ground Conditions

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Abstract

Microtunnelling operations involve a complex interaction of processes that require a variety of supporting equipment and personal experience. They require the integration of different construction processes such as supply chain management for the machine or for material handling. Breakdowns of critical processes might directly affect the productivity of the microtunnelling project. The objective of this research is to evaluate and analyze the factors that affect the productivity of microtunnelling operations. The method of process simulation can be used before a microtunnelling project actually is executed; thereby the problems at the different phases can be anticipated and analyzed. For this purpose, a SysML (System Modeling Language) model describing the microtunnelling process is developed in the first step. Subsequently, the simulation software AnyLogic is applied to create the simulation module based on the SysML formalization. An actual microtunnelling project at the city of Recklinghausen, Germany, is used for the validation of the developed simulation module. After validation, the simulation module is expanded with consideration between different soil compositions and used in order to evaluate the impact of the soil. As a result, the efficiency of microtunnelling is assessed by help of the developed simulation module for different ground conditions. A sensitivity analysis is also carried out with consideration various combinations of resources. The results highlight the identification and analysis of the various resources that affect the productivity in microtunnelling operations.

Keywords: Microtunnelling operations; Computer simulation; Simulation model.
1 INTRODUCTION

The productivity of microtunnelling is affected by many dynamic, uncertain variables and disturbances, such as weather, space congestion, crew availability, regulatory requirements, design changes and rework. In addition, tunnel construction with microtunnelling is a complex operation process. Therefore, if the construction process is reasonably arranged, then it can help to control and adjust the construction operation more efficiently. For this purpose, the need to better understand the construction process, the factors that affect the productivity.

Simulation is a widely used tool for the design, development, analysis and optimization of technical processes for more than 40 years. In general, process simulation methodology is going to develop the simulation model representing the logic of various activities required to construct a facility, the resources involved in carrying out the work (labors, equipments, management, etc.), the technical processes and unit operations [6]. The simulation model can help to analyze causes for delay, bottlenecks and reduction of productivity during construction operations in general. The use of simulation for researching construction operations was first proposed by Halpin in the 1970s [13]. He developed the CYCLONE (CYCLic Operations NEtwork) and it became the starting point of modern construction simulation languages and made process simulation methodology popular. Subsequently, different simulation languages have enhanced the CYCLONE such as INSIGHT by Paulson [19], RESQUE by Chang [14], UM-CYCLONE by Ioannou [4], HSM by AbouRizk [20], STROBOSCOPE by Martinez [21], and RBM by AbouRizk and Shi [22]. Researchers have also applied the CYCLONE in construction operations e.g. many authors have used CYCLONE in optimizing planning of earthmoving operations [15] [16] [17]. Some studies have applied CYCLONE on MTBM to evaluate the effects of bottleneck and different soil composition on the productivity [10] [12]. Various authors have used CYCLONE to analyze the effect of soil conditioning on the productivity of TBMs [11] [18]. In the end of 2012 and early 2013 numerous studies have attempted to analyze the effect of different types of disturbances on the productivity of EPB-shields based on SysML methodology combined with AnyLogic simulation software [8] [9].

The purpose of this study is to evaluate the factors that affect the productivity of microtunnelling operations by using a simulation. The operation analysis is performed by using simulation module, which is developed by helping of SysML methodology and AnyLogic simulation software. The results highlight the correlation between soil
composition and productivity. Therefore a sufficient amount of data collected from repeated operations in different types of soil conditions are analysed. In addition, a series of sensitivity analyses is performed by varying combinations of resources to measure their effect on the productivity.

2 MICROTUNNELLING ANALYSIS

Before the simulation model for microtunnelling is built, the resources, equipments and construction processes required during tunnel construction with MTBM must be analyzed. The most important resources (main resources) are identified in this paper as following: the labor crews, the jacking station system, the control container, the separation plant, the microtunnelling machine, the lubrication mixture and the pump system. The other resources are also identified, but they are considered as secondary resources. These involve construction equipment e.g. loader, crane, navigation system.

In tunnel construction operations with MTBM, three different classifications of the labor crew are normally required. The first is the Operator defined as a one laborer. The Operator is involved in the task of job management and also operates the entire the equipments in the job site e.g. control container, loader, crane. The second crew is named Crew 1 and defined as a one laborer working on the surface. The Crew 1 in the task of the mixing of the lubricant fluid, rigging the pipe section and preparing the pipe section before jacking pipe section forward. The last crew is named Crew 2 defined as three labors working in the shaft. The Crew 2 is involved the help to replace jack collar, connect or disconnect cables, hoses.

The core process of tunnel construction with MTBM can described as follows: The Operator and Crew 2 receive the signal from the shaft bottom, the crane is maneuvered, picks up a new pipe, which is stockpiled at a location near the top of the starting shaft and lower pipe into the launch cradle. When the pipe is positioned on the launch cradle, the jack collar, cables and pipelines may be replaced and connected in order to prepare for further operation. Subsequently, the pipe section may be jacked forward. When pipe jacking forward is finished, the jack collar is retracted, cables and pipelines are disconnected. When the process is completed, the preparation for the next pipe is started and the sequence repeats itself. The cycles repeat as required until the length of the tunnel is excavated.
3 SIMULATION MODEL DEVELOPMENT

The SysML is used to develop the simulation model. The SysML methodology offers a graphic method can be used for the analysis, modelling and representation of construction processes. Within this paper the SysML model representation the processes and elements are developed. The SysML model consist three type of diagrams: "block definition diagram (bdd)", "state machine diagrams (stm)" and "sequence diagram (sd)". The reader is referred to [1] for a more detailed description of the SysML methodology.

3.1 Block Definition Diagram

The bdd is used in order to show the structure elements in the microtunnelling construction. Each element in the bdd is considered as a stand-alone block, which can have a specific behavior, attributes, constraints and requirements [7]. Applied on the microtunnelling construction, a first hierarchical order is established by the distinction between some of main resources. The next level of hierarchy is provided by the secondary resources. The main and secondary resources are identified in the section 2. The main blocks of an MTBM are illustrated in Figure 1.

3.2 State Machine Diagram

In order to capture the processes of the microtunnelling construction elements, the technique of state machine is applied. The intrinsic processes of every block repre-
sented in the bdd is described within a separate stm. Each stm explains the behavior of an activity in terms of transitions between states triggered by events. A stm diagram example illustrates the behavior of microtunnelling in Figure 2.

### 3.3 Sequence Diagram

In order to identify the interaction between blocks the sq is used. Normally, the tunnel construction with MTBM can be separated into two stages. The first stage (namely preparation stage) is included in the preparation process before jacking pipe forward. The preparation stage can be described in three main steps, as follows:
- Step 1: The Crew 1 is working on the surface and will attach the pipe section to the crane. The Operator will lift, lower it into the shaft and laid on the jacking frame. The Crew 2 is working in the shaft will setup the pipe section for the installation.
- Step 2: Mixing the lubricant on the surface is executed by Crew 1. Setting up the slurry lines and hydraulic hoses on the MTBM is performed by Crew 2.
- Step 3: After the pipe section is jacked, the Crew 2 will disconnect the slurry lines and hydraulic hoses.

The second stage (namely excavation stage) is performed by the Operator. The Operator will handle the control container in order to steer the jacking pipe section process with excavation, and removing of the spoil.

### 4 DEVELOPMENT OF SIMULATION MODULE

In this study the simulation module is developed using the AnyLogic simulation software by XJ Technologies [3]. In order to upgrade the flexibility of the established model, the element blocks are implemented and transcribed into AnyLogic software.
Table 1: Duration information of job site: BV Recklinghausen V8

<table>
<thead>
<tr>
<th>Activity number</th>
<th>Activity</th>
<th>Minimum value (min)</th>
<th>Mode value (min)</th>
<th>Maximum value (min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Attaching pipe section</td>
<td>0.42</td>
<td>0.5</td>
<td>1.0</td>
</tr>
<tr>
<td>2</td>
<td>Lifting pipe go to shaft</td>
<td>0.5</td>
<td>0.7</td>
<td>1.0</td>
</tr>
<tr>
<td>3</td>
<td>Lower pipe</td>
<td>0.85</td>
<td>1.25</td>
<td>1.6</td>
</tr>
<tr>
<td>4</td>
<td>Laying pipe</td>
<td>1.2</td>
<td>1.65</td>
<td>2.7</td>
</tr>
<tr>
<td>5</td>
<td>Replace jack collar</td>
<td>2.5</td>
<td>3.33</td>
<td>4.0</td>
</tr>
<tr>
<td>6</td>
<td>Cables connect</td>
<td>43.0</td>
<td>52.3</td>
<td>82.3</td>
</tr>
<tr>
<td>7</td>
<td>Jacking forward</td>
<td>120</td>
<td>155</td>
<td>271</td>
</tr>
<tr>
<td>8</td>
<td>Retract jack collar</td>
<td>3.33</td>
<td>4.5</td>
<td>5.5</td>
</tr>
<tr>
<td>9</td>
<td>Disconnect cables</td>
<td>15.0</td>
<td>19.0</td>
<td>22.3</td>
</tr>
<tr>
<td>10</td>
<td>Time for clean spoil container</td>
<td>18</td>
<td>23</td>
<td>25</td>
</tr>
<tr>
<td>11</td>
<td>Time for mixing bentonite</td>
<td>19.5</td>
<td>21.2</td>
<td>25.5</td>
</tr>
</tbody>
</table>

This is achieved by transcribing the element blocks within SysML model into named Active Object Classes (AOC). The process interaction inside of each block as well as the behaviour between each block is rebuilt by using state charts, which are provided by AnyLogic.

The use of the simulation module will be applied in this paper in order to analyze the microtunnelling processes.

5 PROJECT DESCRIPTION AND DATA COLLECTION

In this research, data is collected from the project BV Recklinghausen V.8, located at the city of Recklinghausen, Germany. The subproject BV Recklinghausen V.8 was is one part of big project 11km of water treatment, ensuring proper drainage, flood protection in Recklinghausen City, Germany. The length of the tunnel is 145.0m and is located wholly within marl. The depth to the axis was approximately 8.7m, grade of the tunnel was 2.6 degree and type of pipe DN1200 is used. Pipe size is 1.2m for the internal diameter, 1.56m external diameter, and 4.0m length. The location of the construction site offers good access. Excavation is carried out by microtunnelling machine AVN 1200T using hydraulic spoil removal.

The input duration information data use in running the simulation model was collected in the job site by manual timing. The duration information for various activities are shown in Table 1.
6 SIMULATION RESULTS

6.1 Verification of Simulation Module

Before using the simulation module, it is important to confirm that the simulation module represents an indication of the logic and structure of the MTBM. In order to verify, a complete simulation module without consideration of soil impacts and disturbances is used. A total of 36 simulations is executed with the simulation module. Figure 3 shows the change of productivities in 36 cycles. A pattern can be easily obtained from these simulations. The Figure 3 shows that the productivities between theoretical data and the output data are quite similar. The average productivity for installing one 4.0 m pipe section is 236.77 min, as shown in red line in Figure 3. The average simulated duration from 36 cycles, show as black line in Figure 3 is 249.35 min, it means that 5.045% higher than the average productivity obtained in job site, which is clearly in a reasonable range of typical microtunnelling project. In addition, the 3D model is performed by using simulation module as well. The logistics of tunnel construction with MTBM in the 3D simulation is represented. Consequently, it is concluded that the simulation module correctly reflects the logic and structure of the MTBM and the accuracy of input data.

6.2 Simulation with Different Soil Compositions

After verifying the simulation module with actual data, the model can be enhanced with different soil conditions. The encounter such a soil variety would be probably rare in actual practice for a single microtunnelling operation. Therefore, the assumption that the results of the module test could be used for simulation of impacts of similar types of soils and the tunnel will be encountered with different type of soils. In order to select a specific type of soil mostly encountered in real situations, three types of soil marl/clay, fine sand, sand and gravel are chosen. The minimum and
maximum values of the durations of pipe jacking according to the nature of the soil (chosen according to [2]) are shown in Table 2.

Table 2: Summary of penetration rates for each type of soil [2]

<table>
<thead>
<tr>
<th>Type of soil</th>
<th>Minimum value (min)</th>
<th>Maximum value (min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fine sand</td>
<td>19</td>
<td>45</td>
</tr>
<tr>
<td>Sand and gravel</td>
<td>35</td>
<td>157</td>
</tr>
<tr>
<td>Clay/marl</td>
<td>69</td>
<td>292</td>
</tr>
</tbody>
</table>

Figure 4: Simulation results for different soil compositions

Figure 4 shows the productivity and the operation time of the MTBM in three cases. In the first case, the soil encountered at the construction site is fine sand. The rate of progress is 1.98 m/h, the project will be finished in 73 hours. The percentage of working time and waiting time of MTBM are 23.1% and 76.9%, respectively. The second case, the length of the tunnel is wholly within sand and gravel. The rate of progress is 1.6 m/h, the project will be finished in 89.5 hours. The percentage of working time of MTBM is 37.7% and waiting time is 62.3%. In the third case, the soil encountered along the tunnel is clay and marl. The rate of progress is 1.13 m/h, the project will be finished in 127.7 hours. The percentage of working time
and waiting time of MTBM are 57.8% and 42.2%, respectively. For all cases, the disturbances are not considered, the resources are always available, the equipments are not maintained during the construction.

### 6.3 Simulation with Variation of Resources

The sensitivity analysis is focused on manpower and Output Crew Quota (OCQ). The combinations of adding 1 labor to Crew 1, adding and reducing 1 to 2 labors to Crew 2 with different OCQ is simulated. The OCQ may be defined as the productivity of the crew and calculated by formula:

\[ OCQ = \frac{T_{at}}{T_{tp}} \]  

Where \( T_{at} \) is the actual time spent by the number of labors executing the task; \( T_{tp} \) is the time plan for executing the task. For instance, normally the Crew 2 consists of 3 labors and the \( T_{tp} \) is 19 min in order to complete the "disconnect cables" task, with 2 labors the Crew 2 needs the \( T_{at} \) of 24 min to finish. The ratio of the times based on formula 1 is \( OCQ \). The OCQ is used within the paper based on the data and experience from the construction site.

<table>
<thead>
<tr>
<th>Resource information</th>
<th>Parameters</th>
<th>Productivity information</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of Operator</td>
<td>Number of Crew 1</td>
<td>Number of Crew 2</td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td>3</td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td>4</td>
</tr>
<tr>
<td>1</td>
<td>1</td>
<td>5</td>
</tr>
<tr>
<td>1</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>1</td>
<td>2</td>
<td>2</td>
</tr>
<tr>
<td>1</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>1</td>
<td>2</td>
<td>4</td>
</tr>
<tr>
<td>1</td>
<td>2</td>
<td>5</td>
</tr>
</tbody>
</table>

Sensitivity analysis is carried out by using simulation module to analyze resource optimization as shown in Table 3. The highest productivity results from one Operator, 2 laborers in Crew 1 and 5 laborers in Crew 2. But the effective capacity (according to author opinion) results from 1 Operator, 1 laborer in Crew 1 and 3 laborers in
Crew 2. If too many laborers are working, they will occupy more space and equipment. Therefore interfere with each other. The productivity is improve only slightly. In addition, the cost for too many labors working in the construction site must be considered.

7 CONCLUSIONS

Within the paper, the current state of an approach to analyze the causes, which effect to productivity as well as prediction the progress of tunnel construction with MTBM are described. The states in order to develop the simulation module are presented. The simulation module description of the construction operation by using MTBM is established by using SysML methodology combined with AnyLogic simulation software. The use of simulation module to analyze the relationship between soil conditions and microtunnelling productivity is presented.

The simulation module named MiSAS (Microtunnelling: Statistics, Analysis and Simulation) was recently developed. The applied simulation environment is AnyLogic simulation software. The MiSAS is based on Java and utilises discrete event, system dynamic method, which are supported by the AnyLogic simulation software. The analysis different type of disturbances, soil composition to rate of progress or prediction the progress are integrated in the MiSAS. The MiSAS simulation module and the results will be presented in the near future.

REFERENCES


TBM Performance Prediction by Process Simulation

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Abstract

Tunnelling processes are complex and therefore difficult to plan. In order to avoid unplanned downtime, it is necessary to analyse the tunnelling process during planning. Process simulation is an efficient tool that allows to analyse the productivity of the tunnelling operations in advance. Based on a functional analysis of different projects, a generalized function model has been developed. Focusing on logistical processes, interdependencies and downtime patterns are analysed.

In this paper, the authors will present a modular simulation approach, which will be used in the next step to simulate the supply chain of the system. The consequences of possible process interruptions and disturbances are shown within a representative example. In order to analyse how tunnelling is affected by the TBMs supply chain, several disturbances are triggered within the model. By observing their consequences, planners can estimate the need for additional logistic equipment or system changes.

Keywords: Modular simulation, TBM, tunnelling, performance prediction
1 INTRODUCTION

Process Simulation has been rarely used as a planning instrument in TBM tunnelling. Most studies were confined to an academic surrounding although there have been some very fruitful collaborations where academic knowledge benefitted actual tunnelling projects in terms of utilizing simulation to compare different technical options [1]. Simulation as a tool to develop project scenarios using a probabilistic approach based on macroscopic data is more widely applied in project management [2]. In order to make the use of simulation attractive to the industry, the complexity and modelling effort must be reduced. One way to achieve this is by using a modular model structure [5]. In order to develop the models architecture, systematic modelling techniques such as Process Oriented Analysis, the Delft Systems Approach or IDEF0 are to be applied [8]. The multi method simulation software Anylogic by 

XJtechnologies is used to implement the proposed models and run simulations with them. While on one hand observing subsystem behaviour to optimize technical details, simulation can also be applied to generate a macroscopic view of a project over its whole duration [9], [10]. Especially the effect of different disturbances and their interacting influences on the project duration allows closing the gap between deterministic planning and experience based estimations on tunnelling performance. To demonstrate the capabilities of such a simulation system, the possible application for the analysis of a TBMs supply chain, including the segment transfer, is introduced.

2 MODELLING APPROACH

In order to develop a modular model consisting of exchangeable components, it is necessary to follow a standardized modelling approach [7]. Since the performance of a TBM is not only determined by its technical layout but also largely by organizational influences, its actual control mechanisms must be modelled as well. Comparable requirements for systems modelling outside the construction industry can especially be found in areas where highly complex systems are developed such as software, defence systems or modern production plants. The Process Oriented Analysis (POA) is widely used, especially in Europe, to gain a quick overview of the processes within a company [12]. The POA model consists of flow diagrams reflecting the architecture and state diagrams showing the behaviour.
The Delft Systems Approach has been successfully applied to analyse industrial systems such as airplane factories or container terminals [14]. A function analysis is providing the functional structure of the system, but no time dependent information. The IDEF0 modelling methodology has been widely applied to support the structured function analysis of defence systems [13]. All three methods focus on understanding the architecture and functional structure of systems, while the actual process level is only considered by POA. Modern simulation tools like Anylogic offer a good variety of methods to model the actual processes quite intuitively but require a systematic approach for the development of the overall model architecture. However as all of these methods have originally been designed for the purposes of different specific industries and thus offer different advantages and drawbacks for TBM simulation, a hybrid approach is chosen.

**Figure 1:** Generalised structure and ports of a function

Figure 1 shows the internal structure of the standardisation used to describe the functions of TBMs. It originates from the Delft Systems Approach. Since this does not specify how the actual processes are modelled and how their control flow is exchanged, this is done following the POA approach using statecharts. This basic structure of a function, derived from above methodologies, provides rules for the decomposition of material flows, resource administration and control structures. Physical flows can only be exchanged between functions within the same parent object or the direct parent object. Each function contains its control structure which is interacting only with the functions own subfunctions and with the parent functions control structure.
The control structure is responsible for directing operations, distributing material and assigning resources to processes [15]. In case resources are available to different functions of the same or different hierarchy levels, they must be modelled in the parent structure of the highest hierarchy level of their usage. The functional analysis follows a top down approach, starting with the top level function which is decomposed step by step revealing the individual subfunctions [4] (see Figure 2).

A functional model still does not contain any information about time dependent system behaviour. Behavioural information within a function is added using state charts and flow objects, providing the systems time dependent behaviour. Statecharts offer a very clear way for behavioural description [3], [6]. They support hierarchical structures which often greatly increases clarity.
3 MODULE EXAMPLE

3.1 The Segment Transfer System

As one of the most promising applications for discrete event simulation in TBM tunnelling is the optimization of the supply chain, one of its functions, namely the segment transfer is shown in detail within the next section. Choosing the right segment transfer system is a frequent task for designers and planners. The transfer of segments from a vehicle of any kind to the ring building area can be done in various ways. Which way is chosen, has, among other consequences, a large impact on the vehicles waiting time within the TBMs backup system. This paper compares a direct segment crane to a system consisting of a crane and a segment feeder.

![Figure 3: Schematic representation of alternative segment transfer systems](image)

Segments which are transported using the segment feeder will be unloaded from the vehicle by the segment crane and placed onto the last position of the segment feeder. The feeder works as a buffer and hands them over to the erector (figure 3 solid path). Since the feeder can hold one whole ring, the waiting time for the transport vehicle is only determined by the unloading sequence.

In case a direct crane is used, a segment can only be handed over, as soon as the previous one has been picked up by the erector. Segments can be buffered by unloading them onto the invert first and then delivering them to handover from here (figure 3 dotted line). In this case both, the unloading process and the delivery process, compete for the same resource, namely the crane. Therefore, the waiting time is also being influenced by the ringbuilding process.
3.2 Individual Module Comparison

It is obvious that the two different transfer structures differ with regards to the function model. While the model of the segment feeder system consists only of the functions for unloading and handover, the functional structure of the segment transfer using a direct crane is more complicated. Due to the processes related to buffering segments in the invert, there are more steps involved. This is a noteworthy example for the importance of a performing functional analysis of the simulated system instead of a structural one. The structure of the direct crane system seems simpler, containing the crane as the only active component. But the structure of processes taking place is more complex. The internal structure of the functions is depicted in figure 4.

![Diagram of internal function structure of different transfer systems](image)

**Figure 4:** Internal function structure of different transfer systems

4 SIMULATION ENVIRONMENT

The implementation of the TBM supply chain model will be illustrated in this section. The supply chain simulation model is comprised of two main modules: the general tunnelling cycle and the segment supplying cycle. Their interaction and the detailed segment transfer are currently analysed in separate models which will be integrated in future.

The general tunnelling cycle is simulated based on statecharts. The main elements of a state chart are states and transitions. Each state (rectangular) indicates a condition and will be connected to other states by using transitions (arrow). Branches (diamond) are used to represent a decision in a statechart.
Figure 5: General Tunnelling cycle [11]

Figure 5 illustrates the general tunnelling cycle. The four states of this cycle are: Excavation (1), RingBuild (2) and Stoppages (3, 4). The Stoppages are distinguished in planned and unplanned standstills. The tunnelling cycle starts with Excavation (1). After Excavation, in case no disturbances happened and enough segments have been transported to the backup system, the RingBuild (2) will start. The segment supplying cycle is modelled using standard elements of the Anylogic Enterprise Library and Rail Library (see Figure 6).

Figure 6: Segment supplying cycle

A train is used to supply segments from the jobsite to the backup system. At the beginning of tunnelling, the train enters the cycle (a) and moves to the gantry crane to be loaded (b and c). Then it moves to the backup system (d). If the previous ring is not yet built completely or the segment crane (g) is disturbed, the train must wait to be unloaded (e). After RingBuild is finished or the segment crane functional again the train can be unloaded. The unloaded train moves back to the job site to be loaded again (j).
5 CHARACTERISATION OF THE SIMULATED PROJECT

The following case study is based on the tunnelling of a 8000 meters stretch. The tunnel diameter is ten meters and a ring consists of six segments and one key segment (6+1). The segments width is two meters. Five different geological formations can be found along the planned alignment, leading to different excavation rates (table 1). The time distribution of the excavation process is assumed to be triangular.

Table 1: Excavation rates in different sections

<table>
<thead>
<tr>
<th>Section</th>
<th>Length [m]</th>
<th>Excavation speed [mm/min]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Min.</td>
</tr>
<tr>
<td>1</td>
<td>1500</td>
<td>17</td>
</tr>
<tr>
<td>2</td>
<td>500</td>
<td>20</td>
</tr>
<tr>
<td>3</td>
<td>1200</td>
<td>30</td>
</tr>
<tr>
<td>4</td>
<td>4600</td>
<td>50</td>
</tr>
<tr>
<td>5</td>
<td>200</td>
<td>15</td>
</tr>
</tbody>
</table>

After excavating two meters, the ring building process starts. Its duration is assumed to be triangular with minimum, maximum and mode of 40, 50 and 45 min. After every three rings the TBM supply lines and other installations must be inspected and extended. To simulate this, the tunnelling stops and the planned stoppage phase starts (figure 5). Another planned stoppage is scheduled after every completed 100 cycles. In this phase the conveyor belt and its installation is inspected and extended. The durations of planned stoppages are shown in table 2.

Table 2: Planned stoppages duration

<table>
<thead>
<tr>
<th>Process name</th>
<th>Duration [min]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Min.</td>
</tr>
<tr>
<td>Pipes extension</td>
<td>120</td>
</tr>
<tr>
<td>Conveyor band extension</td>
<td>360</td>
</tr>
</tbody>
</table>

The distance between the gantry crane and the shaft entry (tunnelling start point) amounts to 100 meters. Table 3 illustrates the working parameters for the segment supply which are modelled as a triangular distribution.
Table 3: Segment supplying parameters

<table>
<thead>
<tr>
<th>Name</th>
<th>Min.</th>
<th>Max.</th>
<th>Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>Train speed [km/h]</td>
<td>5</td>
<td>10</td>
<td>8</td>
</tr>
<tr>
<td>Loading time (gantry crane) [min]</td>
<td>18</td>
<td>25</td>
<td>21</td>
</tr>
<tr>
<td>Unloading time (segment crane) [min]</td>
<td>30</td>
<td>40</td>
<td>33</td>
</tr>
</tbody>
</table>

6 SIMULATION RESULTS

6.1 Influence of Segment Buffer Size

The first simulation experiment focuses on the effect of an increased segment buffer size. In a first simulation run the backup contains a buffer for one ring. Disturbances within the supply chain directly affect the tunnelling performance. That entails, one ring segment per cycle is supplied to the backup system and built. For the next cycle another ring segment is transported. The left side of figure 7 shows the simulation results for 8000 m tunnelling without an extra buffer. Total tunnelling time amounts to 466 days. The time distribution is shown in table 4. In total about 10% of the project time are lost due to disturbances affecting the supply chain and tunnelling operations. The tunnelling performance is only limited by the supply chain after reaching a higher distance than without the buffer.

Table 4: Process time distribution without and with buffer

<table>
<thead>
<tr>
<th>Process name</th>
<th>Duration [days]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>No buffer</td>
</tr>
<tr>
<td>Excavation</td>
<td>155</td>
</tr>
<tr>
<td>RingBuild</td>
<td>125</td>
</tr>
<tr>
<td>Stoppage (planned)</td>
<td>141</td>
</tr>
<tr>
<td>Stoppage (unplanned)</td>
<td>45</td>
</tr>
</tbody>
</table>

In the next step this model is simulated with an additional buffer for one ring within the backup system. The simulation results show that in this case, the makespan is reduced to 433.5 days.
It is obvious that the existence of a segment buffer within the backup system helps to
isolate the TBM operation from disturbances in the supply chain. This way the
amount of downtime due to supply chain problems can be reduced by roughly two
thirds.

6.2 Comparison of Segment Transfer Systems

Furthermore, different segment transfer methods have been compared. The simulation
experiments in this case study show that, from a logistic point of view, the segment
feeder is clearly advantageous compared to a direct crane.

Figure 8: Unloading duration with different segment transfer types

There might be other areas (i.e. invert cleaning) with conflicting requirements. Being
able to quantify this advantage in terms of time saving allows planners to better
balance these conflicting requirements. Figure 8 shows the varying waiting time of
the train.
7 CONCLUSION AND OUTLOOK

A tool which allows planners to compare the performance of different systems and subsystems can deliver valuable information to support decisions. Therefore, the next steps of this research will work towards practical and easy exchangeability of modules within the simulation environment. To generate realistic simulation results, it is necessary to identify key performance factors which are used to parameterize the model. It is crucial to determine the relevant parameters which are affecting performance [4]. To estimate project completion times or overall jobsite efficiency, macroscopic indicators can be used. But if process simulation shall be a tool to actually improve technology, the simulation study must go down to a high level of detail to make causes for delays visible. It is possible to visualize the consequences of technical problems or other disturbances within the system and to test possible countermeasures virtually for their efficiency.

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Back Analysis and Inverse Problems
Application of the Extended Kalman Filter for Soil Parameter Identification during Tunnel Excavation

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Abstract

The numerical tunnel excavation model is represented by a nonlinear state-space system. The system state, a vector whose elements are the soil parameters, is not known exactly \textit{a priori}. For identification of the system true state, the extended Kalman filter (EKF) is employed using a number of synthetic settlements at observation positions. The EKF propagates the \textit{a priori} state estimate and covariance matrix based on the stationary state transition equation. Provided that the new calculated data are available, the state estimate and covariance matrix are updated to obtain the \textit{a posteriori} estimate and covariance matrix of the state. These state estimate and covariance matrix will then be assigned to the new \textit{a priori} state estimate and covariance matrix. As the filter iterates, the state estimate gets closer to the true state and the covariance becomes smaller until least squares convergence criterion is ascertained. The finite element (FE) model is used reliably for the identification process. However, for such a complex geotechnical analysis, long computational time is a drawback. To speed up the process, an artificial neural network model of the finite element model is suggested.

\textbf{Keywords:} Extended Kalman filter, parameter identification, mechanized tunneling
1 INTRODUCTION

1.1 State-space Representation for Parameter Identification

The tunnel excavation is represented as a static non-linear Gaussian system of state transition equation and observation equation. The system state is a vector whose elements are the quantities of interest - the soil parameters. This state is not known exactly. For identification of the system true state, the state transition equation (Eq. 1a) is formulated as stationary, i.e. the next state is merely the current known state, disturbed by an amount of uncertainty. As this state is fed to the observation equation (Eq. 1b), which is a mathematical approximation of the tunnel excavation model, it produces a set of calculated settlements at a number of specified positions around the tunnel chamber and on the tunnel surface. This observation equation is also contaminated with noise characterizing the inexactness of the measurement procedure.

\[
\begin{align*}
\mathbf{x}(k+1) &= \mathbf{x}(k) + \mathbf{w}(k), \\
\mathbf{y}(k) &= \mathbf{h}(\mathbf{x}(k)) + \mathbf{v}(k),
\end{align*}
\]

where \( \mathbf{x} \) and \( \mathbf{y} \) represent the system state and observation data, respectively. The state transition noise \( \mathbf{w} \) and observation noise \( \mathbf{v} \) are uncorrelated and white Gaussian having zero-mean and covariances \( \mathbf{Q} \) and \( \mathbf{R} \), respectively.

The Kalman filter method was developed by Rudolf E. Kalman in early 1960s [1]. Since that time it has been widely employed in signal processing, mechanical systems, etc. as a very successful method for state estimation. Since late 1980s, some applications of the Kalman filter method have also arisen in the field of geotechnical engineering for parameter identification of elastic and ideally elastoplastic material parameters [2, 3, 4], in situ stresses in rock mass [5], and geometrical parameters [6]. According to the categorization of back analysis methods introduced by Giorda [7], the Kalman filter belongs to the probabilistic back analysis approach. In this work, we present the use of the EKF to identify a set of important geomechanical parameters of the Hardening Soil constitutive model [9] of the homogeneous soil surrounding a tunnel under excavation.
1.2 The Extended Kalman Filter

The a priori state estimate and its covariance matrix at time $t_{k+1}$ given observation data up to time $t_k$ are estimated as

$$\hat{x}(k+1\mid k) = E[x(k+1)\mid y(1), y(2), \ldots, y(k)],$$

$$P(k+1\mid k) = E \{x(k+1) - \hat{x}(k+1\mid k)\} \{x(k+1) - \hat{x}(k+1\mid k)\}^T.$$  

Whenever observation data at time $t_{k+1}$ are available, the a posteriori estimate and its covariance matrix, which better represent the model under study, can be estimated as

$$\hat{x}(k+1\mid k+1) = E[x(k+1)\mid y(1), y(2), \ldots, y(k+1)],$$

$$P(k+1\mid k+1) = E \{x(k+1) - \hat{x}(k+1\mid k+1)\} \{x(k+1) - \hat{x}(k+1\mid k+1)\}^T.$$  

The EKF estimates a posteriori mean and covariance based on the recursive least squares estimation method. First, the EKF is initiated with best a priori knowledge of the considered model, which can be obtained by in situ tests or laboratory experiments.

$$\hat{x}(0\mid 0) = E[x(0)],$$

$$P(0\mid 0) = E \{x(0) - \hat{x}(0\mid 0)\} \{x(0) - \hat{x}(0\mid 0)\}^T.$$  

The initial state of the model parameters is chosen based on engineering experience or preliminary examination of the geotechnical model under study. The closer are the initial parameter values to the true ones, the better it is to initiate the EKF. In case of lack of preliminary knowledge, the initial covariance matrix is assigned rather arbitrarily large due to the lack of confidence in the choice of the state vector.

Before observation data of the model are available, the EKF propagates the mean and error covariance of the state through time. The time updates of the mean and covariance of the state are calculated as

$$\hat{x}(k+1\mid k) = \hat{x}(k\mid k),$$

$$P(k+1\mid k) = IP(k\mid k)I^T + Q,$$

(2a)

(2b)

where $I$ is the state transition matrix which is identity because we formulate the state transition equation (Eq. 1a) as stationary.
As soon as the observation data are available, measurement updates of the state and covariance can be performed following the below equations

\[
\hat{x}(k+1|k+1) = \hat{x}(k+1|k) + K(k+1)\{y_{\text{exp}} - h(\hat{x}(k+1|k))\}, \tag{3a}
\]

\[
K(k+1) = P(k+1|k)H^T(k+1)\{H(k+1)P(k+1|k)H^T(k+1) + R\}^{-1}, \tag{3b}
\]

\[
P(k+1|k+1) = P(k+1|k) - K(k+1)H(k+1)P(k+1|k), \tag{3c}
\]

where \(H\) is the sensitivity matrix, whose elements are the partial derivatives of observation data with respect to the state variables, \(K\) is the Kalman gain matrix, and \(y_{\text{exp}}\) is the expected observation data.

In the implementation of the EKF, the error covariance matrix of estimation \(P\) is enlarged with a modification weight \(W\) in every iteration to obtain fast convergence as proposed by Hoshiya et al., 1993 [4]:

\[
P(k+1|k+1) = P(k+1|k+1)W. \tag{4}
\]

The time update equations (Eq. 2a and 2b) and measurement update equations (Eq. 3a, 3b, and 3c) are evaluated iteratively until convergence is ascertained. In every iteration, the EKF attempts to minimize the cost function, which is the trace of state error covariance matrix \(P(k+1|k+1)\) due to the recursive least squares optimality criterion that the EKF employs. Thus, when the EKF converges, the cost function is minimized after a number of iterations. However, as we have artificially scaled up the covariance matrix by a chosen weight as described in Eq. 4, this cost function can not be used to examine convergence in a minimizing sense. Instead, we examine the original weighted least squares cost function below along iterations:

\[
J(k+1) = \{y_{\text{exp}} - h(\hat{x}(k+1|k))\}^T R^{-1}\{y_{\text{exp}} - h(\hat{x}(k+1|k))\}. \tag{5}
\]

2 APPLICATION

2.1 The Artificial Neural Network as the Forward Model

The FE model was provided by the C2-subproject within the collaborative framework of the German Research Foundation funded SFB837 project [8]. In this model, which is named the Scenario 1, a shield tunnel boring machine (TBM) advancing in homogeneous soil is modelled using the FE analysis software PLAXIS 3D 2011. The homogeneous soil behaves according to the Hardening Soil constitutive model [9]. For detailed description of the tunnel excavation FE model the reader is advised to
refer to the paper [10]. A set of stiffness and failure parameters is chosen for identification: stiffness for un-/reloading $G_{ur}$, secant stiffness in standard drained triaxial test $E_{50}$, cohesion $c$, and friction angle $\phi$.

The FE model is used reliably for the identification process owing to the application of the advanced material model and the computational program. However, for such a multi-step nonlinear plasticity FE analysis, long computational time is a drawback. This time-consuming running of the forward model causes one to spend very long time to obtain the estimation results, thus it makes the process of adjusting the filter settings more tedious. To overcome this difficulty, we build an artificial neural network (ANN) model using calculated data produced by the finite element model. The supervised-learning takes a number of uniform random sets of the Hardening Soil parameters, in which every parameter is in a certain lower and upper bounds, as inputs, and FE analysis response of settlement at the two observation positions (taken after every second excavation steps) as targets for training the ANN model.

A single-hidden-layer multilayer perceptron (MLP) [11] is chosen for the network architecture. In order to represent the nonlinear relationship between the targets and the inputs we select the hyperbolic tangent sigmoid function for the activation function of each neuron in the hidden layer, and linear function for the activation function of each neuron in the output layer. The synaptic weights are trained using the Levenberg-Marquardt backpropagation implemented in MATLAB Neural Network Fitting Tool.

Network output is evaluated by a series of nested activation functions linking neurons in the input layer, through the hidden layer, to the output layer

$$h_o(x) = \varphi_o \left( \sum_k \omega_{ok} \varphi_h \left( \sum_i \omega_{ki} x_i \right) \right),$$  \hspace{1cm} (6)

where $\varphi$ is the activation function, $\omega_{ok}$ is the synaptic weight from neuron $k$ in the hidden layer to the output neuron $o$, similarly $\omega_{ki}$ is the synaptic weight from the neuron $i$ in the input layer to the neuron $k$ in the hidden layer, and $x_i$ is the $i$th element of the input vector $x$.

Quality of the trained network can be validated by the mean squared error (MSE) performance index and the regression plot. MSE is the average squared difference between network outputs and targets, the lower value of MSE means that the network is better trained. The regression plot shows the correlation between the network outputs and targets, regression value closer to 1 means a better fit.
2.2 Results

Identification results obtained with the ANN model are not exactly the same as those obtained with the FE model. Compared with identification of the parameters with the FE model (Fig. 1(b)), the parameters identified using the ANN model (Fig. 1(a)) converge faster to the true values. This is because the ANN model is smoother than the FE model owing to the use of the smooth activation functions. Therefore, approximation of the sensitivity matrix $\mathbf{H}$ of the ANN model is more accurate which makes the EKF more easily to arrive at the exact estimates of the parameters. Faster and smoother convergence rate of identification with ANN model compared with identification with FE model can also be observed from the plot of progressing cost function $J$ over iterative steps (Fig. 2).
However, no matter how good the convergence with the ANN model is, we should repeat the identification procedure using the FE model since it is the numerical model we want to fit and use for further analysis in other sub-projects. To this end, the use of the ANN model has great advantage in setting up and re-configuring the EKF which is a very time-consuming task because of long computation time of the FE model.

Due to low sensitivity of the cohesion $c$ to the measurement data, the EKF showed difficulty to drive this parameter to the true value. Therefore, at the moment we exclude the cohesion $c$ from the parameter list that needs identification.

3 CONCLUSION

Fast and robust convergence of the parameter identification process has shown that the EKF has prospects to be a reliable method for material parameter identification in tunnel excavation and geotechnical engineering in a broader sense. The important advantageous feature of the EKF method, namely the utilisation of the model and measurement uncertainties, which cannot be avoided in large scale geotechnical structures, makes this method very attractive and powerful.

A disadvantage of the EKF for parameter identification of nonlinear systems is that the system needs to be linearized about the current state during the whole identification process. The linearization of the system is prone to inaccuracy specially for highly nonlinear systems. In further work, the authors will investigate and employ other derivatives of the Kalman filter, such as the unscented Kalman filter and the ensemble Kalman filter. These new filters obviate the need for linearization by applying directly nonlinear transformations of an ensemble of individual representatives of the state means and covariances.

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Soil-Model Parameters Identification via Back Analysis for Numerical Simulation of Shield Tunneling

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Abstract

This paper presents a three-dimensional numerical model for analyzing via finite element method (FEM) the mechanized tunneling in urban areas. The numerical model is meant to represent the typical characteristics of the tunneling process by means of slurry shield tunnel boring machine (TBM). The Hardening soil (HS) model as it is implemented in the finite element code PLAXIS 3D, is employed for assessing the predictions made regarding the tunnelling construction process. Further, a parameter optimisation technique (direct back analysis) is applied to examine the possibility to fit simulated results and observed (measured) data, expressed in the form of displacements. A sensitivity analysis (SA) has been carried out for ranging the chosen for the back analysis soil constitutive parameters. It is shown based on the applied back analysis procedure that in case the numerical model is too large it is worth to explore equivalent reduced numerical model for soil–model parameter identification.

Keywords: Mechanised tunnelling, numerical simulation, model parameters identification, direct back analysis, particle swarm optimisation (PSO)
1 INTRODUCTION

Besides economic viability, the objective by a tunnelling in urban areas is to keep the ground deformations, especially the surface settlements, as low as possible. Therefore the closed face shield tunnelling has become a well-established tunnel construction method especially in urban areas, because of its relatively higher excavation speed, and relatively low — 0.5% or less — average amount of the ground losses. Examples of 3D numerical simulations of mechanised tunnelling are given in [8, 4] among many others. In order to have an adequate numerical model for making reliable predictions it is required to know the soil–model properties along the investigated tunnel length. This is of main importance in the cases where there is significant disagreement between the expected (i.e. a priori calculated) and the real measurements during tunnelling, due to e.g. change of the soil layers or different type of obstacles in front of the tunnel face (see [7]). In the present paper a back analysis procedure is proposed and applied to a mechanised shield tunnelling project in order to assess the possibility to identify soil-model parameters based on observations during the tunnel excavation.

2 SIMULATION OF MECHANISED SHIELD TUNNELLING

Detailed description of the relevant physical subsystems needed to be modelled within the simulation of mechanised tunnelling by means of slurry shield tunnel boring machine (slurry shield TBM) is given in [9]. The investigated tunnelling project consists of shallow and deep tunnels of different length whose vertical planes of symmetry coincide. The geometry (cylindrical shape of the tunnels), material properties, initial and excavation conditions are considered symmetric with respect to the vertical plane containing the tunnel axes of symmetry and therefore only one-half of the model is analysed. In Figure 1 there are given the model dimensions and the finite element discretization.

In Table 1 are given the used input parameters of the adopted HS model [1, 10]. The slurry shield TBM is 9 m long, simulated via plate elements. Plate elements are also used for the concrete tunnel lining. The parameters of the linear elastic model assigned to the plate elements are given in Table 2.

The action of the grout pressure on the surrounding subsoil is simulated via non-uniformly distributed load applied to the soil elements by deactivating the plate elements at the place grouting acts. The used values are from 145 kN/m² at the tunnel
crown to 230 kN/m$^2$ at the tunnel invert. The support pressure at the tunnel face is simulated by means of a non-uniformly distributed pressure, 115 kN/m$^2$ at the tunnel crown and 200 kN/m$^2$ at the tunnel invert.

The contact between the shield skin (plate elements) with the surrounding ground (soil elements), and between the tunnel lining (plate elements) and the ground is simulated via soil shear strength reduction with 40% at the contact zone. To simulate the soil-structure interaction a special joint elements called in PLAXIS interfaces are applied to the plate elements on their side in contact with the soil.

The excavation process is modelled by means of a quasistatic formulation of the corresponding mathematical model that results in a step-by-step procedure. In the first calculation phase the initial conditions are applied according to the $K_0$ procedure in PLAXIS and employing Jaky’s equation $K_{0nc} = 1 - \sin \varphi$, [3]. The steps composing a single excavation stage are: excavation of the soil in the tunnel front, deactivation of the finite elements at along 1.5 m; application of face support pressure at the tunnel face; activation of the TBM shield, i.e. activation of the plate element of 1.5 m; application of grouting pressure at the back of the TBM; installing (activation) new concrete lining ring along 1.5 m.

Figure 1: Dimensions of the main FE-model (left) and FE discretization (right).
Table 1: HS–model parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\varphi$</td>
<td>35.0</td>
<td>[$^\circ$]</td>
</tr>
<tr>
<td>$\psi$</td>
<td>5.0</td>
<td>[$^\circ$]</td>
</tr>
<tr>
<td>$c$</td>
<td>10.0</td>
<td>[kN/m²]</td>
</tr>
<tr>
<td>$E_{ref}$</td>
<td>35 000</td>
<td>[kN/m²]</td>
</tr>
<tr>
<td>$E_{oed}$</td>
<td>35 000</td>
<td>[kN/m²]</td>
</tr>
<tr>
<td>$E_{ur}$</td>
<td>100 000</td>
<td>[kN/m²]</td>
</tr>
<tr>
<td>$p_{ref}$</td>
<td>100</td>
<td>[kN/m²]</td>
</tr>
<tr>
<td>$m$</td>
<td>0.70</td>
<td>[-]</td>
</tr>
<tr>
<td>$R_f$</td>
<td>0.90</td>
<td>[-]</td>
</tr>
<tr>
<td>$\nu_{ur}$</td>
<td>0.20</td>
<td>[-]</td>
</tr>
<tr>
<td>$\gamma_{unsat}$</td>
<td>17.0</td>
<td>[kN/m³]</td>
</tr>
<tr>
<td>$R_{inter}$</td>
<td>0.60</td>
<td>[-]</td>
</tr>
</tbody>
</table>

3 SOIL PARAMETER IDENTIFICATION

3.1 Sensitivity Analysis

Solving numerically, e.g. via finite element method (FEM), nonlinear elasto-plastic 3D problems related to mechanised tunnelling simulations is often expensive with regard to computational resources and time. This yields difficulties in applying back analysis to identify model parameters. A helpful tool to assess the model performance is the sensitivity analysis (SA) performed prior or posterior the back analysis. SA can be used to a) quantification of the numerical model; b) evaluation of the importance of each input parameter and c) of the confidence level of the identified input parameters. In the present study it is used a derivative based local SA. The scaled SA analysis indicates the amount of information provided by the $i$-th observation for the estimation of the $j$-th parameter. The forward- finite difference scheme to calculate the

Table 2: Parameters of the employed linear elastic model

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Lining</th>
<th>TBM</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>$d$</td>
<td>0.20</td>
<td>0.35</td>
<td>[m]</td>
</tr>
<tr>
<td>$E$</td>
<td>30 000</td>
<td>210 000</td>
<td>[MPa]</td>
</tr>
<tr>
<td>$\gamma$</td>
<td>24</td>
<td>38</td>
<td>[kN/m³]</td>
</tr>
<tr>
<td>$\nu$</td>
<td>0.10</td>
<td>0.30</td>
<td>[-]</td>
</tr>
</tbody>
</table>
scaled sensitivity $SS_{i,j}$ reads:

$$SS_{i,j} = \left( \frac{\Delta y_i}{\Delta x_j} \right) x_j = \left( \frac{y_i(x_j + \Delta x_j) - y_i(x_j)}{\Delta x_j} \right) x_j. \quad (1)$$

For the purpose of this study $\Delta x_j = 0.1 x_j$. The overall model sensitivity to a given model parameter $x_j$ is assessed by a composite scaled sensitivity defined as [2]:

$$CSS_j = \sqrt{\frac{1}{n} \sum_{i=1}^{n} (SS_{i,j})^2} \quad (2)$$

where $n$ is the number of the observations (measurements).

### 3.2 Soil Model Parameters Identification

In this particular geotechnical problem – mechanised tunnelling in homogeneous, normally consolidated soil – we are interested in identification of the basic constitutive input parameters related to the soil stiffness $E_{50}^{ref}$, $E_{oed}^{ref}$, $E_{ur}^{ref}$ and to the soil strength $\phi$, $\psi$, $c$ of the adopted HS model.

The friction angle and dilatancy angle are assumed to be related as $\psi = \phi - 30^\circ$, and $\psi = 0$ if $\phi < 30^\circ$. According to the experimental results presented in [11] and [12] we use the following empirical relations:

$$E_{oed}^{ref} = E_{50}^{ref} < \frac{1}{2} E_{ur}^{ref} \quad (3)$$

Subsequently the input parameters of the HS model to be identified directly are reduced to four, namely $\phi$, $c$, $E_{oed}^{ref}$ and $E_{ur}^{ref}$.

In order to perform the back analysis we have used synthetic measurements. Such measurement data were collected based on the simulation of the excavation of solely the shallow tunnel. It means we solved the forward problem as described above with geometry shown in Figure 1 and model parameter values given in Table 1 and Table 2. We have extracted the vertical displacements at points $O_{12}$ and $S_{12}$ after each second excavation stage (i.e. each 3rd meter) from excavation stage 6 to the 30th excavation stage. Therefore we have $N = 2 \times 13 = 26$ observations on hand for identification of the chosen model parameters.

Next, because the solution of the main model needs too much computation time it was decided to use for performing the model parameter identification a sub–model,
Figure 2: Vertical displacements in the observation points O₁₂ and S₁₂ during the excavation calculated with the small FE-model, and with the main FE-model.

called hereafter "small model" that has the same performance as the main model, Figure 2. The small model geometry and the finite element discretization are shown in Figure 3.

For the unknown soil constitutive parameters we have defined very conservatively a search space \((0.5 \bar{x}_j; 1.5 \bar{x}_j)\) for all parameters except \(\varphi\) whose search interval is \((0.9 \bar{\varphi}; 1.3 \bar{\varphi})\), where \(\bar{x}_j\) are the nominal parameters values used to obtain the synthetic observation data. The mean squared error is used to formulate the objective function for the optimisation problem:

\[
f(X) = \frac{1}{N} \sum_{i=1}^{N} \left( \gamma_{i}^{calc}(X) - \gamma_{i}^{meas} \right)^2
\]

with \(X = (\varphi, c, E_{ref}^{\text{ref}}, E_{dur}^{\text{ref}})\) – vector of the input parameters of the HS model to be identified; \(\gamma_{i}^{calc}(X)\) – calculated data, i.e. vertical displacements obtained by solving the forward problem using the parameter set \(X\); \(\gamma_{i}^{meas}\) – measured data i.e. vertical displacements obtained by solving the forward problem with the nominal parameter set \(\bar{X}\), \(N\) – total number of measurements (\(N = 26\) in this case). In Figure 4 there is presented the concept of the adopted direct back analysis. For solving the optimisation problem defined as minimising the objective function in Eq. 4 with the constraint in Eq. 3 we used PSO algorithm. The PSO procedure was first introduced by [6] and has two primary operations – velocity \(V_k(t)\) and position \(X_k(t)\) update:
Figure 3: Dimensions of the small FE-model (left) and FE discretization (right).

\[
V_k(t) = \omega V_k(t-1) + c_1 r_1 (X^L_k - X_k(t-1)) + c_2 r_2 (X^G - X_k(t-1))
\]

(5)

where \( k = 1, 2, 3, \ldots, K_p \), and \( K_p \) is the total number of particles, \( X^L_k \) is the best previous position of the \( k \)-th particle at current iteration, and \( X^G \) is the best particle among all the particles in the swarm. The parameters \( c_1 \) and \( c_2 \) are two positive constants — cognitive and social parameter, respectively, or known also as acceleration coefficients, \( r_1 \) and \( r_2 \) are two independent random numbers in the range \([0, 1]\). The parameter \( \omega \) is an inertia weight [13] — linearly decreasing over the time. The used PSO parameters are listed in Table 3.

4 RESULTS

The advantage of the mechanised tunnelling is to keep the displacements in the ground as low as possible. Therefore one have to measure displacements at that places where they are expected to be large and therefore can be measured reliably with the e.g. extensometers, inclinometers, levelling instruments. In Figure 5 there are given the vertical and the horizontal displacements by the excavation of the shallow tunnel. We have selected the observation points \( O_{12} \) (on the ground surface)
and $S_{12}$ (at the tunnel invert) as they belong to the zone with significant vertical displacements. The meaning of the subscript 12 is that the observation points mark the position of the 12th excavation stage and are located at $12 \times 1.50$ m from the beginning of the tunnel.

In Figures 7 and 8 it is presented the performance of the PSO regarding the maximum and minimum values of the HS model parameters subject to identification. After 120 iterations the PSO has converged according to the stop criterion. The deviation between the optimised and exact parameters $\varphi$, $E_{oed}^{\text{ref}}$ and $E_{ur}^{\text{ref}}$ is small. Remarkable is that the parameter $c$ which has the smallest sensitivity (Fig. 6) has the largest final relative error – 22.80 % (Fig. 7), while the parameter $\varphi$ which has the largest sensitivity has the smallest final relative error – 0.91 %.

5 CONCLUSIONS

For the constitutive input parameters identification of the HS model a direct back analysis was carried out by using the PSO for minimising the disagreements between the observations (in this study synthetic) data and the simulated displacements during mechanised tunnelling. For performing the direct back analysis the FE-code 3D PLAXIS has been coupled with MATLAB where the adopted PSO (PSO with inertia, [13]) was implemented.
Table 3: PSO parameters

<table>
<thead>
<tr>
<th>PSO parameter</th>
<th>Value [-]</th>
</tr>
</thead>
<tbody>
<tr>
<td>$K_p$</td>
<td>20</td>
</tr>
<tr>
<td>$\omega_{\text{max}}$</td>
<td>0.9</td>
</tr>
<tr>
<td>$\omega_{\text{min}}$</td>
<td>0.4</td>
</tr>
<tr>
<td>$c_1$</td>
<td>0.5</td>
</tr>
<tr>
<td>$c_2$</td>
<td>1.25</td>
</tr>
<tr>
<td>$T_{\text{max}}$</td>
<td>300</td>
</tr>
<tr>
<td>stop criterion – deviation of $f(X_k)$</td>
<td>$10^{-26}$</td>
</tr>
</tbody>
</table>

Figure 5: Vertical displacements (Z-direction) by the excavation of the shallow tunnel (left) and horizontal displacements (Y-direction) by the excavation of the shallow tunnel (right).

Most tunnels in urban areas are long, linear structures, and values for model parameters can be obtained by performing field measurements or from laboratory tests. However, in tunnelling it is often technically difficult to collect sufficient data needed for reliable numerical simulation. The back calculated soil constitutive parameters help to make the numerical predictions more reliable. Such predictions can then be used for the revision and improvement of the tunnel design e.g. excavation procedure, support pressures and improvement of the properties (chemical, mechanical) of the support materials – bentonite and grout [5]. The number, the place, and the type of the measurements, are strongly dependent on the complexity of the investigated geotechnical problem.
Figure 6: CSS calculated by Eq. 2 in \( O_{12} \) (left) and in \( S_{12} \) (right).

Figure 7: Identification of the soil angle of internal friction \( \varphi \) (left) and the cohesion \( c \) (right).

Figure 8: Identification of the soil stiffness for primary loading \( E_{ref}^{oed} \) (left) and for unloading/reloading \( E_{ur}^{ref} \) (right).
One current shortage of the adopted parameters identification procedure via direct back analysis results from the iterative computation time used for repeated solution of the forward problem. By using a suitable PSO parameters like number of particles, number of maximum iterations, etc, it is possible to reduce the number of calls of the forward model improving the performance of the PSO algorithm. In addition, proper sub-modelling can be used to reduce the calculation time required to solve the forward problem.

It is foreseen to investigate the maximum distance of the TBM before the observation cross section, needed for a successful parameters identification. The next step will be to considered the uncertainties in the measurements, and their influence on the identification process. Next, the reported here investigation will be performed with a model simulating the excavation of the deep tunnel shown in Fig. 1.

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REFERENCES


Some Thoughts about Solving Non-linear Inverse Problems

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Abstract

Some automatic and numerically cheap non-linear inverse problem solution strategies are suggested on the basis of the following three interrelated simplifying methods. (i) The critical points arisen from the noise may hinder the global minimisation. To "clean" the merit function from the noise, the concept of the noise-free approximation of the real-life merit function (follower) is used. The two merit functions, the real-life and the follower (constructed with measured and simulated data, resp.) are similar on a great part of the compact parameter domain in case of small noise. Where the two merit functions are different, an error domain of the parameters is found. (ii) The interpolation of the follower merit function is possible on the basis of the computed real-life merit function values on the similarity domain assuming that the latter is invariant. The numerical work (i.e. the # of the interpolation points) can be decreased by the elimination of some parameters through sub-minimisation. (iii) The elimination may help to change the shape of the merit function. If all parameters are eliminated except one, then the transformed merit function (depending on one parameter only) can be used in minimisation, reliability testing and regularisation (if needed). The new algorithms illustrated by some examples related to the basic geotechnical testing.

Keywords: Global minimisation, sub-minimisation, reliability, parameter error
1 INTRODUCTION

The solution of the non-linear inverse problems is the global minimum of a Least Squares objective or merit function. There is a general observation that in case of real-life inverse problems, the number of the non-important local minima may increase exponentially with the parameter number due to the noise.

Generally a classical minimisation method [1] is used to find the global minimum in the following algorithm: a set of local minima are determined from various starting vectors and the smallest is selected. Then a confidence domain is determined (see e.g. [1]) assuming random noise and a linearized model.

This method is non-automatic and can be tedious if the number of the parameters is greater than even one. The concept of uniqueness of the solution is neither defined nor tested. The noise can be and is generally partly non-random. Two complementary techniques are suggested to overcome these difficulties.

2 DEFINITIONS AND METHODS

2.1 Noise-free Approximation of the Real-life Merit Function, Minimisation

The first technique is the definition of a closest noise-free merit function, called follower merit function. The solution of the inverse problem is the global minimizer $p_{\text{min}}$ of the Least Squares objective (or merit) function which is related to the norm of the error vector $h(t,p) = u_m(t_i) - u(t,p)$. In the unweighted case:

$$F(p) = \sum_{i=1}^{N}[u_m(t_i) - u(t_i, p)]^2$$  \hspace{1cm} (1)

where $t_i$ are sampling times (i=1,2..N), subscript $m$ indicates measured value and $p$ is the parameter vector containing $M$ parameters which may influence (partly linearly and partly non-linearly) the solution $u(t, p)$ of the model. The so called follower merit function $F'(p)$ is defined by the data simulated with $p_{\text{min}}$:

$$F'(p) = \sum_{i=1}^{N}h_i(p)^2 = \sum_{i=1}^{N}[u(t_i, p) - u(t_i, p_{\text{min}})]^2$$  \hspace{1cm} (2)

The $z_i$ is the noise can be expressed from the following decomposition:

$$u_m(t_i) = u(t_i, p_{\text{min}}) + z_i$$  \hspace{1cm} (3)

Using this, the two merit functions have the following relationship:

$$F(p) = \sum_{i=1}^{N}[h_i(p) - z_i]^2 = F'(p) - \sum_{i=1}^{N}[2h_i(p)z_i] + F(p_{\text{min}})$$  \hspace{1cm} (4)
Some thoughts in inverse problem solution

It is assumed that the noise is relatively small in the sense that the parameter domain has a part ‘far’ from the global minimum where $|\vec{h}_i(p)| > |\vec{z}_i|$ is met. A geometrical parameter error domain [2] is defined by the natural contour value $c$ of the follower merit function as follows irrespective of the type of the noise:

$$F'(p) = \sum_{i=1}^{N} \vec{h}_i(p)^2 \leq c \approx F(p_{\text{min}})$$

where $|\vec{h}_i(p)| \leq |\vec{z}_i|$ is met. The follower can be approximated by the real-life merit function on the complement parameter domain (Fig 1):

$$F'(p) = \sum_{i=1}^{N} \vec{h}_i(p)^2 > c$$

Having has less critical points, it is worthy to minimize the follower. This can be done far from the minima where the follower can be approximated by the real-life merit function (‘similarity domain”), and, as a result, the non-important critical points can be skipped. This domain is bounded by a contour of the follower merit function [2], its complement is a geometrical error domain where the non-important critical points cannot be skipped (which can be a confidence domain but it is independent of the type of noise).

Two possible minimisation strategies can be suggested entailing a mesh generation in the subspace in the non-linearly dependent parameters assuming the compactness of their domain. In the first algorithm a new starting vector is found by generating a local mesh (lattice) in the subspace of some ‘important’ non-linearly dependent parameters around a point where a classical minimisation method ends (see the 1st example in section 3). In the second algorithm the mesh is generated on the whole domain of the non-linearly dependent parameters, the minimum valued mesh point is bracketed (see examples 2 to 3 in section 3).
The minimal sections can be determined in both cases. The minimal distance:

\[ |D_i| \geq |z_i|, \quad \forall i \]

where \( D_i \) is the increment of the error vector on the parameter vector increment \( \Delta_i \).

The maximum distance – which is practical to be applied - is limited by the size of the parameter error domain. It is assumed that the shape of the follower (and the geometric parameter error domain) is determined basically by the model and, can geometrically be explored “in advance”.

2.2 The Split Inverse Problem, the Minimal Section

The parameter vector is \( p \) divided into two parts, \( p^1 \) and \( p^2 \):

\[ p = [p^1, p^2], \quad p^1 \in R^J, \quad p^2 \in R^{M-J} \]  

The split or hierarchical inverse problem is as follows ([3]):

\[ F(p^1, p^2) = \min!, \quad p_2 = p_2^* \]  

\[ F[a(p^2), p^2] = \min! \]

where the first minimisation is made with respect to \( p^1 \) and the second is made with respect to \( p^2 \) such that the relation \( p^1 = a(p^2) \) is determined beforehand:

The split inverse problem is equivalent to the original one if the Hesse matrix of the original objective function with respect to \( p = [p^2, p^1] \) is positive definite at least in a vicinity of the global minimum (which is assumed in this work).
Some thoughts in inverse problem solution

The statement follows from the fact that if the Hesse matrix with respect to \( p \) is positive definite then the Hesse matrix with respect to \( p^1 \) and its Schur complement (i.e. the Hesse matrix for the first and the second minimisations) are positive definite. This topologically means a non-degenerate minimum.

If the sub-minimisation is solved on the whole domain of \( p^2 \) then an \( M-J \) dimensional section \( F(a(p^2),(p^2)) \), called the minimal section of the merit function with respect to the parameter vector part \( p^2 \) is resulted. If the model depends linearly on \( p^1 \) then \( p^1 \) can be found in one minimisation step.

2.3 The Reliability Tests

The solution of the inverse problem is unique if (1) the global minimum of the noise-free merit function is single and is not degenerate and, if (2) the local minima of the follower noise-free merit function are not deeper than the minimum of the real-life merit function \( F(p_{\min}) \). The solution of the inverse problem is precise enough if the parameter error domain (or confidence domain) of the solution is contained by the parameter domain [4] to [7].

The reliability can be tested by the one-dimensional minimal section of a parameter \( p^i \), called the Critical Sensitivity Section (CSS) of \( p^i \). The projection onto the two dimensional space \( F,p^i \) is called the Critical Sensitivity Curve (CSC) of \( p^i \). Some properties of CSSs – by definition – are as follows (see Fig 1).

The CSS of parameter \( p^i \) contains all important minima, therefore, it can be used for global uniqueness test. The point of the minimal section of parameter \( p^i \) where \( p^i = p^i_{\min} \) is the global minimum \( p_{\min} \). As a result, the solution of an ill-posed inverse problem (with quasi-degenerate global minimum) can be approximated from the CSS of parameter \( p^i \) if the “good value” \( p^i_{\min} \) is known.
The intersection of the line $F(p_{\text{min}}) = \text{const}$ and the CSS provides the size of the parameter error domain defined by a level line of the follower merit function in direction of $p_i$ (see Fig 1). Concerning the classical analytical methods, the local uniqueness condition is automatically tested if the standard deviation is computed from the covariance matrix ([1]). A “distribution free” 88% confidence interval is defined by the Chebisev inequality:

$$P\left(|x_i - p_i| \geq \sigma_i \right) \leq \frac{\sigma_i^2}{a^2}$$

(11)

where $x_i$ is the expected value and $\sigma_i$ is the standard deviation of the parameter $p$

3 EXAMPLES, DISCUSSION

The suggested methods were demonstrated by validating two models for each of three tests (Table 1). The merit function value $F(p_{\text{min}})$ in a normalized form was used for the discrimination among the model-versions.

For the staged oedometric relaxation test (MRT), in model H a consolidation model and a relaxation model are summed, the relaxation is not included in model HC [5]. For the staged compression tests (MCT), in model AC a consolidation model and a creep model are superimposed, the creep is not included in model A [5, 6]. For the water pressure dissipation test a one dimensional, coupled, linear consolidation model is used, the identified initial condition is monotonic for model II and is non-monotonic for model I [7].
Some thoughts in inverse problem solution

In the first example ([4]) the model H is fitted on multistage oedometric relaxation test (MRT) data with the conjugate gradient method. Where a local minimum is found by conjugate gradients, a coarse local grid is generated in the subspace of two important parameters, in order to determine a new starting vector. The blue and the coloured lines in Figure 2 are the projections of the CSSs related to the coarse and the fine local grids onto a 2-dimensional plane, respectively.

The second example is the global minimisation of the long and short stages of the same test (MRT). The CSSs of parameter $c_v$ have two minima for the long stage and are quasi-degenerated for the short stages (Figs 3, 4).

The third example is the evaluation of the multistage oedometric compression test (MCT) with models AC and A. The CSS of $c_v$ has one minimum for model A and has two minima for model AC. The geometrical parameter error domain and the confidence intervals are comparable (Fig 4).

In the second and third examples the consideration of the creep and relaxation halves the fitting error (Figs 4, 5). The short stages were evaluated with the simplest models HC and A, the solution was “regularized” by using the CSS of $c_v$ of the short stage and by using the value of $c_v$ identified from the last, long stage. The identified compression curve points are situated “above” the standard one determined by the compression test in accordance with the expectations following from the time dependency of the constitutive law (Figs 6, 7).
The last example is the evaluation of the dissipation test with two data series measured just above the CPT tip, subtracting either a non-precise or a precise prior pore water pressure $u_0$ value from the measured data. Different solutions are found for model I with non-connected confidence domains in these two cases, moreover, the solution of model II is different from these as well (Fig 8). The solution with the smallest fitting error $F$ has the best fit by eye (and is in agreement with the solution determined by $t_{50}$ [8]). This result indicates the importance of the goodness of the model (i.e. two dimensional model is needed in tip-close position) and the model qualification (e.g. using a randomness test for the residuals).

4 CONCLUSIONS

The concept of the noise-free approximation of the real-life merit function was successfully used to elaborate a global and a local minimisation method (to skip the non-important critical points) and, to determine some mathematically precise reliability criteria. The concept of the split inverse problem was successfully used to elaborate some numerical techniques to eliminate the linearly dependent parameters, to test the reliability criteria and to “regularize” the ill-posed inverse problems (e.g. in case of too short data series). The suggested techniques were illustrated by some examples of the standard laboratory and in situ testing.

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Benchmarking of Optimization Algorithms

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² Ruhr-Universität Bochum, Laboratory of Foundation Engineering and Soil and Rock Mechanics, Bochum, Germany

Abstract

In this paper, we would like to present an approach for impartial and quantitative benchmarking of optimization algorithms with respect to characteristics induced by the forward calculation. Starting from a brief introduction into the employed approach, a strategy for optimization algorithm benchmarking and rating is introduced. Due to the professional background of the authors this rating strategy is illustrated on a selection of different search methods in regard to expected characteristics of geotechnical parameter back calculation problems.

Keywords: Optimization, Parameter Back Calculation, Benchmarking
1 MOTIVATION

For numerical simulations, it is essential to use a model parameter set which generates a realistic system response. In practice, parameter back calculation based on the direct approach is often used for this purpose in which mathematical optimisation algorithms are a critical component. Many different optimization algorithms are known and are available within the literature (for an overview, see e.g. [11] or [7]). These algorithms use a variety of approaches to perform the search for optimal parameter combinations. However, the performance and convergence of the different optimization algorithms itself varies strongly, and also depends strongly on the optimization problem to solve. Exorbitant computational costs or, in the worst case, an improper or random parameter set will be returned if an unsuitable optimization algorithm is selected. Therefore, a suitable optimization algorithm has to be carefully selected for each problem.

For optimization algorithms, the performance by means of “finding the optimum reliable” and “low computational cost” cannot be assessed in a closed mathematical form in most of the cases. However, we may use a statistical analysis of the solution obtained from optimization runs to make conclusions on some properties of the optimization algorithm and the search-performance. For this approach, the success rates of finding the optimum is treated as a stochastic value and statistical measures are applicable. Hereafter the approach used and some results are presented.

2 EMPIRICAL BENCHMARKING APPROACH

The parameter identification approach, using the direct back analysis method, consists of an iterative procedure controlled by an optimization algorithm. The model parameters are iteratively changed in such a way to achieve better agreement between the model results and the measured values, e.g. the field measurements. This agreement (or disagreement) is measured by the objective function \( f(x) \). The aim of the optimization algorithm is therefore to iteratively minimize the objective function value.

The computational cost caused by this iterative process is mainly influenced by the number of forward calculations (usually numerical simulations) requested by the optimization algorithm. The processor usage of the optimization algorithm on the other hand is usually neglectable (\(<<1\) sec). A typical single forward calculation used in Geotechnics (e.g. Finite Element model of an excavation pit) requires often 5 min
calculation time or more. Minimizing the computational cost of an optimization sequence is therefore equivalent with minimizing the number of forward calculations. For the statistical approach applied here no “real life” forward calculation has been used to avoid high calculation time. As shown in Fig. 1 the normal calculation scheme for an objective function value has been altered using a well defined and well known test function. The introduction of the test function has no influence on the iterative process controlled by the optimization algorithm. From view of the optimization algorithm an objective function value is still calculated based on a parameter vector.

As stated by [2] and also by the experience of the authors (e.g. [6, 8, 14]), many objective functions from the field of (geo-)technics have a globally convex shape, in which often secondary (locally optimal) solutions are present. Furthermore many objective functions show a certain ‘roughness’ or ‘noise’ at a smaller scale. Additionally for some parameter vectors a forward calculation can fail reproducible (e.g. no convergence in the Finite Element Method) and no objective function value can be calculated accordingly.

In view of these facts two test functions have been chosen for the benchmarking presented: Firstly the Moved axis parallel hyper-ellipsoid function which has no secondary optima and one global optimum [10]. Secondly the Ackley test function in its generalized form, which shows several secondary optima of varying objective function values and a single global optimum (see Fig. 2 and [1] for details).
Both test functions exhibit no roughness or failed parameter vectors. In order to incorporate both characteristics the original test function \( f(x) \) is superposed with a noise field \( r(x) \) according to Eq. (1) and (2). In Eq. (2) \( \text{srnd}(x) \) is a pseudo-random number generator returning equal-distributed numbers ranging from 0.0 to 1.0, while for one and the same argument \( x \) always the same number is returned. The control variable \( \tau \) is the noise scaling factor. According to Eq. (3) a parameter vector \( x \) is considered “failed” if the pseudo-random number for \( x \) is smaller or equal to a predefined failure probability \( p_f \).

\[
\begin{align*}
  f^*(x) &:= f(x) + r(x) \quad (1) \\
r(x) &:= \tau \left( \frac{1}{2} - \frac{1}{n} \sum_{i=1}^{n} \text{srnd}(x_i) \right) \quad (2) \\
\text{srnd}(x) &\leq p_f \quad (3)
\end{align*}
\]

The majority of optimization algorithms will not find the exact location of the test function global optimum \( x^* \), but will rather move asymptotically towards \( x^* \) due to underlying paradigms. The optimization sequence is considered to be successful, if the parameter set \( x_{\text{min}} \) with the smallest objective function value is located within \( \Psi \) as defined by Eq. (4).

\[
\Psi = \left\{ x_{\text{min}} \mid \| x_{\text{min}} - x^* \|_2 \leq d_{\Psi} \right\} \quad (4)
\]

The search range \( \Omega \) and \( d_{\Psi} \) has been chosen as follows (please note that the relative size of \( \Psi \) for both test functions is equal compared to \( \Omega \)):

- Ackley Test Function: \(-1.0 \leq x_i \leq +2.0\) with \( d_{\Psi} = 0.1 \)
- Moved axis parallel hyper-ellipsoid function: \(-10.0 \leq x_i \leq +20.0\) with \( d_{\Psi} = 1.0 \)

To assess the probability of which an optimization algorithm is able to converge within \( \Psi \) on a test function for given values of \( p_f \) and \( \tau \), a large number of optimization sequences is repeatedly run. For each sequence the start parameter sets are chosen randomly within \( \Omega \) and the number of forward calculation is limited to 500. The quotient of successful sequences over the total number of sequences is considered as success rate \( p \). The number of optimization sequences is increased until the success rate is stabilizing, what usually corresponds to some 10’000 runs.
3 RESULTS

The empirical benchmarking approach described above has been applied to 5 selected optimization algorithms. Namely, Monte-Carlo method (MC), a gradient descent method (GD) (e.g. [12, 11]), an evolutionary-genetic algorithm (EG) [e.g. 11], the Simplex-Nelder-Mead optimizer (SNM) [9] and the particle swarm optimizer (PSO) [5, 4]. For each algorithm both test functions have been used with \( n = \{2, 3, 4, 6, 8 \text{ and } 10\} \) unknown parameters.

The diagrams of Figure 3 show on the vertical axis the success rate \( p \) over the noise control variable \( \tau \) and, respectively, the failure rate \( p_f \). The main conclusions are:

- The MC method performs well for \( n = 2 \). For higher \( n \) it clearly suffers from the “curse of dimensionality” [3]
- The GD performs works very nice for smooth objective function topologies with no secondary optima. It performs very badly if secondary optima are present, as it is the case for the Ackley test function. The GD is also not robust to noise and failing forward calculations.
- The SNM optimizer is much more robust than the GD. Nevertheless, due to its local character the success rate for the Ackley test function is \(~50\%\) even for the ideal case of \( \tau = 0 \) and \( p_f = 0 \).
- The EG algorithm class is very popular among many researchers due to its high robustness. This robustness is also visible in Figure 3. The major drawback of this method is its need for a large number of forward calculations as also stated by [13].
- For both test function the PSO (10 particles) used shows the best performance values. It outperforms clearly all other tested algorithms including the EG method. This finding is in agreement with the experience of other researchers, e.g. [13].

The results of Figure 3 show clearly how different the optimization algorithms behave for the two test functions. This illustrates how the nonlinearity of the objective function has a strong influence to the performance of the optimization algorithm.
4 CONCLUSION AND OUTLOOK

In the present paper an approach for impartial and quantitative benchmarking of optimization algorithms has been briefly presented and applied to 5 selected optimization algorithms. Of all the optimization algorithms tested, the PSO shows the best performance. However, if an objective function with a severely different nonlinearity is present, or if the number of unknown parameters \( n \) increases strongly the PSO may be outperformed by other algorithms.

The next step will be to define a rating function based on the performance profiles to provide an objective rating method for optimization algorithms.

\[
\text{Optimization algorithm} \quad \begin{array}{c|c|c}
\text{failure probability} p_f & \text{noise scaling factor } \tau & \text{failure probability} p_f \\
\hline
\text{Monte-Carlo} & \begin{array}{c}
-2.0 \ldots +1.0
\end{array} & \begin{array}{c}
-10.0 \ldots +20.0
\end{array} \\
\text{Gradient Descent} & \begin{array}{c}
-2.0 \ldots +1.0
\end{array} & \begin{array}{c}
-10.0 \ldots +20.0
\end{array} \\
\text{Simplex Nelder Mead} & \begin{array}{c}
-2.0 \ldots +1.0
\end{array} & \begin{array}{c}
-10.0 \ldots +20.0
\end{array} \\
\text{Evolutionary Genetic} & \begin{array}{c}
-2.0 \ldots +1.0
\end{array} & \begin{array}{c}
-10.0 \ldots +20.0
\end{array} \\
\text{Particle Swarm Optimizer} & \begin{array}{c}
-2.0 \ldots +1.0
\end{array} & \begin{array}{c}
-10.0 \ldots +20.0
\end{array}
\end{array}
\]

\textbf{Figure 3:} Results of the 5 tested optimization algorithms
REFERENCES


Comparison of Optimization Algorithms for Identification of Parameter Values in Unsaturated Soils

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Abstract

Four optimization algorithms, namely the particle swarm optimization algorithm, the differential evolution algorithm and the two hybrid versions of each of these algorithms (i.e. the versions incorporating an additional routine for local search), are compared for the identification of parameter values in unsaturated soil models. The hybrid versions of the algorithms are implemented on a cluster of parallel computers with the objective of reducing runtime for computationally demanding search problems. Algorithms performance is compared in terms of convergence rate and computational time. The paper demonstrates the potential of all algorithms for the inverse analysis of tests in unsaturated soils and, in particular, highlights the effectiveness of the hybrid versions of the algorithms when implemented on parallel computers. The parallel version of the hybrid differential evolution algorithm shows the best performance in terms of both convergence rate and computational time.

Keywords: Parameter identification, unsaturated soils, particle swarm optimization, differential evolution, parallel computing, hybrid algorithm
1 INTRODUCTION

The development of constitutive models for unsaturated soils is a particularly challenging research topic, mainly because of the complex coupled hydro-mechanical behaviour of this material. This topic has attracted the interest from constitutive modellers over recent years and the literature is nowadays rich of constitutive formulations that can describe relatively well the basic aspects of the stress-strain and soil-water retention behaviour of unsaturated soils. The calibration of these constitutive formulations is however complicated by the need of selecting a large number of parameter values, which creates an obstacle to the application of these formulations into practice. The present paper compares four different optimization algorithms, which can help to identify parameter values in unsaturated soil models from laboratory and field tests. These four algorithms, which belong to the class of evolutionary algorithms, are the particle swarm optimization (PSO) algorithm [1], the differential evolution (DE) algorithm [2] and two hybrid versions of the PSO [3] and DE [4] algorithms, respectively. The two hybrid versions have been implemented on a cluster of parallel computers to reduce runtime and to demonstrate the application to the analysis of real engineering problems.

2 OPTIMIZATION ALGORITHMS

In this section, the above four optimization algorithms are briefly illustrated but the reader is invited to refer to [1-2, 4-5] for a detailed description of their characteristics.

2.1 PSO

The particle swarm optimization (PSO) algorithm is a population-based stochastic global optimization algorithm proposed by Kennedy and Eberhart [1]. In this paper, the algorithm is referred to as the basic PSO (bPSO) to differentiate it from its hybrid version described in section 2.3. The bPSO algorithm is based on the intelligence of a swarm made up of a number of particles. These particles “fly” over the search space looking for the optimum “position” corresponding to the minimum of the objective function. Each particle knows its current velocity and position, and has a memory of its best previous position. Each particle updates its position during subsequent iterations by taking into account its best previous position and the best position ever achieved across the entire swarm (the “social knowledge” of this position is shared among particles). The algorithm is characterized by the definition of updating rules, which drive the particles towards the optimal region as the algorithm progresses and, eventually, towards the final solution.
2.2 DE

Differential evolution (DE) is a simple and powerful population-based evolutionary algorithm proposed by Storn and Price [2]. In this paper, the algorithm is referred to as the basic differential evolution (bDE) algorithm in order to differentiate it from its hybrid version described in section 2.3. The bDE algorithm contains an initialization stage and an evolutionary stage, which is characterized by three evolution operators: mutation, crossover, and selection. The mutation operation serves as a search mechanism that makes the population robust. It generates a trial solution vector by adding a weighted difference of vectors to a target vector. The crossover operation has the objective of increasing the diversity of the population while the selection operation directs the search toward the optimal region.

2.3 Hybrid Versions of bPSO and bDE

The bPSO and bDE algorithms are effective in finding the global optimum of the search space but they show slow convergence rates during the latest stages of the optimization process, which can lead to a significant reduction in searching efficiency. A hybrid optimization approach is therefore adopted to improve the efficiency of the bPSO and bDE algorithms. The corresponding hybrid versions of these algorithms are here referred to as the hmPSO (hybrid moving-boundary PSO) algorithm and the hDE (hybrid DE) algorithm [3-4]. These hybrid algorithms incorporate a local search method (the Nelder-Mead simplex algorithm) to carry out quick explorations of the search space around the region of potential optima at a much lower computational cost.

2.4 Parallel Versions of hmPSO and hDE

For real engineering applications, the optimal set of parameter values can be attained by the hmPSO or hDE algorithms after thousands of objective function evaluations, which can take days or even months of computations. To reduce runtime, an asynchronous parallel version of both the hmPSO and hDE algorithms has been developed in this work using a client-server model [3-4]. The client-server model consists of three main components: a) the server, b) the particle clients and c) the local search clients. The server is the centre of the model, which is responsible for data sharing and system management. The clients are instead responsible for undertaking the actual numerical simulations (particle clients) and for running the Nelder-Mead simplex local searches (local search clients). Each client communicates with the server but there is no communication among clients (including among particle clients).
3 NUMERICAL EXAMPLES

To compare the performance of the above four algorithms (PSO, hmPSO, bDE and hDE), two optimization analyses have been executed as explained in the following. The parallel computer “Shenteng7000”, with 1288 IBM HS21SM blade servers, was used for the parallel computations. This is a Linux cluster hosted at Supercomputing Center of the Chinese Academy of Sciences in Beijing, China.

3.1 Sequential Optimization on Single Processor

The first example attempts to identify the values of five parameters in the Mualem-van Genuchten water retention and permeability model from 1D infiltration tests in an unsaturated soil column [5]. The population size is set equal to 40 for all four algorithms. A known set of model parameter values is first used to produce fictitious “experimental” data via the analytical solution of the chosen infiltration problem. These data are then interpreted by the four optimization algorithms to find the original set of model parameter values. Figure 1 and Figure 2 indicate that the hDE algorithm shows the best performance, followed by the hmPSO, bDE, bPSO algorithms. With the exception of bPSO, all algorithms find the solution with satisfactory accuracy.

3.2 Parallel Optimization on Multiple Processors

The second example attempts to find parameter values in the Barcelona Basic Model (BBM) by inverse analysis of pressuremeter tests in unsaturated soils [3]. This example provides a more realistic comparison of the optimization algorithms than the previous case. It also requires much longer computational times because each objective function evaluation involves the finite element simulation of multiple pressuremeter tests at different suction levels, which takes an average of 5 minutes. Because of this, parallel versions of the hmPSO and hDE optimization algorithms have been developed to speed up solution. Similar to the previous case, a series of fictitious “experimental” data is first produced from finite element models of pressuremeter tests at different suction levels using a given set of BBM parameter values. These fictitious experimental data are then interpreted by the optimization algorithms to search for the original set of BBM parameter values. In this case, the population size is set equal to 48 for all optimization algorithms.
**Comparison of Optimization Algorithms for Identification of Parameter Values in Unsaturated Soils**

**Figure 1:** Logarithm of the objective function against number of function evaluations

**Figure 2:** Logarithm of the objective function against computational time
Figure 3 and Figure 4 compare the efficiency of the two hybrid algorithms in terms of number of objective function evaluations and computational time. The performance of the two algorithms is very different. Figure 3 shows that the hDE algorithm finds the optimal solution after only 8754 objective function evaluations, while the hmPSO is still searching for the solution after 40000 evaluations. Figure 4 shows similar variations of the logarithm of the objective function against computational time.

![Figure 3](image1.png)  
**Figure 3:** Logarithm of objective function against number of function evaluations

![Figure 4](image2.png)  
**Figure 4:** Logarithm of objective function against computational time
4 CONCLUSIONS

The paper compares the performance of four different optimization algorithms applied to the problem of identifying parameter values in unsaturated soil models. The algorithms are the particle swarm optimization algorithm, the differential evolution algorithm and the hybrid versions of both these algorithms (i.e. the versions incorporating an additional Nelder-Mead simplex routine for local search).

Results show that the differential evolution algorithm performs generally better than the particle swarm optimization algorithm in terms of convergence rate and computational time. This is true for all three versions of the algorithm, i.e. the basic version, the sequential hybrid version and the parallel hybrid version. In particular, the results show that the parallel hybrid differential evolution algorithm may be effectively applied to the analysis of real geotechnical problems such as, for example, the calibration of unsaturated soil models from laboratory tests (e.g. from infiltration column tests) or field tests (e.g. from pressuremeter tests).

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Kalman Filter – from Control Applications to Identification in Geomechanical Problems

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Abstract

Kalman filter represents a powerful estimation tool, which enables estimation of the system/model states which are not directly available for measurement. A special property of the Kalman filter is, that even in the presence of white-noise type uncertainties it provides reliable state estimates. Kalman filter was developed first for the purpose of application in control systems. Owing to its properties with certain modifications and extensions it can be successfully implemented with static and nonlinear problems as well. In geomechanics, measurement of geotechnical material constants which constitute a material model usually represents a very difficult task even with modern test equipment. Back-analysis has proved to be a more efficient and more economic method for identifying material constants because it needs measurement data such as settlements, pore pressures, etc., which are directly measurable, as inputs. This work will present the Kalman filter with its implementation for the state estimation as an underlying problem, as well as the extended Kalman filter – local iteration procedure incorporated with finite element analysis software for parameter identification in certain geomechanical problems related to tunnel excavation. Identified parameters include elastic constants of linear elastic material models, elasto-plastic constants of inelastic material models, and model geometry parameters.

Keywords: Kalman filter, state estimation, parameter identification
1 INTRODUCTION

Development of reliable models plays an essential role in investigation of a system behavior. Under a system one can generally understand an object or a process, which may undergo changes either due to interactions with the environment or due to internal influences related to system properties and condition. In the first approach a system can be regarded as a black box (Fig. 1) with its inputs $u$ and outputs $y$, which represent the influence of environment and the system response to it, respectively.

Figure 1: General system representation

Many dynamic systems can be reliably modeled by a set of differential equations. In control system theory, this set of equations is often represented in a general matrix state space form

$$\dot{x}(t) = f(t, x(t), u(t)); \quad y(t) = h(t, x(t), u(t))$$  \hspace{1cm} (1)

by its state and output (also called the measurement) equations, respectively, where $x(t)$ represents the vector of the system states, and $f$ and $h$ are arbitrary functions of the states, inputs and outputs. In many applications the observation of states is indispensible, for example for a control system implementation, or for parameter estimation. The Kalman filter approach is addressed in this paper for the state observation purposes in dynamic control systems as well as for a static system parameter estimation.

2 KALMAN FILTER AS ESTIMATOR FOR DYNAMIC SYSTEMS

Kalman filter was first developed for implementation with control systems as well as in signal theory [1]. Under that name it has been used since the works of R.E. Kalman in the 1960s, and actually it represents a solution of the Wiener problem [2], which overcomes some difficulties, especially of numeric nature. The idea behind the Kalman filter relies on the prediction of random signals, separation of random signals from random noise and detection of signals of known forms in the presence of random noise [1]. In control systems the Kalman filter is often used as an efficient tool for estimation of the system states.
2.1 Kalman Filter for Continuous Time Systems

Given a continuous time system described by a set of linear differential equations in matrix form:

\[
\dot{x} = Ax + Bu + w; \quad y = Cx + v
\]

with state vector \( x \), inputs \( u \), outputs \( y \) and the process and measurement noise \( w, v \) respectively, satisfying:

\[
E(w) = E(v) = 0 \quad \text{(white noise)} \tag{3}
\]

\[
E(ww^T) = Q, \quad E(vv^T) = R \quad \text{(covariance)} \tag{4}
\]

the Kalman filter estimates the system states, so that the steady-state error covariance \( P \) should be minimized.

\[
P = \lim_{t \to \infty} E \left( \{ x - \hat{x} \} \{ x - \hat{x} \}^T \right) \tag{5}
\]

Kalman filter equations (6) are of a similar form as for a state observer, e.g. Luenberger observer [3]. This estimator uses the known inputs \( u \) and the measurements \( y \) to generate the output \( \hat{y} \) and the state estimates \( \hat{x} \)

\[
\hat{x} = A\hat{x} + Bu + K(y - C\hat{x}), \quad \hat{y} = C\hat{x} \tag{6}
\]

The Kalman gain \( K \) is determined by solving an algebraic Riccati equation. In (6) \( \hat{y} \) estimates the true plant output.

2.2 Discrete-time Version of the Kalman Filter

Starting point for the state estimation of a discrete-time system using Kalman filter is a state space model (6), which represents a discrete-time equivalent of the continuous time system (1):

\[
x[k+1] = \Phi x[k] + \Gamma u[k] + w[k], \quad y[k] = Cx[k] + v[k] \tag{7}
\]

where the matrices \( \Phi \) and \( \Gamma \) are determined using the matrix exponential. Process and measurement noise \( w \) and \( v \) respectively, are random sequences with zero mean (8) and no time correlation (9). These conditions in a discrete-time form are represented in a similar way as in (3):

\[
E[w[k]] = E[v[k]] = 0 \tag{8}
\]

\[
E[w(i)w^T(j)] = E[v(i)v^T(j)] = 0, \quad i \neq j. \tag{9}
\]
Covariances or mean square noise levels are defined as:

\[ \mathbb{E}[w[k]w^T[k]] = Q, \quad \mathbb{E}[v[k]v^T[k]] = R. \]  

(10)

Kalman filter equations are then expressed as:

\[ \hat{x}[k] = \bar{x}[k] + K[k](y[k] - C\bar{x}[k]), \quad \bar{x}[k + 1] = \Phi\hat{x}[k] + \Gamma u[k] \]  

(11)

and the Kalman gain \( K \) is obtained from the solution of the Riccati equation [4], [5]:

\[ L_{est}[k] = P[k]C^T R_v^{-1}, \]

\[ P[k] = M[k] - M[k]C^T(CM[k]C^T + R_v)^{-1}CM[k], \]  

(12)

\[ M[k + 1] = \Phi P[k] \Phi^T + \varepsilon R_w \varepsilon^T. \]

Block diagram of a closed loop feedback gain control system with the Kalman estimator is represented in Fig. 2. Based on the known input and output measurements, the Kalman filter estimates the states and feeds them back multiplied by the system feedback gain matrix \( L \) in order to form the control input for the plant.

\[ \text{Figure 2: Closed loop feedback gain control system with Kalman filter} \]

More details on implementation of the Kalman filter in control of dynamic systems can be found e.g. in [6], [7].
3 KALMAN FILTER FOR PARAMETER ESTIMATION IN GEOTECHNICAL PROBLEMS

In appropriately adapted form, Kalman filter can be used for identification of parameters in geotechnical problems. Under assumption that those parameters do not change in time, a stationary transition of state estimates (13) is suitable for this purpose:

$$x(k + 1) = x(k) + w(k),$$  \hspace{1cm} (13)

which corresponds to a discrete-time formulation of the state equation in (7) with $\Phi = I$ and with no control input employed. In this case $k$ represents the number of iterations. The measurements, or observations, required for the state estimation are related with the parameters to be estimated by appropriate constitutive law, e.g. relation between applied loads and displacements:

$$K(x)q = f.$$  \hspace{1cm} (14)

Measurement data can involve displacements, stresses, pore water pressures, etc. The choice of the type of measurements depends on the particular geotechnical problem and the available measurement tools. Here $K$ represents the stiffness matrix, $q$ is the displacement vector and $f$ the load vector. For simulation purposes instead of real measurements, the observation data are obtained through solution of the forward problem using FEA, whereat the observation vector at observed positions at the time instant (iteration) $k$ is related with the displacement vector $q$ in terms of equation, which is of the similar form as the output equation in (7)

$$y(k) = Cq(k) + v(k).$$  \hspace{1cm} (15)

With $q$ from (14) we obtain the observation vector in a general nonlinear form (1):

$$y(k) = CK^{-1}(x(k))f + v(k) = h(x(k)) + v(k).$$  \hspace{1cm} (16)

With the state equation (13) and the observation equation (16), the underlying state space model for the estimation of the state vector $x$ is defined. For such a nonlinear geomechanical system, the estimation problem requires a modified structure of the Kalman estimator. The extended Kalman filter (EKF), which linearizes the system at the current state, can be employed for that purpose. For highly nonlinear systems another modification of the Kalman filter, the unscented Kalman filter (UKF) can be applied. More details about the EKF and UKF in the context of geomechanical problems can be found in our papers [8], [9], [10] as well as in [11], [12].
4 APPLICATION

As an illustration of the parameter estimation using Kalman filter, a plain strain model (2m width and 0.5m depth) with 3 layers represented in Fig. 3 is considered. A strip load of 2 MPa is applied on the soil. Due to symmetry only the right-hand half of the structure is considered for the FEA.

The soil is assumed to obey an isotropic linear elastic material law. A set of predefined model parameters is used for synthetic generation of measurement data by numerical FE simulation. Observation points for vertical displacements (settlements) are depicted as round and square dotted spots in Fig. 3 at the distance of 10 cm. The state vector to be estimated by the Kalman filter consists of the 6 model parameters: elasticity moduli $E_i$ and the Poisson's ratios $\nu_i$ for each of the three layers ($i = 1, 2, 3$), respectively.

The progressing of the EKF parameter identification process along iterations is depicted in Fig. 4. It can be seen that after a finite number of iterations, all estimated parameters converge to the final values, which coincide with the true model parameters. In this example the assumed initial values for the Young's modulus and Poisson's ratio are selected to be equal for all three layers. Through the estimation procedure all the parameters converge to their true values independently of the initial value. The convergence was confirmed also with other initial parameter values and other sets of observation data. The number of observation points and the initial parameter values influence considerably the convergence rate.
Figure 4: Parameter estimation with Kalman filter for a three-layer test model

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Geotechnical Models and Geostatistics
Characterization of Rock Mass Conditions and Quality Control of Tunnel Support

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Abstract

Tunneling and subsurface engineering in hard rock require accurate knowledge about the in-situ rock conditions and the structural fabric, both for the planning and the construction phase. The predicted underground conditions, which are based predominantly on interpolation of exploration results from the surface or from boreholes, often differ from the actual rock conditions encountered during tunnel advance. A detailed and objective determination of the effective underground rock mass conditions can be performed with the Slim Borehole Scanner (SBS) directly during tunnel driving.

The SBS allows the investigation, classification and computational visualisation of the surrounding rock mass of underground excavations and to determine the orientation and structure of discontinuities, to measure their aperture, and to address the rock types. Monitoring of the roof rocks with the SBS also assists the systematic quality control of the support and the inspection of damages. The SBS offers relevant advantages by its speedy and cost-efficient method of determining the structure, of evaluating the rock mass parameters with relevance to safety, and of an objective documentation during tunnel driving. The implementation of the acquired data enables an adaption and continuous refinement of the computer models.

Keywords: Slim Borehole Scanner, SBS, rock mass classification, tunnel support
1 INTRODUCTION

The Slim Borehole Scanner (SBS) is a digital optical probe with a slim diameter of only 23 mm. Its original design was developed in form of a wireless and self-sustaining explosion-proof (I M1 EEx ia IIt) tool for the roof control in underground coal mines in a firedamp-endangered atmosphere [3]. For application in tunnelling or ore mining a normal version without EX-proof certification is available.

The Slim Borehole Scanner (SBS) records 360° digital images of the complete wall of slim boreholes with a diameter from 25 mm to 45 mm, drilled for example for anchors and roof bolts. Due to its lightweight and small-sized construction the SBS can easily be inserted into the slim boreholes by hand using prolongable carbon-fibre rods (Figure 1). The tool allows to monitor the surrounding rock mass of tunnels, to determine the structure of discontinuities (e.g. joints), to measure their aperture and to address the different rock types. The orientation of the discontinuities and their position is acquired from the measurements and potential wedges in the roof of a tunnel can be determined.

Figure 1: Utilization of the Slim Borehole Scanner (SBS) for inspection of a roof bolt hole.

2 TOOL DESCRIPTION

The Slim Borehole Scanner (SBS) uses a ring of 20 high-performance micro LED’s to illuminate the borehole wall. The optical unit consists of a hyperbolical mirror for reflecting the image from the wall, a specially adapted lens system and a colour CMOS sensor for the digital image acquisition (Figure 2). From this sensor a ring of pixels is processed resulting in a line-scanning of the borehole wall. The resolution is up to 0.16 mm per pixel (related to a borehole with a diameter of 1”). The image
acquisition is triggered by a magneto-mechanical odometer. Besides the triggering, this system ensures the exact depth measurement and thus the determination of discontinuity spacing, aperture, rock bed thickness etc. A mobile PC or PDA allows the communication and calibration of the SBS. The tool operates completely autarkic by an integrated power supply and memory unit.

Figure 2: Optical head of the Slim Borehole Scanner.

3 IMAGE VISUALISATION AND ANALYSIS

The recorded images from the borehole wall can be directly viewed on site as a preview on a PDA. The further analysis is made with a software application for PC that has been specially developed for the SBS data. This software comprises a database for the utilisation and management of all relevant borehole and image information, as well as of tools for editing, analysing and interpreting the images of the borehole.

Special software functions for analysing the borehole image have been implemented. Determination of the position and the orientation of discontinuities and a semi-automatic measurement of their aperture or mineralization thickness are easily achieved. Furthermore, the rock types and the geotechnical characteristics like fractured zones or open cavities can be marked along the borehole (Figure 3).

Integrated into the software is a 3-dimensional visualisation of the location, the boreholes, the recorded image and the discontinuities and gives a spacial illustration of the borehole and the discontinuity orientation.

The developed software also allows a comparison between images of the same borehole from different (e.g. older and newer) measurements, which is important e.g. for the recognition of the alteration of the opening width of structures with time.
Figure 3: Interpreted borehole image showing rock lithology and geotechnical evaluation, position and orientation of joints and bedding planes (left image), and freely rotatable 3D-visualisation of the borehole image and the discontinuities (right image).

4 DATA INTERPRETATION

The advantage of the borehole measurements lies in the speedy acquisition of objective data on rock mass characteristics and the structural fabric. These data can be acquired from holes drilled into the roof or sides or even the tunnel face. Inspections into the tunnel face give important information on the conditions of the rock mass in the forefront of the tunnel. With the SBS the disaggregating of the surrounding rock, which is generated by changes in the stress field during tunnel driving, can be excellently observed and documented by the recorded borehole images [2]. The acquired data on the rocks, the rock mass conditions and the joint systems assist the rock mass classification / rating of the surrounding rocks and its adaption during tunnel advance. From the measured data about orientation of discontinuities like joints or bedding planes, and observations about their characteristics like mineral fill or opening width, the calculation of potential wedges in the roof, sides or face can be carried out [1] (Figure 4). Size, weight, geometry and friction on the surfaces determine the safety factor. This allows optimizing the support scheme according to the encountered actual structural situation.
Characterization of Rock Mass Conditions and Quality Control of Tunnel Support

Figure 4: Calculation of potential wedges from measured joint data.

If the measurements are carried out in a regular pattern, the visualisation of the various parameters (Figure 5) assists the engineer in selecting the most efficient support classes for the tunnel sections.

Figure 5: Visualisation of surface conditions of discontinuities (plan view).

The Slim Borehole Scanner (SBS) can be used also for special investigations, like measuring the shotcrete thickness, verifying its binding to the rock mass, or checking the success of repair measures like injections.
5 CONCLUSION

The Slim Borehole Scanner (SBS) is an objective measuring and visualisation tool with a broad range of application. Beginning with the geological and geotechnical evaluation of the surrounding rock mass during the driving of a tunnel, its benefits of application also comprise the optimisation of the support scheme, the tunnel monitoring and the diagnosis of damaged spots up to the optimisation of the reparation methods. The SBS also shows its benefits for the inspection of the quality and thickness of the shotcrete and the quality control of injection measures.

REFERENCES


Cross-anisotropic Rock Modelled with Discrete Methods

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Abstract

In rock-engineering practice, layered rocks with cross-anisotropic features are frequently encountered. Alpine tunnel construction sites are often confronted with problems related to cross-anisotropic rocks like phyllites and schists. The assumption of isotropic behaviour is not acceptable to realistically describe the behaviour of such rocks. Rising popularity of discrete methods calls for an appropriate formulation of such materials within their framework. Analytical derivation of parameters of lattice structure which globally exhibits cross-anisotropic behaviour is presented. For this purpose, directionally isotropic lattice, of which links transmit normal and shear load, is considered. Normal and shear stiffnesses are derived as functions of link orientation in such a way that globally observed elastic behaviour recovers cross-anisotropic properties of the continuum. The procedure is based on employing the microplane model as a limit of isotropic and infinitely dense lattice. The result is still a good approximation of less ideal lattices. The aforementioned lattice arrangements can be found as contact configuration in spherical packings used in the Discrete Element Method. Discrete elements with orientation-dependent stiffness are used to show match between theoretically predicted global moduli and behaviour during element tests. A circular tunnel in cross-anisotropic rock subject to radial pressure is also simulated with discrete elements.

Keywords: Rock mechanics, cross-anisotropy, discrete methods
1 INTRODUCTION

Rocks composed of parallel layers (sedimentation or schistosity, e.g. slates and schists) belong to the cross-anisotropic materials (often referred as transversely isotropic or as transverse isotropic materials). In Austria, this subject is of great interest. A great number of tunnel projects, among them the European infrastructure project of Brenner base tunnel, are currently under constructions. These Tunnels must be totally or partly driven through cross-anisotropic rocks like schiefer, quzr phyllite, black phyllite and Bündner schist which are typical for the Alp region.

To define the orientation of the foliation planes in the 3-D space, the strike angle \( \alpha \) and the dip angle \( \beta \) are used. The different definitions of these angles found in the literature can be puzzling. The authors used following definitions (see Figure 1) strike angle \( \alpha \) is the clock-wise angle between the axis \( y \) and the contour line, i.e. the line obtained from the intersection of the foliation plane with a horizontal one. The \( y \)-axis coincides with the axis of the considered cylindrical cavity, e.g. tunnel or drill-hole. Dip angle \( \beta \) is the smallest angle between a horizontal line which is perpendicular to the contour line and the line of maximum dip of the foliation plane.

Cross-anisotropic materials exhibit rotational symmetry in mechanical response about the unit vector \( \mathbf{n} \) normal to the foliation (Figure 1). This means that the material is isotropic in the plane normal to this vector. For the case where \( \mathbf{n} \) coincides with the global \( z \)-axis, this symmetry is captured by the stiffness tensor

\[
\mathbf{C} = \begin{pmatrix}
C_{11} & C_{12} & C_{13} & 0 & 0 & 0 \\
C_{12} & C_{11} & C_{13} & 0 & 0 & 0 \\
C_{13} & C_{13} & C_{33} & 0 & 0 & 0 \\
0 & 0 & 0 & C_{44} & 0 & 0 \\
0 & 0 & 0 & 0 & C_{44} & 0 \\
0 & 0 & 0 & 0 & 0 & \frac{C_{11}-C_{12}}{2}
\end{pmatrix}
\]  

(1)

with five independent components.

In the Cartesian coordinate system \( x, y, z \), \( \mathbf{n} \) is given, in respect to the angles \( \alpha \) and \( \beta \), as \( \mathbf{n} = (\cos \alpha \sin \beta, -\sin \alpha \sin \beta, \cos \beta) \).
1.1 Lattice

The objective of this paper is to model a linear-elastic cross-anisotropic medium using a discrete model. Under discrete models we understand the Discrete Element Method (DEM) with spherical particles, though we are only interested in the static solution in the linear domain. Therefore, an arrangement of DEM particles with contacts in-between them can be seen as a lattice structure, where the contacts (lattice elements) describe interactions with neighboring particles (lattice nodes). We will thus speak of lattice elements, and only turn to the DEM terminology when required.

Constructing the lattice from an arrangement of spherical particles is shown in Figure 2; the procedure is of geometrical nature, parametrized by the contact radius $I_R$ [5] which determines the density of the lattice. $I_R$ is equal to 1 if only geometrically adjacent spheres are to be considered in contact, while greater values will allow “contact” between spheres which some distance between them.

We describe the cross-anisotropic behavior by the stiffness tensor. With continuum models, the constitutive law is written in terms of the stiffness tensor itself; however, discrete models are not homogeneous and we have to make distinction between local (microscopic) and global (macroscopic) levels of description. In the local sense, each lattice element will be characterized by the normal and shear stiffnesses $k_n$, $k_t$.

Globally, though, the behavior is given by complicated interplay between numerous lattice elements.

Our task is therefore to investigate the relationship between local and global lattice behavior: providing a mathematical description of how the global behavior (expressed as a stiffness tensor) depends on local characteristics (stiffnesses of lattice elements) and how are they playing together (lattice geometry). Subsequently, we will inverse
the previous result, i.e. for some desired stiffness tensor, we will determine local characteristics (stiffnesses) leading to the behavior described by that tensor. To this end, we make use of the following assumptions: the lattice is isotropic, i.e. the orientation of the lattice elements is in average uniformly distributed; the displacement of lattice nodes does not deviate from the mean displacement, in another words, the lattice is deformed uniformly, as whole. The latter is commonly referred to as Voigt hypothesis [4] and is an important restriction for the lattice behavior. Denser lattices (with greater $I_R$) fulfill this assumption by themselves better than loose lattices, as they have more contacts restricting deviation of individual nodes from the surrounding deformation. We therefore expect our results to better describe the real behavior in the case of dense lattices.

![Figure 2: Dependence of lattice density on contact radius $I_R$: (a) sphere packing for constructing the lattice, (b) lattice with small $I_R$, (c) lattice with bigger $I_R$](image)

### 1.2 Microplane and Lattice

The transition between discrete, i.e. lattice [3] and continuous description will be done via the microplane theory [2]. This theory describes each material point as an infinite number of microplanes oriented uniformly in all possible directions at that material point, each of them characterized by volumetric, deviatoric, normal and shear moduli (in our case, we only use the latter two, $E_N$ and $E_T$). The cross-anisotropic nature is introduced by supposing dependency of those moduli on the respective microplane orientation such that symmetries of a cross-anisotropic medium are satisfied. The stiffness tensor is obtained by integration of the moduli over all microplanes.

The lattice structure has only a finite number of nodes and a finite number of isotropically-oriented lattice elements in each node. The stiffness tensor is obtained by summation of stiffnesses over all elements. We can write the lattice stiffnesses $k_n$, $k_i$ as functions of some yet unknown moduli $E'_N$, $E'_T$ and let the lattice density grow.
without bounds. After limit transition, we obtain the stiffness tensor of the infinitely dense lattice by integration of the unknown moduli over all lattice elements. By imposing equality of microplane and lattice stiffness tensors, we obtain the values of the unknown lattice moduli $E'_N$, $E'_T$ (proportional to the microplane moduli); using those moduli when constructing the lattice ensures that the stiffness tensor of the discrete lattice will be equal to the stiffness tensor of the microplane model. Consequently, for a given stiffness tensor, we can compute lattice moduli which will lead to the response characterized by that stiffness tensor.

1.3 Tests
The stiffness tensor of a lattice is obtained from element tests (in our case, uniaxial unconfined compression and simple shear tests) on a periodic lattice. It is subsequently compared with the prescribed values, and the dependence between accuracy and lattice density is shown. The second numerical example is modeling of a tunnel subjected to radial pressure. For space reasons, this paper does not show all steps of the derivation in form of equations. These will be soon published separately.

2 NOTATION
$E_N$, $E_T$: microplane normal and tangential moduli
$E'_N$, $E'_T$: lattice normal and tangential moduli
$k_n$, $k_t$: lattice normal and tangential stiffnesses
$C_{mn}$: stiffness matrix component
$\varphi \in (0,2\pi)$: azimuth angle in spherical coordinates
$\theta \in (0,\pi)$: inclination angle in spherical coordinates, from the pole

3 MICROPLANE STIFFNESS
We consider the microplane model by [2] where all microplanes only have normal and shear moduli $E_N$, $E_T$. The stiffness tensor is obtained by integration of the moduli over all possible orientations of microplanes given by the unit vector $\mathbf{n}$. For our purposes, we will integrate over angles $\theta$, $\varphi$ (Figure 3.a), $z$-axis being coincident with the cross-anisotropy axis; this lets us write microplane moduli as $E_N = E_N(\theta)$ and $E_T = E_T(\theta)$, independent of the azimuth $\varphi$. We will assume that those moduli can be given as combination of in-plane ($E_N^a$, $E_T^a$) and out-of-plane ($E_N^b$, $E_T^b$) ones,
\[ E_N(\vartheta) = \sin^2 \vartheta E_N^a + (1 - \sin^2 \vartheta) E_N^b \]
\[ E_T(\vartheta) = \sin^2 \vartheta E_T^a + (1 - \sin^2 \vartheta) E_T^b \]  
(2)

as shown in Figure 3.b. We notice that the moduli \( E_N^a, E_N^b, E_T^a, E_T^b \) are only four in number; therefore, our resulting stiffness tensor will have only four independent components, in contrast to five independent values for a general cross-anisotropic material.

![Figure 3](image)

**Figure 3:** a) Polar coordinate system; \( r, \vartheta, \varphi \), i.e. radius, inclination, azimuth respectively, b) elliptic distribution of microplane moduli

Writing microplane orientation vector \( \mathbf{n} = (\sin \vartheta \cos \varphi, \sin \vartheta \sin \varphi, \cos \vartheta) \), and with the Jacobian equal to \( \sin \vartheta \), stiffness tensor components are written after some algebra as

\[
C_{ijkl} = \frac{3}{2\pi} \int_0^{2\pi} \int_0^\pi E_N(\vartheta)n_in_jn_kn_l + E_T(\vartheta) \left( \frac{n_in_k}{4} \delta_{il} + \frac{n_jn_l}{4} \delta_{jk} + \frac{n_i}{4} \delta_{jl} + \frac{n_k}{4} \delta_{ij} - n_in_jn_kn_l \right) \sin \vartheta \sin \varphi \sin \vartheta \, d\vartheta \, d\varphi \, d\gamma
\]

(3)

After laborious integration, we obtain stiffness tensor components

\[
\begin{pmatrix}
C_{11} & C_{111} \\
C_{33} & C_{333} \\
C_{13} & C_{233} \\
C_{12} & C_{1122}
\end{pmatrix} = \begin{pmatrix}
12 & 35 \\
36 & 8 \\
6 & 6 \\
20 & -8 \\
6 & -12 \\
16 & -6 \\
12 & -2 \\
20 & -6
\end{pmatrix}
\]

\[
\begin{pmatrix}
E_N^a \\
E_N^b \\
E_T^a \\
E_T^b
\end{pmatrix} = \mathbf{A}_{CE} \begin{pmatrix}
E_N^a \\
E_N^b \\
E_T^a \\
E_T^b
\end{pmatrix}
\]

(4)

We chose \( C_{44} \), out-of-plane shear modulus, as the fifth, dependent value, since it is difficult to measure experimentally [1]; it is written as combination
The matrix $A_{CE}$ in Eq. (4) is invertible; thus we obtain the solution of the inverse problem (when $C_{ij}$ are given) as

$$
\begin{pmatrix}
E^a_N \\
E^b_N \\
E^a_T \\
E^b_T
\end{pmatrix} = A_{CE}^{-1}
\begin{pmatrix}
C_{11} \\
C_{33} \\
C_{13} \\
C_{12}
\end{pmatrix}.
$$

(5)

4 MICROPLANE-LATTICE MODULI PROPORTION

Components of the stiffness tensor were derived for microplane moduli $E_N$, $E_T$, which are necessarily proportional to lattice contact moduli $E'_N$, $E'_T$ via a dimensionless factor $\mu$, which expresses relative “stiffness density” of the lattice, and has a geometrical meaning shown below. We will show the derivation only for the normal moduli; the procedure for tangential moduli is identical. Considering lattice constructed from spherical packing, we suppose intra-nodal stiffnesses given as $k_n = E'_N \pi r^2/(2r')$, where $\pi r^2$ is a fictious contact area, divided by contact length $2r'$. In general $k_n = E'_N \pi r^2/r'$ with some algorithm-dependent constant $\hat{\pi}$ (we assume no special value of $\hat{\pi}$, in the case mentioned it is equal to $\pi/2$). Microplane moduli are then related to intra-nodal stiffness via some unknown $\alpha$ as

$$
E_N = \alpha k_n = E'_N \alpha \hat{\pi} r^2/2r' = \mu E'_N.
$$

(7)

To find the value of $\mu$ value, we suppose that the lattice is in average isotropic, i.e. lattice element length $2r'$ is in average orientation-independent; the difference between $r$ and $r'$ accounts for possibly non-unit contact radius $I_R$ as explained above (Figure 2). The lattice occupies volume $V$ and has $N$ contacts, the only orientation-dependent values are the moduli $E'_N (\Theta)$ and $E'_T (\Theta)$.

By comparing stiffness tensors for lattice after limit transition (Eq. (38) in [3]) with microplane stiffness integral, we obtain, with some algebra,

$$
\mu = \frac{E_N}{E'_N} = \frac{E'_T}{E'_T} = \frac{2}{3} \frac{N r^2 r'}{V} \hat{\pi}
$$

(8)

with $r^2 r'$ denoting the average value of $r^2 r'$ over all contacts.
The geometrical meaning of the equation can be seen better if we consider the special case of both $r$ and $r'$ being constant; averages can be then omitted, giving

$$\mu = \frac{1}{6} \frac{N(2r')(2\tilde{r}^2)}{V} = \frac{1}{6} \frac{N(l_i A_i)}{V}.$$  \hspace{1cm} (9)

We see that $\mu$ is dimensionless, giving proportion of $N$ contact “volumes” (area $A_i = 2\tilde{r}^2$, length $l_i = 2r'$) to the overall lattice volume $V$.

Since all relationships for $C_{ij}(E_*)$ in (4) are linear in $E_*$, macroscopic lattice stiffness can be written as $C_{ij}(E_*) = \mu C_{ij}(E'_*)$. In particular, the Eq. (4) and its inverse become

$$\begin{pmatrix}
C_{11} & C_{12} & C_{13} \\
C_{21} & C_{22} & C_{23} \\
C_{31} & C_{32} & C_{33}
\end{pmatrix}
= \mu A_{CE}
\begin{pmatrix}
E_{N}^{a} & E_{N}^{b} & E_{T}^{a} & E_{T}^{b} & E_{T}^{b}
\end{pmatrix},

\frac{1}{\mu} A_{CE}^{-1}
\begin{pmatrix}
C_{11} & C_{12} & C_{13} \\
C_{21} & C_{22} & C_{23} \\
C_{31} & C_{32} & C_{33}
\end{pmatrix}.

(10)

5 ELEMENT TESTS

Random dense (porosity equal to 0.5) periodic packing of spheres with equal radius is considered. The lattice structure is created by finding contacts between particles, using varying contact radius $I_R$. The cross-anisotropy axis coincides with the global $z$-axis. Given laboratory values of $C_{11}$, $C_{33}$, $C_{13}$, $C_{12}$, we use the current packing geometry to determine $\mu$ and lattice moduli via Eq. (10) and assign stiffnesses via Eq. (2). The goal is to compare the stiffness tensor obtained in three different ways: the prescribed values (back-calculated from lattice stiffnesses); from the current lattice by summation of stiffnesses, using Eq. (35) in [3]; from simulated lattice response, with stiffnesses in Eq. (10), loaded in different ways, as described below.

The test is run for a range of contact radii $I_R$, to show that denser lattice has a stabilizing effect, being closer to the Voigt hypothesis, as mentioned in the introduction.

5.1 Computing the Stiffness Tensor

We simulate homogeneous loading of periodic lattice structure to obtain parameters of the cross-anisotropic material. For each axis, we perform an unconfined uniaxial compression test to obtain normal modulus and Poisson’s ratios, and a shear test to obtain shear modulus (Figure 4). In total, six tests are run. Out of the 12 values obtained (three normal moduli, three shear moduli, three Poisson’s ratios, each measured twice), only five are independent; redundant values are used to check correctness.
5.1.1 Normal moduli and Poisson’s ratios

To find normal moduli from simulations, we make use of the normal compliance relationship for orthotropic material, of which cross-anisotropic material is a special case:

\[
\begin{pmatrix}
\varepsilon_{xx} \\ \varepsilon_{yy} \\ \varepsilon_{zz}
\end{pmatrix} =
\begin{pmatrix}
1 & -\frac{v_{yx}}{E_y} & -\frac{v_{yz}}{E_z} \\
-\frac{v_{xy}}{E_x} & 1 & -\frac{v_{yz}}{E_z} \\
-\frac{v_{xz}}{E_x} & -\frac{v_{xy}}{E_y} & 1
\end{pmatrix}
\begin{pmatrix}
\sigma_{xx} \\ \sigma_{yy} \\ \sigma_{zz}
\end{pmatrix}
\]

and prescribe strain \( \hat{\varepsilon} \) in one direction with zero lateral stresses and evaluate the response. For instance, for the \( x \)-compression we prescribe (prescribed values are denoted with \( \hat{\cdot} \))

\[
\varepsilon_{xx} = \hat{\varepsilon}_{xx} = \hat{\varepsilon}, \quad \sigma_{yy} = \sigma_{zz} = 0,
\]

and use measured response \( \sigma_{xx}, \varepsilon_{yy}, \varepsilon_{zz} \) to compute

\[
E_x = \frac{\sigma_{xx}}{\hat{\varepsilon}_{xx}}, \quad \nu_{xy} = -\frac{\varepsilon_{yy}}{\sigma_{xx}} = -\frac{\varepsilon_{yy}}{\hat{\varepsilon}_{xx}}, \quad \nu_{xz} = -\frac{\varepsilon_{zz}}{\hat{\varepsilon}_{xx}}.
\]

5.1.2 Shear moduli

The shear test is purely deformation-controlled. The periodic cell is prescribed shear strain \( \hat{\varepsilon} \). Shear moduli are found from shear stiffness equations.
5.1.3 Stiffness tensor

Components of the stiffness tensor are found by inversion of the orthotropic compliance matrix in Eq. (11) using symmetries \( E_y = E_x, \nu_{xz} = \nu_{yz} \) and abbreviating \( e = E_x/E_z, \; m = 1 - \nu_{xy} - 2e\nu_{xz}^2 \):

\[
\begin{pmatrix}
C_{11} \\
C_{12} \\
C_{13} \\
C_{33} \\
C_{44}
\end{pmatrix}
= 
\begin{pmatrix}
E_x \left(1 - e\nu_{xz}^2\right) \\
(1 + \nu_{xz})m E_x \\
E_x \nu_{xy} + e\nu_{xz}^2 \\
E_x \nu_{xy}m \\
E_z m \\
G_{yz}
\end{pmatrix}.
\]

5.2 Results

Values of input parameters for \( I_R = 1.8 \) are shown in Table 1. Resulting stiffnesses for all values of \( I_R \) are given in Table 2. As expected, higher values of \( I_R \) lead to a better agreement with simulated results.

Table 1: Input and derived values for the six element tests performed for \( I_R = 1.8 \).

<table>
<thead>
<tr>
<th>Input values</th>
<th>Derived values</th>
</tr>
</thead>
<tbody>
<tr>
<td>number of spheres</td>
<td>avg. number of contacts per sphere</td>
</tr>
<tr>
<td>sphere radius (m)</td>
<td>lattice density scaling ( \mu )</td>
</tr>
<tr>
<td>interaction radius ( I_r )</td>
<td></td>
</tr>
<tr>
<td>( C_{11} ) (MPa)</td>
<td>( E_N^a ), (MPa)</td>
</tr>
<tr>
<td>( C_{33} ) (MPa)</td>
<td>( E_N^b ), (MPa)</td>
</tr>
<tr>
<td>( C_{13} ) (MPa)</td>
<td>( E_T^a ), (MPa)</td>
</tr>
<tr>
<td>( C_{12} ) (MPa)</td>
<td>( E_T^b ), (MPa)</td>
</tr>
<tr>
<td>prescribed strain ( \hat{\varepsilon} )</td>
<td></td>
</tr>
</tbody>
</table>
Table 2: Stiffness tensor resulting from element tests with the contact radius \( I_R = 1.8 \). **Bold** values are prescribed.

<table>
<thead>
<tr>
<th>Stiffness tensor components</th>
<th>( C_{11} )</th>
<th>( C_{33} )</th>
<th>( C_{13} )</th>
<th>( C_{12} )</th>
<th>( C_{44} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>microplane Eq. (10)</td>
<td>130 MPa</td>
<td>55 MPa</td>
<td>28 MPa</td>
<td>40 MPa</td>
<td>32.3 MPa</td>
</tr>
<tr>
<td>lattice [3]</td>
<td>130 MPa</td>
<td>55.2 MPa</td>
<td>28 MPa</td>
<td>39.8 MPa</td>
<td>32.2 MPa</td>
</tr>
<tr>
<td>error</td>
<td>0.06%</td>
<td>0.5%</td>
<td>0.06%</td>
<td>-0.4%</td>
<td>-0.04%</td>
</tr>
<tr>
<td>element tests</td>
<td>122 MPa</td>
<td>52.7 MPa</td>
<td>26.7 MPa</td>
<td>37.6 MPa</td>
<td>30.6 MPa</td>
</tr>
<tr>
<td>error</td>
<td>-6%</td>
<td>-4%</td>
<td>-4%</td>
<td>-5%</td>
<td>-5%</td>
</tr>
</tbody>
</table>

![Graph showing stiffness vs. contact radius](image)

**Figure 5:** Values of the \( C_{11} \) stiffness for element tests.

6 CIRCULAR TUNNEL IN LAYERED ROCK

[1] deals with the following problem: circular tunnel with radius \( r_0 \) in cross-anistropic material is loaded with radial pressure \( p_0 \). The material behavior is given by a set of elastic parameters \( E_x, E_z, \nu_{xy}, \nu_{xz} \) and estimated out-of-plane shear modulus \( G_{yz} \). The orientation of the foliation is determined by strike angle \( \alpha \) and dip angle \( \beta \). The numerical values are given in Table 3; notice the negative values of \( E_T^{\nu}, \) and \( E_N^{\nu}, \) which are only meaningful in the lattice context of which stiffness tensor components are still positive.
Table 3: Input and derived values for the circular tunnel problem.

<table>
<thead>
<tr>
<th>Input values</th>
<th>Derived values</th>
</tr>
</thead>
<tbody>
<tr>
<td>number of spheres</td>
<td>average number of contacts</td>
</tr>
<tr>
<td>9100</td>
<td>14.5</td>
</tr>
<tr>
<td>sphere radius (m)</td>
<td>lattice density scaling $\mu$</td>
</tr>
<tr>
<td>0.24</td>
<td>1.32</td>
</tr>
<tr>
<td>interaction radius $I_r$</td>
<td>$E_N^{a'}$ (GPa)</td>
</tr>
<tr>
<td>1.5</td>
<td>4</td>
</tr>
<tr>
<td>foliation strike angle $\alpha$</td>
<td>$E_N^{b'}$ (MPa)</td>
</tr>
<tr>
<td>270°</td>
<td>-70</td>
</tr>
<tr>
<td>foliation dip angle $\beta$</td>
<td>$E_T^{a'}$ (MPa)</td>
</tr>
<tr>
<td>25°</td>
<td>-26.5</td>
</tr>
<tr>
<td>$E_x$ (GPa)</td>
<td>$E_T^{b'}$ (GPa)</td>
</tr>
<tr>
<td>5</td>
<td>1.17</td>
</tr>
<tr>
<td>$E_z$ (GPa)</td>
<td>$G_{yz}$ (lattice) (GPa)</td>
</tr>
<tr>
<td>2</td>
<td>1.52</td>
</tr>
<tr>
<td>$V_{xy}$</td>
<td></td>
</tr>
<tr>
<td>0.25</td>
<td></td>
</tr>
<tr>
<td>$V_{xz}$</td>
<td></td>
</tr>
<tr>
<td>0.125</td>
<td></td>
</tr>
<tr>
<td>$G_{yz}$ (estimate) (GPa)</td>
<td></td>
</tr>
<tr>
<td>1.21</td>
<td></td>
</tr>
<tr>
<td>$p_0$ (MPa)</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td></td>
</tr>
</tbody>
</table>

Stiffness tensor components are computed using Eq. (15). Lattice moduli (Eq. (10)) are used with Eq. (2) to obtain stiffnesses for the individual lattice elements. Scaling parameter $\mu$ in Eq. (7) is computed from the geometrical arrangement via Eq. (9).

The DEM model consists of a regular arrangement of particles at the tunnel wall, while the rest of the medium is the result of random central-attraction deposition. Particle radius was chosen such that the tunnel wall has 20 particles around the perimeter. Simulated domain was cropped circularly to $10 \times r_0$, and is periodic along the tunnel axis. All particles (including the ones on the boundaries) are free to move.

The pressure $p_0$ is applied on the tunnel wall as force on each of the particles. The resulting deformation curve, shown in Figure 5, corresponds qualitatively to the results of [1] obtained with the finite element method.
Figure 6: Radial displacements of the tunnel wall (innermost layer of particles).

7 CONCLUSIONS

A method of capturing cross-anisotropic behavior with discrete lattice-like models was presented. We have established relationship between the moduli of dense lattices and the microplane model under the assumption of the Voigt hypothesis and elliptical distribution of orientation-dependent stiffnesses. This relationship let us determine local lattice stiffnesses so that the global behavior corresponds to the desired stiffness tensor; the agreement depends on the density of the lattice, which is given by the contact radius $R_I$. Larger values of $I_R$ give very good agreement. An example of deformation of circular tunnel in layered rock subject to radial pressure was given. Thus, our results find an application in modelling deformation of layered rock with discrete models, which are becoming increasingly popular in engineering practice.
ACKNOWLEDGEMENTS

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REFERENCES


Hydromechanical Modelling of an Underground Excavation with Plastic/Viscoplastic Constitutive Equations Associated with Higher Order Continuum Methods

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Abstract

The long term behaviour of underground galleries is technically and economically a great stake for EDF, especially in the framework of nuclear waste storage in deep geological formation. For this reason, EDF/CIH has developed a constitutive model, called $L&K$, able to describe both instantaneous and delayed mechanical effects in geomaterials. In addition to the strong link between instantaneous and delayed behaviours, non-elastic strains are deduced from a specific non-associated flow rule that takes into account the evolution of the dilatancy with respect to the stress state. This model was developed and validated along three theses: two consecutive ones to establish the concepts and the mathematical formulation [1, 2], and a third one to apply the model in hydromechanical coupled conditions [3]. All the constitutive equations are not necessarily detailed in the present paper, the reader will refer to [3] for more information about the model. In order to illustrate the relevance of the model, an application to the Meuse/Haute-Marne Argillite is given through an example of coupled hydromechanical modelling taking into account higher order continuum methods to regularise strain localisation problems encountered [4].

Keywords: Hydromechanical modelling, nuclear waste storage, local second gradient model.
1 INTRODUCTION

The long term behaviour of underground excavations is a social and economic stake, especially in the context of nuclear waste storage in deep geological formation. Several experimental galleries have been dug in the underground research laboratory (URL) of Bure (France) to understand the behaviour of the host rock (Callovo-Oxfordian argillite) under several coupled loads (see [5, 6] for instance). In this paper, a coupled hydromechanical modelling that takes into account advanced constitutive equations associated with higher order continuum methods is presented. Creep phenomenon has already been studied for COx-argillite [7], but both creep and hydromechanical effects are quite rare in the literacy [5]. Constitutive equations used in this study (L&K model, see [2]) offer a coupling between instantaneous and delayed behaviours as well as a specific evolution of the dilatancy. Strains in geomaterials are strongly coupled with pore pressure thanks to Biot’s equations [8]. The main novelty of this work comes from the use of higher order continuum methods [4] associated with advanced constitutive equations in coupled hydromechanical conditions.

2 CONSTITUTIVE EQUATIONS

Constitutive equations considered in this works come from the L&K model [2]. This model based on physical observations and micromechanical aspects is split into two non-elastic mechanisms by following the classical strains partition: an instantaneous elastoplastic mechanism inspired by Hoek&Brown’s works and a delayed viscoplastic mechanism based on the so-called Perzyna’s theory. Another crucial point is the specific evolution of the dilatancy.

2.1 Poroelasticity

A classical non-linear hypoelastic framework is followed for the elastic mechanism. Constitutive equations are coupled with hydraulic quantities through Biot’s equation:

\[
\sigma_{ij} = \sigma'_{ij} - b p_w \delta_{ij}
\]  

(1)

\(\sigma_{ij}\) is the Cauchy stress tensor, \(\sigma'_{ij}\) the effective stress tensor, \(b\) the Biot coefficient, \(p_w\) the pore pressure and \(\delta_{ij}\) the Kronecker tensor.

Water diffusion is described by the usual Darcy’s law.
2.2 Elastoplastic Mechanism

2.2.1 Yield surface

The yield surface equates with a generalised Hoek&Brown’s criterion both on a compression path and on an extension path as well (hereafter, for the sake of conciseness, variables $X(\xi^{\text{ep}})$ are denoted $X$):

$$F^{\text{ep}}(\sigma, \xi^{\text{ep}}) = s_{II}H(\theta) - \sigma_c H_c(\theta) + B^{\text{ep}}I_1 + D^{\text{ep}}a^{\text{ep}}$$  \hspace{1cm} (2)

$$A^{\text{ep}} = -\frac{m^{\text{ep}}k^{\text{ep}}}{\sqrt{6}\sigma_c H_c} \quad B^{\text{ep}} = \frac{m^{\text{ep}}k^{\text{ep}}}{3\sigma_c} \quad D^{\text{ep}} = s^{\text{ep}}k^{\text{ep}} \quad k^{\text{ep}} = \left(\frac{2}{3}\right)^{\frac{1}{2\sigma^{\text{ep}}}}$$  \hspace{1cm} (3)

Strain hardening/softening is induced by functions $a^{\text{ep}}, m^{\text{ep}}$ and $s^{\text{ep}}$ not detailed in the present paper (see Figure 1). $H(\theta)$ introduces the effect of the third invariant of the deviatoric stress tensor on the yield surface. $s_{II} = \sqrt{s_{ij}s_{ij}}$, $s_{ij}$ is the deviatoric stress tensor. $\xi^{\text{ep}}$ is the internal variable of the elastoplastic mechanism.

![Figure 1: L&K model. Definition of the different thresholds.](image)

2.2.2 Plastic flow rule

The rate of plastic strains is given by:

$$\dot{\varepsilon}^{\text{p}}_{ij} = \dot{\lambda} G^{\text{ep}}_{ij} \quad G^{\text{ep}}_{ij} = \frac{\partial F^{\text{ep}}}{\partial \sigma_{ij}} - \left(\frac{\partial F^{\text{ep}}}{\partial \sigma_{kl}} n_{kl}\right) n_{ij} \quad n_{ij} = \frac{\beta' s_{ij}}{s_{II}} - \frac{s_{ij}}{3 + \beta'^2} \quad \beta' = -\frac{2\sqrt{6}\sin \psi}{3 - \sin \psi}$$  \hspace{1cm} (4)
ψ is the dilatancy angle. It follows specific laws according to the stress state and hardening/softening functions which are not discussed in the present paper. More information about the elastoplastic mechanism may be found in [3].

2.3 Viscoplastic Mechanism

The rate of viscoplastic strains is given by following the framework of Perzyna (distance between the stress state and the viscoplastic surface):

\[
\dot{\varepsilon}_{ij}^{vp} = \langle \phi(F^{vp}) \rangle G_{ij}^{vp} \quad \langle \phi(F^{vp}) \rangle = A_v \left( \frac{F_{vp}}{P_a} \right)^{n_v}
\]

(5)

The viscoplastic surface \( F^{vp} \) which has a similar expression of \( F^{ep} \) given by Equation 2, undergoes a positive hardening from the initial yield threshold to a maximal viscoplastic threshold. The internal variable of the viscoplastic mechanism is the viscoplastic distortion. The viscoplastic mechanism has an influence on the instantaneous mechanism through its internal variable. When the stress state is above the maximal viscoplastic threshold, the rate of \( \xi^{ep} \) is given by the sum of the rates of plastic and viscoplastic distortions. For more information about the model, refer to [3].

3 HYDROMECHANICAL MODELLING

3.1 Material Parameters

Great depth, influence of water on measures and important scale effects make COx argillite properties not so obvious to determine. Main parameters inspired by [5] are listed in Table 1.

Table 1: Main parameters for COx modelling.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young modulus ( E )</td>
<td>4 GPa</td>
</tr>
<tr>
<td>Poisson ratio ( \nu )</td>
<td>0.12</td>
</tr>
<tr>
<td>Volumetric mass ( \rho )</td>
<td>2470 kg/m(^3)</td>
</tr>
<tr>
<td>Compression strength ( \sigma_c )</td>
<td>32 MPa</td>
</tr>
<tr>
<td>Porosity ( \Phi )</td>
<td>18%</td>
</tr>
<tr>
<td>Biot coefficient ( b )</td>
<td>0.6</td>
</tr>
<tr>
<td>Permeability ( K_{int} )</td>
<td>( 1.10^{-20} ) m(^2)</td>
</tr>
</tbody>
</table>


Constitutive parameters (*L&K model*) are calibrated on traditional laboratory tests including tri-axial compression tests, extension tests and creep tests. The former allow to draw stress-strain curves as well as volumetric strain curves. Elastoplastic thresholds presented in Figure 1 are defined by stress-strain curves whereas dilatancy parameters are calibrated on volumetric strain curves. Extension tests allow to calibrate the parameter $H_0^e$ involved in the function $H(\theta)$ (see Equation 2). Viscoplastic parameters are calibrated by using creep tests. Figure 2 shows creep tests (LaEGO lab test -5607-2, Exp. Data) for two different deviatoric stress levels and the corresponding numerical calibration (Num. Calibration).

![Graph showing creep tests](image)

**Figure 2:** Calibration of viscoplastic parameters on creep tests.

For the sake of conciseness, constitutive parameters are not detailed.

### 3.2 Framework of the Study

#### 3.2.1 Objectives

This study can be seen as a continuation of previous works by [3]. In this paper, two kinds of hydromechanical computations have been carried out with two different objectives:

1) In underground applications, the so-called *Excavation Damaged Zone* is often considered as the area in which the permeability of the rock mass varies by a factor of more than 100 during the excavation step. In coupled computations, the permeability often remains unchanged whereas the porosity varies with strains according to coupled constitutive equations. Hence, in a first application, we intend to evaluate the effect of a variable permeability on pore pressures during an excavation step. We assume the permeability to
vary with respect to the porosity according to the following relations [9]:

\[
K_{\text{int}}(\phi) = K_{\text{int},0} \left(1 + \chi (\phi - \phi_0)^3\right) \quad i f \ (\phi - \phi_0) \leq 10^{-2} \quad (6)
\]

\[
K_{\text{int}}(\phi) = K_{\text{int},0} (1 + 10^{-6}\chi) \quad i f \ (\phi - \phi_0) \geq 10^{-2} \quad (7)
\]

Initial permeability and porosity are respectively denoted \(K_{\text{int},0}\) and \(\phi_0\). A parametric study is carried out on the parameter \(\chi\).

2) In a second application, we assume the permeability to remain unchanged and a concrete lining, 40 cm thick, is installed during the excavation step. A parametric study is carried out on the installation time.

Hereafter, we use \(L&K\) constitutive equations for the argillite and a linear elastic model for the concrete lining.

3.2.2 Geometry, boundary, initial and loading conditions

Computations are performed in 2D taking into account a complete hydromechanical coupling. The GMR gallery has a horseshoe shape with a mean radius equal to 2.3 m. Only a half model is meshed according to symmetry assumptions (Figure 3). For the sake of conciseness, boundary conditions are not detailed in this paper (see Figure 3).

Figure 3: GMR gallery. Geometry, boundary, loading and initial conditions. On the left, without concrete. On the right, with the concrete lining installed.

We assume the initial stress state to be equal to the \textit{in situ} one. The major stress is perpendicular to the gallery’s axis. Initial displacements are zero.

Loading conditions are the followings:
1) In the first application (without concrete), the excavation is carried out by decreasing nodal forces on the wall (see $F_{nodal}$ in Figure 3). Nodal forces are derived from an equilibrium step in which initial fields are input and displacements on the wall are locked. The excavation lasts 20 days and after this step, nodal forces remain zero and delayed effects on displacements and pore pressures are observed.

2) In the second application, the concrete lining is installed during the excavation step (see circles in Figure 3). Delayed effects on pore pressures, stress state in the concrete and displacements are observed.

3.2.3 The second gradient dilation model
Computations are carried out with the finite element open source software Code_Aster [10]. The use of classical constitutive equations to model strain softening in geomaterials often fall short of providing converged solutions without meeting with the so-called strain localisation problem. In this case and to circumvent these problems, higher order continuum methods are generally needed. In this paper, we choose to work within the framework of the second gradient dilation model [4] seen as a simplified local second gradient model and as a constrained micromorphic dilation model as well. Second gradient models need an enhanced kinematics and thus additional material parameters to be defined. In the particular case of the second gradient dilation model, only one parameter is introduced (namely, $a_1$). We work with $a_1=1.10^6$ Pa.m$^2$ mainly for convenience but also because of some previous works by [4].

3.3 Main Results
3.3.1 First application - Variable permeability
In the first application, the permeability varies with respect to the porosity. A parametric study is carried out on the scalar $\chi$ (see Section 3.2.1). Figure 4 shows pore pressures obtained during the excavation as well as during the creep step from 20 to 200 days (curves on the left). An increase of the parameter $\chi$ causes the same evolution of the permeability and thus an increase of pore pressures. Negative values are justified by the hydraulic boundary condition on the wall and the high dilatancy that occurs during the excavation. Permeability maps are presented on the right at the end of computations. An increase of the parameters $\chi$ causes an increase of the $EDZ$ as defined in Section 3.2.1.
3.3.2 Second Application - Concrete installation time

In the second application, the permeability remains unchanged and a concrete lining is installed during the excavation step. In Figure 5, the concrete installation time is denoted $T_{cl}$. Figure 5 shows maps of the major principle stress in the lining at the end of computations for $T_{cl}=10$ days and $T_{cl}=20$ days and typical stress curves from the installation time to the end of computations. An increase of $T_{cl}$ causes a decrease of the compression state in the lining. If the concrete is installed right behind the rock face, the stress state is multiplied by a factor of more than 2.5 in relation to a lining installed at the end of the excavation step.

Figure 5: Maps of the major principle stress in the concrete at the end of computations for $T_{cl}=10$ days and $T_{cl}=20$ days on the left. Typical stress curves from the installation time to the end of computations on the right.
4 CONCLUSION

This study has demonstrated the ability to perform coupled hydromechanical modelling with quite advanced constitutive equations (L&K model) associated with higher order continuum methods (the second gradient dilation model). Even if comparisons with in situ available data have not been discussed in this paper, results obtained are quite in agreement with experimental observations and highlight the weight of delayed effects in modelling argillites’ behaviour. Possible prospects for this work deal with unsaturated conditions, thermal coupling and 3D effects.

REFERENCES


[10] Code_Aster, EDF R&D, finite element software (GNU GPL licence), 
Electrical Monitoring of Ambient Geological and Hydraulic Conditions Beyond Tunnels and Underground Facilities

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Abstract

AEPI methods (pat. pend.) are designed for reconnaissance of geologic and hydraulic conditions beyond underground facilities, particularly tunnels, galleries and boreholes. Techniques of radially oriented Resistivity (→R/ρ) and IP measurements along selective perimeter locations provide insights into ambient ground.

PRINCIPLE OF AEPI

Geologic and anthropogenic tunnel environments characterized by heterogeneous or anisotropic settings regarding ρ-/IP-properties cause deformation of electrical 3D-AEPI-fields in principle. Special 4P-layouts can be used to delineate ambient volumes of significant contrasts compared with ρ-/IP-properties of the geologic background: e.g. water-bearing, fault or karstified zones, cavities, flow paths and foundation structures. AEPI currents are fed parallel to the construction axis through electrode connections into the rock, thus creating constant oriented flow of electrical charge. As shown by example, an adjacent relatively conductive water-bearing rock mass in a distance from a tunnel e.g. causes significant zonular distributions of AEPI-potentials, building up maximum gradients along slices oriented transversal to the induced current and perpendicular to the electrical boundary. Potential spectra induced gradually differ in accordance with the boundary distance.

Keywords: Azimuthal Electrical Perimeter Investigation Methods (AEPI), Azimuthal Electrical Perimeter Monitoring/Sounding (AEPM/AEPS), Ambient Electrical Resistivity Tomography (AERT)
1 AZIMUTHAL ELECTRICAL PERIMETER INVESTIGATION

1.1 Methods and Applications

AEPI methods generate oriented flow of electrical charge in circumjacent settings of underground constructions through at least two ground-coupled perimeter electrodes aligned in the construction axis. In consequence of this geoelectrical measure, vector spaces of electrical fields span beyond the perimeter locations of current donation (A and B) in dependence on the ambient settings of the surrounding ρ- and IP-conditions.

By receiver electrodes C, within C-ring-alignments (CR) and through CR-groups (CRG), positioned at the perimeter interface to the surrounding ground (s. Fig. 1), more or less complex electrical images of the ambient conditions are taken through the application of Azimuthal Electrical Perimeter Monitoring (AEPM), Azimuthal Electrical Perimeter Sounding (AEPS) and Ambient Electrical Perimeter Tomography (AERT).

![Figure 1: An adjacent relatively conductive rock mass or fault zone in a distance from a tunnel causes a significant zonular distribution of AEPI-potentials, building up maximum gradients along a perimeter slice oriented transversal to the induced current flow and perpendicular to the electrical boundary (C1-3; potential electrodes).](image)
According to the modelled geological situation in Fig. 1, a small fractional array of three C-electrodes in regular spacing (CR) can be used during induction of flow of electrical charge to image existence and direction of the parallel conductive rock boundary or fault zone beyond the tunnel. Since the regular spaced C-ring-alignment in example of Fig. 1 is directed towards the electrically contrasting formation or fault zone, the levels of potential amplitudes at C2- and C3-position are equal but differ both from the potential measured at C1. Hence the vertical boundary is clearly mirrored regarding existence and direction through the induced polarization of AEPI-potentials between left-sided minimum determined at C1 compared with the opposite maximum derived between C2 and C3.

1.1.1 Azimuthal Electrical Perimeter Monitoring (AEPM)

AEPM is a qualitative and long-term AEPI-application to survey potential changes of ambient geologic, anthropogenic or hydrogeological conditions in real-time. The number of needed C-electrodes thereby depends on the task: e.g. in case of monitoring of relative changes of the distance between groundwater table beneath a tunnel and the particular monitoring point, one single static C-electrode in use can be sufficient (s. Fig. 2).

**Figure 2:** AEPM: The distance of groundwater table beneath a tunnel influences the 3D-distribution of AEPI-field as shown in three different modelling slices; vertical symbols in the tunnel contours indicate the vertical direction of maximum polarization of electrical potential perpendicular to the groundwater table and to the induced current flow.
Thus AEPI potentials detected at any fixed axial measurement position rise or decline as a result of increase or decrease of the groundwater in real-time. Potential amplitudes acquired at constant positions over time gradually differ in principle in the degree, the distance between perimeter and groundwater table changes.

1.1.2 Azimuthal Electrical Perimeter Sounding (AEPS)

AEPS is a method to survey ambient geologic, anthropogenic or hydrogeological conditions with azimuthal and radial resolution along circular electrode alignments (CR), oriented transversal to the induced current flow. The radial investigation depth is controlled by axial distances, varied between single CRs and according A-, B-/B1-electrodes (s. Fig. 3 and 4). Homogeneous and axisymmetric underground settings

**Figure 3:** Perpendicular potential slices in sequence of increasing distance to nearest point of current transmission (electrode A or B/B1); succession from left to right as shown in Fig. 4 (CR1 vs. CR3).
Electrical Monitoring of Ambient Geological and Hydraulic Conditions

Figure 4: Schematic principle of AEPS at three locations using TBM as electrode A; ring-arrays of C-electrodes (CR) marked by bold dotted lines perpendicular to the tunnel axis; streamlines of electric field in the surrounding homogeneous ground.

and construction features are hereby characterized by equal electrical perimeter potentials within each CR-alignment. AEPS can be carried out in conjunction with BEAM4 applications using a TBM for an advanced ahead Single-Point Resistance logging, designed for the geoelectrical acquisition of qualitative hydraulic and lithologic information of ahead geological conditions with a resolution related to the tunnel diameter (s. Fig. 4).

1.1.3 Ambient Electrical Resistivity Tomography (AERT)
AERT is a method to survey ambient geologic, anthropogenic or hydrogeological conditions with azimuthal, radial and axial resolution via at least two circular perimeter alignments CR, forming a CR-group (CRG) of azimuthal and axial spread. Moreover current transmission is carried out through different axial and azimuthal A- and B-/B1-electrode positions (s. Fig. 5). Homogeneous and axisymmetric underground settings and construction features are hereby characterized by equal electrical potential amplitudes within each CR in case of axial current donation. The individual CR-levels of electrical potentials gradually differ in compliance with the physical law, i.e. due to their non congruent axial positions in relation to the used positions of electrodes A and B (s. Fig. 3 and 4).
Figure 5: Schematic layout sketch for AERT data acquisition, using basically two CR as a group (CRG) and different A-to-B distances for current transmission; schematic current path-lines between A- and B-electrode positions for a homogeneous or axisymmetric surrounding, dotted lines marking the according radial investigation ranges.

During AERT data acquisition, the axial positioning of electrodes A and B/B1 is varied stepwise with regard to different radial investigation ranges like used for AEPS. Azimuthal shifts between A- and/or B-electrodes are carried out to further delineate e.g. small anomalous volumes compared to the tunnel diameter in axial resolution. The derived potential gradients between C-electrodes from different CR positions are used to achieve enhanced 3D-resolution, particularly regarding the exploration of shapes of anomalies.

REFERENCES


Computational Failure Analysis in Subsurface Engineering
Numerical Analysis of Swelling Deformations in Tunnelling – a Case Study

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Abstract

This paper presents a numerical analysis of swelling deformations in the Pfaendertunnel near Bregenz/Austria, which is a well-known example for swelling in claystones. The constitutive model used in this study employs a semi-logarithmic reduction of swelling strains with stress level and exponential convergence with final swelling strains over time. Input swelling parameters for the analysis were derived from laboratory swelling tests. Due to the wide range of experimental results, upper and lower bounds of swelling parameters were used in the analysis. Calculated invert heave was in particular sensitive to the choice of the maximum swelling pressure, which governs both the swelling strain at stress point level and the size of the swelling zone below the tunnel invert. Good match with the in situ measurements was obtained for a maximum swelling pressure of 1500 kPa, which represents the upper edge of the experimental values and is significantly lower than the overburden pressure of about 4900 kPa.

Keywords:  Swelling rock, clay swelling
1 INTRODUCTION

Swelling phenomena have caused major difficulties in many tunnelling projects in the last 100 years. When water is allowed to infiltrate the swelling rock mass after tunnel excavation, chemical processes within the rock matrix can be initiated which result in large volume increase. As water aggregates at the tunnel invert, typically large invert heave deformations evolve if no or a flexible invert lining is installed. In the case of a rigid support concept, large swelling pressures may develop. The most prominent rock types exhibiting swelling behaviour are certain types of claystone and rocks containing anhydrite. This paper focuses on the derivation of calculation parameters from laboratory swelling tests in order to back analyse in situ measurements with a novel constitutive model for swelling rock.

2 SWELLING ROCK MODEL

As the details of the model (which has been implemented by T. Benz, NTNU Norway as a user-defined soil model for the finite element software PLAXIS) are unpublished as yet, the main features of the model are briefly explained in the following section.

2.1 Stress Dependency of Swelling

The relationship between final swelling strains $\varepsilon_i^{(t=\infty)}$ and the axial stress in the direction of swelling is given by Grob’s [2] semi-logarithmic swelling law (Figure 1).

$$
\varepsilon_i^{(t=\infty)} = -k_{qi} \cdot \log_{10} \left( \frac{\sigma_i}{\sigma_{q0i}} \right)
$$

$k_{qi}$ is the (axial) swelling parameter, $\sigma_i$ is the axial stress and $\sigma_{q0i}$ is the maximum swelling stress in that direction. Swelling strains are calculated in the coordinate system of principle stresses without any interaction of swelling in the different directions [5].

2.2 Time Dependency of Swelling

The model is based on exponential convergence towards the final swelling strain with time [5]. The swelling strain increment is determined by the parameter $\eta_i$ (Figure 1).
The influence of elastic and plastic volumetric strains, $\varepsilon_v^{el}$ and $\varepsilon_v^{pl}$, can be taken into account by the parameters $A_{el}$ and $A_{pl}$. Positive volumetric strains (loosening of the material) result in faster approach of the final swelling strain, while negative volumetric strains delay or may even stop the evolution of the swelling strains. This approach accounts for the dependency of the swelling rate on the penetration rate of water, which changes with the permeability of the rock mass.

$$\varepsilon_i^{q(t+\Delta t)} = \varepsilon_i^{q(t)} + \left(\frac{\varepsilon_i^{q(t=\infty)} - \varepsilon_i^{q(t)}}{\eta_q(t)}\right) \cdot \Delta t$$

$$\eta_q(t) = 1 / \left(A_0 + A_{el} \cdot \varepsilon_v^{el} + A_{pl} \cdot \varepsilon_v^{pl}\right)$$

Figure 1: Semi-logarithmic swelling law and influence of $\eta_q$ on evolution of swelling strains

2.3 Plastic Strains and Iterative Procedure

Plastic strains are calculated according to a Mohr-Coulomb failure criterion with tension cut-off. Shear strength is defined by effective friction angle, $\varphi’$, and effective cohesion, $c’$. The direction of the plastic strain increment is defined by a non-associated flow rule, using the angle of dilatancy $\psi$. The elastic, plastic and swelling strain increments add up to the total strain increment:

$$\Delta \varepsilon = \Delta \varepsilon^{el} + \Delta \varepsilon^{pl} + \Delta \varepsilon^q$$

An implicit backward-Euler-scheme is used on stress point level to find the stress state which satisfies the constitutive equations for the given total strain increment.
3 PFÄNDERTUNNEL CASE STUDY

3.1 Project Description

The 6.7 km long first tube of the Pfändertunnel near Bregenz (Austria) was constructed in 1976-1980 according to the principles of the New Austrian Tunnelling Method (NATM). The Pfänderstock consists of various sedimentary molasse rocks (sandstone, conglomerate, claystone, marl) which were deposited in the area north of the Alps. Significant invert heave of up to 30 cm was observed after about 75% of the tunnel length was excavated. These observations lead to detailed laboratory investigations of the swelling characteristics of the Pfaenderstock material, an extensive monitoring program and to the installation of additional anchors in the tunnel invert.

3.2 Laboratory Swelling Tests

The marl (claystone) layers were identified as the rock type causing the swelling of the tunnel invert due to their high content of Montmorillonite. Czurda & Ginther [1] reported results of swell heave tests in oedometric conditions, i.e. the evolution of vertical strain was monitored under constant vertical stress. They distinguished between undisturbed molasse marl (series A) and the fault zone material (series B, Figure 2). Series A samples showed higher maximum swelling potential, but lower maximum swelling pressures than the samples of series B.

![Swelling test results](image)

Figure 2: Swelling test results after [1]
This notable difference was attributed to relaxation and swelling of the series B samples before the samples could be tested[1]. For the back analysis two swelling parameter sets are considered, which represent the upper and lower boundary of the test results. The time swelling parameters $A_0$, $A_{el}$ and $A_{pl}$ are calibrated to match the in situ time-swelling curve.

### 3.3 Numerical Model and Material Parameters

The 2D plane strain finite element model used in this study is shown in Figure 3. Tunnel geometry and basic material parameters of the marl layer ($E = 2.5$ GPa, $\phi' = 34^\circ$, $c' = 1000$ kPa) have been taken from [4]. Tunnel overburden is ~200 m above the tunnel crown, which is representative of the cross section at km 5+373. Linear elastic plate elements are used for the shotcrete lining, with $E = 7.5$ GPa for the young and $E = 15$ GPa for the cured shotcrete. The final concrete lining is modelled with volume elements assuming linear elastic behaviour and a stiffness of $E = 30$ GPa. The final lining thickness varies between 50 cm at the invert and 25 cm at the crown.

Swelling parameters are listed in Table 1 and illustrated in Figure 2. Sets 1a, 1b and 2a only employ $A_0$ for the time dependency of swelling, while in set 2b evolution of swelling with time is entirely governed by elastic volumetric strains.

After top heading / invert excavation (assuming pre-relaxation factors of 75% and 37.5%, respectively), the concrete invert arch is installed. Swelling is confined in the model to an area of 15 m x 15 m below the tunnel invert. After a swelling phase of 65 days, the final lining is activated, followed by another swelling phase of 115 days. John reported that the decision on invert anchoring and pre-stressing was based on the swell heave deformations observed up to this point [3]. In the cross section considered here this resulted in applying a pattern with 2.2 m anchor spacing.

**Table 1:** Swelling parameters

<table>
<thead>
<tr>
<th>parameter</th>
<th>set 1a</th>
<th>set 1b</th>
<th>set 2a</th>
<th>set 2b</th>
</tr>
</thead>
<tbody>
<tr>
<td>swelling potential $k_q$ [%]</td>
<td>3.0</td>
<td>3.0</td>
<td>0.75</td>
<td>0.75</td>
</tr>
<tr>
<td>max. swelling stress $\sigma_{q0}$ [kPa]</td>
<td>1000</td>
<td>1500</td>
<td>4000</td>
<td>4000</td>
</tr>
<tr>
<td>$A_0$</td>
<td>5.0e-3</td>
<td>2.5e-3</td>
<td>3.0e-3</td>
<td>0.0</td>
</tr>
<tr>
<td>$A_{el}$</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>9.0</td>
</tr>
<tr>
<td>$A_{pl}$</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
</tr>
</tbody>
</table>
3.4 Results

3.4.1 Evolution of invert heave with time

Figure 4 compares the time-swelling curves calculated with the different parameter sets with the measured invert heave in km 5+373[3]. The measurements plot close to a straight line in logarithmic time scale, which cannot be reproduced exactly by the exponential approach employed in the model. The match with the measured invert heave is, however, sufficient from a practical point of view.

Set 1a delivers too little invert heave (10 mm), and the development of deformations completely stops after activating the prestressed anchors. Increasing the maximum swelling stress by 50% (set 1b) yields ~50% more deformation and a better match with the measurements. While such a significant influence may be expected, it should be noted that experimental results for these two sets plot so close to each other that either of the two parameter sets appears justified (Figure 2).

Surprisingly, sets 2a and 2b – which represent much smaller free-swell deformations – deliver more invert heave than sets 1a and 1b. This is a result of the higher maximum swelling stress of sets 2a and 2b, which activates swelling in deeper rock layers. Modelling the evolution of swelling with time entirely in dependence on elastic volumetric strains (set 2b) results in a slightly more prolonged time-swell-curve than using a constant value of \( A_0 \) (set 2a).
Figure 4: Development of invert heave with time

3.4.2 Distribution of swelling strains over depth

The size of the rock mass which is affected by swelling depends primarily on the maximum swelling stress. For set 1b ($\sigma_{q0} = 1500$ kPa) the swelling zone is confined to about 2 m below the tunnel invert, which matches well with the sliding micrometer measurements in cross section km 5+820 (Figure 5). The swelling zone with sets 2a and 2b ($\sigma_{q0} = 4000$ kPa) is much deeper due to the higher maximum swelling pressure, even though similar invert heave is obtained with both parameter sets.

Figure 5: Profile of vertical displacements, a) numerical analysis at $t = 7180$ d, b) measurements km 5+820 [3]
4 CONCLUSIONS

This paper presented the results of a back analysis of measured swelling deformations in the Pfaendertunnel (Austria). A constitutive model based on Grob’s swelling law and exponential convergence with final swelling strains over time was used for the numerical calculations. Input swelling parameters were derived from laboratory swelling tests.

Different sets of swelling potential $k_q$ and maximum swelling stress $\sigma_{q0}$ delivered very similar swelling deformations at the tunnel lining, as increasing $\sigma_{q0}$ is roughly equivalent to increasing $k_q$. However, good match with the measured displacement profile below the tunnel invert was only obtained with $\sigma_{q0} = 1500$ kPa, which represents the upper edge of the experimental results on undisturbed molasse marl. Using higher values of $\sigma_{q0}$ delivered too large swelling zones. The invert heave measurements plot close to a straight line in logarithmic time scale, which cannot be exactly reproduced by the exponential approach of the constitutive model. The match with the measured evolution of swelling, however, is sufficient from a practical point of view.

REFERENCES


Damage Analysis for Wells in CO\textsubscript{2} Storage Sites

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Abstract

This paper presents the innovative application of interface damage laws to the analysis of well integrity. For this problem we formulate a simple and quite general solution procedure, based on analytical integration in space and numerical integration in time. As a representative application example, we present an analysis of the effects of cement shrinkage on the integrity of well interfaces. The performance of the proposed algorithm is assessed in terms of convergence rate and solution accuracy.

Keywords: Underground CO\textsubscript{2} storage, oil/gas wells, damage mechanics, well integrity.
1 INTRODUCTION

The long-term safety of the underground storage of CO$_2$ requires a careful evaluation of the risk of carbon leakage through existing wells. However, the studies of well integrity taking into account the mechanical processes are relatively rare and conventional elasto-plastic laws are employed for the well materials. To our knowledge, models of damage mechanics have never been used for the cement sheath. This is in contrast with the important role played by cracking and debonding effects [4] and with the cyclic nature of the expected hydro-mechanical loads, realistically leading to the degradation of physico-mechanical parameters.

Moreover, as observed in field and laboratory studies [2, 6], the leakage risk induced by continuum damage in the cement is very small if compared to the effects of damage localized at sheath-rock and sheath-casing interfaces.

In the present paper, in view of these considerations, we propose the application of damage interface laws in the analysis of well integrity. The corresponding problem solution is obtained through analytical integration in space and numerical integration in time. As a representative application example, we present an analysis of the effects of cement shrinkage on well integrity.

Figure 1: Configuration of the well problem and assumed boundary conditions
2 SETTING OF THE WELL PROBLEM

The problem at hand is illustrated in Figure 1. The analysis is focused on the eventual formation of the so-called “micro-annulus” at well interfaces, that is, the debonding at the inner (casing-sheath) or outer (sheath-rock) contacts.

Hence, softening relations between traction and relative displacement are used to model the response at the aforementioned surfaces of material discontinuity, while a linear and isotropic elastic response is assumed for both the involved continuum materials, that is, the steel and the cement composing the casing and the sheath, respectively.

In axisymmetric and plane strain conditions, the equilibrium equation and the expressions of strains in the annular layer \( i = 1, 2 \) read:

\[
\frac{d\sigma_{ri}}{dr} = \frac{\sigma_{ri} - \sigma_{\theta i}}{r} \quad \text{and} \quad \varepsilon_{ri} = \frac{du_i}{dr}, \quad \varepsilon_{\theta i} = \frac{u_i}{r} \tag{1}
\]

where we have denoted by \( \sigma_{ri} \) and \( \sigma_{\theta i} \) the radial and hoop stresses, respectively, and we have neglected the effects of body forces. Equations (1)\(_2,3\) relate the radial strain \( \varepsilon_{ri} \) and the hoop strain \( \varepsilon_{\theta i} \) to the radial displacement \( u_i \).

For a simplified simulation of the effects of temperature changes and of cement shrinkage, we consider the additive decomposition of strains in elastic and (given) inelastic components, that is, \( \varepsilon_{\theta i} = \varepsilon_{e \theta i} + \varepsilon_{a \theta i} \) and \( \varepsilon_{ri} = \varepsilon_{e ri} + \varepsilon_{a ri} \). These relations, combined with linear isotropic elastic laws, lead to:

\[
\varepsilon_{\theta i} = \frac{(1 - v_i^2)\sigma_{\theta i} - v_i(1 + v_i)\sigma_{ri}}{E_i} + \varepsilon_{a \theta i}(1 + v_i) \tag{2}
\]

\[
\varepsilon_{ri} = \frac{(1 - v_i^2)\sigma_{ri} - v_i(1 + v_i)\sigma_{\theta i}}{E_i} + \varepsilon_{a ri}(1 + v_i) \tag{3}
\]

for the Young modulus \( E_i \) and the Poisson coefficient \( v_i \) of the annular layer \( i = 1, 2 \).

The well-known general solution for the stresses reads [5, 7]:

\[
\sigma_{\theta i} = C_{Ai} - \frac{C_{Bi}}{r^2} \quad \text{and} \quad \sigma_{ri} = C_{Ai} + \frac{C_{Bi}}{r^2} \tag{4}
\]

for the two integration constants \( C_{Ai} \) and \( C_{Bi} \) characterizing the annular layer \( i = 1, 2 \). Hence, substituting stress solutions (4) in constitutive law (2) and using (1)\(_3\), a general solution is obtained for displacements:

\[
u_i = r \left[ \frac{1 - v_i^2}{E_i} \left( C_{Ai} - \frac{C_{Bi}}{r^2} \right) - v_i(1 + v_i) \left( C_{Ai} + \frac{C_{Bi}}{r^2} \right) + \varepsilon_{a \theta i}(1 + v_i) \right] \tag{5}\]
We note that these solutions can be easily extended to take into account a non-zero initial stress state. As shown in Figure 1, the considered well consists of $N = 2$ annular layers (1: casing, 2: sheath) and $J = 2$ interfaces ($\Gamma_1$: sheath-casing, $\Gamma_2$: casing-rock). The equations governing this problem can be outlined as follows:

- The two conditions imposed at the inner and outer boundaries, that is, a given pressure $(-\sigma_0)$ eventually imposed by injected fluids, and the given displacement $\bar{u}_f$ imposed by the rock formation, respectively,

$$\sigma_1(r_0) = \sigma_0 \quad \quad u_f = \bar{u}_f$$  \hspace{1cm} (6)

where $u_f$ is the displacement of the outer face of the sheath-rock contact.

- The $2J = 4$ conditions imposing the continuity of tractions at interfaces

$$\sigma_1(r_1) = \sigma_{\Gamma_1} \quad \quad \sigma_2(r_1) = \sigma_{\Gamma_1} \quad \quad \sigma_2(r_2) = \sigma_{\Gamma_2} \quad \quad \sigma_f = \sigma_{\Gamma_2}$$  \hspace{1cm} (7)

where $\sigma_{\Gamma_1}$ and $\sigma_{\Gamma_2}$ are the tractions at casing-sheath and sheath-rock surfaces, respectively (see detail in Fig. 1), and $\sigma_f$ is the (unknown) reaction applied by the rock formation on the well.

- The $J = 2$ definitions of the displacement jumps $u_{\Gamma_1}$ and $u_{\Gamma_2}$ at casing-sheath and sheath-rock surfaces, respectively,

$$u_{\Gamma_1} := u_2(r_1) - u_1(r_1) \quad \quad u_{\Gamma_2} := u_f - u_2(r_2)$$  \hspace{1cm} (8)

- The $J = 2$ constitutive equations relating tractions and displacement jumps at interfaces

$$\sigma_{\Gamma_1} = \hat{\sigma}_{\Gamma_1}(u_{\Gamma_1}) \quad \quad \sigma_{\Gamma_2} = \hat{\sigma}_{\Gamma_2}(u_{\Gamma_2})$$  \hspace{1cm} (9)

where $\hat{\sigma}_{\Gamma_1}$ and $\hat{\sigma}_{\Gamma_2}$ are generic interface damage laws (see Sect. 4 below for a model example).

- The $2N = 4$ general solutions for radial stress $\sigma_{2}(r_2)$ evaluated at the boundaries of the annular layers:

$$\sigma_{ri}(r_j) = C_{Ai} + \frac{C_{Bi}}{r_j^2} \quad \text{for} \quad (i; j) = \begin{cases} (1; 0, 1) \\ (2; 1, 2) \end{cases}$$  \hspace{1cm} (10)
The \((2J - 1) = 3\) general solutions for displacements (5) evaluated at the interfaces:

\[
\varepsilon_{ij} = r_j \left[ \frac{1 - \nu_i^2}{E_i} \left( C_{Ai} - \frac{C_{Bi}}{r_i^2} \right) - \frac{\nu_i(1 + \nu_i)}{E_i} \left( C_{Ai} + \frac{C_{Bi}}{r_i^2} \right) + \varepsilon_i \right]
\]

for \((i; j) = (1; 1)\) and \((2; 1, 2)\) \(\cdots\) (11)

3 WELL PROBLEM SOLUTION

As shown in the previous section, the considered well problem is governed by \((2N + 6J + 1) = 17\) equations in the following \((4N + 4J + 1) = 17\) unknowns:

- the displacement \(u_f\) of the outer face of the sheath-rock contact;
- the rock reaction \(\sigma_f\);
- the \(2N = 4\) radial stresses at the boundaries of annular layers \(\sigma_{ri}(r_j)\) for \(i = 1, j = 0, 1\) and for \(i = 2, j = 1, 2\);
- the \((2J - 1) = 3\) displacements at the interfaces \(u_i(r_j)\) for \(i = 1, j = 1\) and for \(i = 2, j = 1, 2\);
- the \(2N = 4\) integration constants \(C_{Ai}, C_{Bi}\) for \(i = 1, 2\).
- the \(J = 2\) tractions \(\sigma_{\Gamma_1}, \sigma_{\Gamma_2}\) at interfaces;
- the \(J = 2\) displacement jumps \(u_{\Gamma_1}, u_{\Gamma_2}\) at interfaces;

For example, the four constants \(C_{Ai}, C_{Bi}\) can be obtained from the analytical solution of the system consisting of the following four equations:

- the equation obtained from the inner boundary condition (6)\(_1\) combined with the stress expression (10) for \(i = 1, j = 0\);
- the equation obtained from combination of the traction continuity conditions at casing-sheath interface (7)\(_{1,2}\) with stress expressions (10) for \(i = 1, j = 1\) and for \(i = 2, j = 1\);
- the two equations obtained from definitions of displacement jumps (8) combined with inner boundary condition (6)\(_2\) and with the displacement expressions (11) for \(i = 1, j = 1\) and for \(i = 2, j = 1, 2\).

In the so-obtained analytical solution, omitted for space reasons, the unique unknown quantities are the displacement jumps \(u_{\Gamma_1}\) and \(u_{\Gamma_2}\).
This solution can be used to calculate the radial stresses at interfaces given by (10) for $i = 2, j = 1, 2$, and hereafter denoted by $\hat{\sigma}_{r21}(u_{\Gamma_1}, u_{\Gamma_2})$ and $\hat{\sigma}_{r22}(u_{\Gamma_1}, u_{\Gamma_2})$, which, in turn, can be substituted in the equilibrium conditions obtained from combination of (7)2,3 and (9), leading to:

$$\hat{\sigma}_{r21}(u_{\Gamma_1}, u_{\Gamma_2}) = \hat{\sigma}_{\Gamma_1}(u_{\Gamma_1}) \quad \hat{\sigma}_{r22}(u_{\Gamma_1}, u_{\Gamma_2}) = \hat{\sigma}_{\Gamma_2}(u_{\Gamma_2})$$

(12)

As a consequence of the damage laws appearing on the right sides, the equation system (12) is non linear. Hence, an iterative procedure is employed to obtain its solution at $t_{n+1}$ in terms of the solution at $t_n$. In particular, omitting the sub-index “$n + 1$” to simplify the notation and introducing the vector $u_{\Gamma} := [u_{\Gamma_1}, u_{\Gamma_2}]^T$, the equation system (12) is written in residual form at $t_{n+1}$:

$$R(u_{\Gamma}) := \begin{bmatrix} \hat{\sigma}_{r21}(u_{\Gamma_1}, u_{\Gamma_2}) - \hat{\sigma}_{\Gamma_1}(u_{\Gamma_1}) \\ \hat{\sigma}_{r22}(u_{\Gamma_1}, u_{\Gamma_2}) - \hat{\sigma}_{\Gamma_2}(u_{\Gamma_2}) \end{bmatrix} = 0$$

(13)

A Newton-Raphson procedure is then employed to solve the residual system (13) with respect to the unknown displacement jumps:

$$u_{\Gamma}^{(k+1)} = u_{\Gamma}^{(k)} - \left( \frac{\partial R}{\partial u_{\Gamma}} \right)^{-1}(k) R^{(k)}$$

(14)
for the iteration number \( k \). We note that in (14), the (analytical) expression for the inverse of the gradient of residuals (13) will also include the interface tangent stiffnesses:

\[
K_{t1} := \frac{d\hat{\sigma}_T}{du_T}, \quad K_{t2} := \frac{d\hat{\sigma}_T}{du_T} \quad (15)
\]

The numerical procedure used to solve the application example presented in Section 5 below is schematically illustrated in Figure 2.

### 4 MODEL EXAMPLE

In the application presented in Section 5 below we consider a cohesive interface law based on the following relation between interface traction \( \sigma_T \) and the mode-I relative displacement \( u_T \) [1]:

\[
\sigma_T = (1 - D)K_u u_T + D[1 - H(u_T)]K_u u_T \quad (16)
\]

where we have denoted by \( H(\cdot) \) the Heaviside function, by \( D \) the damage variable and by \( K_u \) the stiffness modulus characterizing the linear elastic response assumed for the undamaged or compressed interface.

Denoting by \( D_n \) the damage value at time instant \( t_n \) and by \( \langle u_T \rangle \) the positive part of the displacement jump, the damage at time \( t_{n+1} \) is obtained from the evolution law [3]:

\[
D = \max \left[ D_n, \min \left( 1, \frac{\beta}{\eta(1 + \beta)} \right) \right] \quad \text{with} \quad \beta := \frac{\langle u_T \rangle}{u_{T0}} - 1 \quad \text{and} \quad \eta := 1 - \frac{u_{T0}}{u_{Tc}} \quad (17)
\]

where the displacement jump values corresponding to first cracking and full opening are defined as

\[
u_{T0} := \frac{\sigma_u}{K_u} \quad \text{and} \quad u_{Tc} := \frac{2G_c}{\sigma_u} \quad (18)
\]

respectively, in terms of the undamaged tensile strength \( \sigma_u \) and of the mode-I fracture energy \( G_c \). Finally, the tangent stiffness modulus is calculated as follows:

\[
K_t = \begin{cases} 
(1 - D)K_u + DK_a & \text{if } D = D_n > \frac{\beta}{\eta(1 + \beta)} \text{ or } D = 1 \\
(1 - D)K_u + DK_a - \frac{K_u}{\eta(1 + \beta)^3} \frac{(\langle u_T \rangle)^2}{u_{T0}^2} H(u_T) & \text{otherwise}
\end{cases} \quad (19)
\]
for

\[ K_a = \begin{cases} 
0 & \text{if } u_\Gamma > 0 \\
K_u & \text{if } u_\Gamma \leq 0
\end{cases} \]  

(20)

Further details can be found in the aforementioned references [1, 3].

**Table 1:** Effects of cement sheath shrinkage on well integrity. Geometric and material parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Casing internal radius</td>
<td>8.5 \cdot 10^{-2} m</td>
</tr>
<tr>
<td>Casing external radius</td>
<td>9.5 \cdot 10^{-2} m</td>
</tr>
<tr>
<td>Sheath external radius</td>
<td>11.5 \cdot 10^{-2} m</td>
</tr>
<tr>
<td>Steel Young modulus</td>
<td>200 GPa</td>
</tr>
<tr>
<td>Steel Poisson coefficient</td>
<td>0.3</td>
</tr>
<tr>
<td>Cement Young modulus</td>
<td>11 GPa</td>
</tr>
<tr>
<td>Cement Poisson coefficient</td>
<td>0.2</td>
</tr>
<tr>
<td>Undamaged interface stiffness</td>
<td>1.0 \cdot 10^5 GPa/m</td>
</tr>
<tr>
<td>Undamaged interface strength</td>
<td>1.0 MPa</td>
</tr>
<tr>
<td>Interface fracture energy</td>
<td>0.009 kN/m</td>
</tr>
</tbody>
</table>

5 NUMERICAL EXAMPLE

It is often observed that cement shrinkage due to hydration can lead to the formation of a micro-annulus at the sheath-rock interface, thus increasing the risk of CO2 leakage [4, 8]. Therefore, as a first application example, we employ the proposed numerical procedure to evaluate the effects of shrinkage on well integrity. The shrinkage of cement slurry is simulated by imposing to the sheath an increasing homogeneous state of negative inelastic strain \( \varepsilon_a^{\theta_2} = \varepsilon_r^{\theta_2} \), up to a very small maximum value. We assume the rock as rigid (\( \bar{u}_f = 0 \)) and the absence of injected fluids in the well (\( \sigma_0 = 0 \)).

The considered geometric and material parameters are reported in Table 1. The same parameter values are set for both the interfaces, which are assumed to be significantly weaker than the continuum cement, as it is often observed as a consequence of execution defects, such as mud deposition [8].

As an effect of shrinkage strain, increasing tensile radial stresses are initially observed at both the interfaces (Fig. 3a) as well as in the whole casing and sheath (solid curve in Fig. 4a). However, after the attainment of the tensile strength at the sheath-rock contact, the subsequent softening response (Fig. 3b) induces an inversion in the rate of radial stress also at the casing-sheath surface (Fig. 3a). In this way, as shown also by the dash-dot curve in Figure 4a, the inner interface reaches a compressive state exhibiting no damage, in contrast with the sheath-rock contact which attains a state of practical full damage (Fig. 3c). The corresponding displace-
ment jump (about 10 μm in Fig. 3b,d) is consistent with the minimum values of micro-annulus thickness observed in laboratory and field studies [2].

As shown in Figure 4b, the cement shrinkage leads also to an increase in tensile hoop stress, up to values close to the strength of the (continuum) cement, consistently with the frequent observation of radial cracks in the sheath.

In Figures 3b,d and 4c, it can be observed that the considered value of stiffness $K_u$ realistically ensures a practically rigid response of undamaged and compressed interfaces.

**Figure 3**: Effects of cement sheath shrinkage. Radial stress at both the interfaces versus shrinkage strain (a) and displacement jump (b). Damage (c) and displacement jump (d) at interfaces versus the imposed shrinkage strain. Interfaces: 1) casing-sheath; 2) sheath-rock. Solutions obtained for 1000 time steps (lines) and 15 time steps (markers)
To assess the rate of convergence of the algorithm, the ratio $\|R^{(k)}\|/\|R^{(1)}\|$ is plotted in base $-10$ logarithmic scale versus the iteration number in Figure 4d. In spite of the quite severe setting of the interface parameters, namely the small fracture energy and the large undamaged elastic stiffness, a superquadratic rate of convergence is attained after some difficulties experienced at the first iterations. In this way, no more than 5 iterations are required to obtain a residual norm less than the assumed tolerance ($10^{-12}\|R^{(0)}\|$). We have also assessed the accuracy of the algorithm by verifying the indifference of the solution to the size of time steps (Figure 3).

Figure 4: Effects of cement sheath shrinkage. Spatial distributions at different time instants of radial stresses (a), hoop stresses (b) and displacements (c). Convergence profiles of the numerical algorithm (d). Interfaces: 1) casing-sheath; 2) sheath-rock
6 CONCLUDING REMARKS AND FUTURE DEVELOPMENTS

We have proposed the innovative application of interface damage mechanics to the analysis of well integrity. A simple and quite general procedure has been formulated for the solution of this problem, implementing analytical integration in space and numerical integration in time.

Besides the application example presented herein, this procedure can be used to investigate other potential reasons for debonding at well interfaces, such as cycles in temperature and pressure of the injected fluids [4] or cyclic deformations of the rock formation.

Furthermore, the proposed method can incorporate damage interface laws accounting for fluid pressures, in order to simulate the upward propagation of debonding at well interfaces due to the transmission of the higher fluid pressures acting in the reservoir.

In a multi-scale computational method able to manage the interactions between wells and CO\textsubscript{2} storage site, the presented developments offer a possible formulation and solution of the relevant small-scale problem.

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Damage Plasticity Model for Intact Rock

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Abstract

The finite element method (FEM) is frequently used for numerical simulations of underground constructions, e.g. deep tunnel excavations. Such construction processes are complex 3D problems and reliable estimates of displacements and stresses in the ground-support system are difficult to obtain. A key feature is modeling the constitutive behavior of the rock mass involved. Rock mass is composed of intact rock and discontinuities, i.e. bedding planes, joints etc. [1]. Intact rock is a frictional-cohesive material, like soils and concrete [2], showing a highly complicated and distinct nonlinear mechanical behavior, including development of irreversible deformations, degradation of stiffness as well as strain softening. The majority of the available constitutive models for intact rock formulated in the framework of continuum mechanics are either based on the flow theory of plasticity, on the theory of damage mechanics or on a combination of both theories. In the present contribution a 3D model for intact rock, based on a combination of plasticity and damage theory, is proposed. The material parameters as well as model parameters are identified using an optimization procedure, based on experimental data. The rock model is validated by numerical simulation of laboratory experiments, i.e. triaxial compression tests at various levels of lateral confinement. The presented rock model is found to be capable of capturing key features of the constitutive behavior of intact rock.

Keywords: Intact rock; rock model; constitutive model; damage plasticity model; deep tunnels
1 INTRODUCTION

Rock mass is a frictional-cohesive material, like soils and concretes, but in contrast to other frictional-cohesive materials the mechanical response of rock mass is strongly affected by discontinuities, which are typically present at the macro scale [2]. Thus a common approach is to separately investigate intact rock by classical laboratory experiments (e.g. triaxial compression tests) and discontinuities by empirical methods (e.g. rock mass classification systems) and/or laboratory experiments focusing on the discontinuities themselves (i.e. direct shear test of joints).

Constitutive models for rock mass in the framework of the FEM can be formulated by two different approaches; the discontinuum approach, where discontinuities are modeled directly, and the continuum approach, where an equivalent continuum is modeled. For deep tunnels and in the absence of distinct single discontinuities the continuum approach is commonly adopted [3, 4].

Nowadays the Hoek-Brown failure criterion [1, 5] - often in a smooth formulation proposed by Menétérey and Willam [6] - adopted in linear elastic, perfectly plastic models - is one of the most popular FEM rock models in practical rock mechanical engineering. The Hoek-Brown criterion combines a failure criterion for intact rock, similar to the Leon failure criterion for concrete [7], and an empirical approach for consideration of discontinuities by down scaling of the strength parameters based on a rock mass classification system [4]. Alternatively, the Mohr-Coulomb failure criterion fitted to the Hoek-Brown criterion (e.g. [5]) is frequently applied, e.g. [8, 9]. These relatively simple rock models have very limited capabilities; hardening as well as strain softening as found in experiments cannot be modeled and linear elastic behavior is predicted for loading with predominant hydrostatic compression, while in [9, 10] the deformation properties are clearly shown to depend on the level of hydrostatic pressure.

In the field of concrete engineering advanced material models, based on the failure criterion proposed by Leon [7], were developed during the last decades. Etse and Pivonka [11, 12] proposed a 3D concrete model in the framework of the flow theory of plasticity. Grassl and Jirásek [13] proposed an advanced model, based on the combination of the flow theory of plasticity and theory of damage mechanics. Valentini [14] reviewed and evaluated both models, validating the superior performance of the model by Grassl and Jirásek.

In the present contribution a damage plasticity model with an adjusted plastic potential for intact rock is presented. The model is validated by 3D numerical simulations
of laboratory experiments on intact rock, i.e. triaxial compression tests on Kareliya granite, carried out by Stavrogin, Tarasov and Shirkes [2]. The model presented is the first step in the development of a damage plasticity model for rock mass, which is currently in progress [15].

For the large number of 13 required parameters for the presented intact rock model a hand fitting procedure becomes almost impossible. Thus, a sequential identification procedure for determining the required parameters, based on triaxial compression tests, is presented. An optimization algorithm suggested by Summerer [17] combining an evolutionary [16] and gradient based algorithm is employed. The optimization scheme was enhanced for treating non-convergent sets of parameters.

2 DAMAGE PLASTICITY MODEL FOR INTACT ROCK

The damage plasticity model for concrete proposed by Grassl and Jirásek [13] is used for intact rock. The plastic potential is adjusted for intact rock by replacing the exponential function for \( m_g(\bar{\sigma}^m) \) in [13] with a linear function.

The isotropic intact rock model combines linear elasticity, single surface plasticity with nonlinear isotropic hardening and non-associated plastic flow and scalar isotropic damage describing strain softening. To avoid mesh size dependent results the softening behavior is regularized using the specific mode I fracture energy \( G^I_{\text{f}} \) and the characteristic length of the finite element \( l_{\text{char}} \). The essential equations of the damage plasticity model are given as follows:

**constitutive equations:**

\[
\sigma = (1 - \omega(\alpha_d)) \bar{\sigma} = (1 - \omega(\alpha_d)) \varepsilon : (\varepsilon - \varepsilon^p) \tag{1}
\]

**yield function:**

\[
f_p(\bar{\sigma}^m, \bar{\rho}, \bar{\theta}; \alpha_p) = \left[ \frac{1 - q_h(\alpha_p)}{f_{cu}^2} \left( \bar{\sigma}^m + \frac{\bar{\rho}}{\sqrt{6}} \right)^2 + \sqrt{\frac{3}{2} \frac{\bar{\rho}}{f_{cu}}} \right]^2 + \frac{q_h^2(\alpha_p)}{f_{cu}^2} m_0 \left( \bar{\sigma}^m + \frac{\bar{\rho} r(\bar{\theta})}{\sqrt{6}} \right) - q_h^2(\alpha_p), \tag{2}
\]

\[
m_0 = 3 \frac{f_{cu}^2 - f_{iu}^2}{f_{cu} f_{iu}} \frac{e}{e + 1}, \quad e = \frac{1 + \varepsilon}{2 - \varepsilon}, \quad \varepsilon = \frac{f_{iu} f_{bu} - f_{cu}^2}{f_{bu} f_{cu}^2 - f_{iu}^2} \tag{3}
\]
\[
\begin{align*}
\bar{r}(\hat{\theta}) &= \frac{4 \left(1-e^2\right) \cos^2(\hat{\theta}) + (2e-1)^2}{2 \left(1-e^2\right) \cos(\hat{\theta}) + (2e-1) \sqrt{4 \left(1-e^2\right) \cos^2(\hat{\theta}) + 5e^2-4e}} , \quad \hat{\theta} = \left[0, \frac{\pi}{3}\right] \\
q_h(\alpha_p) &= \begin{cases} 
\frac{f_{cy}}{f_{cu}} + \left(1 - \frac{f_{cy}}{f_{cu}}\right) \alpha_p \left(\alpha_p^2 - 3\alpha_p + 3\right) & \text{if } \alpha_p < 1 \\
1 & \text{if } \alpha_p \geq 1
\end{cases}
\end{align*}
\]

Evolution of the strain-like internal hardening variable:
\[
\dot{\alpha}_p(\bar{\sigma}^m, \bar{\rho}, \bar{\theta}; \alpha_p) = \dot{\gamma} \frac{1}{x_h(\bar{\sigma}^m)} \left(1 + \frac{\bar{J}_2}{3} \frac{f_{2cu}}{f_{2} + 10^{-4}} \cos^2 \left(\frac{3}{2} \bar{\theta}\right)\right) \left\| \frac{\partial g_p}{\partial \bar{\sigma}} \right\| ,
\]

\[
x_h(\bar{\sigma}^m) = \begin{cases} 
A_h - (A_h - B_h) e^{-R_h(\bar{\sigma}^m)/C_h} & \text{if } R_h(\bar{\sigma}^m) \geq 0 \\
E_h e^{R_h(\bar{\sigma}^m)/F_h} + D_h & \text{if } R_h(\bar{\sigma}^m) < 0
\end{cases}
\]

Flow rule and plastic potential:
\[
\dot{\varepsilon}^p = \dot{\gamma} \frac{\partial g_p}{\partial \bar{\sigma}} , \\
g_p(\bar{\sigma}^m, \bar{\rho}; \alpha_p) = \left\lfloor \frac{1 - q_h(\alpha_p)}{f_{2cu}} \left(\bar{\sigma}^m + \bar{\rho} \right)^2 + \sqrt{\frac{3}{2}} \frac{\bar{\rho}}{f_{cu}} \right\rfloor^2 \]

Evolution of the isotropic damage variable:
\[
\omega(\alpha_d) = 1 - e^{-\alpha_d/\varepsilon_f} \leq 0.9999 , \\
\varepsilon_f = \frac{G_f}{f_{tu} l_{char}} - \frac{f_{tu}}{2E}
\]

Evolution of the strain-like internal softening variable:
\[
\dot{\alpha}_d = \begin{cases} 
0 & \text{if } \alpha_p < 1 \\
\frac{\dot{\varepsilon}^p_{vol}}{x_s(\dot{\varepsilon}^p_{vol})} & \text{if } \alpha_p \geq 1
\end{cases}
\]
\[ x_s(\dot{\varepsilon}^{p,\text{vol}}) = \begin{cases} 1 + A_s R_s^2(\dot{\varepsilon}^{p,\text{vol}}) & \text{if } R_s(\dot{\varepsilon}^{p,\text{vol}}) < 1 \\ 1 + A_s \left( 4 \sqrt{R_s(\dot{\varepsilon}^{p,\text{vol}})} - 3 \right) & \text{if } R_s(\dot{\varepsilon}^{p,\text{vol}}) \geq 1 \end{cases} \] (12)

\[ R_s(\dot{\varepsilon}^{p,\text{vol}}) = \frac{\dot{\varepsilon}^{p,\text{vol}}}{\varepsilon^{p,\text{vol}}} \], \hspace{1cm} \varepsilon^{p,\text{vol}} = \sum_{I=1}^{3} \langle -\dot{\varepsilon}_I^p \rangle \] (13)

\( \sigma \) and \( \bar{\sigma} \) denote the nominal and effective stress tensor, respectively. \( C \) is the elastic stiffness tensor, \( \varepsilon \) and \( \varepsilon^p \) are the total and the plastic strain tensor. \( \bar{\sigma}^m, \bar{\rho}, \bar{\theta} \) and \( J_2 \) denote the effective mean stress, the effective deviatoric radius, the effective Lode angle and the second invariant of the effective deviatoric stress tensor. \( \varepsilon^{p,\text{vol}} \) is the volumetric plastic strain and \( \varepsilon^p, I = 1, 2, 3 \) are the principal plastic strains.

The model involves 13 parameters; 8 material parameters, namely Young’s modulus \( E \), Poisson’s ratio \( \nu \), the compressive yield stress \( f_{cy} \), uniaxial compressive strength \( f_{cu} \), the biaxial compressive strength \( f_{bu} \), the uniaxial tensile strength \( f_{tu} \), the dilatancy parameter \( m_{g0} \) and \( G^f_1 \) and 5 model parameters \( A_h, B_h, C_h, D_h \) and \( A_s \).

3 \hspace{1cm} OPTIMIZATION SCHEME FOR PARAMETER IDENTIFICATION

A sequential optimization procedure for identification of the 13 required parameters of the damage plasticity model for intact rock from triaxial compression tests is presented. Displacement driven triaxial compression tests for at least 3 various levels of lateral confinement (given by the radial stress \( \sigma_r \)) with measurements of the axial stress \( \sigma_a \), the axial strain \( \varepsilon_a \) and the radial strain \( \varepsilon_r \) are required. \( f_{bu} \) and \( D_h \) cannot be identified from triaxial compression tests. Biaxial compression or triaxial extension tests are required to determine \( f_{bu} \) and experiments including tensile loading to determine \( D_h \). If those are not available for the specific type of intact rock \( f_{bu} = 1.16 \cdot f_{cu} \) and \( D_h = 1 \cdot 10^{-6} \) is assumed, as used for concrete [14]. Furthermore, it is not possible to determine both \( G^f_1 \) and \( A_s \), as multiple combinations of both lead to the same results. It’s recommended to determine \( G^f_1 \) for the specific type of rock from experiments, e.g. wedge splitting tests.

The employed optimization algorithm presented by Summerer [17] consists of an evolutionary algorithm proposed by Kučerová [16] followed by a gradient based algorithm. The objective value to be minimized during optimization is defined as the sum of squared, scaled and weighted errors of results from numerical simulation of the triaxial compression tests compared to the experimental results [17]. During
evolution based optimization an artificial, very large objective value is specified for parameter sets leading to convergence problems during the numerical simulations. The sequential procedure includes the following steps:

1) hand fitting of $E$, $\nu$ and $f_{cu}$: $E$ and $\nu$ are taken as the secant of the first data point in the $\varepsilon_a - \sigma_a$ and the $\varepsilon_a - \varepsilon_r$ curves, respectively; $f_{cu}$ is found as the peak axial stress in uniaxial compression;

2) identification of $f_{tu}$ using the combined evolutionary, gradient based optimization algorithm by minimizing the error of only the peak axial stress;

3) identification of $f_{cy}$, $m_{g0}$, $A_h$, $B_h$ and $C_h$ using the evolutionary, gradient based optimization algorithm, for minimizing the deviation of the $\varepsilon_a - \sigma_a$ and the $\varepsilon_a - \varepsilon_r$ relations in the prepeak regime (experimental results are cut off at the peak and strain softening is omitted in numerical simulations);

4) identification of $A_s$ using the evolutionary, gradient based optimization algorithm for minimizing the deviation of the $\varepsilon_a - \sigma_a$ and the $\varepsilon_a - \varepsilon_r$ relations in the postpeak regime (the prepeak regime is independent to $A_s$);

4 RESULTS

The damage plasticity model for intact rock as well as the optimization procedure for parameter identification are validated by 3D numerical simulations at integration point level of triaxial compression tests at various levels of lateral confinement on Kareliya granite and non-burst-prone (NBP) Donbass sandstone, conducted by Stavrogin, Tarasov and Shirkes [2]. The diameter and the height of the cylindrical samples are 30 mm and 80 mm, respectively. $G_I^f = 0.10 \text{ Nmm/mm}^2$ is assumed, as no other information is available. Three levels of lateral confinement with $\sigma_r = 0$, 25 and 100 MPa are chosen for parameter identification. The experimental results not used for identification ($\sigma_r = 10$ and 50 MPa) are used for validation. The determined material and model parameters are given in tables 1 and 2. Figure 1 shows the results of the laboratory experiments and the numerical simulation.

<table>
<thead>
<tr>
<th>Table 1: Material parameters for Kareliya granite [2]</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E$ ($N/mm^2$)</td>
</tr>
<tr>
<td>43781</td>
</tr>
</tbody>
</table>
Table 2: Model parameters for Kareliya granite [2]

<table>
<thead>
<tr>
<th>A_h</th>
<th>B_h</th>
<th>C_h</th>
<th>D_h</th>
<th>A_f</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5948</td>
<td>4.96 \cdot 10^{-3}</td>
<td>73.9250</td>
<td>1 \cdot 10^{-6}</td>
<td>138.80</td>
</tr>
</tbody>
</table>

![Graph showing experimental and numerical results of triaxial compression tests of Kareliya granite](image)

Figure 1: Experimental and numerical results of triaxial compression tests of Kareliya granite [2]

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Determination of Uniaxial Tensile Strength of Rock Materials with Numerical Methods

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Abstract

For a realistic material description with a damage-plasticity constitutive law (DPCL), needed for advanced numerical investigations, the uniaxial tensile strength ($\sigma_t$) of a material is of high importance. Difficulties in performing direct uniaxial tension tests on rock specimens lead to the indirect determination of the tensile strength by means of the Brazilian test. However, the stress states at failure are not comparable. Additionally, the tensile failure of brittle materials depends on the load amplitude and the distribution of flaws inside the specimen. Experimental results of Brazilian tests are compared with numerically calculated Brazilian tests to obtain the critical stress field in the specimen. The highest maximum principal stress is regarded as the critical quantity for the evaluation of the Brazilian test. The DPCL is fitted to the critical stress states available from all tests and evaluated for the $\sigma_t$ needed as an input parameter. Numerical calculations of the Brazilian test with the DPCL show an improved material description.

Keywords: Brazilian test, uniaxial tensile strength, damage-plasticity
INTRODUCTION

Due to the difficulty in performing direct tensile tests on rock materials an indirect tensile test, the Brazilian test, is performed [5]. The Brazilian test is widely used as the standard testing mechanism to obtain the uniaxial tensile strength because of the simplicity of specimen preparation and execution [3, 5, 14]. A cylindrical specimen is uniaxially compressed in radial direction to induce tensile stresses perpendicular to the loading direction. With the failure load and the geometry the Brazilian tensile strength (BTS) is calculated and deemed representative for the tensile strength of the material [4, 5]. According to theory the tensile stress that corresponds to a specific failure load \( P \) is at the center of the cylindrical specimen [4]. However, contrary to the uniaxial tensile test, the stress field within the specimen is inhomogeneous and multiaxial, which is the reason why classical calculations are inadequate to describe the tensile strength realistically. Additionally the uniaxial tensile strength of materials with a low uniaxial compressive strength to uniaxial tensile strength ratio is underestimated by the Brazilian test [6]. This suggests an influence of the uniaxial compressive strength \( \sigma_c \) on the uniaxial tensile strength \( \sigma_t \), which is determined by the Brazilian test. Marikides et al. [13] obtained closed form full-field solutions for the stress and displacement field in either plane stress or plane strain. However, the validity of these results is limited by Hudson et al.’s finding [7], introducing that the cracks initiate directly below the loading points for flat steel-platen loading and 10 degree radiuses end-cap loading. In general the tensile strength obtained depends on the contact conditions between specimen and jaws [7]. The stress field depends only on the geometry and the applied load. Hooper et al. [8] pointed out that the tensile stresses can only be determined by a three-dimensional analysis because of the influences of the tensile stresses in the contact region. However, most calculations and analyses have been performed for two-dimensional domains only [9, 10, 13, 16].

METHODS

2.1 Theoretical Framework

A procedure is suggested to determine the real \( \sigma_t \) numerically by using experiments with no uniaxial stress state and a yield function. Positive values are assumed for tensile stresses. The yield function \( F(\sigma) \) in the three-dimensional principal stress space is required in order to identify the stress tensor \( \sigma \) which is critical when
Determination of a Corrected $\sigma_t$

inelastic deformation or failure initiates. When eq. (1) is fulfilled, the stress state causes inelastic deformation or failure.

$$F(\sigma) = 0$$

(1)

This equation must be valid for at least one local stress state in a specimen at the moment of failure during an arbitrary test. Once the function is setup with the required parameters, every possible critical stress state, including $\sigma_t$, can be determined immediately. The chosen yield function of the constitutive equation for this work is described in eq. (2) [1, 11, 12].

$$F = \frac{1}{1 - \alpha(r_\sigma)}(\bar{q} - 3\alpha(r_\sigma)\bar{p} + \beta(\tilde{\varepsilon}^{pl}_c, \tilde{\varepsilon}^{pl}_t, r_\sigma)\langle \hat{\sigma}_{max} \rangle - \gamma(-\hat{\sigma}_{max}) - \sigma_c(\tilde{\varepsilon}^{pl}_c) = 0$$

(2)

$p$ is the hydrostatic pressure, $\bar{q}$ is the Mises equivalent effective stress and $\alpha$ can be expressed in terms of $r_\sigma$, the ratio of initial equibiaxial compressive yield stress to initial uniaxial compressive yield stress. The function $\beta$ denotes the ratio of the actual uniaxial compressive strength $\sigma_c(\tilde{\varepsilon}^{pl}_c)$ to the actual uniaxial tensile strength $\sigma_t(\tilde{\varepsilon}^{pl}_t)$. $\gamma$ depends on the ratio of the second stress invariants on the tensile and the compressive meridians. $\hat{\sigma}_{max}$ is the maximum principal effective stress, where $\langle \cdot \rangle$ denotes the Macauley brackets. The compressive and tensile accumulated plastic strains are denoted by $\tilde{\varepsilon}^{pl}_c$ and $\tilde{\varepsilon}^{pl}_t$. They are zero at the initiation of failure or inelastic deformation. The constitutive equation can be used to model softening and stiffening, both of which are not needed for the purpose of this work. This means that all parameters which are functions of the compressive and tensile accumulated plastic strains $\tilde{\varepsilon}^{pl}_c$ or $\tilde{\varepsilon}^{pl}_t$ are constant. The elastic behavior is linear [1, 11, 12].

2.2 Numerical Optimization

The yield function from (1) can be rearranged as a function of the desired parameter $\sigma_t$ and other material parameters $\sigma_c, r_\sigma$ and $\gamma$, if $\bar{p}, \bar{q}$ and $\hat{\sigma}_{max}$ are calculated for a critical stress $\sigma^{crit}$. The critical stress state $\sigma^{crit}$ is obtained by means of FE simulations of experiments. Once $\sigma^{crit}$ has been determined, the material parameters can be adjusted such that the yield condition (now expressed as a function of the
material parameters) is satisfied. Eq. (3) is then solved using the Gauss-Newton scheme [2].

\[ F|_{\sigma_{\text{crit}}} (\sigma_c, \sigma_t, r_\sigma, \gamma) = 0 \] (3)

2.3 Experiments

Experiments yielding stress states comparable to those obtained from the uniaxial tensile test are preferential. It is beneficial to use additional critical stress states provided by very different experiments, ideally ones that directly yield \( \sigma_c \). Considering \( \sigma_t \) as the parameter of interest leaves \( r_\sigma \) and \( \gamma \) undetermined. While \( \gamma \) is estimated as suggested in [1] the parameter \( r_\sigma \) is allowed to vary in the optimization and is therefore determined in the same way as \( \sigma_t \). All tests are evaluated with 3D implicit numerical calculations for the critical stress state at the failure load. All data used were taken from a database containing 65 results for each testing method.

2.3.1 Brazilian test

To obtain an appropriate stress state, the Brazilian test according to ISRM [15] was chosen. Specimen and jaws are shown in Figure 1.

![Figure 1: The Brazilian test results are obtained from tests according to ISRM [15]](image)

Despite the inhomogeneous and multiaxial stress state in the specimen of the Brazilian test, it failed due to tensile stresses at the point of maximum tensile stress. For a very brittle material such as rock the critical stress state is the maximum
principal (tensile) stress in the specimen at the failure load [6]. The Brazilian Tensile Strength (BTS) [15] is widely used as \( \sigma_t \) despite its inexpediency.

2.3.2 Uniaxial compression test

To obtain \( \sigma_c \), the uniaxial compression test was chosen. However, close to platens a multiaxial and non-uniform stress field is possible because of the friction between platens and specimen. Therefore, the stress states were evaluated in the middle section of the specimen.

3 RESULTS

3.1 Evaluation for Principal Stresses

The critical stress state in the Brazilian test specimen is biaxial with the magnitude of the compressive stress component about three times the tensile stress component. Therefore the \( \sigma_t \) cannot be simply taken from the Brazilian test without taking the compressive stress into account. Figure 2 shows the yield function in plane stress where the circle marks the average stress state of all Brazilian test results. Figure 2a shows the effect of introducing the BTS as \( \sigma_t \) into a yield function. It is evident, that the stress state in Figure 2a does not represent the uniaxial tensile strength and therefore at best only a qualitative approach to the \( \sigma_t \). A numerically calculated Brazilian test with an incorrect yield function would fail before the external load could reach the failure load, thereby underestimating it. Figure 2b shows the yield function with the corrected \( \sigma_t \). It is evident, that in this case the critical stress state of the Brazilian test satisfies the yield function at failure as requested.

3.2 Calculation of a Brazilian Test with the Optimized \( \sigma_t \)

Figure 3 shows the failure load normalized by the experimental mean over the displacement normalized by a user specified maximum of Brazilian tests calculated with the BTS and the corrected \( \sigma_t \). The load data is compared against the experimental reference mean and its standard deviation. The calculation with the BTS clearly underestimates as expected the failure load, whereas the calculation with the corrected \( \sigma_t \) provides a significantly better result and matches the experimentally determined mean of the failure loads.
Figure 2: The yield function in plane stress with the stress state of a Brazilian test with a) the BTS and b) the corrected $\sigma_f$.

Figure 3: The load displacement curve of the Brazilian test calculated with the optimized $\sigma_f$ is more accurate.
4 CONCLUSIONS

A method to obtain the ultimate tensile strength ($\sigma_t$) of a rock material is developed and validated. It is further shown that the tensile strength obtained by the Brazilian test is inaccurate and would lead to wrong results when used in numerical calculations. The strength of the proposed method is the ability that different experiments can be used to fully describe any smooth yield function. Numerical calculations with the so determined yield function additionally satisfy the failure criterion at the failure load for the chosen experiments automatically.

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Damage Analysis and Risk Assessment for Existing Buildings
3D Numerical Analysis of Tunnelling Induced Damage: the Influence of the Alignment of a Masonry Building with the Tunnel Axis

Jori M.J. Kappen¹, Giorgia Giardina¹, Max A.N. Hendriks¹ and Jan G. Rots¹
¹ Delft University of Technology

Abstract

The development of infrastructure in major cities often involves tunnelling, which can cause damage to existing structures. Therefore, these projects require a careful prediction of the risk of settlement induced damage. The simplified approach of current methods cannot account for three-dimensional structural aspects of buildings, which can result in an inaccurate evaluation of damage.

This paper investigates the effect of the building alignment with the tunnel axis on structural damage. A three-dimensional, phased, fully coupled finite element model with non-linear material properties is used as a tool to perform a parametric study. The model includes the simulation of the tunnel construction process, with the tunnel located adjacent to a masonry building.

Three different type of settlements are included (sagging, hogging and a combination of them), with seven different increasing angles of the building with respect to the tunnel axis. The alignment parameter is assessed, based on the maximum occurring crack width, measured in the building. Results show a significant dependency of the final damage on the building and tunnel alignment.

Keywords: Damage assessment, Tunnelling, Masonry, 3D Finite Element Analysis, Building-tunnel alignment
1 INTRODUCTION

Due to the rapid expansion of cities around the globe, the demand for higher capacity infrastructure in densely populated areas is constantly increasing. As a consequence, underground excavations for the construction of tunnels, train stations or parking garages are multiplying in urban areas. These projects inevitably lead to settlements, which can affect the neighbouring structures. Recent tunnelling projects have received a great amount of media attention, due to settlement induced damage [3].

To prevent such damage, a preliminary assessment of the damage risk to surface buildings needs to be performed [4]. In this paper, finite element analyses are carried out on a 3D model of building, soil and tunnel, in order to account for three-dimensional effects usually neglected in the traditional assessment. Parametrical studies are performed to quantify the influence of characteristic building features. In particular, the paper investigates the effect of building alignment with the tunnel axis on structural damage, while undergoing tunnelling induced settlements.

2 METHODOLOGY

The sensitivity study is performed with the aid of a three-dimensional, fully coupled finite element model.

2.1 The Numerical Model

The model includes a typical Dutch masonry house, under which a circular tunnel is bored (Figure 1). The soil is modelled as a solid block of 200 × 100 × 50 m\(^3\) with linear-elastic material properties; a Young’s modulus linearly increasing with the depth is assumed. The tunnel is 20 m deep, 100 m long and has a diameter of 8 m. The tunnel lining consists of weightless curved shell elements, with linear concrete material properties. The volume loss inducing the surface settlement is simulated by applying a radial pressure to the lining, which causes a contraction of the tunnel lining elements. Since the presented study is focused on the simulation of the surface building structural response, a relatively simple model of soil and tunnelling induced volume loss is accepted. A staged analysis is performed, in order to account for the tunnel progression. This is implemented by simultaneously removing soil elements and adding tunnel lining elements, to which a radial pressure is applied.

The building has a square footprint and is approximately symmetric, along both central axes. The walls of the building are modelled with shell elements with a size
of $0.4 \times 0.4 \text{ m}^2$ and a thickness of 0.3 m. Floors are omitted due to their negligible addition to the global stiffness of the building; dead and live loads are applied to the building walls. A total strain rotating smeared crack model with linear tension softening is adopted for the masonry to account for stress and stiffness redistribution after cracking. Interface elements between the building and the soil describe the soil-structure interaction in normal and tangential direction. For the tangential direction, a distinction is made between a smooth and a rough interface: the smooth interface neglects shear transfer between the soil and the building, while the rough interface is modelled through a non-linear Mohr-Coulomb friction criterion. For the normal direction, linear compressive behaviour with a tension cut-off criterion is adopted in both cases. Figure 2 illustrates the material constitutive relations. The material parameters are summarised in Table 1.

### Table 1: Material parameters

<table>
<thead>
<tr>
<th>Property</th>
<th>Symbol</th>
<th>Unit</th>
<th>Soil</th>
<th>Masonry</th>
<th>Interface</th>
<th>Lining</th>
</tr>
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<tbody>
<tr>
<td>Young’s Modulus</td>
<td>$E$</td>
<td>N/m²</td>
<td>$5 \times 10^{7}$</td>
<td>$6 \times 10^{9}$</td>
<td>$3 \times 10^{10}$</td>
<td></td>
</tr>
<tr>
<td>Gradient</td>
<td>$m$</td>
<td>N/m³</td>
<td>$1 \times 10^{7}$</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>$\nu$</td>
<td>-</td>
<td>0.3</td>
<td>0.2</td>
<td>0.2</td>
<td></td>
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<tr>
<td>Density</td>
<td>$\rho$</td>
<td>kg/m³</td>
<td>2000</td>
<td>2400</td>
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<td></td>
</tr>
<tr>
<td>Thickness</td>
<td>$t$</td>
<td>m</td>
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<tr>
<td>Tensile strength</td>
<td>$f_t$</td>
<td>N/m²</td>
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<td></td>
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<tr>
<td>Fracture energy</td>
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<td>$k_n$</td>
<td>N/m³</td>
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<tr>
<td>Shear stiffness</td>
<td>$k_t$, smooth</td>
<td>N/m³</td>
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<td>$5 \times 10^7$</td>
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<tr>
<td></td>
<td>$k_t$, rough</td>
<td>N/m³</td>
<td></td>
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<tr>
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<td>N/m²</td>
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<tr>
<td>Dilatancy angle</td>
<td>$\psi$</td>
<td>°</td>
<td>0</td>
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</tr>
</tbody>
</table>
2.2 Set-up of the Variational Study

The sensitivity study consists of the variation of the angle between the central axis of the building with respect to the tunnelling axis. The building is rotated clockwise in eight steps of 22.5°, from $\alpha = 0$ to $\alpha = 180^\circ$ (Figure 3). Previous studies have indicated that the position of a building with respect to the tunnelling axis can be of substantial influence on tunnelling-induced damage (e.g. [1]). Therefore, three different positions of the building have been included in this study, i.e. $x = 0$, 12 and 24 m. These positions are hereafter referred to as sagging zone (P1), combined settlement profile (P2) and hogging zone (P3) respectively (Figure 4). Note that due to the plane symmetry of the building, a building rotation of $\alpha = 0^\circ$ is equal to a building rotation of $\alpha = 180^\circ$.

Figure 2: Constitutive relations: (a) masonry and (b) interface in normal direction; (c) smooth and (d) rough interface in tangential direction.

Figure 3: Set-up of the sensitivity study [2]
Tunnelling Induced Damage: the Influence of Building Alignment

Figure 4: Position of the building with respect to the tunnel: analysed cases

3 RESULTS

For all the 54 performed analyses, the structural damage is assessed by measuring the maximum crack strain $\varepsilon_{cr,max}$ occurring in the building as a consequence of the tunnelling. All the analysis stages are included in the evaluation. The maximum crack width is estimated through the following relation:

$$w_{cr,max} = h \cdot \varepsilon_{cr,max}$$

where $h$ represents the crack bandwidth of the masonry model.

The results are plotted in Figure 5 for both the cases of smooth and rough soil-structure interface. In general, a more severe damage occurs when the model allows for the transmission of horizontal displacements from the soil to the above structure, i.e. in case of rough interface. This is in agreement with the results of previous studies, e.g. [1]. For both interface types the effect of the alignment parameter is qualitatively similar.

3.1 Buildings in the Sagging Zone

In case of sagging deformation (P1) the increment of the angle $\alpha$, up to $135^\circ$, results in a more severe structural damage. The progression of the longitudinal settlement trough seems to play a dominant role, since the building is located directly above the tunnelling axis, where the maximum settlement arises. For $\alpha = 90^\circ$, the weakest structural element of the building, i.e. the façade with openings, is mainly subjected to the longitudinal settlement profile. For $90^\circ < \alpha < 135^\circ$, the damage is worsened by the effect of the stresses transmitted by the stiffer blind wall, especially when the tunnel face approaches the building (Figure 6).
3.2 Buildings in the Combined Zone and the Hogging Zone

Contrary to the cases with the building in the sagging zone, for the cases in the combined settlement profile (P2) and in the hogging zone (P3), the condition \( \alpha = 0^\circ \) is the most sensitive to the differential settlement (Figure 7). This sensitivity decreases by increasing \( \alpha \) up to 67\(^\circ\), and increases again after 90\(^\circ\). The more parallel the façades with openings are to the tunnelling centreline, the less is the damage experienced by the building. This can be related to the dominant settlement trough. Since the building is located at a certain distance from the tunnelling axis, it is more affected by the progression of the transversal settlement trough than the longitudinal trough. Therefore, the façades in the transversal direction undergo more differential settlements, which in time can be related to the emergence of cracks.

3.3 The Influence of Boring Direction

Despite of the symmetry of the building plan with respect to the tunnel axis direction, the curves plotted in Figure 5 are not symmetrically dependent on the alignment angle about \( \alpha = 90^\circ \). This behaviour is mainly due to the influence of the tunnel face position with respect to the façade with openings. In other words, the different damage between the pairs \( \alpha = 22^\circ \) and 157\(^\circ\), \( \alpha = 45^\circ \) and 135\(^\circ\), and \( \alpha = 67^\circ \) and 112\(^\circ\) measures the influence of the boring direction, e.g. northbound versus southbound, on the building response. Especially for the rough interface in the sagging zone, the maximum obtained crack width in the building can differ significantly, up to a factor 3.
Tunnelling Induced Damage: the Influence of Building Alignment

(a) P1-A1, settlement contour plot
(b) P1-A1, crack strain contour plot
(c) P1-A6, settlement contour plot
(d) P1-A6, crack strain contour plot

Figure 6: Comparison between cases P1-A1: 22° and P1-A6: 135°, stage 12, rough interface

(a) P2-A1, settlement contour plot
(b) P2-A1, crack strain contour plot
(c) P2-A3, settlement contour plot
(d) P2-A3, crack strain contour plot

Figure 7: Comparison between cases P2-A1: 22° and P2-A3: 67°, stage 12, rough interface
4 CONCLUSION

The object of this work is the numerical assessment of tunnelling induced damage to buildings. The paper discusses the results of a sensitivity study in which three-dimensional finite element analyses are used to evaluate the influence of the building alignment with a tunnel axis. The variations consist of three different locations and seven different angles of the building with respect to the tunnelling axis. The study is focused on masonry structures; a total strain rotating crack model with linear tension softening simulates the masonry behaviour. This allows to evaluate the structural damage in terms of maximum crack width.

The results show that for buildings in the sagging zone, a low alignment angle is the least sensitive to differential settlements, while the maximum measured crack width increases by increasing the angle. A building in the combined settlement profile or in the hogging zone displays an opposite behaviour: cases with low alignment angles are the most susceptible to damage, while increasing the angle to 90° lowers the maximum measured crack width. Also the boring direction appears to be very influential, especially for a building with a rough interface in the sagging zone.

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Cut-slope Versus Shallow Tunnel: A Risk Management Perspective During Construction

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Abstract

Choice between cut-slope and shallow tunnel alternatives is a common decision-making problem in the construction of highways or railways in mountainous regions. This paper outlines a decision making process that focuses on construction risks. Five indicators, including construction cost, duration, casualties, time overruns and economic loss, are adopted to evaluate the alternatives. Landslide and tunnel collapse risks are analyzed quantitatively for these alternatives. On the basis of reliability theory, the probabilities of landslide and tunnel collapse are estimated. The extent of likely impacted area is delimited based on failure potential. Then according to the exposed elements within the impacted area, the consequences are estimated using Monte-Carlo simulation and event tree approach. A computer code was developed for quantitative risk assessment and multi-objective decision making, and finally an example is presented to illustrate the entire process.

Keywords: Decision making, Cut-slope, Shallow tunnel, Quantitative risk assessment, Construction risks, Normal and exceptional risks
Evaluation of Building Stiffness in the Risk-assessment of Structures Affected by Settlements

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Abstract

The paper presents three competing approaches to consider building stiffness parameters in the building damage assessment due to tunnelling induced displacements more effectively. Current methods often consider the building stiffness in a simplified and unrealistic way. Hence, remedial measures are frequently overdesigned. With access to data of a tunnelling project, three approaches using the principles of System Identification and Back Analysis are compared. Unavoidable uncertainties, stemming from idealisations, not precisely known material parameters and measurement errors, are quantified by cross-checking the approaches outcome among each other. In order to evaluate the building stiffness in the risk-assessment of structures affected by displacements, the potential of each approach will be discussed.

Keywords: Tunnelling, building stiffness, damage risk, damage assessment, System Identification, Back Analyses, DInSAR Monitoring
1 INTRODUCTION

The safety of adjacent structures plays a decisive role in the whole process of designing intra-urban tunnelling projects. Depending on the specific type of building, structures possess different possible damage potentials to displacements caused by shield tunnelling. Currently, it is common practice to apply simplified approaches to assess the sensitivity of buildings due to displacements. The most established one is the Limiting Tensile Strain Method (LTSM) from Burland et al.. Aiming at a preliminary assessment of the buildings’ damage potential, the quality of estimation of the equivalent building stiffness strongly affects the results.

In the framework of the Collaborative Research Center SFB 837 - founded by the German Research Foundation (DFG) – there is the possibility to use measurements of a satellite and a ground based displacement monitoring at a current tunnelling construction site. Having access to the associated building data (structural analyses, construction drawings), a more detailed analysis of the building stiffness is carried out. Furthermore, applying the method of System Identification, a Back Analysis from deformations to the building stiffness is intended. A further focus of the research is on the handling of unavoidable uncertainties of the specific building types and the displacement measurements. To accomplish the risk-assessment more precisely, new approaches of evaluating building stiffness are presented and illustrated in the paper.

2 METHODS FOR THE ASSESSMENT OF STRUCTURAL DAMAGE DUE TO TUNNELLING INDUCED DISPLACEMENTS

In 1956 Skempton & MacDonald [9] developed the first purely empirical approach for an assessment of settlement induced building damages. In their work, the angular distortions $\rho/L$ – solely caused by shear deformation – are proposed to be regarded as an evaluation criterion. Burland & Wroth (1974) and Burland et al. (1977) realised the deficits of an approach only based on shear and introduced a new semi-empirical approach for damage assessment, the Limiting Tensile Strain Method (LTSM) [2]. This commonly used approach schematized buildings as an equivalent weightless rectangular beam based on Timoshenko’s equations (1955). For the assessment of possible occurring damages, Burland & Wroth suggest to derive the limiting tensile strain $\varepsilon_{\text{lim}}$ in relation to the equivalent beam’s deflection ratio $\Delta/L$. $\varepsilon_{\text{lim}}$ indicates the onset of cracking and $\Delta$ is the displacement of a point relative to the line connecting two reference points with the distance L (Figure 1). Assuming the neutral axis in the middle of the beam, Burland & Wroth expressed the analytical Eqs. (1) and (2).
The building stiffness in Eqs. (1) and (2) is estimated by the second moment of area multiplied by the Young’s modulus of an equivalent beam of height H and width B. In that case the building dimensions are equal to the equivalent beam, roof structures get ignored. Neglecting the building stiffness’ effect on the shape of the settlement trough, this approach leads to a conservative damage assessment of buildings. The greenfield settlement trough itself – as a loss-incurring event – can be determined with the empirical approach by Peck (1969).

Finally, in supplementation to Burland’s LTSM method, Boscardin & Cording (1986) [1] developed a damage classification system which accomplishes the relationship between the limiting tensile strain $\varepsilon_{\text{lim}}$ and expected building damages (Figure 2).
Furthermore, in their studies the damaging effect of horizontal strains was added to the LTSM method. Until today the fundamental methods from Burland et al. and the extension from Boscardin & Cording are well established in the damage assessment of overlying buildings affected by tunnelling. There is a large volume of published studies modifying the original method according to Burland et al., or investigating the performance of the LTSM method using real case studies. The focus of the modifications, and the scientific findings of the case studies respectively, illustrate clearly the decisive influence of the buildings stiffness on the accuracy of the damage assessment. On the basis of more than 100 2D Finite Element (FE) analyses, Potts & Addenbrooke [8] expressed the influence of building stiffness on the shape of the settlement trough by establishing so-called modification factors $M^{\text{DR,i}}_\text{hog}$ and $M^{\text{ch},i}$. Taking into account the soil-structure interaction, the modification factors represent

a.) the relationship between the deflection ratio $\Delta/L$ of the settlement trough influenced by the building stiffness and the deflection ratio $\Delta/L$ in greenfield conditions and the

b.) the relationship between the horizontal strain $\varepsilon_h$ influenced by the building stiffness and the horizontal strain $\varepsilon_h$ in greenfield conditions.

According to [8], modification factors are calculated separately for sagging and hogging or respectively for strain and compression conditions (Eqs. (3) and (4)).

$$M^{\text{DR hog}} = \frac{DR_{\text{hog/sag}}}{DR_{\text{hog/sag}}} \quad (3)$$

$$\rho^* = \frac{EI}{E_s (L/2)^4} \quad \text{(relative bending stiffness)} \quad (5)$$

$$M^{\varepsilon_{ht}} = \frac{\varepsilon_{ht/c}}{\varepsilon_{ht/c}} \quad (4)$$

$$\alpha^* = \frac{EA}{E_s (L/2)} \quad \text{(relative axial stiffness)} \quad (6)$$

Considering the ratios $\rho^*$ and $\alpha^*$, in which the stiffness of buildings is related to the soil one, design charts provide the modification factors [8]. In Eqs. (5) and (6) $L$ denominates the length, $E$ the Young’s modulus, $I$ the second moment of area and $A$ the cross-section area of the building. $E_s$ is the secant stiffness of the soil that would be obtained at 0.01 % axial strain in a triaxial compression test performed at the half tunnel depth. Applying the obtained modification factors to the deflection ratio $\Delta/L$ and the horizontal strain $\varepsilon_h$ in greenfield conditions according to Burland et al., a simplified consideration of soil-structure interaction effect is given. Thereby, the classification of a damage category gets more realistic and less conservative. As already mentioned, the building stiffness is obtained by the Young’s modulus $E$ and the second moment of area $I$ of the equivalent beam. Variations of the building
stiffness caused by the combination of stories, floor slabs and the wall design between the stories are neglected. To consider these variations of the building stiffness, Voss (2003) [10] and Finno et al. (2005) [4] suggested a refined model of the equivalent beam. For this purpose they developed a laminated beam, taking into account the fact that modern buildings are often designed with floor and roof diaphragms that are able to distribute the lateral actions between the structural elements. The laminated beam considers layers of rigid plates and core materials, which are deformed by shear forces affected by the stiffness parameter $G_A$. More details on the model can be found in [4].

All approaches have in common, that the approximation of the building stiffness $EI$ is highly simplified.

3 INVESTIGATION AND EVALUATION OF BUILDING STIFFNESS

The depicted approaches illustrate the central position of the building stiffness $EI$ in the damage assessment. The equivalent beam is not sufficiently accurate to take into account the actual complexity of the building’s behaviour [3]. However, it is not entirely certain that the provided results are on the safe side, sometimes the damage risk will be underestimated. To determine the complex influences on the building stiffness parameters ($EI$, $EA$ and $GA$) more accurate, detailed analyses of existing buildings of several eras are necessary. With these results the current damage assessment practice could be improved. Requirements for the research project related to the building stiffness parameters are defined as:

- Output in format $EI$, $EA$ and $GA$ for the application in depicted approaches
- Investigation of single buildings, to reach a categorization according to year of construction, construction type and materials
- Transferability of results in general with quantification of probability of occurrence and uncertainty

3.1 System Identification and Back Analysis

In the framework of the Collaborative Research Center SFB 837, an extensive database, among others of a current tunnelling construction site, is available. This database is well-suited for an extended investigation of building stiffness, especially to identify individual systems, using System Identification and Back Analysis methods. These methods follow the simple scheme of a Black Box calculation (Figure 4).
Due to the degree of complexity of real buildings and the intention to categorize the building stiffness parameters to several buildings types, the development of physical models has to take into account uncertainty. Therefore, it is recommended to consider an error value $\delta X$ or an error function $\delta x(t)$ in the target values $\tilde{E}I(u,y(t)), \tilde{E}A(u,y(t))$ and $\tilde{G}A(u,y(t))$.

$$\tilde{E}I(u,y(t)) = EI(u,y(t)) + \delta x(t) \cdot \delta X$$  \hspace{1cm} (7)
$$\tilde{E}A(u,y(t)) = EA(u,y(t)) + \delta x(t) \cdot \delta X$$  \hspace{1cm} (8)
$$\tilde{G}A(u,y(t)) = GA(u,y(t)) + \delta x(t) \cdot \delta X$$  \hspace{1cm} (9)

### 3.2 Methodological Procedure of System Identification and Back Analysis

Three competing approaches are developed for the description of physical models:

- a.) analytical
- b.) numerical-iterative
- c.) statistical

Consistent to the work of Burland et al. (section 2), the analytical approach also converts the buildings into equivalent beams. Taking into account measured displacements (output $u(t)$) and the greenfield settlements (input $y(t)$), $EI$ and $GA$ are approximated with differential equations of the beam and its individual boundary conditions. Dependent on horizontal movements the building stiffness parameter $EA$ is separately determinable with the same model. However, compared to alternatives this approach leads to higher proportions of $\delta x(t)$, or respectively $\delta X$ due to its high level of simplifications.
In order to describe the boundary conditions and the displacements in the physical model more precisely, a superposition of several equivalent beams is possible. Moreover, according to the principal of virtual forces and the balance between internal and external work, a derivation of the building stiffness parameters is independent from the design of physical models. Employing plate elements the analyses are also extensible to multidimensional systems. Influences on the building stiffness like openings etc. are smeared across the equivalent beam.

The basics in the numerical approach are similar to those of the analytical approach. However, the description of the physical model is carried out applying the Finite Element Method (FEM). Due to a high degree of redundancy, for instance related to a continuous elastic foundation, the solution process must be an iterative one. Following the procedure of Meier et al. [6], the measured values are compared to the values after each calculation. If a defined limit value is exceed, a new calculation with changed stiffness parameters is carried out. Initial parameters of the building stiffness are estimated on the basis of empirical values. Interface-elements model the connection between soil and structure in a simplified way (Figure 5). The performance of interface-elements allows a user-defined configuration of the greenfield settlement trough in the building’s model. Simultaneously the effect of soil-structure interactions is modelled precisely. These properties are achieved with modified spring elements, within a shifted spring characteristic in magnitude of local greenfield settlement values. The horizontal movements in this approach are included by the coefficient of friction of transvers springs. However, the influence of horizontal movements is, already in case of low building stiffness’, negligibly small [3].

Figure 5: Basic principles and features of Interface-Elements
In the statistical approach the sample size of input data and the preferably homogenous boundary conditions of shield tunnelling play a decisive role. If these requirements are satisfied, numerous factors of buildings are compared to measured displacements. The most important factor, according to the approaches of Burland et al. and Potts et al., is the deflection ratio $\Delta/L$ in areas of sagging and hogging. Assuming that all buildings are affected by identical displacement reactions, a grouping with similar building stiffness parameters is possible. Including factors like the proportion of windows etc., the comparative analysis permitted quantitative information of the building stiffness parameters. Of course, in real cases tunnelling is not homogenous, so that a standardisation of the shield tunnelling parameters is necessary. The initial point of a standardisation is the relation to stated reference values. These can be variable factors like the supporting pressure or the cover.

To avoid enormous efforts, a suitable method for an automatic and dynamic grouping of buildings is the discriminant analysis [7]. Related to features for instance the deflection ratio $\Delta/L$, a discriminant function with the option of a subsequent grouping of more buildings is expressed. Similar to an artificial neural network, the quantification of the building stiffness parameter gets more precise by increasing the size of sample.

### Table 1: Comparison and evaluation of System Identification based approaches

<table>
<thead>
<tr>
<th>Approach</th>
<th>Accuracy</th>
<th>Complexity / effort of implementing</th>
<th>Transferability to damage assessment methods</th>
</tr>
</thead>
<tbody>
<tr>
<td>analytical approach</td>
<td>strongly influenced by the quality of physical models and displacement measurements</td>
<td>high processing speed due to large simplifications; many calculations in a very short time</td>
<td>direct applicability of results in existing damage assessment</td>
</tr>
<tr>
<td>numerical-iterative</td>
<td>consideration of complex structures and other building stiffness influencing details</td>
<td>high numerical efforts and computationally intensive iterationsteps</td>
<td>transfer of numerical results into an equivalent beam using the Rompello method (2012)</td>
</tr>
<tr>
<td>statistical approach</td>
<td>strongly influenced by quality and size of sample</td>
<td>time consuming data management; rapid data processing of new samples after establishing a discriminant function</td>
<td>only quantitatively improvement using factors etc.</td>
</tr>
</tbody>
</table>

As already described by Meyerhof (1953), the settlement induced displacements decrease over the building’s height. Hence, in the numerical-iterative approach and in the statistical approach the influence of strain and compression over the building’s height will be analysed more detailed.

Each of the mentioned approaches has its specific advantages and disadvantages. To verify the building stiffness parameters a cross-check within the approaches is planned.
3.3 Existing Shield Tunnelling Database

An extensive database of a current tunnelling construction site is the basis to put the approaches (section 3.2) successfully into practice. Beside the often more than 100 years old building’s documentation, area-wide and precise displacement values are available. In addition to approx. 1200 terrestrial measurements, approx. 16000 spaceborne measurements are processed in the framework of the SFB 837. The applied method to measure displacements on earth surface is called Persistent Scatterer Interferometry (Ferretti et al. 2001), which is based on the Differential Interferometric Synthetic Aperture Radar (DInSAR) technique. To attain area wide and accurate measurements at a construction site of a tunnelling project, data provided from the TerraSAR-X satellite (German Aerospace Center) are used [5].

Based on available machine data of the shield tunnelling process (advancement speed, supporting pressure, etc.), the standardisation of the shield tunnelling influences in the statistical approach is realised. Additional data from other tunnelling construction sites are appropriate to improve the assessment of building stiffness parameters and the categorisation of typical buildings.

4 CONCLUSION

State of the art in tunnelling induced damage assessment is to model buildings as simple beam. However, the behaviour of real structures is much more complex. Hence, a detailed knowledge of the building’s specific stiffness parameters is essential for a realistic and economic damage assessment. The three approaches, proposed and evaluated with respect to their performance in this paper, are individually or combined suitable to connect building stiffness parameters with different types of overlying structures. For the first implementation a huge amount of displacement and building data of a current tunnelling construction site are analysed. However, further data are needed to generalise the results and to minimise the total amount of uncertainty.

The common used, but simplified approaches [2], [8] – outlined in the beginning of this paper – can still be used. But recalculated building stiffness parameters EI, EA and GA of specific building categories should be employed.

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Topography Influence on Subsidence due to Horizontal Underground Mining Using the Influence Function Method

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Abstract

The classic influence function method is widely used in mining subsidence prediction, but it is limited to predict the subsidence of the horizontal stratiform underground mining under flat surfaces. After studying the original method and the influence of topography on subsidence under simplified mining conditions, this paper tries to improve the influence function method in order to take topographic variations into account. An adjustment of the influence radius is firstly developed. Then, other improvements concern the max subsidence value and influence angle. After this, the influence function method can well fit numerical simulation results which are used to stand for the field mining conditions in Lorraine. This improved method can be the foundation of further work.

Keywords: Topography influence, horizontal underground mining, influence function, max subsidence value, influence angle
1 INTRODUCTION

Mining subsidence affects the security and functions of farmlands, buildings, roads and so on. Many methods have been developed to predict such subsidence such as empirical, analytical and profile function, or physical and numerical model methods [7, 16]. The influence function method is the most used method but it is limited. One of his limitations is the fact that it considers only flat surface and flat stratiform ore body. Some researchers have studied how the subsidence is affected by topography, especially in mountainous regions, and tried to modify the influence function to fit field data [2-5, 8-13]. All of them showed that when the surface dip angle goes greater, the maximum vertical subsidence value increases and the subsidence basin becomes asymmetrical. In this paper, we suggest some improvements of the influence function method so that it can deal with non-horizontal surface conditions.

2 THE INFLUENCE FUNCTION METHOD

The influence function method aims to calculate the vertical subsidence induced by the extraction of a horizontal stratiform underground mining layer. Other movements and deformations can then be derivated form the vertical movement.

In practice, there are many mathematical functions, which can be adopted as an influence function. The two most widely used functions are listed in Table 1 [7, 15, 16]. The influence function depicts the vertical subsidence of several surface points (in the influence range of it) due to one elementary mining zone. The final full-scale vertical subsidence basin can then be calculated as the superposition of all the elementary subsidence due to all excavated mining elements.

Referring to Figure 1, due to a horizontal underground mining, in a flat terrain, the subsidence curve shapes are symmetrical. The slope and horizontal displacement have similar shapes, as well as curvature and horizontal deformation. The inflection points of the vertical subsidence are at the ribs of the extraction area.

Table 1: Some influence functions used in subsidence prediction

<table>
<thead>
<tr>
<th>Author</th>
<th>User (Year)</th>
<th>Function</th>
<th>Remark</th>
</tr>
</thead>
<tbody>
<tr>
<td>Knothe</td>
<td>Knothe (1953,1957) Zenc (1969) Whittaker and Reddish (1989)</td>
<td>[ \frac{1}{R^2} e^{-\frac{r^2}{R^2}} ]</td>
<td></td>
</tr>
<tr>
<td>Litwiniszyn</td>
<td>Litwiniszyn (1957) Liu Baochen and Liao Guohua (1965) Sroka A.Hejmanowski (2006)</td>
<td>[ \frac{n}{R^2} e^{-n\pi \left(\frac{L}{R}\right)^2} ]</td>
<td>With n =1 or n =2, in relation to strata conditions</td>
</tr>
</tbody>
</table>
3 TOPOGRAPHY INFLUENCE ON SUBSIDENCE

3.1 Data Sources

The present work makes reference to simplified numerical simulation models (the overlying strata of models are simplified to one continuous stratum, as well as the surface shape to a global slope) to analyze the topography influence only. All strata are isotropic and have the characteristics mentioned in Table 2.

Table 2: The physical and mechanical characteristics of the model

<table>
<thead>
<tr>
<th>stratum</th>
<th>density kg/m³</th>
<th>Y modulus GPa</th>
<th>P ratio</th>
<th>cohesion MPa</th>
<th>tension MPa</th>
<th>friction °</th>
</tr>
</thead>
<tbody>
<tr>
<td>roof</td>
<td>2540</td>
<td>0.4</td>
<td>0.22</td>
<td>3.0</td>
<td>2.0</td>
<td>22</td>
</tr>
<tr>
<td>ore body</td>
<td>1700</td>
<td>3.5</td>
<td>0.35</td>
<td>3.0</td>
<td>2.4</td>
<td>25</td>
</tr>
<tr>
<td>floor</td>
<td>2590</td>
<td>2.1</td>
<td>0.40</td>
<td>1.0</td>
<td>2.0</td>
<td>38</td>
</tr>
</tbody>
</table>

3.2 Subsidence Characteristics changed by Topography

Under these simplified conditions, Figure 2 shows the vertical and horizontal displacements obtained in Flac 3D, in the inclined main section. The mean depth of
models is 200 m, the extraction zone is a 300 m long square and the excavation thickness is 5 m. With the increase of the surface angle, the maximum vertical subsidence increases, and moves in the downward direction of the surface; the positive horizontal displacement value increases, meanwhile the negative value decreases; the influence range decreases downward but increases upward. Both the vertical subsidence and horizontal displacement become asymmetrical.

4 IMPROVING THE INFLUENCE FUNCTION METHOD

The influence function method has been introduced into a code initially developed in our Laboratory [15]. All the following improvements depend on this code.

4.1 Adjustment of the Influence Radius

The classic influence function (takes Litwiniszyn’s function as example, see Table 1) can be used in plane surface for calculating several surface points’ subsidence (in the influence radius) induced by one mining element. But when the topography is rough, this function must be calibrated. Above all the next improvements, the influence function should use a varying influence radius ($R$).

Referring to Figure 3, when the surface is not horizontal, varying influence radius should be used for calculation ($R_{xa}$ for point $P_xa$, $R_{xb}$ for $P_xb$), which are given by the influence angle of the mining element and the elevation difference between the surface point and the mining element. For comparison, we use the same influence radius ($R_0$) when surface is flat. We can then change the original function into Eq.(1).

$$f(r,R(H(x),\varphi)) = \frac{n}{(H(x)\cdot\tan \varphi)} e^{-n \pi \left(\frac{r}{H(x)\cdot\tan \varphi}\right)^2}$$

(1)

Where $H(x)$ is the elevation difference between the mining element and the surface point under consideration and $\varphi$ is the influence angle of the mining element.
4.2 Improvement of the Influence Function Method

4.2.1 Improvement due to varying max subsidence

Let us remember first that one surface point’s subsidence is the integration of the product of the influence function by the expected maximum subsidence ($W_{max}$).

According to numerical simulations, Figure 4 shows the relationships between the subsidence coefficient ($q=W_{max}/m$, where $m$ is the mining thickness) and the mining depth. It can then be depicted as Eq.(2). In our case, $a \approx 0.2$, and $b \approx 45$. Then the influence function can be improved as in Eq.(3).

$$q(H) = a + \frac{b}{H}$$  \hspace{1cm} (2)

$$W_{ps} = \int_{ex-zone} m \cdot q(H(x)) \cdot f(r, R(H(x), \varphi))dx$$  \hspace{1cm} (3)

**Figure 4:** The subsidence coefficient ($q$) vs mining depth

4.2.2 Improvement due to varying influence angle

When analyzing the numerical simulation results, two different influence angles appear on both sides of the ground surface: upward and downward. They vary as a linear function with the change of surface dip angle, referring to Figure 5. The slopes of the two curves have almost the same absolute values, and same intersection points. Therefore, we can use a single piecewise equation to depict the change of the influence angles with the surface dip angle. Then, a piecewise influence function can be used as in Eq.(5). Here, $k \approx 0.27$ and $c \approx 52.5^\circ$.

$$f_i(r, R) = \begin{cases} f(r, R(H(x), k\alpha + c)) & \text{-- x in the downward direction} \\ f(r, R(H(x), -k\alpha + c)) & \text{-- x in the upward direction} \end{cases}$$  \hspace{1cm} (5)

where $\alpha$ is the surface dip angle.

Synthesizing the both previous improvements, the final subsidence can be calculated by the new piecewise influence function, as in Eq.(6).

$$W_{ps} = \int_{ex-zone} m \cdot q(H(x)) \cdot f_i(r, R)dx$$  \hspace{1cm} (6)
Figure 5: Variation of the influence angles with surface dip angle (D in the legend means average mining depth of model)

4.3 Comparison

Figure 6 shows that, taking the vertical subsidence as example, the influence function method works much better with the varying max subsidence and the varying influence angle taken into account than when the method is just adjusted according to the influence radius.

Figure 6: Numerical simulation vs influence function method with adjusted influence radius only vs influence function method with adjusted influence radius, improved influence angle and improved maximum subsidence value (surface dip angle=15)

5 CONCLUSION

The classic influence function method is suitable in the prediction of the subsidence induced by the extraction of a horizontal stratiform underground mined ore layer under a flat surface. An adjustment of the influence radius has been introduced into this method first. The new radii depend on both the influence angle of the unit mining element and the elevation difference between the surface points and the mining element under calculation.
The final subsidence of one surface point due to an entire mining zone can be calculated by the integration of the product of the expected maximum subsidence value by the influence function. Based on the results of numerical simulations, varying maximum subsidence values and influence angles have been introduced when the surface is not horizontal. The maximum subsidence values should be inversely proportional to the mining depth and two influence angles for every unit mining element are employed for surface upward and downward directions. After the adjustment of the influence radius and the improvements introduced for the max subsidence values and the influence angles, better results are achieved with the so-improved influence function, which are closer to numerical simulation results.

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A 3D Temporal Evolutionary Numerical Model of a Masonry Building in Barcelona subjected to Tunnelling Subsidence

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Abstract

The aim of this work is to deepen the knowledge of the numerical simulation used in the prediction of building damage produced by tunnelling subsidence. This paper analyzes the structural response of a one-storey building subjected to real ground movements experienced during the construction of the L9 Metro line tunnel in Barcelona, bored by a Earth Pressure Balance Machine (EPB) Tunnel Boring Machine (TBM). A 3D temporal evolutionary numerical model is used to predict the damage in the building resulting from tunnelling subsidence. The real structural damage presented is compared with the predictions of the numerical model. This last task can be done since the measures of real ground movements given by the monitoring instruments, such as retro-reflective prisms and total stations, are available for this research. Main model parameters have been determined by means of characterization experiments developed on the site and in the laboratory, thus giving a higher significance to the analysis.

Keywords: Settlements, structural damage, sensitivity, tunnelling, numerical model, 3D analysis
1 INTRODUCTION

Tunnels are underground constructions that permit the integration of transportation infrastructures in a city due to the general lack of space on their surfaces. Therefore, most of these infrastructures are usually built beneath buildings and historical constructions. This fact makes extremely important to carry out reliable predictions of the existing risk of damage due to the ground subsidence generated by tunnelling works.

Common prediction of the damage is developed during the design of the tunnel and it comprises several steps. Primary assessments start with the use of empirical approaches that estimate a category of damage in the buildings according to the maximum tensile strain reached in it, assuming that the building is represented as an elastic beam which conforms to a Gaussian settlement profile (see references [1] to [6]). The use of Finite Element methods in 2D and 3D are optimal when more detailed predictions of damage are pretended, for instance, to assess the location and width of crack patterns.

This paper shows a 3D temporal evolutionary numerical model of a real case consisting of a set of one-storey masonry dwellings from the 1920's in the neighbourhood of Bon Pastor (Barcelona). These buildings are located over a stretch of the L9 metro tunnel bored by a TBM-EPB. The tunnel of 12m diameter, 0.35m of segment thickness and only 9m of overburden was bored in deltaic ground conditions due to the vicinity of the Besòs river. All these factors entailed an important ground subsidence (the maximum monitored settlement was about 40mm) that produced the appearance of several cracks in the buildings. The tunnel lining, the soil and the excavation process are also included in the model, thus doing a step forward on the research carried out in [7] and [8], where primary 2D and 3D numerical simulation were performed. Monitoring data of ground movements, damage surveys and material properties are available to give a higher significance to the analyses.

2 DESCRIPTION OF THE CASE STUDY

2.1 Background

The L9 Metro line in Barcelona is a reference tunnelling project in Europe due to its length (more than 40Km), large excavation diameters (9.4m and 12m) and the wide variety of geological and hydrological conditions encountered. Nowadays, 8Km of the line are already in service in the north part of the city.
The set of one-storey masonry buildings selected for this study represent a common building typology (Figure 1) frequently used in the 20's due to the construction of residential complexes for workers coming from south Spain due the World Exposition that took place in Barcelona during the 1929. The poor bearing capacity of the soil, the presence of groundwater, the low depth of the tunnel and the initial structural state of the adjacent buildings are key factors in the present case study [7].

![Figure 1: One-storey masonry dwellings from the 1920's located in the outskirts of Barcelona](image)

### 2.2 Geometrical Survey and Characterization of Materials

The masonry dwellings have a squared plant of dimensions $8m \times 8m$ and a terrace in the front part. The structural components comprise the facade, which acts as a load bearing wall, two interior bearing walls and three columns. The facade has a thickness of 200$mm$, the interior bearing walls, 140$mm$, and the rest of walls, 40$mm$.

The in-place strength of mortar was determined by means of penetrometer tests. These tests, in addition to X-Ray diffraction techniques, showed a notable quality difference between the mortar used in the facade and in the columns (lime mortar with 1.7$MPa$ of compressive strength and $CV^1= 76\%$ for the facade; cement mortar with 28.7$MPa$ of compressive strength and $CV= 10\%$ for the columns).

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$^1$ Coefficient of Variation of results
The compressive strength of the ceramic bricks was 18MPa (CV=35.6%). The characteristic compressive strength was then equal to 4.87MPa and the vertical modulus of elasticity was 2678.5MPa. For columns, these values raised up to 11.39MPa and 6264.5MPa respectively. For further details on the determination of these magnitudes, see [7] and [8].

3 EXPERIENCE IN 3D MODELLING OF BUILDINGS SUBJECTED TO TUNNELING SUBSIDENCE

The use of 3D models for the simulation of tunnelling problems has been a matter of interest through time. The improvement of computer features makes easier the resolution of more complex issues and situations. As it was mentioned previously, the present model includes the ensemble simulation of the masonry dwellings, the soil tunnel lining and the excavation process, since it is considered that the longitudinal settlement profile can cause important cracking damage [9], in particular when the tunnel heading is still far from the buildings. Damage mechanisms can combine a mixture of shear deformation, arching and bending behaviour [10].

The advance of the TBM-EPB through the soil will be simulated according to the methodology proposed in [11], [12], [13] and [14], where the excavation was implemented by the application of loads on the lining to produce its shrinkage and hence to simulate the volume loss of a linear elastic soil. Although this technique may induce unrealistic strains and stresses in the vicinity of the tunnel, it was shown that, in case of deep tunnels, the produced ground movements were reasonably similar to the measured ones.

Recent works as [15] show numerical analyses in 3D of a masonry building with a pile foundation subjected to tunnelling subsidence in soft soil. This study remarks that aspects like the soil-structure interaction can be well simulated by using macro-models for masonry in a stage analysis.

4 DESCRIPTION OF THE NUMERICAL MODEL

The numerical model simulates the structural behaviour of a group of 12 dwellings distributed in two rows of 6 houses over an area of 370m x 235m and 35m of depth of alluvium soil. Such a high extension of soil is taken to avoid the transmission of boundary effects produced by the model limits. The dimensions of each dwelling are 3m of height and 8m wide. All of them have two windows of 1m x 1m and a door of
2\(m\) of height and 0.80\(m\) wide. This pattern is repeated along the dwellings. The tunnel stretch of 300\(m\) of radius is also included.

The first dwelling (from now on, DW1; see Figure 2) was the one subjected to the most severe subsidence since the tunnel axis is underpassing just beneath of it. For this reason, this dwelling is modelled in more detail, thus containing all its inner partition walls and column with their specified thickness (Figure 3). A non-linear analysis is performed here with a high density mesh. Facades of dwellings DW2 and DW3 are equally simulated, since damage was also reported in the surveys.

On the other hand, the rest of dwellings are included in linear elastic regime to transfer boundary conditions to the most detailed part of the model (the affection on these walls was not noticeable). Hence, the mesh density decreases gradually when getting far from DW1 in order to reduce the computational costs.

![Figure 2: Aerial view of the tunnel underpassing the dwellings](image-url)
4.1 Model Mesh

Model mesh is formed by 4 different element types. In order to model the set of buildings, 33078 curved hyperbolical shell elements are used in DIANA®, since they permit the calculation of plastic and cracking strains. From these, 21554 are quadrilateral elements with 4 nodes (Q20SH) that perform the non-linear analysis in the DW1 and the adjacent walls. All the other elements (11524) are triangular with 3 nodes (T15SH), performing a linear elastic analysis. Element size is 100mm for the quadrangular elements, thus decreasing the element size till 600mm in the zones out of the influence of the tunnel.

The tunnel lining mesh is formed by 13062 quadrangular plane shell elements (Q20SF) of 900mm of element size that permit only linear elastic calculations. The ground is modelled using 183818 three-side iso-parametric solid pyramid elements (TE12L). The generation of this 3D model was done by filling the interior of a low-density contour 2D mesh. Therefore, this mesh was successfully connected to lining and building elements with an increase of the mesh density in the zones of interest. The contact between the soil and the building is modelled by a set of 1153 two-node spring elements (SP2TR) for each coordinate direction (3459 springs in total, see Figure 4), which allow gapping between soil and masonry wall.
4.2 Applied Loads and Modelling of the Tunnel Excavation

The self-weight is determined according to a typical value of density for masonry (1800Kg/m$^3$). The roof load of buildings is estimated in 2.5 KN/m$^2$ acting in a surface of 16m$^2$ ($8m \times 8m$). Loads are distributed on the bearing walls and columns according to their contributive area.

Tunnel excavation and heading advance is modelled by applying successively pressure loads on the rings to produce its shrinkage towards the concentric direction. Therefore, the soil elements which are directly connected to the lining elements will also deform, thus producing a field of ground displacements that fits well to a Gaussian transversal and longitudinal profile of settlements ([13], [14]).

The first attempts to model the settlement trough were done by fixing the tunnel invert and mid-height lateral points. However, the present case shows a very shallow tunnel ($z/D=1.25$) and thus, the application of concentric pressure gave an excessive flat Gaussian profile which did not correspond to the reality. A solution for this problem was solved when fixing all the points of the tunnel from the invert till approximately 3/4 of the height. Pressure loads were equally applied and the shape of the Gaussian profile become more similar to the real one. The data available from 15 retro-reflective prisms installed on building facades to control vertical and horizontal movements permitted the calibration of these results.
The excavation process was modelled in 3 phases: 1) When tunnel heading was close to the building but not yet beneath of it (the structure is mainly subjected to the longitudinal settlement profile); 2) the tunnel heading was just beneath the DW1 and 3) the tunnel heading had completely underpassed the building and the settlements were stabilized. It must be emphasized that the soil in the interior of the tube is not modelled to simplify the analysis.

4.3 Materials

The non-linear analysis of DW1 and its adjacent walls is performed using the Total Strain Rotating Crack model (from now on, TSRC). This is a fracture model with distribute cracking usually employed to evaluate the non-linear behaviour of brittle materials. Masonry is therefore treated assuming isotropic properties.

According to the data of the material characterization and considering the properties in the weak axis of masonry, the parameters of the walls and the columns are shown in Table 1. The constitutive model of masonry is given by an exponential function to represent the softening of the material when reaching the post-peak tensile stress. A parabolic model is used for the compressive behaviour.

<table>
<thead>
<tr>
<th>MASONRY WALLS PROPERTIES</th>
<th>(E) (**)</th>
<th>(G) (*)</th>
<th>(n) (**)</th>
<th>Density (**)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2678.5 MPa</td>
<td>1071.4 MPa</td>
<td>0.20</td>
<td>1800 Kg/m³</td>
<td></td>
</tr>
<tr>
<td>(f_t) (**)</td>
<td>(f_c) (**)</td>
<td>(G_{ft}) (**)</td>
<td>(G_{fc}) (**)</td>
<td></td>
</tr>
<tr>
<td>0.05 MPa</td>
<td>3.24 MPa</td>
<td>0.02 N·mm/mm²</td>
<td>10 N·mm/mm²</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>MASONRY COLUMNS PROPERTIES</th>
<th>(E) (*)</th>
<th>(G) (*)</th>
<th>(n) (**)</th>
<th>Density (**)</th>
</tr>
</thead>
<tbody>
<tr>
<td>6264.5 MPa</td>
<td>2505.8 MPa</td>
<td>0.20</td>
<td>1800 Kg/m³</td>
<td></td>
</tr>
<tr>
<td>(f_t) (**)</td>
<td>(f_c) (**)</td>
<td>(G_{ft}) (**)</td>
<td>(G_{fc}) (**)</td>
<td></td>
</tr>
<tr>
<td>0.05 MPa</td>
<td>7.59 MPa</td>
<td>0.02 N·mm/mm²</td>
<td>10 N·mm/mm²</td>
<td></td>
</tr>
</tbody>
</table>

The soil is modelled as a linear elastic material with a coefficient of Poisson equal to 0.2. and a Young of \(E=1000MPa\). This value is unrealistic for the type of soil of this case study. However, it helps to adjust correctly the settlement field, especially in
zones far from the tunnel, where lower values of $E$ lead to an excessive amount of settlements due to low depth of the tunnel. Tunnel lining is also modelled as a linear elastic material rigidly connected to the soil, with a Poisson coefficient of 0.2 and Young modulus equal to $30,000 MPa$. Self-weight of lining and soil is not considered. The springs in the $Z$-axis direction are modelled as non-linear rigid supports connecting the base of the building and the soil. These springs have no tensile strength to avoid the appearance of unreal traction loads in the foundations due to the higher stiffness of the building respect to the soil. The springs in the horizontal axes are modelled as linear springs to avoid the creation of structural mechanisms.

5 RESULTS OF THE SIMULATION AND COMPARISON TO SURVEYS

The damage described in the surveys showed several diagonal and vertical cracks with origin in the corners of windows and doors till the lintel of the facade (see Figure 5). The right corner of the DW1 was the zone subjected to the most severe settlement. Thus, horizontal cracking appeared also in the lateral wall of DW1 since it avoided a higher descent of the facade corner. Minor cracking appear also on the base of windows.

![Figure 5](image)

**Figure 5:** Damage occurred due to tunnelling works (extracted from [7]).

The comparison between the cracking patterns of Figure 5 and the results obtained from the non-linear analysis show good agreement. Diagonal cracks in the interior walls appear as it was described in the surveys. Horizontal cracking of the lateral walls is also well simulated in the numerical analysis (Figure 6). Regarding the cracks in the facade wall given by the model (Figure 7), vertical cracks appear starting from the corners of windows and doors. In DW1, diagonal cracking can be seen between the door and the right window.
It must be taken into account that the building starts to crack when tunnel heading is not yet beneath the building due to the influence of the longitudinal settlement profile. However, the application of the phased analysis didn't show significant changes of cracking patterns in comparison when the shrinkage of the tunnel was done just in one phase, thus not considering the temporal component.

**Figure 6:** Numerical results of cracking patterns in DW1

**Figure 7:** Numerical results of cracking patterns in the facade

### 6 CONCLUSIONS

The analysis of these masonry buildings by using the Total Strain Rotating Crack model together with a linear elastic model of the lining and the soil has been proved to be an effective method to predict cracking patterns in the structure. The use of the technique of shrinking the tunnel produces a settlement field that agrees with registered monitoring data. The application of the pressure loads on a rigid enough lining allows to run a phased analysis which can reproduce satisfactory the underpass of the TBM without the need of removing elements simulating the excavation of the soil.
ACKNOWLEDGEMENTS

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REFERENCES


Three Dimensional Finite Element Analysis of a Strutted Sheetpile Supporting a Deep Excavation

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Abstract

This paper presents the results of three dimensional (3D) finite element analysis of a strutted sheetpile excavation in layered soil. The finite element analysis was prepared to model the Ford Engineering Design Center (FEDC) excavation in Evanston, Illinois. Available geotechnical data from this documented case history included borehole logs, laboratory test results, and field SPT / CPT measurements. The finite element model matched the geometric description and construction sequence of the deep excavation using interpreted soil parameters from the documented site investigation. The developed model estimated deflections around central and corner locations. Modeled deflections were compared to recorded field measurements from three inclinometers installed near central and corner locations of the excavation. The error in the estimated deflections was less than 30\%, thus, the developed 3D model of the deep excavation is considered adequate. This paper highlights the capabilities of 3D finite element model to capture the behaviour near central and corner sections, the importance of properly modeling the excavation sequence in order to achieve realistic modeling results, and the importance of using such case histories to improve interpretation of soil parameters from laboratory or in-situ tests.

Keywords: Deep excavation, 3D finite element, field measurements, construction sequence
1 INTRODUCTION

Deep excavation projects are usually conducted in areas of scarcity of land where excavation areas could be surrounded by high-rise buildings; therefore, a great concern is directed to the ground movements associated with the retaining system of such excavations. Numerical methods have been widely applied to estimate ground movements associated with deep excavations (Ou et al., 1996; Lee et al., 1998; Finno et al., 2006; and Lin and Woo, 2007). This study presents the development of a three-dimensional finite element model using PLAXIS version 2010 of the Ford Engineering Design Center (FEDC) excavation in Evanston, Illinois. Available geotechnical data for this documented case history (Blackburn, 2005) including borehole logs, laboratory test results, and field SPT / CPT measurements was used to interpret the model input soil parameters. The development of the model input soil parameters is discussed in this paper. Ground deformations were monitored in this site around central and corner locations (Blackburn, 2005). The developed model estimated deflections around central and corner locations, which were compared to the recorded field measurements. Modeling the excavation sequence is presented herein along with discussion on its importance to achieve realistic modeling results.

2 EXCAVATION SITE AND SHORING WORKS DESCRIPTION

Ford Engineering Design Center (FEDC) was constructed on the Northwestern university campus in Evanston, Illinois. The building was founded on drilled caissons, and consisted of five above-grade levels and two basement levels. Sheetpile walls with walers and internal bracings were used as a shoring system to support a total excavation depth of 9.1m. The initial ground elevations around the site were not equal. The east and west sides of the site were approximately at elevation (+5.5)m, while the north and south sides were at elevation (+3.7)m and (+3.5)m, respectively. FEDC building was founded in close proximity (within 5m) to the technological institute (Tech) building, which was founded on a shallow foundation system of strip and isolated footings. The strip and isolated footings of the Tech building were founded at elevations ranging from (+0.4)m to (+1.7)m. Further details on the neighbour building foundations are presented in Blackburn (2005).

Shoring works consisted of XZ85 section sheetpile walls supported by two levels of internal bracing. The two levels of internal bracings consisted of beams (diagonal corner bracing) and pipes (cross-lot bracing). The cross-lot bracings were supported in the center by H-pile supports. The two levels of internal bracings were preloaded.
Preloading forces are included in Blackburn (2005). Figure 1 shows the layout and cross sections of lower internal bracing system. Two levels of waler beams were attached to the sheetpiles. Further details of the shoring works system are presented in Blackburn (2005).

![Figure 1: Lower level bracing layout (after Blackburn, 2005).](image)

### 3 SUBSOIL INVESTIGATION

Site investigation conducted in the FEDC site included five (5) boreholes (with SPT measurements) that extended to depth of 25m. Field tests included seventeen (17) vane shear tests and six (6) pressure meter tests. In addition, five (5) cone penetration tests (CPT) were performed. Laboratory testing included fifty eight (58) unconfined compressive tests. Subsoil investigation data is documented in Blackburn (2005).

Cone penetration test (CPT) results along with borehole logs presented in Blackburn (2005) were used to interpret the soil stratigraphy. Soil classification based on CPT results were interpreted in accordance with Robertson (1990). The resulting stratigraphy consisted of the following layers:

1) **Top Sand:** Medium dense, fine to coarse sand layer between elevations (+5.3)m and (+1.0)m.

2) **Clay Crust:** Stiff silty clay layer between elevations (+1.0)m and (-1.0)m.

3) **Upper Clay:** Soft to medium stiff clay layer between elevations (-1.0)m and (-5.0)m.

4) **Middle Clay:** Medium stiff to stiff clay layer between elevations (-5.0)m and (-12.0)m.
5) Lower Clay: Stiff clay layer between elevations (-12.0)m and (-16.0)m.
6) Sand/Silt/Clay mixture: Dense, sand/silt/clay mixture extending from elevation (-16.0)m to the bottom of investigated depth.

4 INSTRUMENTATION

Four inclinometers were installed adjacent to FEDC excavation as shown in Figure 2. The recorded data of three inclinometers (I1, I2, and I5) reported in Blackburn (2005) were considered in this study. The alley between the project area and the Tech building was monitored by Inclinometers 1 and 2 extending to elevation (-22.0)m. Inclinometer 5 was placed near the Sheridan road about 5m from the west sheetpile side and extended to elevation (-19.2)m. Inclinometer 3 was installed near a utility shaft area, which was not considered in this study.

Strain gages were installed on the internal bracings to evaluate the average strain, and thus estimate the internal stresses. Gages were installed prior to preloading, yet after the strut placement on the waler. Variation of average strain of internal bracings was monitored along with the surrounding temperature, which was then used to account for internal stresses generated within bracings in the numerical model. Further details on the strain gages locations, data, and internal stresses consideration in finite element modeling is discussed in Blackburn (2005) and Helwa (2012).
5 CONSTRUCTION DETAILS

Details on excavation works and bracings installation are presented in Blackburn (2005) and are summarized as follows:

1) Levelling the site area to elevation (+3.4)m.
2) Installation of sheetpiles.
3) Excavation of corner areas to elevation (+1.0)m. Meanwhile, center of north and south walls were supported by access ramps.
4) Installation of upper waler and upper diagonal corner bracings at elevation (+1.5)m.
5) Excavation proceeded to elevation (-1.5)m excluding access ramps areas.
6) Installation of lower waler and lower diagonal corner bracings at elevation (-1.0)m.
7) Installation of two levels of cross-lot bracings (elevations (+1.5)m and (-1.0m)). Access ramps were removed simultaneously.
8) Excavation to elevation (-3.8)m.

6 NUMERICAL MODELING

6.1 Geometry

The excavation area has plan dimensions of 43.5m x 34.9m, and depth of 9.1m. The model dimensions are 350m x 350m in the horizontal direction and 20m in the vertical direction to eliminate end effects. The project area and underlying soil layers were assumed horizontal. A plan view of the FEDC model is shown in Figure 3. Distribution of internal bracings followed patterns presented in Section 2. Elevations of internal bracings and walers followed construction details presented in Section 5. Sheetpile walls were modeled as anisotropic plate elements with axial to lateral wall stiffness ratio of 20 (default Plaxis value for sheetpiles). It is noted that other axial to lateral stiffness ratios have been recommended in various studies for other retaining wall systems (Zdravkovic et al., 2005 and Helwa, 2012). Internal bracings were modeled as node to node elements, while waler beams were modeled as beam elements. Input parameters for sheetpile walls, internal bracings, and waler beams are presented in Helwa (2012).
Figure 3: FEDC Plaxis 3D model.

6.2 Excavation and Construction Stages

The authors tried to simplify the construction stages by ignoring the existence of access ramps in the model; however, the resulting deflections trend with depth did not match the field measurements trend. Among the trials to enhance the model performance, the authors decided to model the access ramps and their staged removal simultaneously with the cross-lot bracings installation, which resulted in better simulation of the problem under study. Therefore, it was concluded that proper detailed modeling for construction stages is essential to achieve realistic modeling results. A final consolidation stage was important to account for pore-water drainage during the considered project duration (about 5 months) between site levelling and excavation to elevation (-3.8)m. Excavation and construction of retaining walls for FEDC site was modeled in thirteen stages followed by a consolidation stage as summarized in Table 1. Construction details of the utility shaft area near Inclinometer 3 and the elevator pit within the excavation area were not considered in this study, as they do not affect the readings of Inclinometers 1, 2, and 5 considered herein.

6.3 Constitutive Model and Model Parameters

Hardening soil model was used in the current study. A basic feature of the Hardening soil model is the dependency of soil stiffness on stress level. Therefore, all input stiffness values were related to a reference confining stress ($P_{ref}$), defined at the center of each soil layer.
Table 1: FEDC modeled construction stages.

<table>
<thead>
<tr>
<th>Phase No.</th>
<th>Phase Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Initial phase using Ko procedure.</td>
</tr>
<tr>
<td>2</td>
<td>Tech building foundation deployment and loading.</td>
</tr>
<tr>
<td>3</td>
<td>Open excavation to elevation (+3.4)m.</td>
</tr>
<tr>
<td>4</td>
<td>Sheetpile installation.</td>
</tr>
<tr>
<td>5</td>
<td>Excavation to elevation (+1.0)m, excluding access ramps areas.</td>
</tr>
<tr>
<td>6</td>
<td>Upper waler and diagonal corner bracing installation at elevation (+1.5)m.</td>
</tr>
<tr>
<td>7</td>
<td>Excavation to elevation (-1.5)m, excluding access ramps areas.</td>
</tr>
<tr>
<td>8</td>
<td>Lower waler and diagonal corner bracing installation at elevation (-1.0)m.</td>
</tr>
<tr>
<td>9</td>
<td>Access ramps excavation to elevation (+1.0)m.</td>
</tr>
<tr>
<td>10</td>
<td>Upper cross-lot bracing installation at elevation (+1.5)m.</td>
</tr>
<tr>
<td>11</td>
<td>Access ramps excavation to elevation (-1.5)m.</td>
</tr>
<tr>
<td>12</td>
<td>Lower cross-lot bracing installation at elevation (-1.0)m.</td>
</tr>
<tr>
<td>13</td>
<td>Excavation to elevation (-3.8)m.</td>
</tr>
<tr>
<td>14</td>
<td>Consolidation stage (t=158 days).</td>
</tr>
</tbody>
</table>

Undrained effective stress analysis (Method A in Plaxis) was followed, in which effective stiffness and effective strength parameters are assigned to soil clusters. Pore-water pressures can be predicted with this type of analysis, and thus can be followed by a consolidation phase.

Input soil parameters included bulk unit weight ($\gamma_t$), soil permeability ($k_x$ and $k_y$), oedometer loading stiffness ($E_{oed}$), triaxial loading stiffness ($E_{50}$), triaxial unloading/reloading stiffness ($E_{ur}$), power (m) for stress-dependent stiffness formulation, Poisson’s ratio for loading and unloading ($\nu_{ur}$), angle of shearing resistance ($\phi$), dilatancy angle ($\psi$), $K_o^{nc}$-value, interface parameter ($R_{inter}$), and failure ratio ($R_f$).

The bulk unit weight ($\gamma_t$) was estimated as the average of unit weights calculated using the in-situ water content measurements (assuming specific gravity ($G_s$) of 2.7) and that calculated using CPT correlation presented by Mayne et al. (2009). Empirical correlation referenced by Kulhawy and Mayne (1990) relating soil permeability to void ratio for cohesive soils was used to estimate the soil vertical permeability ($k_y$). The soil void ratio was calculated using the measured in-situ water contents...
(assuming $G_s = 2.7$) and calculated bulk unit weights. Soil layers were assumed isotropic, thus vertical ($k_y$) and horizontal ($k_x$) permeabilities were considered equal. Note that the permeability coefficient was an input parameter in soil clusters inducing pore-water pressures that was later significant in the consolidation phase; therefore permeability coefficients were defined for clay layers only.

The oedometer loading stiffness ($E_{oed}$) was assumed equivalent to the constrained modulus ($M$) interpreted from CPT measurements. The constrained modulus evaluated at the middle of each layer was substituted in the input data with the corresponding reference pressure ($P_{ref}$), which is equivalent to the effective vertical overburden pressure. Constrained modulus was determined from CPT measurements using the correlation suggested by Senneset et al. (1989): $M = \alpha (q_t - \sigma_{vo})$, where $q_t$ is the corrected cone tip resistance, $\sigma_{vo}$ is the total overburden pressure, and $\alpha$ is a coefficient ranging from 4 to 15 in most clays. For this study, the value of ($\alpha$) was back calculated by comparing the modeled and measured deflections. A value of 7 for ($\alpha$) resulted in good match between modeled and measured deflections as shown later in Figure 4. For the top sand layer and bottom sand/silt/clay mixture, $E_{oed}$ values were evaluated using measured SPT values (Kulhawy and Mayne, 1990).

Triaxial loading stiffness ($E_{50}$) was estimated using the factor referenced in PLAXIS material model manual relating $E_{50}$ to $E_{oed}$ such that $E_{50} = 1.25E_{oed}$. Triaxial unloading/reloading stiffness ($E_{ur}$) was estimated using the factor referenced in PLAXIS material model manual relating $E_{ur}$ to $E_{50}$ such that $E_{ur} = 3E_{50}$. Power ($m$) values were taken equal to those assumed by Blackburn (2005). Drained unload/reloading Poisson’s ratio ($\nu_{ur}$) was assumed 0.3 for all layers.

For the top sand layer and bottom sand/silt/clay mixture, no laboratory tests were reported in Blackburn (2005) for angles of shearing resistance. Angles of shearing resistance of 35˚ and 31˚ were assumed for the top sand layer and bottom sand/silt/clay mixture, respectively. For the clay layers, no laboratory tests were reported in Blackburn (2005) for the angles of shearing resistance or clay index properties. Accordingly, the correlation presented by Skempton (1957) was used to estimate the soil plasticity index (PI) using measured field vane undrained shear strength. Interpreted PI values were then used to determine the angle of shearing resistance using correlation presented by Mitchell (1976). Dilatancy angle ($\psi$) was determined in accordance with guidelines provided in PLAXIS material model manual. For clay layers, the dilatancy angle is very small ($\psi = 0$); while for sand layers, dilatancy angle is related to angle of shearing resistance such that: $\psi = \phi - 30^\circ$. 

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Values for $K_o^{nc}$ at normal consolidation was evaluated using Jaky’s (1944) formula such that $K_o^{nc} = 1 - \sin\phi'$. The interface parameter ($R_{inter}$) was taken as the adhesion factor defined by Tomlinson (1957), which equaled the reduction factor applied to undrained shear strength for clay-concrete interface. The undrained shear strength was evaluated using CPT measurements, field vane shear tests, pressure meter tests, and unconfined compressive tests. The failure ratio ($R_f$) was selected as 0.9 for all soil layers as recommended by PLAXIS material model manual. Input hardening soil model parameters are summarized in Table 2.

**Table 2:** Input Hardening soil model parameters.

<table>
<thead>
<tr>
<th>Property</th>
<th>Top Sand</th>
<th>Clay Crust</th>
<th>Upper Clay</th>
<th>Middle Clay</th>
<th>Lower Clay</th>
<th>Sand/silt/clay mixture</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type</td>
<td>Drained</td>
<td>Undrained</td>
<td>Undrained</td>
<td>Undrained</td>
<td>Undrained</td>
<td>Drained</td>
</tr>
<tr>
<td>$\gamma$ (kN/m$^3$)</td>
<td>19</td>
<td>20.4</td>
<td>19</td>
<td>19.6</td>
<td>20.4</td>
<td>20.4</td>
</tr>
<tr>
<td>$k_x$ (m/day)</td>
<td>-</td>
<td>9.1x10$^{-11}$</td>
<td>3.4x10$^{-10}$</td>
<td>2.3x10$^{-10}$</td>
<td>1.1x10$^{-10}$</td>
<td>-</td>
</tr>
<tr>
<td>$k_y$ (m/day)</td>
<td>-</td>
<td>9.1x10$^{-11}$</td>
<td>3.4x10$^{-10}$</td>
<td>2.3x10$^{-10}$</td>
<td>1.1x10$^{-10}$</td>
<td>-</td>
</tr>
<tr>
<td>$E_{50}$ (kPa)</td>
<td>12,500</td>
<td>25,000</td>
<td>8,750</td>
<td>11,250</td>
<td>17,500</td>
<td>50,000</td>
</tr>
<tr>
<td>$E_{soed}$ (kPa)</td>
<td>10,000</td>
<td>20,000</td>
<td>7,000</td>
<td>9,000</td>
<td>14,000</td>
<td>40,000</td>
</tr>
<tr>
<td>$E_{ur}$ (kPa)</td>
<td>37,500</td>
<td>75,000</td>
<td>26,250</td>
<td>33,750</td>
<td>52,500</td>
<td>150,000</td>
</tr>
<tr>
<td>$c$ (kPa)</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>$\phi$ (degree)</td>
<td>35</td>
<td>31</td>
<td>22</td>
<td>24</td>
<td>30</td>
<td>31</td>
</tr>
<tr>
<td>$\Psi$ (degree)</td>
<td>5</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>1</td>
</tr>
<tr>
<td>$v_ur$</td>
<td>0.3</td>
<td>0.3</td>
<td>0.3</td>
<td>0.3</td>
<td>0.3</td>
<td>0.3</td>
</tr>
<tr>
<td>$P_{ref}$ (kPa)</td>
<td>21.32</td>
<td>45</td>
<td>81.30</td>
<td>118.6</td>
<td>147.5</td>
<td>121.0</td>
</tr>
<tr>
<td>$m$-Power</td>
<td>0.5</td>
<td>0.5</td>
<td>0.8</td>
<td>0.85</td>
<td>0.85</td>
<td>0.6</td>
</tr>
<tr>
<td>$K_o^{nc}$</td>
<td>0.4264</td>
<td>0.485</td>
<td>0.6254</td>
<td>0.5933</td>
<td>0.5</td>
<td>0.485</td>
</tr>
<tr>
<td>$R_f$</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
</tr>
<tr>
<td>$R_{inter}$</td>
<td>0.7</td>
<td>0.6</td>
<td>0.9</td>
<td>0.7</td>
<td>0.6</td>
<td>0.6</td>
</tr>
</tbody>
</table>
7 RESULTS

Computed deflections are compared to actual field measurements recorded at Inclinometers 1, 2, and 5 in Figure 4. The match between computed and measured deflections at Inclinometer 5 is not as good as that achieved for Inclinometers 1 and 2. Inclinometer 5 is located on the west side of the excavation adjacent to Sheridan road. Thus, the Inclinometer 5 could have been affected by variable loading conditions that were not taken into account in the numerical model. Measured deflections at Inclinometers 1, 2, and 5 were higher than those computed near ground surface. This may be attributed to construction problems like unmeant dislocating of inclinometer pipe top. With the exception of near ground surface deflections, computed deflections using PLAXIS 3D deviated from field measurements by less than 30%. Thus, the developed 3D model of the deep excavation was considered adequate.

![Figure 4](image-url)  
Figure 4: Computed deflections versus field measurements for Inclinometers 1, 2, and 5.

To investigate 3D effects on developed deflections within the excavation, deflections near the center and corner sections of the north side of the excavation (Inclinometers 1 and 2, respectively) were compared. Deflections near the corner section are less than the center section by about 10%. Note that the 3D effects are suppressed in this case due to the installation of cross-lot bracings near the center section of the wall. Three dimensional finite element models can be applied to capture 3D effects on behaviour of walls near central and corner sections to enhance wall design and to better estimate wall deflections.
8 CONCLUSIONS

1) Proper modeling of excavation sequence is essential to achieve realistic modeling results. Modeling features such as access ramps were essential in this study to get deflections similar to measured values.

2) Anisotropic axial to lateral wall stiffness ratio of 20 for sheetpiles has proven to be suitable for the modeled braced sheetpile walls. Other studies have shown that this ratio vary for other retaining systems.

3) Field measurements are essential to improve the soil model assumed in finite element modeling. A better interpretation of the soil constrained modulus using CPT measurements was achieved by comparing modeled deflections to field measurements.

4) Three dimensional finite element modeling is capable of capturing the difference in wall behaviour near central and corner sections, which is critical in estimating deflections of retained excavations especially in urban areas; and can be used to enhance wall design.

REFERENCES


Effects from a TBM-EPB to Adjacent Buildings: 3D Simulation of a Twin Tunnel

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²Attiko Metro S.A, Greece

Abstract

The aim of this paper is a profound investigation of all relative parameters involved in the TBM-EPB tunnelling. A 3D numerical model simulating a twin tunnel excavated by a TBM-EPB is presented, allowing for all the components that influence the induced ground settlements. The face support, the pressure applied to the steering gap slurry and the tail gap grouting are appropriately simulated. The methodology is validated by comparing the numerical results to in situ measurements recorded during the excavation of the ongoing Thessaloniki subway with a TBM-EPB. Full soil-structure interaction using 3D numerical analysis was carried out, including a 7 storey building founded close to the tunnels. Useful conclusions regarding the effects of relevant parameters have been drawn for an accurate design of shallow tunnels by the TBM-EPB method.

Keywords: TBM-EPB tunnelling, 3D simulation, soil-structure interaction
1 INTRODUCTION
Tunnel boring machines (TBM) combined with the earth pressure balanced (EPB) shield method have been extensively used for the construction of tunnel in urban areas. The ability of the EPB method in controlling the induced settlements by regulating the pressures applied to the tunnel face, beside the shield and behind the final lining renders the method the most effective, particularly in urban areas with loose soils and/or adjacent buildings interacting with tunnels. Recent researches investigating the affecting parameters have been presented ([1], [3-5]). The main objective of the present paper is to extend the existing methods to a full 3D interaction and assess the potential response of ground and adjacent buildings. All relevant components i.e. TBM shield, support of the excavation face, over-excavation, pressure behind the shield, tail gap grouting and progressive hardening of the cement based tail grout were taken into account. The proposed method is being applied to the on going Thessaloniki subway. The good agreement acquired from the comparison of monitored data and numerical results has led to useful conclusions.

2 THESSALONIKI SUBWAY
Thessaloniki subway is consisted of twin tunnels (northbound and southbound lines). The two lines pass through high population density areas at a relatively shallow depth, varying from 8 to20 m and in close distance with adjacent buildings. The excavation passes through a sandy clay formation where two compressed air TBM-EPB, appropriate for conditions under the water level and soils with low permeability, are employed to reduce subsidence. The two TBM-EPB have a conical shield providing a shield over-excavation of 20 mm (steering gap). The stability of the tunnel face is ensured through the excavated soil itself, which is maintained in a compressed state within the excavation chamber. The optimum value of the air pressure is appropriately defined to balance the existing effective pressure at rest and the water pressure at the tunnel face, preventing the seepage flow into the chamber. To prevent ground deformation in the steering gap, heavy bentonite slurry is injected until the prefixed pressure is achieved (PSG system). The tunnel lining, consisting of grade C40/50 concrete prefabricated lining rings, is set in place near the end of trailing fingers. The ring thickness is 300 mm and the segment width is 1500 mm. The gap between the extrados of the lining and the excavation wall is 145 mm and is filled by a two-component grout with resistance accelerator. Figure 1 illustrates in detail the TBM-EPB being used for the construction of Thessaloniki subway.
The ground conditions at the site comprise for about 4.5 m of fill material overlying a formation of sandy clay consisting of two main layers, assigned as CL-1 and CL-2. The groundwater level was encountered at 5.0 m below the ground level. The alignment of the tunnels, the locations of the instruments and the monitored building D91 are shown in Figure 2.

3 NUMERICAL ANALYSIS

3.1 Simulation Procedure

The mesh and the numerical analysis were carried out using the software FLAC\textsuperscript{3D} [2]. A refine mesh including a six floor building with a basement corresponding to the building D91 was included in the mesh. Figure 3 illustrates the mesh consisting of 110640 brick elements for the soil, 135 beam elements and 7134 shell elements for the building and 8256 shell elements for the tunnel lining. No valuable excess of pore pressure was observed during the construction and therefore an effective stress analysis was applied. A constitutive law with double yielding providing the ability of non constant deformation moduli was applied [2]). The parameters required for the application of the double yield model were derived from oedometer tests and are given in Table 1 together with main soil properties of each soil layer.

According to instrumentation data a constant pressure of 180 kPa was applied at the excavation face. Tunnel lining behaviour was considered as linear elastic. Surface forces varying with depth are applied to the lining elements as a result of the ground water table. The gap between the extrados of the lining and the excavation wall was filled by mortar grout pressurized at 180 kPa. Low stiffness and shear resistance is attributed to the grouted region next to the shield, while increasing stiffness is assigned to the grouted elements further from the shield based on the results of unconfined tests. A reasonable value for the slurry shear modulus of three hundred times the shear strength, $G_{sl} = 25$ kPa, was applied while the Poisson’s ratio was taken equal to 0.49.

Figure 1: Schematic illustration of the TBM-EPB section
Figure 2: Horizontal plan at the area of analysis.

Table 1. Geotechnical properties of soil layers

<table>
<thead>
<tr>
<th>Layer</th>
<th>Fill</th>
<th>CL-1</th>
<th>CL-2</th>
<th>GM</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth (m)</td>
<td>0 – 4.5</td>
<td>4.5 – 9.5</td>
<td>9.5 – 30</td>
<td>30 – 35</td>
</tr>
<tr>
<td>Effective cohesion c’ (kPa) / angle of friction, φ’ (deg)</td>
<td>3 / 30</td>
<td>3 / 25</td>
<td>5 / 25</td>
<td>5 / 35</td>
</tr>
<tr>
<td>Poisson’s ration, ν</td>
<td>0.3</td>
<td>0.3</td>
<td>0.3</td>
<td>0.3</td>
</tr>
<tr>
<td>Plastic bulk modulus, $K_t$ (kPa)</td>
<td>4,000</td>
<td>5,000</td>
<td>7,500</td>
<td>20,000</td>
</tr>
<tr>
<td>Ratio of elastic to plastic bulk modulus, R</td>
<td>5</td>
<td>5</td>
<td>5</td>
<td>3</td>
</tr>
<tr>
<td>Cap pressure, $p_c$ (kPa)</td>
<td>100</td>
<td>100</td>
<td>NC*</td>
<td>NC*</td>
</tr>
</tbody>
</table>

Remark: NC means that the cap pressure is equal to the in-situ mean stress

The PSG system was not activated and a limited pressure varying from 20 (northbound) to 40 kPa (southbound) was build up in the over-excavation area as a result of the gap communication with the front face and the tailing part of the shield. The simulation process of the tunnel construction includes a number of stages equal to the number of segment rings (65 per tunnel).

3.2 Numerical Results – Comparison to In Situ Measurements

The cross sectional evolution of surface settlement as the TBM gets across a point of interest is demonstrated in Figure 4. From Figure 4a arises that, with the TBM-EPB method, surface settlements are encountered at a distance of less than $2.0D$ from the
excavation face and that they are fully developed when the face is in a distance of 4.0D ahead. Figure 4b illustrates the development of surface settlement prediction at the monitored cross section with regards to the northbound advancement. When the northbound is 1.21D far from the reference section the settlement profile is given by the dotted line and when the excavation face reaches the reference section the settlement profile is given by the dash line. Further advancement of the tunnel face leads to a progressive increment of the settlements. The final settlement profile occurs when the excavation face is at the distance of 3.63D in front of the section.

Figure 3: Finite difference mesh including building D91.

Figure 4: Development of settlement prediction of southbound and northbound advancement.
The black circular markers, standing for the final recorded settlements across the section, are in excellent agreement with the predicted values. Both the monitored settlement profile and the predicted one are not symmetrical as a result of lower pressures applied during the construction of the northbound tunnel.

4 CONCLUSIONS

A 3D numerical model has been developed for the simulation of a twin tunnel excavation process by a TBM-EPB. A 3D numerical analysis providing a full soil-structure interaction was carried out to evaluate the ability of numerical methods in providing accurate prognosis for the settlements of buildings adjacent to tunnels excavated with the TBM-EPB method. The application of the proposed method to the on going Thessaloniki subway demonstrated that apart from the face supporting pressure, the steering and the tail gap pressures significantly affect the ground movement. It has been also deduced that an accurate application limits the development of subsidence within $2.0D$ behind and $4.0D$ ahead the excavation face.

REFERENCES


Influence of Tunneling on Nearby Existing Building and/or Tunnel: Model Test and Finite Element Analysis

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²Nippon Telegram and Telephone West Corporation, Japan

Abstract

For the model tests of tunneling, a new device has been developed not only for applying inner displacement of tunnel but also for allowing the movements of the tunnel itself with satisfying the equilibrium between tunnel and the surrounding ground. Using this device, model tests of tunnel excavation considering an existing building and existing tunnel are carried out. Non-linear finite element analyses corresponding to the model tests are also conducted using FEMtij-2D software where elastoplastic subloading $t_{ij}$ model is used as a constitutive model of soil. It is found from the model test that there is a significant effect of tunneling on the existing foundation of building even if the tunnel is constructed in deep underground. It is also revealed that in the ground with existing (preceding) tunnel, the earth pressure distribution around the preceding and following tunnels, and ground movements during tunnel excavation depend on the distance and position between the twin tunnels. The numerical analyses perfectly capture the results of the model test in both conditions.

Keywords: Model test, Non-linear finite element analysis, Existing building, Twin tunnels, Surface settlement, Earth pressure, Deviatoric strain distribution
1 INTRODUCTION

Nowadays, urban tunneling in shallow and deep undergrounds is increasing all over the world. However, tunnel excavations inevitably cause ground deformations and may affect existing structures or tunnels. So far, it is clarified through numerical and experimental studies that loosening of ground by tunnel excavation leads to the serious damage of nearby existing structures in shallow ground [4, 5, 6]. Meanwhile, in Japan, a law for public use of deep underground in big city has issued. This law is made based on an idea that tunnel construction does not affect much on the existing structures, if the tunnel is constructed in enough deep underground (more than 40m in depth) and the vertical distance between the foundation of existing structure and the tunnel crown is more than 10m. Also, it is usual that new tunnel is constructed near the existing tunnel in urban area. To investigate the effect of tunneling on existing structure in deep underground and the ground behavior under construction of twin tunnels in which the relative position of the preceding and following tunnel are changed, model tests and numerical simulations are performed. For the present model tests of tunneling, the previous device of circular tunnel excavation [6] is improved. This new device allows the movements of the tunnel itself with satisfying the equilibrium between tunnel and the surrounding ground as well as applies any kind of tunnel shrinkage. The corresponding numerical simulations are performed using FEMtij-2D software, in which the subloading $t_{ij}$ model [3] is used as an elastoplastic constitutive model for the ground material.

2 OUTLINE OF MODEL TESTS

In our previous research [6], a tunnel apparatus to simulate tunnel excavation where the cross section of the tunnel is circular was developed. In that device, absolute displacement is applied in the boundary of the tunnel. But, a real field tunnel is free to move in the vertical and horizontal directions. To consider this important point, in this research further modification of the model tunnel apparatus has been done for allowing the movements of the tunnel together with the soils of the ground. The device allows the movements of the tunnel itself with satisfying the equilibrium between tunnel and the surrounding ground. In this research, model tests of deep tunnel excavation near existing building foundation and twin tunnel excavation are conducted changing the position of the following tunnel and with the new apparatus.
Figure 1 shows the improved tunnel device, in which the excavation part can be moved upward and downward, and left and right without friction by a cylindrical roller bearing and a horizontal slider attached in the device. The weight of the entire model tunnel is balanced with the counter weight applied through the fixed pulley set at the top of the device. As a result, the tunnel excavation can be simulated by leaving it to an equilibrium condition of the vertical and lateral earth pressures controlling the amount of shrinkage of the tunnel diameter. The total diameter of the tunnel is \( B = 10 \text{cm} \) and the device consists of a shim at the center of the tunnel surrounded with 12 segments in the same way as the previous apparatus.

Figure 1: Schematic diagram of tunnel device

Figure 2 shows the layout of the 2D apparatus. Diagram (a) is for tunneling near existing structure with pile foundation, and diagram (b) is for twin tunneling having two model tunnel devices; both tunnel devices have the same dimension as described in Figure 1. In each device, 12 load cells are used to measure earth pressure acting on the tunnel. The load cells are attached with blocks which are placed surrounding the segments of the tunnel. Therefore, earth pressure can be obtained at 12 points on the periphery of the tunnel at a time.

In the model test of the deep ground with existing structure, pile foundation were used as shown in Figure 2(a). The piles were simulated using polyurethane walls, because the model is two-dimensional under plane strain conditions. Young’s modulus of the pile material \( (E = 1.276 \times 10^5 \text{kN/m}^2) \) were chosen to agree with the similarity ratio of 1:100 to its prototype, for example the diameter of tunnel \( B = 10 \text{cm} \) in model tunnel is intended to represent a real tunnel of 10m diameter. The thickness of the pile was 0.5 cm, and the lengths of the pile \( (L_p) \) was 30cm, and the distance between the front and rear piles was 5cm. The depth of the tunnel \( D = 40 \text{cm} \), which corresponds to 40m in
real ground. To impose the existing load, a constant value of dead load of \( q_v = 0.128 \times 9.8 \text{N/cm} \), which is around 1/3 of the ultimate bearing capacity of the pile foundation, is placed on the center of the foundation before performing the tunnel excavation.

**Figure 2:** Circular tunnel device

Mass of aluminum rods, having diameters of 1.6 and 3.0 mm mixed in a ratio of 3:2 in weight, is used as ground material. The unit weight of the aluminum rods mass is 20.4kN/m³, and the length is 50 mm. The mass of aluminum rods are stacked up to a prescribed height after setting single tunnel device or twin tunnel devices according to the purpose of the experiments. The initial ground is made in such a way that the earth pressure becomes similar to the earth pressure at rest adjusting the block of aluminum set at the bottom of the apparatus.

The tunnel excavation is simulated by controlling the shrinkage of the tunnel device and earth pressures around the tunnel periphery is obtained from the load cells of the devices explained above. The resulting surface settlement of the ground is measured using a laser type displacement transducer with an accuracy of 0.01 mm and its position in the horizontal direction is monitored with a supersonic wave transducer. Photographs are taken during the experiments and they are used later as input data for the determination of ground movements with a program based on the technique of Particle Image Velocimetry (PIV).
In the model test of twin tunneling, the preceding (1st) tunnel is excavated first and then the following (2nd) tunnel is excavated. The tests patterns are illustrated in Figure 3. Here, $d_{r1}$ and $d_{r2}$ represent the amount of radial shrinkage of the preceding tunnel and following tunnel, respectively, where at the completion of the tests $d_{r1} = d_{r2} = 40\text{mm}$. The tests have been conducted for a fixed overburden ratio, $D/B=2.0$, where $D$ is the depth from the ground surface to the top of the preceding tunnel and $B$ (=10cm) is the diameter of the tunnel. The vertical distance between the twin tunnels is represented with $S_{T1}$, and the horizontal distance between the twin tunnels is represented with $S_{T2}$.

![Figure 3: Different positions of twin tunnels](image)

3 OUTLINE OF NUMERICAL SIMULATION

The numerical analyses are performed with the same scale of the model tests. As an example, Figure 4 shows a typical mesh used in the finite element analyses for twin tunnels. Both vertical sides of the mesh are free in the vertical direction, and the bottom face is kept fixed. To simulate the tunnel excavation, negative volumetric strain in the tunnel elements is applied which corresponds the amount of radial shrinkage of the tunnel. This is an important simulation technique to consider free movements of the tunnel. Two-dimensional finite element analyses are carried out with FEMtij-2D using the subloading $t_{ij}$ model [3].

Model parameters for the aluminum rod mass are shown in Table 1, and the results of biaxial test for the aluminum rod mass (dots) and the calculated results (curves) are shown in Figure 5. The explanation of the material parameters and the features of the constitutive model are described in the reference [4]. The initial stresses of the ground are calculated by simulating the self-weight consolidation applying body forces.
starting from a negligible confining pressure. In the case of the building loads, the ground is initially formed under geostatic condition, and then concentrated load is applied at the middle node of the foundation.

**Figure 4:** Typical finite element mesh for twin tunnel analyses

**Table 1:** Values of material parameters for aluminum rod mass

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>$\lambda$</td>
<td>0.008</td>
</tr>
<tr>
<td>$\kappa$</td>
<td>0.004</td>
</tr>
<tr>
<td>$N(e_N\text{ at } p = 98kPa}$</td>
<td>0.3</td>
</tr>
<tr>
<td>$R_{CS}=(\sigma_1/\sigma_3)_CS\text{ (comp.)}$</td>
<td>1.8</td>
</tr>
<tr>
<td>$\nu$</td>
<td>0.2</td>
</tr>
<tr>
<td>$\beta$</td>
<td>1.2</td>
</tr>
<tr>
<td>$a/(\lambda-\kappa)$</td>
<td>1300</td>
</tr>
</tbody>
</table>

**Figure 5:** Test results of biaxial test on aluminum rod mass and calculated result
4 DEEP TUNNEL EXCAVATION CONSIDERING BUILDING LOAD

Figure 6 shows the observed and computed surface settlements profiles for the amount of shrinkage of 1mm and 4mm in case of $D/B=4.0$ (tunnel depth $D=40$cm, tunnel diameter $B=10$cm) where pile foundation (pile length $L_p=10$cm) is used to consider building loads. Then, the vertical distance between the pile tip and the tunnel crown is $D_p=10$cm. The figure also represent the result of the greenfield condition for the shrinkage of $d_r=4$mm, which is shown with solid line. The position of the applied dead load and the position of tunnel are depicted at the top and the bottom in the figure. It is seen that the maximum surface settlement occurs at the position of the building load as observed in shallow tunneling in the previous researches [4, 5, 6]. It is revealed that even in the deep underground, tunneling has a significant effect on the existing structure. The numerical simulations can explain well the results of the model test. From these results, it can be said that the surface settlement in real field tunneling may not be maximum just above the tunnel axis when super structures exists nearby the tunnel. In these circumstances, proper prediction of surface settlement is required to prevent excessive damage of nearby exiting super structures. It is also noticed that surface settlement troughs for tunnel excavation in the ground disturbed by existing buildings do not follow the usual pattern of a Gaussian distribution curve even in the deep tunneling, as generally observed for the greenfield condition.

![Surface settlement trough](image)

**Figure 6:** Surface settlement trough

Figure 7 represents the observed and computed deviatoric strain distribution in the ground. As shown in the figures, the shear band in the ground develops during the tunnel excavation and it spreads towards the foundation from the sides of the tunnel. Large deviatoric strain due to tunnel excavation concentrates to the rear pile for the
case of $D/B=1.0$. Since the initial stress in the ground is changed from $K_0$ condition due to the building load, the development of the shear band is different in the left and right side of the tunnel. The effect of the tunneling to the existing structure mainly depends on the distance between the tunnel and the pile tip of the foundation. The computed distribution of deviatoric strain of the numerical analysis shows very good agreement with the results of the model test.

Figure 7: Deviatoric strain distribution

Figure 8 shows the earth pressure distributions around the tunnel. The plots are drawn using twelve axes corresponding to the radial direction of the twelve load cells towards the center of the model tunnel. Here, the dotted curves represent the earth pressure levels before applying the building loads, while the black solid line represents the pressure levels after applying the building loads, and the red solid line represents the earth pressure of the greenfield condition for $d_i=4.0\text{mm}$. The earth pressure at the foundation side increases after applying the building loads. The earth pressure decreases to some extent around the tunnel after performing the tunnel excavation. An unsymmetrical earth pressure distribution is seen around the tunnel. Therefore, the effect of the soil-structure interaction should be properly contemplated in the earth pressure computation around the tunnel lining even in the tunneling of deep underground. The results of the simulations slightly differ from the measured earth pressure levels around the tunnel invert. As a whole, the analysis using the subloading $t_{ij}$ model simulates well the earth pressure distribution of the model test.
5 TWIN TUNNEL EXCAVATION

Figure 9 illustrates the surface settlement profiles for both preceding (1st) tunnel and following (2nd) tunnel excavation in the cases where the 2nd tunnel is situated directly underneath ($S_{T1}=0.25B$), diagonally downward ($S_{T1}=S_{T2}=0.25B$) and same elevation ($S_{T2}=0.50B$) of the 1st tunnel. The soil cover of the preceding tunnel (1st) is $D/B=2.0$ in every case. The vertical axis represents surface settlement, while the abscissa shows the horizontal distance from the intermediate position of two tunnels. A wider settlement trough and larger settlement are observed due to the excavation of the 2nd tunnel directly underneath the 1st tunnel. When the following tunnel sits in the same elevation of the preceding tunnel, final settlement trough is symmetric with respect to the vertical centerline of two tunnels. The numerical analyses capture well the settlement troughs of the model tests.

Figure 10 shows the distributions of observed and computed deviatoric strain in the three cases described above. The distribution of deviatoric strain of the model tests are obtained from the simulation of Particle Image Velocimetry (PIV) technique. The concentration of the strain is represented with the color contrast indicated in the legend. It is seen that the shear band of the ground is developed from the tunnel invert and covered the entire tunnel during tunnel excavation. It is also seen that deviatoric strain occurs towards the preceding tunnel due to the excavation of the following tunnel. The deviatoric strain of the numerical analyses shows very good agreement with the results of the model tests.
Figure 9: Surface settlement profile

Figure 10: Deviatoric strain distribution in ground due to preceding and following tunnels
Figure 11 shows the observed and computed earth pressure distributions around the preceding tunnel. The plots are drawn in the 12 axes corresponding to the radial direction of the 12 load cells towards the center of the model tunnel. The figures represent the value of earth pressure corresponding to the applied amount of shrinkage. It is seen that in $S_{T1}=0.25B$ (directly underneath) for the excavation of the preceding ($1^{st}$) tunnel, earth pressure decreases around this tunnel due to the arching effect, in the same way as the results of the references [1, 2, 5, 6]. As shear band develops surrounding the tunnel (Figure 10) the surrounding ground undergoes to the loose state which reduces stresses in that place. However, the earth pressure at both lateral sides of the tunnel increases and it decreases at the tunnel invert during excavation of the following tunnel. On the other hand, when the following tunnel is constructed at $S_{T1}=S_{T2}=0.25B$ (diagonally downward), earth pressure of the preceding tunnel increases at the right shoulder and the left part of the tunnel invert which is seen in the same figure. When the second tunnel constructed in the same elevation at a distance of $S_{T2}=0.50B$, earth pressure increases at the crown and the invert of the preceding tunnel. Hence, it can be said that the interaction of parallel tunnels should be predicted carefully for proper and safe design in case of the closely spaced tunnels. The numerical analyses perfectly capture the distributions of earth pressure for the excavation of the following tunnel in two different locations.

**Figure 11:** Earth pressure distribution at the preceding tunnel
6 CONCLUSIONS

Through the model tests and numerical simulations, the influence of tunneling on the existing building and/or tunnel is discussed. It is found that there is a significant effect of tunneling on the existing foundation of building even if the tunnel is constructed in deep underground, in the same way as in shallow tunneling. The shear band in the ground develops not symmetrically but toward the pile tip of foundation, so that the maximum settlement does not occur above the tunnel but at the position of the building. In the twin tunneling problem, it is found that for the excavation of the preceding tunnel, earth pressure decreases around this tunnel as expected, however, the earth pressure in the preceding tunnel increases at the directions perpendicular to the following tunnel due to excavation of the following tunnel. A region of large deviatoric strain concentration is seen between the twin tunnels due to excavation of the following tunnel. The numerical analyses perfectly capture the surface settlement, ground deformation and distributions of earth pressure of the model tests.

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Pioneering Real Time Computational Models for Building Damage Prediction during Adjacent Tunnel Excavation

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Abstract

Numerical modelling is commonly employed prior to tunnel excavation to estimate surface settlements and to predict the response of adjacent structures. Unfortunately, geotechnical and building parameters are difficult to determine for the large geographical extent of a tunnelling project. As such, parametric values for modelling purposes are frequently assumed and are rarely revised to provide updated predictions as field data becomes available. Given advances in 'real time' data availability from subsurface- and surface-based monitoring systems, the question arises of how to better fully exploit this data for improved adjacent building protection. To achieve this, integration of numerical models into the monitoring process to provide updated 'real time' building response predictions is explored. This paper extends existing frameworks which utilize geotechnical field data to provide ‘real time’ predictions to also include building considerations.

Keywords: Surface settlements; building damage; monitoring; real-time data
1 INTRODUCTION

Computational modelling, particularly finite element modelling, is frequently employed to assess 'at risk' buildings along a tunnel route so as to determine the required building protection measures to prevent settlement induced damage [14]. At present, these analyses are predominantly limited to pre-construction stages where much uncertainty exists as to the exact selection of geotechnical and building parameters. As such, the reliability of these models is currently limited, despite the investment of significant time and resources during project design. Nonetheless, their results dictate the selected mitigation measures to prevent the occurrence of building damage and, additionally, assist in the development of monitoring schemes to determine and control both ground and building movements during tunnel construction, through the application of additional mitigation measures.

Figure 1: Proposed Integrated System

Despite the vast quantities of monitoring readings commonly produced during tunnel excavation, these actual field measurements are rarely exploited to update the design-stage models. As such, numerical modelling has not, to date, been utilized efficiently
for building damage prediction. To overcome these limitations, this paper explores the current role of computational modelling within the framework of tunnelling projects and investigates the possibility of an integrated system (Figure 1), which exploits the monitoring data produced during tunnel construction for 'real time' building damage predictions. To do so, additional steps are required in a timely manner so as to provide updated building response predictions during tunnel excavation. These steps include parameter calibration and subsequent re-analysis of the numerical models.

2 CURRENT PRACTICE

To date, computational modelling for building protection purposes has been predominantly limited to pre-construction predictions of building response to tunnel excavation. Recent advances, specifically in finite element software capabilities, have enabled realistic modelling of soil behaviour [1], the simulation of complex tunnel construction procedures [3], and precise modelling of specific structures [8]. The accuracy of such predictions relies upon the appropriate selection of model input parameters. In general, ground parameters are determined from geotechnical investigation information and from past tunnelling activities in similar ground conditions. However, these parameters can be difficult to determine due to the heterogeneous nature of subsurface conditions and variations in workmanship and management during tunnel construction [12].

More critically, building parameters are frequently shrouded in uncertainty due to an absence of structural drawings and site reconnaissance generally being restricted to non-destructive testing. This is particularly the case for older buildings where structural layouts, foundation details, material strengths, and previous structural movements are often difficult to determine. Consequently, assumptions must be made but are rarely later confirmed in a systematic way by application of actual response data. This failure to verify may be attributed to the current inability to update computational models in 'real time' but is also indicative of the fact that information about the response of an individual building can rarely be extrapolated to predict the behaviour of other structures along the tunnel route. This is due to the variability of a city's building stock. For example, along the first section of the proposed Metro North route in Dublin, Ireland, wall thickness information was only available for 10.2% of buildings potentially within the tunnel's zone of influence. Within this sampling, wall thicknesses varied from less than 200mm to values in excess of 400mm [5].
2.1 Monitoring Scheme
Based on computational results, a monitoring scheme is typically implemented within the tunnel’s predicted zone of influence where numerical predictions dictate the types of required monitoring data, as well as the location of measuring points. A scheme will usually consist of instrumentation located in three zones (Figure 2): (1) subsurface monitoring – consisting of in-tunnel geodetic prisms to assess tunnel convergence, as well as borehole instruments, such as rod extensometers and inclinometers, to monitor displacements below ground level; (2) ground surface monitoring – in the form of precise levelling studs to quantify movements at ground level, and (3) building monitoring – comprised of BRE (British Research Establishment) levelling sockets, as well as geodetic prisms that are read by nearby Automated Total Station (ATS) to track movements in three-dimensional space. The quantity of monitoring instrumentation is largely dictated by a project’s budget, whilst frequency of readings usually depends on (1) instrument accessibility, (2) automated reading viability, and (3) monitoring goals [10]. The results generally include vast quantities of data, particularly for urban projects beneath historically sensitive structures.

Monitoring information is normally reviewed based upon pre-defined trigger levels, which dictate particular actions. Commonly, a system of three or four trigger levels is adopted: green, amber, red, and black [16]. Green and amber triggers generally require data review to determine necessary actions, whilst red and black triggers denote more drastic actions, such as a construction stoppage or tunnel evacuation. Where necessary, intervention measures such as compensation grouting are subsequently applied. The amount, timing, and location of grout injections are determined from monitoring data but are generally limited to the discreet review of individual datasets. However, arguably determination of the actual cause of reaching a trigger level and the likely effect of intervention activities requires an integrated post-processing approach. Such an approach is not currently undertaken which may be largely attributed to the scale of the problem.

2.2 Typical Project
In order to demonstrate the magnitude of a typical urban tunnelling monitoring scheme, an example of a section of an ongoing project in London City Centre is provided herein. The specific sections of this project considered consist of two tunnels in close proximity, over a distance of approximately 0.8km. Within the zone of
influence of these tunnels, there are over 200 buildings for which the monitoring instrumentation outlined in Figure 2 was adopted. The majority of monitoring instruments positioned on building surfaces are read in 'real time'. This makes those readings available immediately after collection [2]. Based upon readings taken every 10 minutes, the monitoring scheme outputs almost 40,000 readings daily. Typically, readings begin 3 months prior to tunnelling activities (to provide baseline readings) and continue during tunnel excavation and then at a reduced frequency following construction, until settlement values are increasing at a rate of less than 2mm per annum. With project completion expected in 2015, over 58 million individual readings are anticipated for only this section of the tunnel.

<table>
<thead>
<tr>
<th>Instrument</th>
<th>No. of Instruments</th>
<th>Frequency of Readings</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Geodetic Prism</td>
<td>2715</td>
<td>Automated N/A</td>
</tr>
<tr>
<td>2 ATS</td>
<td>100</td>
<td>Manually twice/day</td>
</tr>
<tr>
<td>3 BRE Levelling Stud</td>
<td>1120</td>
<td>Varies</td>
</tr>
<tr>
<td>4 Precise Levelling Stud</td>
<td>845</td>
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<tr>
<td>5 Piezometer</td>
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<td>6 Extensometer</td>
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<tr>
<td>7 Inclinometer</td>
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</tr>
<tr>
<td>8 Geodetic Prism</td>
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</table>

*Figure 2: Monitoring Scheme during Tunnel Excavation*

### 2.3 Impediments

Not only do monitoring schemes consist of an unwieldy dataset that requires substantial processing resources, but the processing power for an average computer does not currently allow for re-analysis of the original models in a timely manner.
Arguably for tunnel projects, the full financial benefits are not being obtained from either existing efforts in numerical modelling or from field monitoring despite significant time and resources invested in these analyses at design stage and furthermore, monitoring schemes frequently costing millions of euro. For buildings in the vicinity of supported excavations, an 'adaptive management approach' has been proposed [6] where monitoring data collection during construction is processed and numerical models are subsequently analysed in real time. However, the proposal of a similar approach for the case of tunnelling, where hundreds of buildings may be present along a route remains a challenge.

3 INTEGRATED SYSTEM

To exploit the vast quantities of monitoring data produced during tunnel construction as a means for updating computational models, a procedure is needed, which calibrates input parameters and subsequently re-analyses the models. Numerical models generally require fundamental ground parameters (e.g. soil friction angle, Poisson's ratio, etc.) and building parameters (e.g. material strengths, soil-structure interface properties, etc.), depending upon the adopted constitutive model. For geotechnical parameters, algorithms may be utilized to perform back analyses using field data to establish in-situ values [13]. These procedures commonly consist of three steps: (1) the error function to assess the difference between the monitored and computed values, (2) the numerical model to simulate the construction process, and (3) the optimization algorithm to reanalyze the numerical model using an iterative process in order to minimise the error function and obtain the in situ parameters. However, these strategies are generally highly complex due to the convergence requirements of the numerical model and, consequently, require significant computing resources. As such, the use of these algorithms in real time is not currently feasible since the timeframe for obtaining updated geotechnical parameters is presently estimated at 8 hours following monitoring data acquisition [6]. For building parameters, the use of similar algorithms for back analysis purposes is possible, but usage in real time is presently not feasible for similar reasons. However, with the likelihood that computer processing power will continue to grow [11], incorporation of these steps as part of an integrated system may be possible in the near future.

Since full-parameter, real-time analysis is not viable, current tunnelling projects utilize simplified methods for updating risk estimations based upon geotechnical field data. For example, the Porto Metro employed the 'Matrix Approach', which used data
relating to the tunnel face's geological conditions to update a risk matrix, initially based upon numerical analyses, which estimated the likelihood of ground surface settlement based upon geological and overburden conditions [9]. To estimate potential damage for individual buildings, settlement predictions were subsequently combined with results of a building vulnerability analysis to produce an overall 'Building Risk Assessment'. However, the numerical model employed for this method based settlement predictions on a 'greenfield' scenario, where simulation of the soil-structure interaction effects was not included. Notably, the consideration of soil-structure interaction effects due to the building's presence is crucial for the accurate estimation of building damage since the problem is an interactive one. Tunnel-induced surface settlements affect adjacent structures, but the building’s weight, geometry and foundation type influences the development of surface settlements [15,7]. Failure to recognise this interaction can lead to the implementation of unnecessary building protection measures and thus, the occurrence of unwarranted costs.

For the Amsterdam North/South Metroline, an interactive system was utilized which exploited geotechnical field data. Based upon results of an earlier numerical sensitivity study for buildings founded on piles, the surface settlement monitoring information relating to volume loss (Vₜ) values was fed back to inform tunnel boring machine (TBM) operations [18]. Whilst this method included soil-structure interaction effects, it was limited to buildings founded on piles. However, many historic building stocks are founded on shallow foundations (e.g. New York, Dublin). Furthermore, the interactive method did not enable earlier building parameter assumptions to be updated based upon field data relating to building movements. As such, methods to date have been limited to geotechnical considerations. Consequently, the extension of current methods to include building parameter considerations is sought.

3.1 Real Time Computational Models

Whilst different buildings may display similar global behaviour to tunnel induced settlements (i.e. tilt/angular distortion), the development of local building damage (i.e. crack formation) may vary significantly depending on individual structures. In fact, relatively small geometrical discrepancies have been revealed to alter local damage predictions in unreinforced masonry buildings, especially for those constructed of weak building materials [17]. As such, the development of an approach based upon
the method employed for the Amsterdam North/South Metroline [18], but extended to account for uncertainty with regard to building considerations, is proposed. To do so, this method proposes the utilization of field data relating to values of building stiffness (E) to provide updated damage predictions, since this parameter has been revealed to play a key role in the response of structures to tunnel induced settlements [15,7]. Building stiffness is composed of axial and bending stiffness components and varies according to building material, structural type, building geometry, as well as pre-existing cracking.

Since building stiffness values may not be determined from field monitoring data solely, as is the case for V₁, the following procedure is proposed to provide updated damage predictions for individual buildings (Figure 3): (1) prior to tunnel construction a sensitivity study of values of V₁ will be conducted where E is assumed; (2) subsurface monitoring during tunnel excavation will subsequently provide field values of V₁, which may be extrapolated with relative reliability over short distances; and (3) subsequently, the applicable model in terms of V₁ will be calibrated against field data relating to building movements to provide an updated damage prediction based on actual building stiffness (i.e. building parameter values are varied until the model simulates the building’s field response). The solution will be approximate since the model is purely calibrated against E and not for fundamental building parameters. However, the method offers a reasonable solution which may be obtained using relatively little computational processing power. However, to implement mitigation measures in a timely manner and address the heaving which occurs in some soils ahead of the TBM [12], model calibration must occur ahead of the tunnel face. This calibration should be conducted as soon as any non-temperature related building movement is detected and results may be subsequently linked to grouting activities.
4 CONCLUSIONS

The need for an integrated approach to building damage prediction is vital for the protection of adjacent buildings during tunnel excavation. Whilst future computer processing power may facilitate entire computational re-analyses, which enable the back analysis of fundamental geotechnical and building parameters, these are not currently viable for real time damage predictions. As such, an extension of an existing method that focuses upon geotechnical considerations to include building considerations is proposed. This involves a computational sensitivity study of values of $V_l$ prior to tunnel excavation and the use of these results in conjunction with field monitoring results relating to building movements to calibrate original models for building stiffness values, thus providing more comprehensive, although not exact, real time building damage predictions. This approach offers an interim solution for more economical usage of computational modelling within tunnel projects and better exploitation of monitoring data.

Figure 3: Proposed Real Time Computational Model
REFERENCES


Prediction of Structural Movements due to Large Diameter Twin TBM Tunnels using Bayesian Updating

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Abstract

The Port of Miami Tunnel project consists of twin highway tunnels linking the port to Miami’s major highways to alleviate port related congestion downtown. Constructed by a hybrid Tunnel Boring Machine (TBM), the excavation was required to pass beneath existing surface structures on Dodge Island. A finite element analysis was performed to evaluate the potential risk of damage to the structures. The range of possible surface settlements calculated from the finite element analysis, using the upper and lower bound geotechnical parameters developed in the project ground investigation, exhibited a high degree of uncertainty. A Bayesian updating approach was developed for systematically evaluating surface settlement data collected during the tunneling operation to predict ultimate ground settlements in advance of the TBM. Once the first measurable surface settlement data was collected by instrumentation installed between the TBM and the structure, the initial predictions were refined using the Bayesian updating approach, which resulted in observed surface settlements within 5mm of observation. Application of Bayesian updating using monitoring data allows for early warning of possibly damaging settlements to provide time for corrections to tunneling methods or contingency measure installation; for the Port of Miami project, it provided validation of anticipated building responses.

Keywords: Bayesian Statistics, TBM Tunneling, 3D Modeling, Surface Settlements, Port of Miami Tunnel
1 INTRODUCTION

Currently, the only access to the Port of Miami for shipping traffic involves navigating busy downtown city streets, causing traffic congestion and limiting economic development of the northern portion of Miami’s Central Business District. To alleviate these issues, the Port of Miami Tunnel is under construction, which will provide a direct underground connection from Port Miami at Dodge Island via Watson Island to I-395 and all other highways. To complete the project, a Public-Private Partnership (PPP) was established between the Florida Department of Transportation, Miami-Dade County, the City of Miami, Meridiam Infrastructure Finance, and Bouygues Travaux Publics as part of the design-build-finance-operate-and-maintain (DBFOM) contract. Bouygues Civil Works Florida (BCWF) acted as the prime contractor for the project.

The project consists of twin 12.5m diameter bored tunnels between Watson Island and Dodge Island constructed from a hybrid Tunnel Boring Machine (TBM) which is required to mine beneath a number of existing structures on Dodge Island. The potentially affected structures include an open-air concrete framed structure, a steel framed passenger loading structure, and steel sheet piling seawall with dead man anchor system. Construction on the Eastbound tunnel bore was completed on July 31st, 2012 and the Westbound tunnel bore is scheduled to be completed in early 2013.

![Figure 1: Existing Dodge Island structures (left) & location relative to tunnel alignment (right).](image)

The TBM travelled from Watson Island, under the Government Cut channel and on to Dodge Island. Geologic conditions encountered by the TBM at the tunnel depth below the structures were complex mixed face conditions through highly
heterogeneous coralline limestone exhibiting varying degrees of cementation with localized zones of loose un cemented sand and silt. A key uncertainty on this project was the behavior of the vuggy limestone which could have led to high volume loss and potentially cause damage to the structures. As the TBM was beneath the channel prior to reaching the structures, surface settlement data was not available to evaluate the performance of the tunneling and the resulting volume loss.

For this project, a finite element analysis evaluated the risks of damage to the structures and a Bayesian updating approach was developed for predicting ultimate ground settlements by systematically evaluating early monitoring data while the tunnel drive is approaching a point of interest. This method can prove useful for providing an early warning of unanticipated magnitudes of ground movements to allow time for contingency measures to be implemented.

2   NUMERICAL MODELING

To evaluate the influence of the tunneling induced ground movements on the existing structures, a three-dimensional finite element analysis was performed. Ground materials were modeled with three-dimensional 4-node tetrahedral elements with a Mohr-Coulomb failure criterion. The structures were modeled with a combination of elastic plate elements, beam elements, and truss elements, as can be seen in Figure 2.

![Figure 2: Discretization of overall finite element model (left) & existing structures (right).](image)

The finite element model was used to perform a potential damage assessment on the Dodge Island structures, and has been summarized elsewhere (see [4]). For this work, the Bayesian priors were developed based on results from the finite element analyses which will be continually updated as new settlement data is collected.
The project Geotechnical Interpretative Report (GIR) provided ranges for geotechnical parameters defined by the calculated mean values, as well as plus and minus one standard deviation. These parameters were input into the finite element model to calculate upper bound, lower bound, and mean settlements. A Beta distribution was fit to the settlements predicted by the finite element analysis to determine the prior density distribution function (the Prior) as shown in Figure 4.

Figure 3: Finite element prediction of surface settlements for mean geotechnical parameters.

3 BAYESIAN UPDATING

Bayes Theorem is described in detail within [3] and is given by equation (1) below. The Bayesian updating approach requires an estimation of the Prior, \( p(\theta) \), which is subjective based on the analyst. With the collection of new data, the Prior is iteratively updated with the observed sampling density, \( f(y|\theta) \), to calculate the posterior density function, \( p(\theta|y) \) (the Posterior). Theoretically, with the collection of enough new data, all answers will converge on the actual value regardless of the chosen Prior.

\[
p(\theta|y) = \frac{f(y|\theta)p(\theta)}{\int f(y|\theta)p(\theta)d\theta}
\]

Where:

\( \theta \) – Maximum predicted settlement at TBM centerline
\( y \) – Observed settlement data normalized to ultimate maximum settlement based on [1] and [5]
\( p(\theta) \) – Prior probability density function
\( p(\theta|y) \) – Posterior probability density function
\( f(y|\theta) \) – Sampling density function of the observed settlements
If we incorporate the Prior developed based on results from the finite element model (i.e. resulting from the GIR ground parameters) and update it with data obtained from in-situ instrumentation as it is collected, we can arrive at a new Posterior for ultimate surface settlements at the TBM centerline (see Figure 4). In practice, the model was iteratively updated, using the Bayesian updating approach, daily to incorporate new data collected from the instrumentation.

**Figure 4:** Prior and posterior distributions for maximum surface settlement.

The in-situ monitoring arrays were not positioned directly on the TBM centerline. As the prior distribution is based on the maximum predicted settlements, which occur at the centerline of the tunnel and well after the TBM has passed, the data were normalized transversely using reference [5] and longitudinally using reference [1].

### 3.1 Prediction & Performance of Settlement

Extensive instrumentation was installed on all the existing structures to evaluate the performance of the tunneling. Mounted three-dimensional optical prisms were installed with settlements recorded twice per day by an automated total station while the TBM was in the vicinity. This instrumentation data was used as the response variable (θ) for the Bayesian updating as a test case to assess the validity of the approach.

The TBM cutter head reached the northern edge of Dodge Island on May 24th, 2012 with measurable settlements first being recorded on the same day. Tunneling generally occurred quickly between May 24 and May 29 (achieving advance rates of approximately 10 meters per day), where the progress was slowed passing beneath the southern end of the existing structure (see Figure 5).
Figure 5: Location of TBM on May 24, 2012 (left) & May 29, 2012 (right).

Utilizing the limited amount of measurable settlement data up to May 24 in the first iteration, the Prior most likely maximum settlement drops significantly from 20mm down to a Posterior of 4.5mm. The Posterior was calculated with instrumentation readings of approximately 2mm or less. This illustrates the method’s ability to provide early prediction of anticipated maximum settlements which would allow time for installation of contingency measures or to validate that observed settlements will be within acceptable ranges.

The model was iterated with each set of daily instrumentation data, and the prediction begins to converge after a few iterations (see Figure 6). Ultimately, settlements of 9mm were observed in the structure at the point where the tunneling progress rate slowed; however, the ultimate settlements were predicted within 5mm by instrumentation installed in front of the structure reporting only slight ground movements.

Figure 6: Longitudinal settlement projection for data up to May 24 (left) & May 29 (right).
4 SUMMARY

The Port of Miami Tunnel Project proved challenging with respect to evaluating the potential risk to the existing surface structures due to the complex geotechnical conditions. Large bore driven tunnels of this size have not previously been constructed in this region and little comparative information was available to reference. Coralline limestone local to south Florida exhibits high degrees of heterogeneity, which make determining appropriate numerical modeling geotechnical parameters challenging. A three-dimensional finite element analysis was conducted to evaluate the potential effects on the surface structures, and a wide range of potential maximum surface settlements was calculated resulting from the uncertainty in geotechnical information. To more accurately predict surface settlements imposed on the structures as the TBM approached, a method for systematically evaluating surface settlement data collected ahead of the TBM was developed using a Bayesian updating approach.

The method relies on calculations from the finite element model to develop prior estimations of expected values and incorporates systematically collected surface settlement data to update the prior probabilities. In this example, the predicted settlements were considered non-detrimental to the integrity of the structure; however, if the ground conditions were worse than what was observed, the settlements could have potentially been unacceptable and would have been predicted in advance. For the Port of Miami Tunnel project, the method would provide validation that the observed settlements are within ranges which are acceptable to the structure.

REFERENCES


Three Dimensional Numerical Analyses for Assessing Mechanized Tunnelling Impact on Existing Structures

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Abstract

This paper presents the methodology and results of comprehensive three-dimensional finite element analyses which were performed to assess the potential impact of tunnelling under a bridge. The finite element models took into account all relevant elements of construction process including soil behaviour, shield tunnelling, precast concrete segmental lining and the tail void grouting. The models also accounted for stage construction and detailed shield-driven tunnel boring machine (TBM) processes including applying balancing face pressure and injecting bentonite slurry through the TBM shield. Three dimensional finite element models developed to ensure minimal impact of tunnelling on the piles supporting the bridge. This was achieved through verifying the adequacy of load carrying capacity and structural integrity of piles during and after construction of tunnels. This study has demonstrated that the predicted tunnelling-induced impacts on the existing structures can be effectively mitigated by using mechanized shield-driven TBM tunnelling.

Keywords: Mechanized Tunnelling, Pile, Three Dimensional Finite Element Analysis, Shield-driven Tunnelling
1 INTRODUCTION

This paper intends to highlight using advanced three dimensional finite element modelling in tunnelling and underground engineering applications. Finite element modelling is an indispensable tool in assessing the impact of underground construction on existing structures and utilities. It enables designers to virtually simulate different stages of construction process with desired level of details. In practice, detailed finite element modelling is performed at the final stages of design to refine designs which were based on simplified or approximate methods.

The project under study is one of the capital projects envisioned to expand the public transportation in the city of Los Angeles. The alignment will be connecting two existing subway lines currently terminated in the downtown area. The preliminary design phase included approximately 1.6 kilometer long 6.7 m diameter twin TBM tunnels in soft ground. Earth pressure balance (EPB) shield tunneling was proposed for excavating the tunnels.

Designing a transit facility in currently congested downtown area brought a number of design challenges into the picture. The geometrical constraints of connecting two existing stations and cost reduction considerations demanded setting up the alignment in close proximity to existing buildings, structures, and utilities. For instance, the proposed alignment passed under an existing bridge in proximity of its pile foundations which are supporting the bridge piers. The alignment of TBM-bored tunnels indicated a minimum of 75 cm separation between the future tunnels and the existing piles.

Three dimensional finite element models developed to ensure the minimal impact of tunneling on the bridge structure. The adequacy of load carrying capacity and structural integrity of piles were ensured during and after construction of tunnels. The finite element model accounted for stage construction and detailed shield-driven TBM processes including applying the balancing face pressure as well as injecting bentonite slurry through the TBM shield. The model took into account all relevant components of the construction process including the nonlinear soil behavior, shield tunneling, segmental lining installation and the tail void grouting.

2 FINITE ELEMENT MODELING APPROACH

There are several studies attempting to estimate and quantify the structural response of piles due to tunnelling-induced ground movement and provide simplified methods such as design charts to estimate the additional forces developed in piles [1]. These
simplified methods are useful in preliminary design stage; however, finite element modelling is adopted in detailed design stage where more realistic and refined estimate is needed. Simplified methods have some drawbacks such as relying on approximate equations for induced ground deformation and the need to estimate the ground volume loss parameter. Finite element modelling consider the complex and dynamic nature of shield-driven tunnel excavation, staged construction, segmental lining installation process, tail void grouting, and hydro-mechanical coupling in the surrounding ground which are essential in modelling the ground behavior and pile response.

During the past three decades, a vast amount of effort has been expended to numerically simulate the shield-driven TBM tunneling processes and construction operation to accurately estimate the induced ground settlement. Among the latest attempts, Kasper and Meschke [2] developed a three-dimensional finite element model to study the influence of the soil and grout material properties and the cover depth on the surface settlements, loading and deformation of the tunnel lining and steering of the TBM. They modeled the TBM as a rigid movable body in frictional contact with soil. Their simulations employed a two-field finite element formulation to solve the strain field and water pore pressure in soil and grout materials. Based on a number of parametric studies, Kasper and Meschke [3] concluded that: 1) strength characteristics and the over-consolidation ratio are major factors influencing the soil deformation in the vicinity of the shield machine and surface settlements, 2) for soils with a high permeability, larger final settlements observed only after full consolidation was observed, and 3) the cover depth of the tunnel is the most important factor in determining the forces developed in the lining.

2.1 Geometry and Mesh Generation

The adopted 3D analysis approach allowed modeling the geometry of tunnel and excavation staging in order to evaluate the full impact of excavation progression on existing structures. The modelling was performed with Midas Geotechnical & Tunneling Analysis System, MIDAS/GTS [5]. The size of the model was determined in such a way to minimize the boundary effects on the analysis results while allowing the analysis to be performed efficiently. The finite element mesh was consisting of tetrahedron solid elements. A small element size of 60 cm was used in the vicinity of the tunnels. In areas far away from the tunnels, the maximum element size was increased to 300 cm.
2.2 Procedure for Modeling EPB TBM

Applying face pressure and shield bentonite slurry pressure, installing segmental rings, and tail void grouting are among features that were considered in the analysis in order to allow an accurate simulation of the EPB tunneling operations. The TBM excavation advances were modeled in 1.5 m intervals which is the length of one ring. The face of excavation was immediately pressurized after excavating each drift in order to reduce the settlement due to face loss. The face pressure was assumed to be constant for ease of application. The applied balancing face pressure was set equal to the horizontal insitu stress at the centerline of the tunnel.

In order to model the conical shield support, compression-only gap elements were used to model the conical shield and the variable gap between the ground and the shield. The maximum gap was considered to be 7.5 cm at the tail of the shield. The length of the shield was assumed to be 4.5 m which is equal to three drifts with 1.5 m in length. Bentonite slurry pressure was applied through the length of shield, i.e. 4.5 m behind the face. This slurry pressure prevents the soil from moving in and reduces the volume of shield ground loss and consequently reduces the ground deformation and settlement. The slurry pressure value was considered as the mean in-situ vertical and lateral stresses at the tunnel’s springline elevation. By increasing bentonite slurry pressures, the crown deflection of tunnels as well as ground convergence would decrease. Theoretically, there is a pressure at which the settlement would completely diminish. Pressures in excess of this value would result in heaving of the ground surface.

Pre-cast concrete segmental rings were installed behind the shield. The first 1.5 m behind the shield representing the ring under installation was assumed without any support; however, prior rings installed provided full support to the excavation. In addition, the thickness of the segments was assumed to be 25 cm along with 5 cm of hardened backfill grout was considered in the model. A reduction factor of 0.80 was applied to the flexural stiffness of the rings to account for the effects of segment joints as suggested in Lee and Ge [4].

The in-situ stresses were initialized through prescribing at-rest lateral pressure coefficient. Surcharges due to the bridge service load and seismic load were applied on the pier columns during the initialization stage. All displacements were reset to zero in the initial stage. Mohr-Coulomb failure criterion was adopted for rock behavior. The displacement degrees of freedom at the bottom face of the model were fixed in all directions; however, only out-of-plane displacements were fixed on the four side faces of the model.
2.3 Verification of Finite Element Results

A check on the order of magnitude accuracy of three dimensional finite element results was made via comparing results with an approximate method described in [1]. Also, convergence studies for different finite element meshes were performed to guarantee the convergence of results.

3 TUNNEL CROSSING UNDER EXISTING BRIDGE

This section discusses the results of advanced 3D numerical studies which were conducted to assess the impact of tunneling-induced ground movements on the existing bridge piles. The proposed tunnel alignment runs between the axes 2, and 3 of the bridge piers and columns as shown in Figure 1. The piers and columns of the bridge are resting on deep foundations including piles and caissons. Soil movement as a result of tunnel excavation induces additional forces in the piles. The additional forces may potentially distress the structural integrity of the piles and the superstructure.

![Figure 1: Sketch of tunnels and piles of existing bridge](image)

The profile of the TBM-bored tunnels indicated a minimum of 75 cm separation between the future tunnels and the existing piles. It is evident that small separation between the bored tunnels and the existing piles will result in a reduction of skin
resistance and tip bearing capacity of the piles depending on the relative location of tunnel with respect to the pile. 

The tunneling excavation causes both axial and lateral deformations in piles located close to the tunnel. The maximum lateral deformation in the pile occurs about the depth of the tunnel’s springline as the surrounding soil medium converges toward the center of the tunnel. As detailed in Chen et al. [1], the vertical soil movement above the tunnel’s springline is generally downward and tends to impose negative skin friction on the pile, causing settlement and possible reduction in the pile load-carrying capacity; however, the vertical soil movement below the tunnel’s springline is upward and will cause pile heave. As a result of pile deformation, additional axial force and bending moments will be induced in the piles. The key factor in pile’s response and induced forces is the ratio of pile length to the tunnel cover depth. The pile behavior is rather different for long piles (piles whose tip is below the tunnel’s springline) and short piles (piles whose tip is above the tunnel’s springline) because maximum soil movements occur about the tunnel springline.

The forces induced in piles as a result of the tunnel excavation were calculated and added to the existing service forces in the piles. Service forces are due to the dead load of the super-structure and traffic loads. Additionally, a lateral load equal to 10% of vertical load was considered at the bridge’s deck level to account for lateral seismic loads. It is noteworthy to distinguish piles belonging to pile groups of 3A and 2D, 3D (shown in Figure 1) when interpreting the results. Piles in pile group 3A are relatively shorter than piles in the rest of pile groups. On the other hand, piles in pile group 2D, and 3D have the minimum separation with the tunnel.

3.1 Pilecap Settlement and Pile Forces

The ground convergence, pile cap settlements and induced forces in the piles can be controlled via applying pressurized bentonite slurry through the shield. By increasing the bentonite pressure, the tunnel convergence, pile disturbances, and ground settlement will decrease. The value of applied pressure was considered as the mean of in-situ vertical and horizontal stresses at the tunnel’s springline elevation.

The induced axial force and bending moment when applying bentonite slurry pressure are presented in Table 1. The bending moment reported in Table 1 corresponds to the bending moment associated with pile deformation transverse to the tunneling direction. The final forces induced as a result of tunneling will remain in the piles permanently.
3D Numerical Analyses for Assessing Mechanized Tunneling Impact on Existing Structures

Table 1: Axial force and bending moment in piles

<table>
<thead>
<tr>
<th>Pile group</th>
<th>Axial force under service load [kN]</th>
<th>Axial force after tunneling [kN]</th>
<th>Bending moment under service load [kN-m]</th>
<th>Bending moment after tunneling [kN-m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>2A</td>
<td>480</td>
<td>915</td>
<td>4.6</td>
<td>28.9</td>
</tr>
<tr>
<td>3A</td>
<td>512</td>
<td>488</td>
<td>5.2</td>
<td>34.0</td>
</tr>
<tr>
<td>2C</td>
<td>612</td>
<td>863</td>
<td>8.7</td>
<td>24.7</td>
</tr>
<tr>
<td>3C</td>
<td>603</td>
<td>885</td>
<td>12.2</td>
<td>21.5</td>
</tr>
<tr>
<td>2D</td>
<td>477</td>
<td>986</td>
<td>7.4</td>
<td>23.3</td>
</tr>
<tr>
<td>3D</td>
<td>512</td>
<td>959</td>
<td>9.1</td>
<td>34.2</td>
</tr>
</tbody>
</table>

3.2 Pile Strength and Load Carrying Capacity

Structural integrity of piles was investigated for combined effects of axial forces and bending moments via interaction diagram curves. As such, interaction diagrams were developed for 70 cm circular section plain concrete piles according to the ACI-318 code provisions. Figure 2 shows the ultimate axial and bending moment pairs observed in each pile group. The ultimate factored forces were obtained by applying a uniform load factor of 1.5 to the results obtained from analysis. As observed, the order of axial force in piles is about the same except for piles in pile group 3A (piles with shortest length). Based on interaction diagram, the largest demand-to-capacity ratio belongs to pile groups 2D and 3D (piles with least separation with the tunnels). The demand-to-capacity ratio is around 0.5.

The load carrying capacity of piles was evaluated considering pile tip bearing as well as frictional skin resistance contributions. The forces developed did not exceed the load carrying capacity of piles.

4 CONCLUSION

Advanced 3D finite element modelling was performed to assess the impact of tunneling on the pile foundations of an existing bridge. The results indicated that tunneling-induced forces in the piles can be mitigated via applying bentonite pressure throughout the shield. It was shown that piles can safely withstand the additional forces due to tunneling. Deformation of piles and settlements experienced under pile-caps were shown to be negligibly small.
**Figure 2:** Interaction diagram curve for maximum forces in piles

**REFERENCES**


Multiphase Models and Coupled Problems in Tunneling
A Dual Random Two-scale Model for Estimating the Thermal Expansion Coefficient of Early-age Concrete

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Abstract

Early-age effects of mass concrete structures are very sensitive to the thermal expansion characteristics of concrete, and the potential of the material to dilate due to the unit temperature change is referred to as the coefficient of thermal expansion (CTE). As a kind of multi-phase composite, the CTE of early-age concrete depends not only on the performance of constituents, but also on their distribution with randomness; besides, along with the hydration reaction of cement, it shows significant time variability. To calculate the CTE in a more accurate way, concrete has been divided into three different scales firstly, according to the scale law. In each scale, a corresponding representative element volume (REV) is described by introducing random parameters. Then, the connections between different scales are successfully realized using two-scale method. Thus, a dual random two-scale model for CTE of early-age concrete is proposed, and a comparison study with experimental results and Rosen-Hashin bounds is conducted, which checks the validity of the presented model in estimating CTE.

Keywords: Early-age, CTE, dual, random two-scale.

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1 INTRODUCE

In the construction of mass concrete structures, such as dam, immersed tunnel, basement, and wharf, crack control at early-age has been one of the very important aspects [2][12]. The increase of the temperature due to the exothermic nature of hydration, followed by non-uniform cooling to ambient temperature, will lead to formation of temperature gradients in concrete. These temperature gradients and resulting non-uniform thermal deformations within the element are responsible for the build-up of considerable thermal stresses, which are recognized as one of the main reasons for cracks at early-age [13]. In response to this mechanism, a temperature-stress coupling model has been established on macro-scale, based on heat conduction theory and continuum mechanics; besides, to solve this problem, related numerical analysis has been conducted using finite element method [1][10]. This macroscopic model depends on related experimental means, assuming that the CTE of concrete is a constant. However, some researchers suggest that CTE of concrete changes dramatically at early-age along with the hydration, which is not a constant [4][16][17]. Therefore, it is necessary to establish a more accurate model, which can completely reflect the development of CTE at early-age and take composition and microscopic constitution of concrete into consideration.

As a kind of multi-phase composite, concrete consist of cement and aggregates, it’s thermal expansion characteristics depend not only on the performance of constituents, but also on their distribution with randomness; besides, along with the hydration reaction of cement, it shows significant time variability. Multi-scale mechanics provides a new approach to solve these problems, which divide concrete into different scales and think that concrete has different composition and microscopic constitution within different scales, and connect each scale with a certain method, called homogenization [5]. Following this approach, some scholars realize the homogenization between scales using micro-mechanics of composites [3][6], however, some assumptions have been made to simplify the calculation, which can’t reflect the random configuration of concrete. Then, based on the asymptotic expansion theory, two-scale method has been proposed[8][9], which is applicable to the composite materials of periodic or random structure with high inclusion content, and take the interaction between inclusions into consideration well [7][14][15].
2 ASYMPTOTIC TWO-SCALE METHOD

As shown in Figure 1, a point $x$ of homogeneous body can be treated as a periodic multiple permutation of REV which is heterogeneous, its size $\varepsilon$ is assumed to be much smaller than the size of the body. When the equivalent homogeneous body is subjected to external forces, its field quantities such as displacement, stress and strain, will vary with the global coordinate $x$; in the meantime, because of the high heterogeneity of local constitution, they will also vary rapidly within the neighbourhood of the $\varepsilon$ [8]. Therefore, introduce the local system $y=x/\varepsilon$, where $x$ represents the homogeneous scale, $y$ is mainly used to reflect the local information; then, express these quantities in a REV as a function of the coordinate $x$ and $y$, thus, the properties of equivalent homogeneous body is successfully connect to local constitution which is heterogeneous.

![Figure 1: Representation of two-scale method](image)

It should be realized that the thermal expansion performance is a kind of thermal-mechanical coupling behaviour, which can be described as following [14]:

\[
\frac{\partial}{\partial x_i} \left[ k_{ij}(x) \frac{\partial \theta^\varepsilon(x)}{\partial x_j} \right] - h(x) = 0, x \in \Omega \\
\theta^\varepsilon(x) = \bar{\theta}(x), x \in \Gamma_1
\]

(1)

\[
\frac{\partial}{\partial x_j} \left[ C_{ijhk}^\varepsilon(x) \left\{ \frac{1}{2} \left( \frac{\partial u^\varepsilon_h(x)}{\partial x_k} + \frac{\partial u^\varepsilon_k(x)}{\partial x_h} \right) - a_{ij}^\varepsilon(x) \theta^\varepsilon(x) \right\} \right] + f_i(x) = 0, x \in \Omega
\]

(2)

Where $u^\varepsilon(x)$ and $\theta^\varepsilon(x)$ represent the displacement vector and the increment of the temperature in a REV corresponding to point $x$ of equivalent homogeneous body;
where

\[ C_{ijk}^e(x) \] and \( a_{hi}^e(x) \) is the stiffness and thermal expansion coefficients; \( k_{ij}^e(x) \) is the coefficients of thermal conductivity; \( h(x) \) is the heat produced by internal heat sources per unit time and unit mass, while \( f_i(x) \) is the internal force; \( \Gamma_1 \) and \( \Gamma_2 \) is the boundary of temperature and displacement, while \( \bar{\theta}(x) \) and \( \bar{u}(x) \) denote the corresponding boundary conditions.

Besides, properties of material composition in a specific REV can be expressed as:

\[ C_{ijk}^e(x) = \frac{\partial}{\partial x_j} - \frac{1}{\varepsilon} \frac{\partial}{\partial y_j} C_{ijk}^e(y) \]

\[ k_{ij}^e(x) = \frac{\partial}{\partial x_j} - \frac{1}{\varepsilon} \frac{\partial}{\partial y_j} k_{ij}^e(y) \]

\[ a_{ij}^e(x) = \frac{\partial}{\partial x_j} - \frac{1}{\varepsilon} \frac{\partial}{\partial y_j} a_{ij}^e(y) \]

Considering local configuration and increment of temperature, an asymptotic expansion of \( \theta^e(x) \) and \( u^e(x) \) is formed with coordinate \( x \) and \( y \):

\[ \theta^e(x) = \theta^0(x) + \sum_{l=1}^{L} \sum_{\langle a \rangle = 1}^{l} \frac{\partial^l \theta^0(x)}{\partial x_{a_1} \cdots \partial x_{a_l}} \]

\[ u^e(x) = u^0(x) + \sum_{l=1}^{L} \sum_{\langle a \rangle = 1}^{l} \frac{\partial^l u^0(x)}{\partial x_{a_1} \cdots \partial x_{a_l}} + \sum_{l=0}^{L} \sum_{\langle a \rangle = 1}^{l+1} \frac{\partial^l \theta^0(x)}{\partial x_{a_1} \cdots \partial x_{a_l}} \]

Where:

\[ a = (a_j \ldots a_l) \quad \langle a \rangle = |a_j + \ldots + a_l| \quad a_j = 1 \ldots n, j = 1 \ldots l \]

\[ N^a(y) = \begin{pmatrix} N^{a_1 \ldots a_l}(y) & \ldots & N^{a_1 \ldots a_n}(y) \\ \vdots & \ddots & \vdots \\ N^{a_1 \ldots a_n}(y) & \ldots & N^{a_1 \ldots a_n}(y) \end{pmatrix} = \{N^{a_1 \ldots a_l}(y) \ldots N^{a_1 \ldots a_n}(y)\} \]

\[ M^a(y) = \{M^{a_1 \ldots a_l}(y) \ldots M^{a_1 \ldots a_n}(y)\} \]

\[ H^a(y) = H^{a_1 \ldots a_l}(y) \]

\[ N^a(y) \] is matrix function, \( M^a(y) \) is vector function, and \( H^a(y) \) is algebraic function, they are all 1-periodic and defined in REV which can reflect the influence of local heterogeneity of composites. In a standard way, putting the expanded \( \theta^e(x) \) and \( u^e(x) \) into Eq. (1) and Eq. (2), and taking into account the derivative rules under two-scale as follows:

\[ \frac{\partial^l}{\partial x_j} = \frac{\partial}{\partial x_j} + \frac{1}{\varepsilon} \frac{\partial}{\partial y_j} \]
Comparing the coefficients of same powers $\varepsilon^n (n = -2, -1, 0, 1, 2 \ldots)$, a series of identities are obtained, then $N^a(y), M^a(y), H^a(y)$ are solved from them. In a specific REV, define homogeneous thermo-elastic tensor $\Theta^H_{ij}$:

$$\Theta^H_{ij} = \frac{1}{|\gamma|} \int_{\gamma} \left( C_{ijhk} \frac{\partial M_m}{\partial y_n} - C_{ijmn} a_{hk} \right) dy$$

(13)

Then, the homogeneous coefficient of thermal expansion is obtained as:

$$a^H_{hk} = \left[ C^H_{ijhk} \right]^{-l} \Theta^H_{ij}$$

(14)

And, $C^H_{ijhk}$ is the homogeneous stiffness coefficient in REV:

$$C^H_{ijhk} = \frac{1}{|\gamma|} \int_{\gamma} \left( C_{ijhk} + C_{ijmn} \frac{\partial N^k_{mh}}{\partial y_n} \right) dy$$

(15)

### 3 DUAL RANDOM TWO-SCALE MODEL FOR CTE OF CONCRETE

#### 3.1 Representation

As a kind of multi-phase composite, concrete has different material composition and microscopic configuration in different scales; besides, along with the hydration reaction of cement, the performance of concrete shows significant time variability. Therefore, to calculate CTE in a more accurate way, in this research, macro-scale has been divided into concrete meso-scale, mortar scale, cement paste scale, according to the scale law, as shown in Figure 2.

![Figure 2: Scale division of concrete](image-url)
Since there is no influence of randomly distributed aggregates and the experiment on cement paste scale is easy to operate with high precision, cement paste is regarded as isotropic and homogeneous, and, its thermal and mechanical parameters are obtained through test; Mortar scale is a two-phase composite consists of cement paste as matrix and sand as inclusion, while concrete meso-scale is composed of equivalent homogeneous mortar as matrix and coarse aggregate as inclusion, also identified as a two-phase composite.

3.2 Cement Paste Scale to Mortar Scale

For mortar scale, a specific REV needs to be described firstly. With randomly distributed sand, a simple RVE is usually insufficient in providing reliable estimates of the performance of mortar. So, each one is further assumed to consist of matrix and sphere which is non-overlapping and distributed randomly, as shown in Figure 3; each sphere is defined by four parameters, they are center position \((y_1^n, y_2^n, y_3^n)\) and radius \(R\) of sphere. With regard to this specific REV for mortar, the size of it is \(10\,\text{mm} \times 10\,\text{mm} \times 10\,\text{mm}\), radius of sphere is \(0.25\,\text{mm}\), and, volume fraction of sphere inclusion is 40%; besides, it’s remarkable that the center position and \(R\) are subject to uniform distribution within their own range. Thus, the REV is further described with stochastic parameters, which can significantly simulate the randomness and uniformity of sand’s distribution in cement paste.

Figure 3: A random sample of REV for mortar

Based on the above description, let \(n\) denotes the number of generated sphere located in REV, and then a corresponding probability distribution model \(w_m^n\) can be defined:

\[
w_m^n = \left( y_1^1, y_2^1, y_3^1, R^1, \ldots, y_1^n, y_2^n, y_3^n, R^n \right)
\]
Thus, any equivalent homogeneous mortar can be viewed as the permutation of this kind of REV:

$$\Omega = \bigcup_{(w^s_m, t \in Z)} e^{(Y^s_m + t)}$$

(17)

Where, $Y^s_m$ represents a normalized REV and $w^s_m$ is the probability distribution model of the sphere defined above.

Based on the test of cement paste, performance of it are easy to determine; then input thermal and mechanical parameters of cement paste matrix and sand inclusion, as well as volume fraction. Generate a specific REV with the probability distribution model $w^s_m$ for mortar and solve it with asymptotic two-scale method. Thus, in this REV, $H^a(y)$, $N^a(y)$, $M^a(y)$ and homogeneous coefficient of thermal expansion $a_{hk}^{h-m}(x, w^s_m)$ is obtained. Generating M samples for random distributions model $w^s_m(s = 1, 2..M)$ and from Kolmogorov strong law of large numbers, the expected-homogenized coefficient of thermal expansion $\hat{a}_{hk}^{h-m}$ can be obtained by:

$$\hat{a}_{hk}^{h-m} = \frac{\sum_{s=1}^{M} a_{hk}^{h-m}(x, w^s_m)}{M}, M \to \infty$$

(18)

### 3.3 Mortar Scale to Concrete Meso-scale

For concrete meso-scale, a specific REV which can significantly simulate the randomness and uniformity of coarse aggregate’s distribution in mortar needs to be described the same way as mortar scale, as shown in Figure 4. With regard to this specific REV for concrete meso-scale, the size of it is $100mm \times 100mm \times 100mm$, radius of sphere is $5-25mm$, and, volume fraction of sphere inclusion is 35%.

**Figure 4:** A random sample of REV for concrete
Based on the above description, a corresponding probability distribution model \( w^s_c \) for concrete meso-scale can be defined the same way:

\[
(w^s_c = (v^1, y^1_1, y^1_2, y^2_1, y^2_2, y^3_1, R^1, R^2, R^3))
\] (19)

Thus, equivalent homogeneous concrete can be viewed as the permutation of REV:

\[
\Omega = \bigcup_{(w^s_c, t \in Z)} (Y^s_c + t)
\] (20)

Based on the computation results on mortar scale, thermal and mechanical parameters of equivalent homogeneous mortar matrix and coarse aggregate inclusion are input, as well as volume fraction. Generate a specific REV with the probability distribution model \( w^s_c \) for mortar and solve it with asymptotic two-scale method again. Thus, in this REV, \( H^a(y, w^s_c), N^a(y, w^s_c), M^a(y, w^s_c) \) and homogeneous coefficients of thermal expansion \( \hat{a}^{h-c}(x, w^s_c) \) is obtained. Generating \( M \) samples for random distributions \( w^s_c (s = 1, 2, \ldots, M) \) and from Kolmogorov strong law of large numbers, the expected-homogenized coefficients of thermal expansion \( \hat{a}^{h-c} \) can be obtained by:

\[
\hat{a}^{h-c}(x) = \frac{\sum_{s=1}^{M} \hat{a}^{h-c}_{hk}(x, w^s_c)}{M}, M \to \infty
\] (21)

4 MODEL VALIDATION

In order to validate the effectiveness of proposed model, mortar scale is calculated based on the experiment [11], and a comparison study with experimental results and Rosen-Hashin bounds is conducted. Thermal and mechanical properties of sand are shown in Table 1.

<table>
<thead>
<tr>
<th>E (GPa)</th>
<th>Poisson’s ratio</th>
<th>CTE (um/m°C)</th>
<th>Volume fraction (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>60</td>
<td>0.2</td>
<td>9</td>
<td>40</td>
</tr>
</tbody>
</table>

As shown in Figure 5, a comparison study with experiment and Rosen-Hashin bounds is performed. First of all, it can be seen that the calculation results of proposed model are in good agreement with experimental data. Secondly, for these two models, the results are slightly lower of experiment. On the one hand, it is on account of that such models are based on a series of simplified assumptions, one of the main assumptions is that mortar is regarded as sand dispersed in the homogenous cement paste with
perfect bond between them, the other one is the assumed no influence of micro-cracking. On the other hand, in the experiment, some air with high CTE may have been entrapped between the membrane and the briquette which can result in measurement error [24]. So, take all the factors into consideration, a good agreement between the predictions and the experimental data was obtained in this study, thus, it can conclude that the proposed model in this research is quiet effective.

![Comparison of two-scale model and experiment for CTE](image)

**Figure 5:** Comparison of two-scale model and experiment for CTE [11]

## 5 CONCLUSION

In this research, concrete macro-scale has been divided into concrete meso-scale, mortar scale, and cement paste scale. Then, a random REV for each scale is described by introducing stochastic parameters, and successfully realizes the connection between different scales using asymptotic two-scale method. Thus, a dual random two-scale model for CTE of early-age concrete is proposed. Then, mortar scale is calculated based on the experiment of cement paste using proposed model, and a comparison study with experimental results and Rosen-Hashin bounds is conducted, which validate the effectiveness of this model proposed in this research.

In the future work, related experiment of cement paste, mortar and concert will carry out, and a completely cross-scale research will also be conducted.
REFERENCES


Approximations and Mesh Sensitivity Induced by the Simplified Modelling of Grouted Bolts by One-dimensional Tension Elements

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¹ETH Zurich, Switzerland

Abstract

The ground reinforcement by bolts is a technique commonly adopted in tunnelling for stabilizing the tunnel walls or the excavation face. One of the methods to investigate the effect of bolts on the tunnel stability is represented by numerical stress analyses which take account of the individual bolts. A rigorous model of the interaction between grouted bolt and surrounding soil should account of the actual geometry of the bolt. Such a model is, nevertheless, very demanding in terms of computer time for typical problems in tunnelling, i.e. the stability of a reinforced tunnel face, where a relatively big number of bolts has to be represented. As usual in this kind of problem, the bolts are modelled by one-dimensional tension elements, which have null diameter and cannot take into account geometrically the diameter of the bolts or the borehole. The present paper deals with the approximations induced by this simplification. More specifically, the paper shows by means of numerical pullout tests in respect of a single bolt in elasto-plastic soil that the behaviour of this model may exhibit mesh-sensitivity. Finally it gives some guidelines concerning the choice of computational mesh in large-scale numerical simulations involving bolts in order to avoid mesh-sensitivity.

Keywords: Ground reinforcement, grouted bolt, pull-out test, numerical stress analysis, one-dimensional tension elements, mesh-sensitivity
1 INTRODUCTION

Rigorous models of the interaction between grouted bolt and surrounding soil account of the actual geometry of the bolt ([1], [3]). Solid elements have to be used to model the bolt, while the possibility of shear failure at the interface between the grouted bolt and the surrounding soil calls for the use of special interface elements. Such models have a large number of static degrees of freedom and are extremely demanding in terms of computer time, if a large number of bolts have to be represented (as in the case of a reinforced tunnel face). In such cases, a simplified model is usually adopted where the bolts are represented by one-dimensional elements with an idealised zero diameter. The interaction of these elements with the soil is dealt with by interface conditions that are incorporated into the numerical formulation of the one-dimensional elements. The present paper deals with the approximations induced by this simplification. More specifically, the paper shows comparative numerical pullout tests in respect both of a model using elastic solid elements for the bolts (Fig. 1a) and a simplified model with one-dimensional elastic tension elements (Fig. 1b). The soil is assumed elastic perfectly plastic with null dilatancy. The results of the comparative analysis show that the simplified model may exhibit mesh-sensitivity. Finally the paper gives some guidelines concerning the choice of computational mesh in large-scale numerical simulations involving bolts in order to avoid mesh-sensitivity.

2 NUMERICAL MODELS

In order to simplify the problem and to gain a better understanding of the numerical results, we investigate here the interaction between a single bolt and the surrounding soil by considering simple models. Figure 1 shows the geometry and the boundary conditions of the models. A cylindrical computational domain under a uniform radial confinement pressure was considered. The axial stress $\sigma_z$ was taken equal to zero. The behaviour of the soil around the bolt was taken as linearly elastic, perfectly plastic with the Mohr-Coulomb yield criterion (Tab. 1). Two elastic bolt models are assumed which are equivalent in terms of the axial bolt stiffness and the maximum shear force at the interface between the soil and the grouted bolt (Tab. 1).
Modelling of Grouted Bolts by One-dimensional Elements: Approximations and Mesh Sensitivity

Figure 1: Problem layout for the numerical pullout test in elasto-plastic soil with bolts modelled, (a), by solid elements or, (b), by cable elements.

Table 1: Parameter values assumed in the numerical pullout tests

<table>
<thead>
<tr>
<th>Parameters for the bolts</th>
<th>Computational case:</th>
<th>(i)</th>
<th>(ii)</th>
<th>(iii)</th>
<th>(iv)</th>
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<tbody>
<tr>
<td>Interface cohesion (c_I) [kPa]</td>
<td>(\infty)</td>
<td>10</td>
<td>10</td>
<td>0</td>
<td></td>
</tr>
<tr>
<td>Interface angle of friction (\varphi_I) [°]</td>
<td>0</td>
<td>25</td>
<td>0</td>
<td>25</td>
<td></td>
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<tr>
<td>Borehole diameter (d) [m]</td>
<td>0.1</td>
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<td></td>
<td></td>
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<tr>
<td>Bolt Young’s modulus (E_b) [GPa]</td>
<td>20</td>
<td></td>
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Additional parameters for the cable elements (Fig. 2b)

<table>
<thead>
<tr>
<th>Parameters for the cable elements</th>
<th>Computational case:</th>
<th>(i)</th>
<th>(ii)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Interface shear stiffness (K_I) [kPa]</td>
<td>10^7</td>
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<td></td>
</tr>
<tr>
<td>Bolt area (A_b) [m^2]</td>
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Additional parameters for the solid bolt elements (Fig. 2a)

<table>
<thead>
<tr>
<th>Parameters for the solid bolt elements</th>
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<th>(ii)</th>
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</thead>
<tbody>
<tr>
<td>Interface shear stiffness (K_d) [kPa/m]</td>
<td>10^9</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Interface normal stiffness (K_{nt}) [kPa/m]</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Bolt Poisson’s number (\nu_b) [-]</td>
<td>0.25</td>
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Parameters for the soil

<table>
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<th>Parameters for the soil</th>
<th>Computational case:</th>
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<tbody>
<tr>
<td>Young’s modulus (E') [MPa]</td>
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<td></td>
</tr>
<tr>
<td>Poisson’s number (\nu) [-]</td>
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<tr>
<td>Friction angle (\varphi') [°]</td>
<td>25</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Cohesion (c') [kPa]</td>
<td>10</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dilatancy angle (\psi') [°]</td>
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Initial stress field

<table>
<thead>
<tr>
<th>Initial stress (\sigma_0) [kPa]</th>
<th>Computational case:</th>
<th>(i)</th>
<th>(ii)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
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</table>
The used one-dimensional element (Fig. 1b) is the so called “cable element” of the FLAC3D code [2]. The cable element is a two-node, straight finite element with one axially oriented translational degree-of-freedom per node (no bending resistance). The numerical formulation of the cable element incorporates an interface condition which accounts for the shear forces developing parallel to the bolt axes in response to the relative motion between the bolts and the surrounding soil. The interface between the grouted bolt and the soil was modelled by interface elements describing a cylindrical surface for the case of solid bolt elements (Fig. 1a). In both models, the interface behaviour was taken to be rigid – plastic with the Mohr-Coulomb yield criterion. The rigidity was materialized by assuming a high value of interface stiffness (cf. Table 1). The computations were carried out for several values of interface shear strength parameters ($c_I, \phi_I$) in order to analyse both of the failure modes that may occur during bolt pullout: the shear failure in the soil around the bolt and the shear failure along the interface between the grouted bolt and the soil. In order to enforce the model to reproduce the first case, the interface shear strength was taken to be equal to infinity. For the second case, we considered interface strength parameters lower than those of the soil. The particular case of interface shear strength parameters equal to those of the soil was analysed as well (see columns (i) to (iv) of Table 1). The numerical solution was carried out using the FLAC2D finite difference code in the case of solid bolt elements (the axisymmetric model, Fig. 2a) and FLAC3D in the case of cable elements (Fig. 2b). In order to investigate mesh dependency effects, several numerical discretizations with more or less coarse grids (i.e., with different grid sizes $e$ close to the bolt) were considered in both analyses.

**Figure 2:** Numerical discretisation in, (a), the axisymmetric analysis with solid bolt elements (finest grid) and, (b), the spatial analysis with cable elements (finest grid).
Every analysis starts with the initialization of the stress state and proceeds with the numerical pullout test: gradual imposition of displacements $u_p$ at the head of the bolt (i.e. to the grid-points or to the structural nodes depending on whether solid bolt or cable elements, respectively, are used) and calculation of the reaction axial forces $F_p$.

3 RESULTS FOR BOLT MODELLING BY SOLID ELEMENTS

Figure 3a shows the maximum pullout force $F_{p\text{max}}$ as a function of the radial grid size $e$ of the elements adjacent to the bolt for different interface shear strength parameters (lines 1-4). According to Figure 3a, line 1, the maximum pullout force $F_{p\text{max}}$ increases linearly with the grid size $e$ if the limit state is associated with failure of the soil (infinite interface shear strength). As explained below, the grid size dependency of the maximum pullout force $F_{p\text{max}}$, which occurs only in the case of infinite interface shear strength, is due to the uniformity of the stress field inside each element and to the relationship between element stresses and nodal forces.

Consider (for the sake of simplicity) a purely cohesive soil. If the interface strength is infinite, the pullout will cause shear failure of the first row of soil elements next to the bolt (see Fig. 4, elements “a”, “b”, “c”, …). At the limit state, the shear stress $\tau_{ry}$ inside each element of the first row will be equal to the soil cohesion $c$. In axisymmetric, numerical analyses, the element contributions to the nodal forces are calculated considering the average radius of every element. They depend, therefore, not only on the inner element radius $r_i (=d/2)$ but also on the outer radius $r_o (=d/2+e)$ and thus on the grid size $e$. In the present case (e.g., for element “a” in Fig. 4),

$$F_{a_{1,y}}+F_{a_{2,y}} = c l 2\pi \frac{r_i+r_o}{2} = c l \pi (d+e) = c l \pi d \left(1+\frac{e}{d}\right). \quad (1)$$

The pullout force increases linearly with the grid size $e$, because it is equal to the sum of the contributions $F_{a_{1,y}}$, $F_{a_{2,y}}$, $F_{b_{1,y}}$, $F_{b_{2,y}}$, … of the elements to the forces of the boundary nodes of the bolt. Note that according to Eq. (1), the spatial discretisation of the problem increases apparently the effective diameter of the grouted bolts from $d$ to $d+e$, i.e. by the factor $1+e/d$. Consequently, the grid size $e$ has to be selected sufficiently small relatively to the bolt diameter in order to reduce the discretisation-induced error.

The erroneously high pullout force and mesh-sensitivity are practically irrelevant if the interface strength is equal or lower than the strength of the soil (curves 2, 3 and 4 in Fig. 3a) because in this case the interface fails before the soil.
For the smallest considered grid size \((e = 0.001 \text{ m})\), the infinite interface strength model (line 1) leads expectedly to the same maximum pullout force like the model with interface elements having the shear strength parameters of the ground.

**Figure 3:** Maximum pullout force as a function of the grid size of the soil elements adjacent to the bolt for bolt modelling, (a), by solid elements or, (b), by cable elements.

**Figure 4:** Explanatory drawing on the mesh dependency of element nodal forces.
The maximum pullout force calculated under the assumption of a purely cohesive interface (line 4) is equal to the expected value \( F_{p_{\text{max}}} = \pi dL' c_I = \pi \times 0.1 \times 3 \times 10 \text{ kN} = 9.4 \text{ kN} \). In the case of frictional interface (lines 2 and 3), however, the numerical maximum pullout force (about 18 and 10 kN for case 2 and 3, respectively) is lower than the force that one might expect on the basis of the prescribed confining stress of 30 kPa \( F_{p_{\text{max}}} = \pi dL' \left(c_I + \sigma_t \tan \phi_I\right) = 22.6 \text{ kN} \) and 13.2 kN. The reason is that the radial stresses (and thus also the frictional resistance) along the interface decrease during to the pullout.

4 RESULTS FOR BOLT MODELLING BY ONE-DIMENSIONAL CABLE ELEMENTS

The behaviour of the one-dimensional cable elements exhibits some similarities to (but also some differences from) the behaviour of the solid bolt elements (Figure 3b). Consider again line 1, which applies to the case of infinite interface strength. The reason for the observed mesh sensitivity is exactly the same like before: The nodal forces resulting from the internal element stresses are proportional to the average radius, which in the present case is equal to \( e/2 \). Assuming for the sake of simplicity a purely cohesive material, the following equation applies for the example of Fig. 4 instead of Eq. (1):

\[
F_{a_{1,y}} + F_{a_{2,y}} = c l 2 \pi \frac{r_I + r_o}{2} = c l \pi e = c l \pi d \left( \frac{e}{d} \right).
\]  

(2)

According to this equation, the effect of the spatial discretisation is equivalent to an apparent change of the bolt diameter from \( d \) to \( e \) (or by the factor \( e/d \)). Contrary to the solid bolt elements, where the discretisation always increases their apparent diameter, the apparent diameter in the case of cable elements may be bigger or smaller than the actual diameter depending on whether \( e > d \) or \( e < d \). This is why the cable element exhibits mesh sensitivity even if assuming a low interface strength (lines 2 to 4 in Fig. 3b). There is always such a small grid size that the pullout force according to Eq. (2) becomes lower than the actual pullout force. The results do not depend on the grid size only if the shear failure occurs at the interface rather than in the soil and this happens only if the grid is sufficiently coarse. The critical grid size \( e_{cr} \) increases with the interface strength (cf. lines 2, 3 and 4 in Fig. 3b). If the interface shear strength parameters are equal to those of the soil (curve 2), the critical grid size \( e_{cr} \) is equal about to the bolt diameter \( d \) (cf. Eq. 2). It is, however, thoroughly possible that the
interface exhibits a considerably higher strength $\tau_m$ than the soil due to the effects of soil dilatancy or of grouting pressure. In this case, the critical grid size $e_{cr}$ may be much bigger than the bolt diameter $d$. It is easy to show, that the following condition must apply in order to avoid mesh sensitivity:

$$e > \frac{\tau_m}{c' + \sigma \tan \phi'} d \quad (3)$$

In the case of finite interface strength and of a sufficiently coarse grid, the maximum pullout force of the cable element (Fig. 3b) is higher than the force of the solid bolt element (Fig. 3a), if the interface has a frictional resistance (lines 2 and 3). This difference does not exist in the case of purely cohesive interface (line 4). The reason is that the radial stress acting upon the bolt decreases during pullout in the case of the solid bolt elements (see last Section) but remains equal to the far-field confining stress in the case of the cable elements.

5 CONCLUSIONS

The simplified one-dimensional bolt model with build-in interface conditions may exhibit mesh-sensitivity or fail to map accurately the frictional resistance of the bolt-soil interface. The grid size of the soil elements adjacent to the bolt has to be selected carefully in order to avoid failure in the soil elements. If the bond strength is high a very coarse calculation grid may be needed in order to avoid mesh sensitivity. With respect to the stress analysis of tunnels reinforced by bolts, the above strategy reduces accuracy of the numerical results close to the limit state.

REFERENCES


Simulation of the Backfilling Process with Annular Gap Grouting Mortar

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²The University of Queensland, School of Civil Engineering, 4072, St Lucia, Queensland, Australia

Abstract

This work focuses on the numerical modeling of the filling process of the annular gap with grouting mortar in the field of mechanized tunneling. The aim of the filling process is to fill the annular gap in a complete way in order to obtain a secure and stable bedding of the tunnel lining. This is achieved by setting up a defined stress state in the shortest possible period of time, e.g. to minimize settlements of the surface. The presented continuum modeling approach describes the processes on a scale which is characterized by the length of the gap between the lining and the soil (macro scale), including pore scale physical effects. For the consideration of micro scale effects with the proposed model, the Constriction Size Distribution (CSD) of the solid granular phase is calculated from the Grain Size Distribution (GSD) during the Finite Element Method (FEM) calculations. Thus the parameter of the mass exchange term can be defined in each time step. The model presented here is suited for the description of the formation of a filter cake and clogging phenomena.

Keywords: Constriction size distribution, filter cake, annular gap grouting mortar
1 INTRODUCTION

Mechanized tunneling techniques are the key to extend an existing infrastructure in densely populated areas. This ecological way of tunneling is dependent on many processes, which have to be coordinated.

One of the processes taking place is the backfilling of the annular gap with grouting mortar. This procedure is necessary, to ensure a stable bedding of the tunnel lining and to minimize settlements on the surface. This is achieved by completely filling the annular gap and therewith recovering the primary stress state of the soil surrounding the tunnel lining. Therefore physics of the backfilling process have to be understood in order to develop a numerical model capturing the backfilling procedure.

An important physical effect to simulate the backfilling process is the infiltration of fluidized fine particle (fines) of the grouting mortar into the surrounding soil. As a first approximation, the relative velocity of the fluid and the fluidized fines is neglected, which means, that fines and fluid are moving with the same velocity through the pore space. In this case, we define the liquid-solid phase transition, i.e. the attachment of fines from the fluid to the rigid skeleton, as infiltration. Due to infiltration, a layer of highly reduced permeability, the so-called filter cake may occur on the border between the surrounding soil and the mortar. In our case the surrounding subsoil has the functionality of a filtering domain. Then a formation of a layer consisting of attached fines inside the subsoil is called internal filter cake, whereas the occurrence of those layer on the border outside the filtering material is called external filter cake. Due to the existing pressure gradient between the grouting pressure and the pore pressure in the subsoil, seepage occurs. As a result of infiltration the permeability of a certain region might be reduced in a way that no more seepage is possible. This phenomenon is called clogging.

On the one hand, the hydraulic behaviour of the subsoil can be numerically simulated by macroscopic models, e.g. using the FEM. These models are in general optimized due to calculation time, but not accurate enough to capture the physics taking place in the annular gap. On the other hand microscopic approaches such as Discrete Element Method (DEM) simulations can be used, which describe the behaviour on the pore scale. Although these models are precise enough to capture infiltration problems, they are extremely time consuming.

Therefore we propose to use a mesoscopic model, described in this contribution. The modeling idea is to use FEM-techniques for the simulation of hydraulic properties. Furthermore the FEM-model is enriched by geometrical microstructural information.
on the level of the material point (integration point in the FEM context), to describe infiltration phenomena in an adequate way.

In this investigation mortar is assumed to consist of spherical particles and to be fully saturated with a fluid. In this case, transport of particle-laden fluid through the porous medium is described using the Theory of Porous Media (TPM)\cite{1,4}, cf. chapter 2. To determine the amount of fines attached to the rigid skeleton, the CSD of the pore network has to be compared to the GSD of the fluidized fine fraction. This is achieved by geometrical considerations, investigated in chapter 3.

2 CONTINUUM MODELING OF INFILTRATION PROCESSES

The TPM is used to describe the transport and the infiltration of fluidized particles in a liquid through a porous medium. First, the fully saturated Representative Volume Element (RVE) is assumed to consist of four constituents $\varphi^\alpha$ with $\alpha = \{f, a, sn, sa\}$, which are described by their volume fractions $n^\alpha = dv^\alpha / dv$. The phase $\varphi^{sn}$ describes the rigid solid skeleton, $\varphi^{sa}$ the phase of fine particles which are attached to the primary fabric, $\varphi^a$ the fine particles dissolved in the pore fluid and $\varphi^f$ the pore fluid. Then, the mass balances of the respective constituents and the mixture are evaluated to describe the transport process. Following de Boer \cite{1}, Ehlers & Bluhm \cite{4}, and Steeb \cite{15} the general partial mass balance in local form can be written as

$$\left(\rho^\alpha\right)' + \rho^\alpha \text{div} \nu_\alpha = \dot{\rho}^\alpha = : \dot{n}^\alpha \rho^{\alpha R}. \tag{1}$$

Here, $\rho^\alpha = n^\alpha \rho^{\alpha R}$ is the partial density, $\rho^{\alpha R}$ the effective density, $\nu_\alpha$ the particle velocity, and $\dot{\rho}^\alpha$ the density production rate of the constituent $\varphi^\alpha$. Moreover, the infiltration of the fine components $\varphi^a$ is realized by a mass exchange term $\dot{n}^a$

$$\dot{n}^a = -\beta c |q|, \tag{2}$$

in case of material incompressible constituents $\rho^{\alpha R} = \rho^{\alpha R}_0$. This can be understood as a constitutive assumption, $\beta$ is a material parameter, $c$ the concentration of fines in the suspension and $q$ the filter velocity according to the Darcy-relationship, cf. Ehlers and Bluhm \cite{4}

$$q = -\frac{k^s}{\eta^{IR}} \text{grad} p. \tag{3}$$

Here $\eta^{IR}$ is the effective dynamic viscosity of the particle-laden suspension depending on the amount of fluidized particles $c(x,t)$ and $k^s(\phi)$ is the intrinsic permeability
of the porous fabric depending on the porosity $\phi(x,t)$ which can be calculated by the equation of Kozeny-Carman, cf. Carrier [2]

$$k_s(\phi) = k_s^0 \left( \frac{\phi^3}{(1-\phi)^2} \right) \left( \frac{1-\phi_0}{\phi_0^3} \right) \quad \text{and} \quad k_s^0 = \frac{1}{C_1 (1-\phi)^2} D_{eq}^2,$$

(4)

with the initial porosity $\phi_0$ and the initial permeability $k_s^0$, which can be calculated using the initial porosity, the so-called Kozeny-Carman constant $C_1$ Irmay [7], Ergun [6] and the equivalent particle diameter $D_{eq}$, Carrier [2].

For the evolution of the effective dynamic viscosity of a suspension $\eta^{iR}$, Einsteins correlation [5] was used

$$\eta^{iR}(c) = \eta^{iR} \left( 1 + \frac{5}{2} c \right),$$

(5)

where $\eta^{iR}$ is the initial dynamic viscosity of a liquid and $c$ the concentration of particles in the suspension. Note, that this correlation is only valid for dilute suspensions. For dense suspensions more precise approaches are available and we refer to Drew and Passman [3].

For this example it is assumed that the mass exchange occurs only within two constituents $\hat{n}^a = -\hat{n}^{sa}$. In this contribution we split the material to $\beta = \psi \phi$, where $\phi = \phi(x,t)$ is the porosity and $\psi$ is a new material function. Applying this condition on Eq. (2) it remains

$$\hat{n}^a = -\psi \phi \left| \mathbf{q} \right|.$$

(6)

By this reformulation of the mass exchange term a physical interpretation for the material parameter is obtained. The expression $(\phi \left| \mathbf{q} \right|)$ gives an information about the total amount of fine particles, transported through the considered pore space. The introduced material function, in the simplest case a material parameter $\psi \in [0,1]$ scales the amount of infiltrated particles in each time step, where $\psi = 0$ means that no infiltration is taking place. In case of $\psi = 1$ all particles which are in the domain are infiltrated.

Note, that for a more detailed derivation of the governing equations we refer to a previous publication [11]. Thus, the following set of equations is representing the Initial Boundary Value Problem (IBVP) of infiltration for the material body $\mathcal{B}$ in the
time interval $T \in [t_0, t]$, which is visualized in Fig. 1

$$\text{div} \left[ \frac{k^s}{\eta^l} \text{grad} \ p \right] = 0, \ \forall \mathbf{x} \in \mathcal{B} \times T,$$

(7)

$$\partial_t (c \phi) + \text{div} \left[ c \frac{k^s}{\eta^l} \text{grad} \ p \right] = \hat{n}^a, \ \forall \mathbf{x} \in \mathcal{B} \times T,$$

(8)

where $k^s$ is the evolving intrinsic permeability, $\phi$ the porosity, $\eta^l$ the evolving viscosity of the suspension, and $p$ the pressure. Additionally the evolution equation for the porosity has to be considered

$$\partial_t \phi = \hat{n}^a.$$  

(9)

Boundary conditions for the filter flux $q$ at the Neumann boundary $\Gamma_N$ and the pressure $p$ at the Dirichlet boundary $\Gamma_D$ as also initial conditions in the material body $\mathcal{B}$ in the time $t_0$ for the concentration $c_0$, the porosity $\phi_0$ and the permeability $k_0^s$ have to be defined

$$q = q \cdot \mathbf{n} = \bar{q}, \quad \forall \mathbf{x} \in \Gamma_N \times T,$$

$$p = \bar{p} \wedge c = \bar{c}, \quad \forall \mathbf{x} \in \Gamma_D \times T,$$

$$c = c_0 \wedge \phi = \phi_0 \wedge k = k_0^s, \quad \forall \mathbf{x} \in \mathcal{B} \times t_0.$$

For the numerical implementation Galerkin finite element scheme was used. Therefore the strong formulation of the IBVP has to be transferred into a weak formulation with the primary variables $\mathcal{P} = \{p, c\}$. This is achieved by multiplication of the regarding strong form equations by test functions and integration of those equations in space. The weak form of the IBVP can be written as

$$\int_{\mathcal{B}} \left[ \frac{k^s}{\eta^l} \text{grad} \ p \right] \cdot \text{grad} \ \delta p \ dv = \int_{\partial \mathcal{B}} q \delta p \ da,$$

(10)

$$\int_{\mathcal{B}} \left[ \partial_t (\phi c) - \partial_t (\phi) + \left[ \frac{k^s}{\eta^l} \text{grad} \ p \right] \cdot \text{grad} \ c \right] \delta c \ dv = 0.$$  

(11)

The boundary- and initial conditions, which were given in context with the strong formulations have also to be fulfilled. Note, that the Neumann boundary condition for the filter velocity $q$ has to be weakly satisfied. To complete the weak IBVP Eq. (9), which is an Ordinary Differential Equation (ODE), has to be solved with conventional methods.
3 GEOMETRIC ANALYSIS OF THE CSD

After setting up the IBVP, the material parameter $\psi$ of the mass exchange term $\hat{n}^a$ has to be specified. The main issue is the identification of mobile and infiltrating particles of the fraction $\phi^a$ in each time step.

One of the first and most famous approaches is the filter rule proposed by Terzaghi [16]. It is applied to proof the stability of a fine grained packing which is located next to coarser packed filter layers. Terzaghi’s rule says, that stability towards erosion occurs if the condition $D_{15}/d_{85} < 4$ is fulfilled, where $D_{15}$ is the 15% fraction of the coarse material and $d_{85}$ the 85% fraction of the fine material.

An approach, which is more detailed taking into account the shape of the GSD curve was proposed by Silveira [14]. There, a more detailed probabilistic analysis is done. It is only valid for soil material which is in its most dens state of compaction. The idea here is to calculate the pore constriction sizes for each possible combination of particles. Combining the result with the probability of occurrence of this pore constriction, Silveira could obtain an approach to calculate the probable path length which each particle can cover before attaching to the porous skeleton. Scheuermann [12] and Schuler [13] proposed different approaches based on Silveira’s idea but improved in some details.

Another approach analysing especially the fine content of the solid fraction to deter-
mine the CSD was proposed by Kenney and Lau [9] and Kenney et al. [8]. In these contributions the determination of a so-called controlling constriction size as also seepage tests to evaluate the instability of the material dependent on its GSD was documented.

Reboul et al.[10] developed a methodology for the calculation of the CSD, based on probabilistic considerations. This approach was calibrated by a numerical assembly of spheres, using the DEM.

However, the approach presented here follows the idea of Silveira, but has certain benefits regarding calculation time and the ability to calculate the CSD depending on the compaction state of the solid. Due to the assumption of spherical particles, the measurement for a single pore constriction are the two biggest circle which can be placed into the area inside the particles which are forming the constriction, e.g. Fig. 2. Following Witt [17], we assume that a pore constellation is always formed by four particles, which form two constrictions. In case of the densest compaction state the constellation collapses to two single constrictions, formed by three particles. In case of the loosest compaction state, both pore constrictions have the same size and overlap each other. The calculation of the compaction state $\Lambda$ of a single pore

![Figure 2](image_url)

**Figure 2:** single pore constellation: a) in the densest compaction state ($\Lambda = 1$), b) in the loosest state of compaction ($\Lambda = 0$)
constellation can be written as proposed by Schuler [13]. In our case the compaction state is assumed to be known and therefore we can calculate the area of the pore constriction \( A_{PE} \) as

\[
A_{PE} = A_{PE\text{max}} - \Lambda (A_{PE\text{max}} - A_{PE\text{min}}),
\]

(12)

where \( A_{PE\text{max}} \) is the biggest and \( A_{PE\text{min}} \) the smallest possible pore constriction area of the constellation. The value for the compaction state lies in the range \( \Lambda \in [0,1] \).

For the calculation of the constriction size distribution, a permutation of all possible constellation has to be done. Therefore four particles with known radii \( r_i \) have to be chosen. Starting to form a constellation the center of the biggest particle (particle A) is placed in the origin of a coordinate system, the center of particle B is located on the same y-value, tangent to particle A. Particle C and particle D are arranged tangential to each other and additional to particle A, respectively to particle B. In this constellation the pore constriction area \( (A_{PE}) \) is calculated as a function of the angle \( \alpha \in [0,\frac{3}{2}\pi] \), by trigonometrical considerations. The minimum and maximum of the function \( A_{PE}(\alpha) \) are necessary entries for Eq. (12). Note that, the minimum of \( A_{PE\text{min}} \) has to be checked for physical plausibility. In case of overlapping particles \( r_A + r_C < AC \) or \( r_B + r_D < BD \), the minimum value has to be chosen in a way, that those particles are tangent to each other. After the calculation of \( A_{PE\text{min}} \) and \( A_{PE\text{max}} \) Eq. (12) is used to derive the pore space which corresponds to the chosen compaction state. Doing this the value for the angle \( \alpha \) is known. The two constrictions of the considered constellation can be derived by solving a non linear set of equations with three unknowns. The probability of occurrence of the constellation \( p_c \) is the same as the probability of occurrence of the four particles \( p_i \), i.e. \( p_c = \prod_{i=1}^{4} p_i \). This information has to be stored and the calculation has to be repeated for each possible constellation of particles from the grain size distribution. Then we obtain the CSD which corresponds to the considered GSD.

The procedure of evaluation of the material parameter \( \psi \) from the CSD is summarized in Fig. 3. Therefore we divide the GSD into the GSD of the fluidized fines and the GSD of the primary fabric. The probability of infiltration of a fraction from the GSD of fines \( p_{inf} \) is the probability of occurrence of a constriction which is smaller than the considered fraction. The material parameter \( \psi \) is the mean value of the probability for infiltration of each fraction of the fluidized fines \( p_{inf}(d_i) \) scaled by the number of particles in this fraction.

Note, that the probability of infiltration is traced back to geometrical considerations.
Figure 3: IBVP for infiltration of mortar into subsoil

and is only valid, if the relative velocity between fluid and dissolved particles is neglected. Otherwise also the hydraulic conditions have to be considered for the determination of the material parameter. In particular it has to be noted, that the probability of infiltration \( \psi = p_{inf}(d_i) = \delta CSD(d_i) \) in case of \( \delta = 1 \) as postulated here, takes only the geometric information into account. In other cases the factor \( \delta \) depends on the hydraulic conditions and has to be determined by experiments.

4 CONCLUSIONS

In this contribution an approach for the numerical simulation of the infiltration process of annular gap mortar was provided. The proposed model consists of a FE-model which is acting on the continuum scale. To improve the simulation microstructural information of the GSD was considered. Furthermore an approach for determination of the CSD from GSD was presented. The obtained material parameter \( \psi \) triggers the infiltration process. An numerical example for the calculation of CSD in case of densest compaction of soil was presented.
A great benefit regarding calculation time to obtain the CSD could be achieved, by skipping physically similar constellations. Therefore the biggest particle is placed in the origin of the coordinate system. Then, many constellations appear to be similar and can be skipped for the derivation of the CSD. Therefore the number of constellations which have to be calculated is drastically reduced.

ACKNOWLEDGEMENT

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Cutting and Material Transport in EPB Shield Machines: 
A Coupled Simulation Approach

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Abstract

The excavation process of EPB shield machines is characterized by complex interactions between the cutting tools and the ground as well as the material transport into the excavation chamber. In this presentation a first attempt is made to couple two partial models for the soil excavation and for the flow of the excavated material within the pressure chamber. As the focus is not on intricate details but on gaining an understanding of the coupling between the processes, a simplified 2D model of the tunnel face and the excavation chamber is adopted. Excavation is simulated using the Discrete Element Method (DEM). This has the advantage of easily capturing fracturing, disintegration and redistribution of material. The fracture process is governed by interaction forces between the particles. The DEM will be used within a layer close to the excavation face. The material transported into the excavation chamber is represented by a non-Newtonian fluid model prescribed by a modified Bingham fluid model. A stabilized Eulerian fluid flow model together with the Characteristic-Based Split method is used for the finite element discretization of an excavation chamber. Using this first prototype coupling strategy, preliminary analysis results concerning the flow conditions in the pressure chamber are presented.

Keywords: Earth pressure balance shield, Tunnel boring machine, cutting wheel, Non-Newtonian flow, discrete element method, finite element method
1 INTRODUCTION

The excavation process at the cutting wheel of tunnel boring machines (TBMs) for soft ground is characterized by complex interactions between the cutting tools and the ground and the subsequent transport of the excavated material into the excavation chamber. In Earth Pressure Balance (EPB) shields, during excavation, the soil is simultaneously modified by conditioning foam, which considerably changes the mechanical properties as well as the permeability of the excavated soil [1]. The soil at the tunnel face is excavated by the cutting tools mounted on the rotating cutting wheel and conveyed through the openings in the cutting wheel into the excavation chamber, where it mixes with the plastic soil already there. The flow of the ground at the cutting wheel and the transport of the excavated material within the excavation chamber of, for example, a mixed shield tunnel boring machine (TBM) is extremely complex. Excavation, mixing of the soil with the bentonite suspension, transport of the material through the cutting wheel and the movement of the mixture within the excavation chamber are intricate processes strongly influencing each other. Here, fundamental research is needed in order to gain more insight.

Modeling of the situation at the cutter head requires the consideration of i) the cutting process and ii) the transport of the excavated material into the excavation chamber. It should be noticed, that due to the foam conditioning of the soil [1], the consistency of the material is changed immediately during and after excavation. Therefore, the material which is conveyed into and transported within the excavation chamber is characterized by a pasty consistency. Here, a first attempt is made to couple two partial models for the soil excavation at the tunnel face and the flow of the viscous earth-foam mixture within the excavation chamber. To this end, a simplified model of the cutting face and the excavation chamber with a reduced two-dimensional geometry is adopted. For the soil, a sand-like material with a low cohesion is assumed. The focus of the presentation is not on intricate details but on gaining an understanding of the coupling between the processes mentioned above and to investigate the feasibility of the proposed hybrid modeling approach. Using this prototype example, we combine two different numerical techniques for the simulation of the coupled cutting-transport processes at the cutting face of EPB shield machines.
2 NUMERICAL METHODS

2.1 Excavation Process

The Discrete Element Method (DEM) is a numerical modelling method in which the material is represented by a number of interacting particles. Essentially, in each calculation cycle, Newton’s second law of motion is solved for all particles. Then, the particle and wall positions are updated, before, in the next step, the chosen force-displacement law is applied. This results in updated contact forces, which allows, again, an updated calculation of the law of motion. For a more detailed description of the method, refer to [2]. In the code that was applied for these calculations, PFC3d by Itasca, each particle is represented by a rigid spherical body.

The force-displacement law is taken into account by choosing a contact model. This defines the interaction between the particles as well as the interaction between particles and walls. For this calculation a linear connection between the acting forces and the displacements is applied. This is described by

\[ F^n_i = K^n U^n n_i \]  

(1)

for the normal direction of the contact and

\[ \Delta F^s_i = -k^s \Delta U^s n_i \]  

(2)

for the tangential direction. Here, \( F^n_i \) relates to the total normal force in each contact \( i \) and \( U^n \) to the total normal displacement, while \( \Delta F^s_i \) relates to the increment of shear force and \( \Delta U^s n_i \) to the incremental shear displacement. \( K^n \) and \( k^s \) mean the normal and shear stiffnesses of the contact, respectively. For the linear model, the normal stiffness is given by

\[ K^n = \frac{k_n^{[A]} k_n^{[B]}}{k_n^{[A]} + k_n^{[B]}}, \]  

(3)

for the tangential stiffness it is

\[ k^s = \frac{k_s^{[A]} k_s^{[B]}}{k_s^{[A]} + k_s^{[B]}}, \]  

(4)

\( k_n^{[A]} \), \( k_n^{[B]} \), \( k_s^{[A]} \) and \( k_s^{[B]} \) being the the normal and shear stiffnesses of the two contacting entities \([A]\) and \([B]\).

Furthermore, contact bonds are implemented, which allows the particles to stick together until a specific tensile force in the contact is exceeded.
2.2 Transport Process

The excavated viscous earth-foam mixture is modelled as a Bingham fluid and its flow within the chamber is described by equations describing the mass and momentum balance of the fluid:

\[ \frac{\partial \rho}{\partial t} = \frac{1}{c^2} \frac{\partial p}{\partial t} = -\nabla \cdot \mathbf{U}, \quad (5) \]

\[ \frac{\partial \mathbf{U}}{\partial t} = -\nabla \cdot (\mathbf{uU}) + \nabla \cdot \tau - \nabla p + \rho \mathbf{g} \quad (6) \]

where \( \mathbf{U} = \rho \mathbf{u} \), \( \rho \) is the density, \( \mathbf{u} \) is the velocity, \( \tau \) is the shear stress, \( p \) is the pressure, \( \mathbf{g} \) is the gravity and \( c \) the sound speed. Different equation describing the compressibility can be adopted in equation 5. The Characteristic Based Split method [4] is used for time discretization to stabilize the numerical instability. Momentum splitting prevents the instability caused by using equal order velocity-pressure interpolation. The discretization along characteristic lines is effective when the convection term dominates the viscous term. After time discretization of the momentum conservation equation, it can be written as:

\[ \mathbf{U}^{n+1} - \mathbf{U}^n = \Delta t \left[ -\nabla \cdot (\mathbf{uU}) + \nabla \cdot \tau - \nabla p^{n+\theta_2} + \rho \mathbf{g} \right]^n \]

\[ -\frac{\Delta t^2}{2} \mathbf{u} \cdot \nabla \left[ -\nabla \cdot (\mathbf{uU}) + \nabla \cdot \tau - \nabla p + \rho \mathbf{g} \right]^n \quad (7) \]

The time discretized momentum equation is then split as follows:

\[ \mathbf{U}^* - \mathbf{U}^n = \Delta t \left[ -\nabla \cdot (\mathbf{uU}) + \nabla \cdot \tau + \rho \mathbf{g} \right]^n - \frac{\Delta t^2}{2} \mathbf{u} \cdot \nabla \left[ -\nabla \cdot (\mathbf{uU}) + \nabla \cdot \tau + \rho \mathbf{g} \right]^n \]

\[ \mathbf{U}^{n+1} - \mathbf{U}^* = -\Delta t \nabla p^{n+\theta_2} + \frac{\Delta t}{2} \mathbf{u} \cdot \nabla (\nabla p^n) \quad (8) \]

The velocity is updated using the equations above. First the predicted velocity is calculated, then the pressure in the next time step is calculated, and finally the velocity is corrected. The equation to update the pressure is shown below:

\[ \Delta \rho = \frac{1}{c^2} \Delta p = -\Delta t \nabla \cdot \mathbf{U}^{n+\theta_1} = -\Delta t \left[ \nabla \cdot \mathbf{U}^n + \theta_1 \nabla \cdot \Delta \mathbf{U}^* - \Delta t \theta_1 \nabla \cdot \nabla (p^n + \theta_2 \Delta p) \right] \quad (9) \]

For the spatial discretization, Finite Element Method is adopted, using triangular element.
A regularized formula [3] of Bingham fluid is used to describe the behaviour of the material in the transport chamber. The formula is written as:

\[
\tau = 2 \left\{ \mu + \frac{\tau_y \left[ 1 - \exp\left(-n\Pi_{\dot{\gamma}}^{1/2}\right) \right]}{\Pi_{\dot{\gamma}}^{1/2}} \right\} \dot{\gamma} \tag{10}
\]

where \(\Pi_{\dot{\gamma}}\) is the second deviatoric invariant of strain rate \(\dot{\gamma}\), \(\tau_y\) is the yield stress, \(\mu\) is the viscosity of the yielded material and \(n\) is the regularization parameter. Figure 1 shows the shear stress-shear velocity relation for different values of the regularization parameter \(n\).

**Figure 1:** Regularized Bingham model: Influence of the regularization parameter \(n\)

### 3 COUPLED NUMERICAL SIMULATION OF THE EXCAVATION AND TRANSPORT PROCESSES

#### 3.1 Excavation Analysis

A simplified simulation model for cutting and transport of the soil material in mechanized tunneling is used to demonstrate the coupled excavation-transport-approach. As the main advantage of the Discrete Element Method lies in the easy capturing of fracturing, disintegration and redistribution of material, it is applied right at the tunnel face in front of the cutting wheel. A sand-like material with low cohesion is
assumed for the soil. Furthermore, in the 2D analysis it is assumed, that the supporting pressure is not re-generated after excavation, an assumption, which is related to a situation, where loss of face stability is encountered in reality.

A snapshot of the simulation model is shown in Figure 2 to illustrate the chosen two dimensional geometry and the configuration in four stages of the excavation process. Due to the large calculation time typically involved in DEM analyses, only a small part of the cutting wheel is investigated here. The total height of the domain is 0.5 m. The red particles on the right hand side symbolize the ground in front of the TBM, the soil that will be cut in the next step. On the other hand, the blue particles on the left represent the suspension that supports and stabilizes the ground. The triangular shape at the top and in between those two materials is the cutting tool. Once the simulation starts the tool will move downwards and cut through the ground with a velocity of 5 m/s. Hence, the total movement from top to bottom requires 0.1 s. Simultaneously, the tool is moved in horizontal direction (i.e. into the soil) with a velocity of 0.1 m/s. After the tool has reached the bottom, it moves again back to the top position. The separated ground particles will then start to move to the left and mix with the suspension and move through the cutting wheel (which is not included in this simulation) into the excavation chamber of the TBM. This process is described
in subsection 3.2.

The generated output of the DEM simulation is the mass flow of ground particles into the excavation chamber, see Figure 3. For a better overview, the height is split into 10 segments each of one-tenth of the overall height. The mass flow through each of these segments $h_1$ (the lowest) to $h_{10}$ (the highest) are shown over the distance the tool moves in horizontal direction in Figure 3. For each section, the mass is calculated by adding up all ground particles moving into the suspension area. It becomes obvious that the mass flow first starts at the top and then, step by step, proceeds to the bottom as the cutting tool moves.

### 3.2 Transport process

The transport model is shown in Figure 4(a). The mass flow output data from the DEM simulation of the excavation process are applied as inflow boundary conditions for the transport process. Consequently, the prescribed velocity at the chamber entry is a function of time and position. Zero pressure is prescribed at the exit of the chamber, representing, in a simplified manner, the situation at the screw conveyer. The parameters for the calculation are chosen as follows: density $\rho = 8000\, \text{kg/m}^3$ (same in DEM and FEM), viscosity $\mu = 10000\, \text{Pa} \cdot \text{s}$, yield stress $\tau_y = 15000\, \text{Pa}$ and
Figure 4: Flow simulation in the excavation chamber: a) Geometry, boundary conditions and pressure field at $t = 0.19\,\text{s}$ and b) face pressure distribution at $t = 0.19\,\text{s}$
the regularization parameter $n$ is adopted as $n = 20$. The transport simulation provides the pressure and velocity field in the chamber as well as on the chamber face at any time. The pressure field in the excavation chamber and the pressure distribution along the height of the pressure chamber at $t = 0.19$ s are shown in Figure 4. At this time instant, the tool has completely almost a full cycle and is located near the top position. A highly nonlinear pressure distribution is obtained, with a maximum pressure at the top of the chamber face and a minimum near the bottom (app. at a level of 0.06 m).

Figure 5 shows the time evolution of the velocity (provided as an input from the excavation analysis) and of the pressure (provided by the flow analysis) at ten points distributed equally along the height of the chamber face for a fully excavation cycle. Note, that after 0.1 s the tool has reached the bottom position and after 0.2 s, the tool has again reached the top position. The plots in Figure 5(a) correspond to the plots shown in Figure 3, however, the velocity is plotted here in logarithmic scale because the velocity of the lower sections is very small compared with the ones of the higher sections.

According to Figure 5a, during the downward movement of the tool, mass flow is generated as soon as the tool reaches the position of the respective level (from 1 to 10). In contrast to the real situation encountered in EPB shields, in the presented simplified analysis, it is assumed, that the supporting medium is not immediately regenerated after the excavation. In other words, after cutting, the soil particles loose their stable configuration and continue to more or less continuously moving into the chamber, which in fact represents a situation associated with the loss of face stability. The continuous influx of the loosened material results in a continuous increase of the pressure in Figure 5b at all 10 levels. This increase continues also after the tool changes its direction, since total mass inflow continues increases (Figure 5a). After reaching a peak value at $t \approx 0.17$s the velocity and the pressure decrease in all 10 levels considered in this figure.

4 CONCLUSIONS

A prototype modeling framework for the coupled modeling of excavation processes at the tunnel face and the subsequent transport processes of the foam-soil mixture within the pressure chamber of EPB shield machines has been presented. The Discrete Element Method has been adopted for the modeling of excavation while a stabilized finite element model for Non-Newtonian fluid flow was developed for the
Figure 5: Temporal evolution of velocity and pressure at 10 equidistant points along the chamber face for a full excavation cycle.
modeling of fluid transport. The presented prototype analysis is restricted to a 2D representation of the tunnel face. Although the current coupled excavation-transport model helps to get some insight into the coupling of two processes simulated by different numerical methods, it is evidently considerably simplified. Using the mass flow according to the excavation model as the input for a model of the flow of the viscous slurry into the pressure chamber, a highly nonlinear spatial distribution of the fluid pressure along the height of the chamber face has been obtained. It should be noted, that the parameters used in the material models for the soil (related to the inter-particle laws in the DEM model) and for the soil-foam mixture have been assumed and not yet validated by experiments. In particular, in the present analysis, a loss of support pressure is assumed after the tool has passed. In association with a assuming soil with little cohesion, loss of face stability is implicitly assumed in the analysis. Despite the simplifications, the presented prototype analyses has shown, that, in principle, the proposed hybrid excavation-transport approach allows for an appropriate coupling of the two major mechanisms interacting at the tunnel face.

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TBM-Ground Interactions
Advances in the Modelling of Excavation and Cutting Tool Wear with the Particle Finite Element Method

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Abstract

This work presents the new advances on the Particle Finite Element Method (PFEM) for the modeling of excavation processes and the wear of the cutting tools. In the PFEM [3, 9, 6], the continuum solid domains are defined by set of particles. These particles are used for the definition of the mesh that discretizes the domain. The most relevant phenomenon to model during a cutting process is the continuous change of the boundary definition of the geomaterial. The characterization of geomaterial boundary is performed by the particle kinematics and established when the particles set is reconnected with a new mesh. For the kinematics description the theory of the continuum mechanics is used. The problem is highly non-linear. Involving large displacements and deformations, constitutive laws for geomaterials and contact mechanics. The contact characterization is crucial. The surface wear and the friction laws will determine the removal of the worn and excavated material.
A lagrangian description of the motion is used to model large material deformations and rapidly changing boundaries. The cut of a geomaterial by a cutting tool is modelled by the physical interaction of two solid domains. An implicit solution scheme is used. Remeshing strategies are employed in order to identify the boundary surfaces and the contact interface. The contact between solid domains is modelled using the contact domain method [7]. The characterization of the contact captures the normal and frictional forces. These forces are the main mechanical input for an Archard-type law [1]. That allows to quantify the excavation and damage caused on the surface of a ground due a mechanical abrasion. The erosion and wear parameters of the grounds under study are needed to effectively model the excavation process. New advances and improvements on the numerical techniques involved in the method have been developed and will be presented in the paper. The results to be presented show how the method can model 2D and 3D ground excavation problems in a simple and efficient way.

Keywords: Particle methods, Contact domain, Excavation, Cutting, Wear.

1 INTRODUCTION

The approach to model excavation and wear presented in this paper is based in the Particle Finite Element Method (PFEM) [3, 6]. The good capabilities of the method for describing free surfaces and changing boundaries was the motivation to apply the method to solid mechanics. In this work we have selected the convenient processes of the original PFEM and we have extended them for the modelling of wear and excavation. The main characteristics of the PFEM were originally developed in the field of fluid dynamics. The first application of the PFEM in solid mechanics can be found in [5].

2 PARTICLE DESCRIPTION

The PFEM uses all the previous background of the standard Finite Element Method (FEM) and introduces new tools to increase the geometrical adaptability to the model. This is done via automatic remeshing. The description of the continuum is based on particles, which usually correspond to the nodes of the finite element mesh. From a cloud of particles and by means of a Delaunay tessellation [4] a mesh can be generated in the complete convex hull. The Alpha-shapes assigned to the particle description are used to recover the solid contour and for the geometrical detection of the contact between multiple domains.
3 CONTACT DETECTION

The geometrical detection of contact is a complex problem in computational mechanics. The geometric search is especially complex when the contact of more than two bodies has to be considered. That can happen when the problem is such that the solids can break, and hence during the solution process, several discrete elements are originated from the initial set up. This is common in excavation problems. The search for an active set of contact constraints is not trivial in this case.

In the PFEM the contact detection is easily performed by means of a remeshing process. From the particle description the proper Alpha-shape is used for the mesh creation. An interface mesh is generated between the multiple domains when they are coming into contact. That is shown in Figure 1.

Figure 1: Geometric contact detection in the PFEM.

The interface mesh generated during the re-mesh stage defines the contact domain. It requires a previous shrinkage of the domains to give consistency to the mesh generation. The interface mesh is used for the contact detection and also to define the contact constraint. The Contact Domain Method, used to compute the contact forces, is founded on the potential of a fictitious continuum domain that works between the contacting bodies. When the constraint is active on the body surfaces, the constraint is computed. The details of the method based on Lagrange Multipliers can be found in [7, 8].
4 WEAR AND EXCAVATION MODELS

After computing contact forces a volume loss rate can be applied to quantify the wear produced in the contact surfaces. The geometry of the boundary is readapted concurrently with the changes of these contact surfaces. Projection algorithms and refining procedures are applied in the excavation zones to represent the large geometry changes of the solid boundaries. The volume loss rate is also applied in excavation. The treatment of the boundary is crucial to model the rapid changes that excavation produces on the geometry. The ground breaks under the action of the cutting tools during the tunneling process. Concerning on tools, wear plays an important role during the excavation work.

Wear is predicted by computing the material that is dug due to the contact interaction. The surface properties of the interacting materials control the wear occurring during the frictional contact (see Figure 2). When a steady state position in the wear mechanisms is reached the wear rate is described by a linear Archard-type equation [1] as:

$$ V_{wear} = k_{abr} \frac{F_N g_T}{H} $$

where $F_N$ are the contact normal forces, $g_T := x$ is the relative sliding distance and $k_{abr}$ is the abrasive wear coefficient, that physically represents the average tangent of the roughness angle divided by $\pi$.

**Figure 2:** A simplified abrasive wear model showing how a cone removes material from a surface.
5 EXAMPLES

Several examples for the normal and frictional contact calibration have been made. Figure 3 shows a numerical test for friction characterization between a cube and a foundation.

The method is applied to model tunneling processes. A large number of 2D and 3D excavation and rock cutting models have been computed. They are presented in order to show the efficiency of the method for modeling wear in these types of problems. Figure 4 shows an example for a TBM Disc model. The impacting forces are depicted with the resulting accelerations on the disc and in the geomaterial. Wear is properly quantified in contact zones.

Figure 3: Numerical calibration of normal and frictional contact.
Figure 4: Wear on a TBM head and on one of its cutting discs.

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Abstract

Every drive with a shield machine provides the user with a large amount of machine data, which may be used to analyse the interaction between the ground and the TBM. A special focus is on the data, which is connected to the movement of the cutting wheel and the shield. Variations of these data are often interpreted as subsoil changes or as a result of adverse effects on the cutting wheel (e.g. clogging or tool wear). However, these data also depend significantly on technical and human factors, which are not connected to the excavation process. Data analyses without extensive revision of these effects therefore may cause misinterpretations. In this first part of the paper, the various influences, particularly from face support, on the machine raw data are specified. As a result a methodology will be presented to extract relevant parts from the raw-data. The importance of the data revision is shown in practical examples, which could have led to false interpretations when analysing the soil-machine-interaction with unrevised machine raw-data. For future projects, the extraction of the excavation-specific data components will allow analyses with improved relevance.
1 INTRODUCTION

Data collected during shield tunneling are frequently used for drawing conclusions about the interaction between machine and the geologic media. Changes in recorded machine raw-data are often attributed to changes in ground conditions. However, the machine data are influenced by numerous factors such as the state of the cutting wheel, the steering of shield or errors / tolerances in measurements. This first part of the paper deals exclusively with quality and processing of the machine raw-data. The status of the documentation of and at the site as well as the effects of certain ground conditions on machine behavior/data is not discussed. Possible sources for errors in raw-data as well as remedial measures are given. Thus, it is possible to define excavation-specific components of machine data which are assumed to be independent of technical or human influence. In this paper the analysis is limited on thrust force on cutting wheel, penetration and total thrust force. It has to be considered, that such processing of raw-data may be necessary for other parameters as well. The analysis of the excavation-specific components itself will be presented in part 2 [1] of the paper.

2 CURRENT PRACTICE OF MACHINE DATA ANALYSES

Some raw-data are significant for evaluating the interaction between machine and ground conditions and refer to either the shield progress or face [2]. Sudden changes in the aforementioned data may reflect a change in ground conditions as the data directly or indirectly depend on those conditions. However, those data must be separated in active and passive parameters:

- An **active parameter** is given by the operator and is thus a pre-determined value. Active parameters include cutting wheel rpm as well as pressure of thrust cylinders which pushes the TBM forward. Additional active parameters are face support pressure and slurry flow rate with the use of mix-shields or the rotation speed of the screw conveyer with EPB shields.

- **Passive parameters** (thrust force on cutting wheel, torque on cutting wheel, advance speed and rate of penetration) are the result of active parameters. The determination of values for active parameters aims for a specific advance speed / penetration under given site conditions. The cutting wheel torque and thrust are, however, not target rather than limiting values for active parameters.
Immediate observation of instant values is mandatory during operations. This, however, cannot be achieved with the use of standard spreadsheets making specialized tunneling information systems necessary [2]. The data of the active parameters are already displayed in real-time in the operator’s cabin [3]. Typically, it is the operator who has continuous access to and supervises these parameters. Additionally, some data are automatically compared to limits which are specified before shield drive [4]. During operations those data are of particular interest which are important for safe tunneling with little settlements (i.e. face support pressure, delivery, densities and pressure for backfill grouting). Of similar importance may be a sudden increase in cutting wheel torque and thrust force while the penetration remains the same or is decreasing. This observation may hint at the necessity of either a cutting wheel inspection with respect to tool wear and clogging or a documentation of changed ground conditions.

3 MEASUREMENT AND STORAGE OF DATA

Today’s TBMs integrated automatic data logging are based on complicated measurement systems. Hundreds of sensors are placed on various locations along the tunnelling system and continuously transmit analogue electrical signals (typically Voltage in mV). The signals are processed by AD converters into physical units such as pressure. Each sensor features a specific range of application (e.g. pressure sensors with a range of 0 – 100 bars) as specified by the manufacturer. Outside the specified range the converted values may be erroneous. With significant overloading, even for a very brief time, above the sensor’s design range the device may be damaged or destroyed. In any case, electrical zero or the linearity of the sensor may experience an offset leading to a need for a recalibration of the sensor (which in many cases is never done). Thus, the use of under-dimensioned sensors may lead to erroneous measurements which cannot be compensated later. Over-dimensioned sensors negatively affect the accuracy of a measurement as a sensor features error margins as a function of its dimension (e.g. ± 0.5 % FS (of the full scale)). In addition, the resolution of the measurements is coarser by using only a small part of the allowable range.

The calibration (the correct reflection of the actual physical status) of the sensors is typically done ahead of tunnelling operations. Control and re-calibration during operations are difficult, because of the large number of sensors. This may lead to unnoticed zero-shifts and changes in linearity of the sensors. Comparable sensors in
laboratory practice need to be checked and re-calibrated on a regular basis because zero shift and changes in linearity occur even without exceeding the sensor’s limit (manufactures specify long-term accuracy of sensors in the range of 0.1% FS per year). Compared to the controlled laboratory conditions, the sensors on a construction site are exposed to very adverse conditions. They include large variations in temperature and humidity, vibrations and damages by force as well an extensive cable length. The sensors endure these conditions over a period of several months which may have negatively affects the sensor’s accuracy. The reliability of the recorded machine data suffers and re-calibration of the sensors is necessary. Sensors adapted to those conditions are necessary to install.

4 EXCAVATION-SPECIFIC DATA COMPONENTS

Ground conditions as well as the current state of the cutting wheel influence the machine raw-data and their change by trend. The actual effect of these two factors may only be assessed by additional investigations. We introduce the excavation-specific component, combining both (ground conditions and state of the cutting wheel) as the respective effects on the raw-data in many cases cannot be clearly distinguished. Additional influences on raw-data stem from operators skills, face support pressure and friction forces. These effects are independent of excavation processes. The raw-data in the database therefore consist of excavation-specific and excavation-independent data components as specified in table 1.

Table 1: Excavation-specific and excavation-independent components of machine raw-data

<table>
<thead>
<tr>
<th>Machine raw-data</th>
<th>Excavation-specific data components</th>
<th>Excavation-independent data components</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Influence from state of cutting wheel</td>
<td>Influence from resistance to excavation</td>
</tr>
<tr>
<td></td>
<td>Technical components</td>
<td>Influence from state of cutting wheel</td>
</tr>
<tr>
<td></td>
<td>Human components</td>
<td>Influence from state of cutting wheel</td>
</tr>
<tr>
<td></td>
<td>Design of cutting wheel (diameter, grade of aperture, type of tools)</td>
<td>Influence from state of cutting wheel</td>
</tr>
<tr>
<td></td>
<td>Actual state of tool wear and clogging</td>
<td>Influence from face support</td>
</tr>
<tr>
<td></td>
<td>Actual state of tool wear and clogging</td>
<td>Influence from friction forces</td>
</tr>
<tr>
<td></td>
<td>Operator’s skills (adjustment of active parameters)</td>
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</table>
It is obvious that machine raw-data has to be partially revised to allow for meaningful interpretations of interactions between TBM and ground conditions. Therefore the excavation-specific data components (see table 1) have to be identified.

4.1 Thrust Force on Cutting Wheel

For shield tunneling without face support, the parameter thrust force on the cutting wheel \(F_{Th}\) represents the sum of friction forces \((F_C + F_B)\) and tool forces \((F_T)\); as the excavation-specific component. This parameter \((F_{Th})\) is also used for analyzing ground conditions when applying face support. The raw-data \((F_{Th})\) are calculated from the pressure and the cross-section of the rams. Three groups of cylinders, each equipped with 2 or more individual cylinders, allow with many TBMs controlled displacement and tilting of the cutting wheel. They also transmit the forces from the cutting wheel and cutting wheel drive to the shield.

With face support, the pressurized chamber is separated from the area with atmospheric condition by the submerged wall. For all shield types with face support, the face support pressure acts not only on the face itself but also on the area of the cutting wheel drive. This force acts opposite to the drive direction and has to be compensated for by the cutting wheel displacement cylinders.

![Figure 1: Schematic presentation of forces acting on cutting wheel of a hydro-shield machine: \(F_T\)-tool forces; \(F_S\)- resulting force of face support pressure acting on area of cutting wheel drive; \(F_B\)- internal friction forces of main bearing; \(F_C\)- internal friction force of hydraulic cylinders; \(F_{Th}\)- thrust force on cutting wheel raw-data.](image)
In order to evaluate the excavation-specific data component (actual tool force \( F_T \)), the force of the slurry on the area of the cutting wheel drive \( F_S \) has to be considered. Even at low support pressures, evaluations show a significant influence of face support force on raw-data. The resulting face support force increases nonlinear with the diameter of machine, as it is presented in figure 2.

![Figure 2: Resulting force from face support pressure at different pressures, depending on cross section diameter (size of effective area)](image)

Calculations of face support forces have been carried out for three projects and the findings are presented in figure 3. The column charts show average values for whole project distance. Raw-data of thrust force on cutting wheel represent 100 % and calculated forces from face support pressure are given as a percentage of these raw-data (friction forces are not considered).

![Figure 3: Face support force portions on total thrust force raw-data (friction force are not considered) for different hydro-shield projects under various geological conditions, different machine sizes and with different face support pressures; major influence is obvious in every project](image)
It was found, that in any case of face support, independent of machine type, geology or used pressures, the influence of face support force on raw-data is very significant. Similar results were found by Festa et al. [5]. Additionally the results given in figure 3 show a trend to lower portions of face support force with increasing excavation resistance of the ground.

Additional forces to be taken into account are various frictional forces. These forces include internal friction of guidance within the main bearing of the cutting wheel drive and friction of the individual thrust cylinders for cutting wheel displacement. It is neither possible nor useful to distinguish between single frictional forces. However, they amount to significant magnitudes as may be seen when closely analyzing instantaneous data of cutting wheel displacement without face contact. Several bar of oil pressure are necessary to move any pistons ($F_C$) as well as the cutting wheel drive ($F_B$). This pressure is termed oil pressure for idle. The “idle pressure” has to be converted to constant frictional force and this force has to be included in the analysis of raw-data. The (theoretically) constant friction force has to be added to the raw-data because parts of tool and face support forces are compensated by those friction forces. As discussed above the computation of the tool force may be summarized in the following formula (1):

$$ F_T = F_{Th} - F_S + F_C + F_B $$

- $F_T$ = tool force [kN]
- $F_{Th}$ = thrust force on cutting wheel raw-data [kN]
- $F_S$ = resulting force of face support [kN]
- $F_C$ = internal friction force of hydraulic cylinders [kN]
- $F_B$ = internal friction forces of guidance within main bearing [kN]

By processing the data of different projects no consistent values for friction forces, but variations of more than 100 % (within each project) were found. A clear reason for this remains ambiguous, so that for further estimations values had to be assumed.
The analyses of the excavation process using the penetration raw-data it is not meaningful without regarding other parameters, because penetration is directly influenced by the active parameters (see section 2). A change of penetration magnitude during excavation hints first of all only a change of active parameters by the operator.

To receive an excavation-specific component in this case, not the technical but the human influence has to be identified. For that reason, scaling of penetration to the tool force is suggested and the obtained parameter is called specific penetration [6].

\[ Pen_{spec} = \frac{Pen_{raw}}{F_T} \]  \hspace{1cm} (2)

\( Pen_{spec} \) = specific penetration [mm/U/kN]
\( Pen_{raw} \) = penetration raw-data [mm/U]
\( F_T \) = tool force [kN]

The relationship between specific penetration and rock mass conditions (UCS, distance of discontinuities) is well known for TBM projects in hard rock conditions, and can be used even for advance rate estimations [7]. Therefore the use of specific penetration values for analyzing shield driven tunnel projects is not a reinvention.

Processing of penetration raw-data is not a splitting of different components but a combination of an active and a passive parameter. Related to evaluations of three different projects the findings show that it may be much easier to use specific penetration as one combined parameter, as to analyze two or more parameters separately (e.g. penetration, tool force and slurry density).
4.3 Total Thrust Force

Total thrust force comprises many different components and forces within the cutterhead and shield of a TBM [3], [8]. Different forces are displayed in figure 4 and have to form an equilibrium. Processing of total thrust force aims for the calculation of shield friction force as the excavation-specific component. Hints for a theoretical calculation are given in Herzog [9]. An additional resistance to the cutting edge of the shield is neglected due to the larger diameter of the excavation than the diameter of the shield [3].

![Figure 4: Schematic presentation of forces acting on shield and cutting wheel of a hydro-shield machine: $F_{Th\ total}$ total thrust force raw-data; $F_S$ resulting force of face support pressure acting on TBM cross section; $F_T$ tool forces; $F_{Sh}$ shield friction force, resulting from earth pressure and bedding stresses; $F_C$ internal friction force of thrust cylinders; $F_B$ pulling force of backup-equipment.]

As shown in figure 4 it is obvious, that for processing of $F_{Th\ total}$ many different components have to be considered. First of all, resulting force of face support pressure ($F_S$) and friction forces ($F_C$) are crucial. In this case, face support force acts on the total cross sectional area of TBM. Due to that fact, resulting forces ($F_S$) have a large influence on total thrust force raw-data. The friction force of the thrust cylinder have to be taken into account, too. Moreover, the pulling force for the backup equipment ($F_B$), the tool forces ($F_T$) and the shield friction force ($F_{Sh}$) have to be subtracted from raw-data.

The magnitude of the tool force was discussed in section 4.1. There are other forces such as friction between brush seal and segments which should be included, their magnitude is however considered negligible.
Theoretically, the shield friction force may be calculated as follows:

\[ F_{Sh} = F_{Th\ total} - F_S - F_C - F_T - F_P \]  

\[ F_{Sh} = \text{shield friction force [kN]} \]
\[ F_{Th\ total} = \text{total thrust force raw-data [kN]} \]
\[ F_S = \text{resulting force of face support pressure [kN]} \]
\[ F_C = \text{internal friction force of thrust cylinders [kN]} \]
\[ F_T = \text{tool force [kN]} \]
\[ F_P = \text{pulling force of backup-equipment [kN]} \]

Calculations have been carried out for one project and the findings are presented in figure 5. The regularity of support force portion on raw-data is shown in figure 5, left side. The black line represents calculated face support force percentage on raw-data. Raw-data of total thrust force is represented by 100%.

**Figure 5:** Left side: portion of resulting face support force (black line) on total thrust force raw-data (100%); right side: calculated average values for each component

The deviation between face support force and raw-data comprises tool force, pulling force of backup-equipment, frictional forces of thrust cylinders and shield friction force. In this case the calculated average amount of these different components represents only approx. 24 % of the raw-data, which can be seen in figure 5, right side.
5 CONCLUSION AND SUGGESTIONS

In this paper (part 1 of 2), options and limits of processing and analyzing of machine data were demonstrated. Thrust forces on cutting wheel, penetration and total thrust force were regarded. It was shown that the relevance of automatically generated raw-data without further processing is very limited for interpretations of soil conditions. Interpretation of automatically generated raw-data with respect to ground conditions may be particularly misleading.

Raw-data may be split into different components. Excavation-specific components (influences of soil conditions and cutting wheel state) in many cases show only little portions on raw-data, whereas excavation-independent components like face support force or friction force show major influence. Thus, it is recommended to separate excavation-specific components as accurately as possible. Although theoretical approaches are given, subsequent evaluation of feasible values is difficult, because of very alternating conditions in the course of shield driven tunnel projects.

Measuring errors may occur as a result of strong vibrations, temperature and moisture deviations in addition to large cable length, influencing complicated measurement setups. Frictional forces may change in course of construction as a result of wear effects and entry of fines (mud, dust, clogging). Thus, the significance even of processed data is influenced negatively and subsequent estimations or even corrections in most cases are hardly to realize.

Despite this persistent inaccuracy, processing of raw-data is very useful, because soil conditions will show larger influence on excavation-specific components as on raw-data. Variations of machine data may be shortly detected as a result of technical or excavation-specific reasons on the one hand. On the other hand, little variations of excavation-specific components will not lead to any variation of raw-data. Continuous observation of the excavation-specific components may form the chance to recognize critical influences (e.g. clogging) much earlier.

For further projects the following suggestions are offered in the following table 2:
### Table 2: Suggestions for machine data interpretations of future projects

**before start of excavation**

- Calibration and check of all sensors measuring excavation relevant parameters (linearity and zero shift for AD conversion) *
- Determination** of internal friction forces within the guidance of main bearing of cutting wheel and hydraulic cylinders by moving the cutting wheel forward and backward
- Determination** of internal friction forces within the thrust cylinders by moving each single cylinder group forward and backward (determination of idle pressures)
- Visualization of excavation-specific components in operator’s cabin as an additional tool for excavation survey

**after start of excavation**

- Regular interval checks of all sensors measuring excavation relevant parameters and recalibration if needed (linearity and zero shift for AD conversion)
- Regular interval checks** of determined values for internal friction forces within the guidance of main bearing of cutting wheel and hydraulic cylinders by moving the cutting wheel forward and backward
- Regular interval checks ** of determined values for internal friction forces within the thrust cylinders by moving each single cylinder group forward and backward (idle pressures)

* Adequate dimension of sensors has to be considered in course of TBM development
** Logging frequency should be raised for determination and check of internal friction forces
It is strongly recommended to use excavation-specific data components for all kinds of analysis, focusing on the interaction of machine data and ground conditions. After identification of technical and human influences it has to be regarded, that excavation-specific components are influenced by ground conditions and state of cutting wheel as well. To compare different projects, cutting wheel design (diameter, degree of aperture, type of tools) has to be taken into account. Influences of tool wear and clogging always have to be regarded within every project. For an exact determination of those influences, a continuous documentation at project site is necessary.

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Abstract

Analyses of Soil-Machine-Interactions require the determination of excavation-specific components from the original TBM-machine data in order to eliminate human or technical effects. It has to be taken into account that the excavation-specific components depend not only on the subsoil but also on the design as well as the actual state of the cutting wheel. Therefore, the informative value of the soil-machine-interaction-analysis depends not only on the revision of the machine data but also on the quality of the subsoil description and the information of the site conditions. Data analyses without sufficient project information may cause misinterpretations. Therefore different influences on the excavation-specific components are specified. This paper will present practical examples of the soil-machine-interaction analyses by using the excavation-specific components of the machine data. Possible misinterpretations, caused by insufficient site data, are also shown. Thereby influences of the subsoil as well as the current state of the cutting wheel are demonstrated. As a result, recommendations for considering various excavation-specific influences are given. For future analyses, quality specifications regarding the basic site data are developed in order to help to improve the value of TBM data analyses.
1 INTRODUCTION

The need of processing TBM-machine data for analyses of soil-machine-interactions has been shown in Part 1 of the paper [1]. Despite of the described revision of technical and human components from the raw-data, changes in the excavation-specific data components are not necessarily connected to changes of the subsoil. While part 1 of the paper described the influences of measurement, support pressure, friction forces and operator skills on the raw-data, part 2 deals with the influences caused by the state of the cutting wheel and the ground conditions itself. Thus the paper is limited to the excavation-specific data components cutting wheel contact force at the tunnel face (in the following: contact force CW) and specific penetration.

2 INFLUENCES ON EXCAVATION-SPECIFIC DATA

The excavation-specific data components of the TBM-data, as a result of the described processing, are influenced by several geological and further technical factors. They mainly depend on the efficiency of excavation during tunnelling. On one hand this depends on the excavation resistance of the subsoil itself and on the other hand on the excavation efficiency of the cutting wheel and therefore on its state. Both influences require information from the construction site.

2.1 Design and State of Cutting Wheel

Analyzing the TBM-data of different projects with regard to the ground conditions one has to take the state of the cutting wheels into account. The cutting wheel design differs greatly concerning the diameter as well as the opening ratio (Fig. 1).

Figure 1: Left: single-rail metro-tunnel (diameter : 7,35 m; opening ratio: 85%). Right: double-rail highspeed-railway-tunnel (diameter : 13m; opening ratio: 30%).
Even if the two TBMs, shown in Figure 1, would have been driven in the same ground conditions, the contact force CW and the specific penetration must be different. With increasing diameter and normally higher number of tools the overall tool force and therefore the contact force CW would be higher. Meaningful quantifications of such differences in contact force CW and specific penetration only hold then true in practice, if both TBMs would have been driven in the same subsoil. Thus, comparisons should be avoided, or at least must be taken very carefully.

In addition, the contact force CW will be influenced by the type of tools, which can change even in the same project (Fig. 2).

Figure 2: Left: Cutting wheel with assembly of drag picks and disk. Right: Same cutting wheel with assembly of drag picks and rippers.

Because of different excavation of varying tool types (drag picks peel, rippers rip, disks cut) the used tool assembly influences the excavation efficiency of the cutting wheel and therefore the excavation-specific components of the TBM-data. While design and tool assembly of the cutting wheel are constant or at least well known, the tool-wear changes more or less rapidly during one project (Fig. 3). Sometimes even damages of the cutting wheel itself can happen.
Like a blunt knife – worn tools hamper the excavation process and therefore lead to increasing contact forces CW, normally connected with decreasing rates of the specific penetration. So the part of the data caused by the wear, will immediately change after tool change. The effect can be shown for a hydro shield, driven in sandy gravels, comparing the excavation-specific data before and after tool changes (Fig. 4). As with worn tools the excavation process can be hampered by cloggings at the front side of the cutting wheel (Fig. 5).

Figure 3: Left: Assembly of a new disc. Right: Worn disc up to the core of the bearing.

Figure 4: Increase contact force CW and decrease of specific penetration with increasing tool wear, starting immediately after tool changes. The highest ratio of the excavation-specific components, caused by the wear (white arrows) occurs before the next tool change.
Analysis of Soil–Machine–Interactions (Part 2)

Figure 5: Clogging material at the front side of the cutting wheel, hampering the penetration of the tools (left white arrow: ripper; right black arrows: drag picks).

The sticky material accumulated between tunnel face and cutting wheel inhibits the penetration of the tools. During hydro shield tunnelling with a totally revised cutting wheel, cloggings occurred, which caused an increase of the contact force CW and a decrease of the specific penetration (Fig. 6). The change of the excavation-specific data can be shown before and after cleaning works. In this case one must keep in mind, that the excavation-specific components will be influenced not only by the clogging but also more or less by wear effects. Analyzing the excavation-specific components of the TBM-data requires as much information about the state of the cutting wheel as possible. Especially influences like tool wear and clogging must be documented during tunnelling.
Figure 6: Increase of contact force CW and decrease of specific penetration with increasing cloggings. The highest part of the excavation-specific components, caused by the cloggings (white arrows) occurs before the cleaning works.

2.2 Ground Conditions

2.2.1 Direct influence: Excavation resistance of soils

The excavation resistance dominantly depends on the strength of the soil, which is more important than the soil type itself. Regularly the excavation resistance should increase with increasing consistencies in fine-grained and increasing degree of packing density in medium- to coarse-grained soils. It should be comparable to the standard penetration resistance, which correlates to the consistency [2] and the packing density [3]. Neglecting clogging and abrasion effects, the contact force CW must increase from the very soft respectively very loose soil in the row up to its equivalent cemented weak rock (Fig. 7).

<table>
<thead>
<tr>
<th>soil</th>
<th>fine-grained soil (clay and silt)</th>
<th>weak rock</th>
</tr>
</thead>
<tbody>
<tr>
<td>consistency</td>
<td>very soft</td>
<td>claystone</td>
</tr>
<tr>
<td></td>
<td>soft-medium</td>
<td>siltstone</td>
</tr>
<tr>
<td>SPT-value [N]</td>
<td>&lt; 4</td>
<td></td>
</tr>
<tr>
<td></td>
<td>2 - 8</td>
<td></td>
</tr>
<tr>
<td></td>
<td>8 - 15</td>
<td></td>
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<tr>
<td></td>
<td>15 - 30</td>
<td></td>
</tr>
<tr>
<td></td>
<td>&gt; 30</td>
<td></td>
</tr>
</tbody>
</table>

increase of excavation resistance (increase contact force CW, decrease specific penetration)

<table>
<thead>
<tr>
<th>soil</th>
<th>medium- to coarse-grained soil (sand and gravel)</th>
<th>weak rock</th>
</tr>
</thead>
<tbody>
<tr>
<td>packing density</td>
<td>very loose</td>
<td>sandstone</td>
</tr>
<tr>
<td></td>
<td>loose</td>
<td>conglomerate</td>
</tr>
<tr>
<td></td>
<td>compact</td>
<td></td>
</tr>
<tr>
<td></td>
<td>dense</td>
<td></td>
</tr>
<tr>
<td></td>
<td>very dense</td>
<td></td>
</tr>
<tr>
<td>SPT-value [N]</td>
<td>&lt; 4</td>
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</tr>
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<td></td>
<td>4 - 10</td>
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<td></td>
<td>10 - 30</td>
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<tr>
<td></td>
<td>30 - 50</td>
<td></td>
</tr>
<tr>
<td></td>
<td>&gt; 50</td>
<td></td>
</tr>
</tbody>
</table>

Figure 7: Tendency of excavation resistance for the main soil types (SPT-values taken from [2] and [3].
The shear strength of soils may also be used for analyzing as it typically correlates to packing density and consistency, respectively. Similarly, the contact force CW must increase and the specific penetration must decrease with increasing shear strength.

As an example, the data of a hydro shield, driven in sandy gravels with varying packing densities, were analyzed to show the influence on the contact force CW and on the specific penetration (Fig. 8). Information, were taken from the standard penetration tests during exploration and from the face mapping. Tunnel sections with different dominating packing densities can be compared to the excavation-specific components of the TBM-data.

**Figure 8:** Contact force CW and specific penetration in tunnel sections with different packing densities.

For the tunnel section with dominantly compact gravel contact forces CW from 2000 up to 2300 kN can be deduced (Fig. 8, left). In comparison values of 3000 up to 3500 kN occurred in dense gravel (Fig. 8 middle), increasing up to 5500 kN in very dense gravel (Fig. 8, right). In this case the high amount of tool wear has to be considered, so that the contact force CW with new tools must be even lower. In addition it has to be kept in mind that the effect of the different tool assembly will probably be superposed to the effect of the packing density. With increasing packing density, rippers work more and more ineffectively in gravel, because the ripping of the grains from the fabric gets more difficult. Thus, damages by force could occur if very high packing densities or boulders would be encountered. Therefore the ratio disks-rippers changed to more disks.
with increasing packing density (Fig. 8). Comparisons of the different gravel sections with neglecting its packing density would lead misinterpretations. One might argue that rippers lead generally to lower contact forces CW and higher specific penetrations and therefore are the most effective tool in all kind of gravel.

Evidently comparing different soil types without considering their strength properties is highly questionable. For example tunnelling in dense sands should generate similar contact forces CW like in dense gravels (analogue to the results of standard penetration tests). So driving the TBM from compact gravels into sands can generate increasing as well as decreasing contact forces CW, dependent from the packing density of the sand (e.g. very loose = decrease; very dense = increase).

The increase of the contact force CW or the decrease of the specific penetration must be the higher, the higher the contrast of the changing strength properties of the ground conditions will be. For example the influence on the excavation-specific components can be shown by a hydro shield leaving gravels, passing mixed face conditions and finally ending in a hard rock section (Fig. 9). Note that the tunnel section with mixed face conditions as well as the soils before was partly grouted.

**Figure 9:** Contact force CW and specific penetration in tunnel sections with changing distributions of gravels and hard rocks.

**cutting wheel:**
- diameter: 13 m
- opening ratio: 30%
- tool assembly: 268 drag picks and 64 disks/rippers (about 95% disks, 5% rippers)

**Subsoil:**
- Soil: sandy gravel
- Rock: limestone
- Mixed Face: sandy gravel, partly grouted
Similar changes of the tool force have been shown by [6] in hardrock conditions with different rock strength properties. Other, mixed face conditions with rocks within soils include coarse fragments, such as cobbles and boulders as well as partly cemented rocks like conglomerate within gravel. They will generate high impacts on the excavation-specific data and particularly increase the tool force.

2.2.2 Indirect Influence: Abrasiveness and Clogging Potential

Soil parameters which have influence on the abrasiveness or the clogging potential can have an indirect impact on the excavation-specific components, even if they would not directly affect the excavation resistance of the soil. The following soil parameters are defined with regard to the wear during shield tunnelling [4]: abrasiveness (LCPC-index), breakability (LCPC-index), equivalent quartz index, cobble proportion, boulder proportion, uniaxial compression, shear strength, packing density. In the same manner, the abrasiveness increases with increasing grain size and decreasing roundness of the grains [5]. Own experiences show that the major influence on abrasiveness is generated by the grade of packing density. Anyway, even parameters that have no high impact on the excavation resistance (cf. chapter 2.2.1), like the equivalent quartz index, may influence the excavation-specific components. A higher abrasiveness will generate increasing tool wear and therefore a decreasing excavation efficiency of the cutting wheel (cf. chapter 2.1).

With reference to Fig. 7 dense gravels with coarser, more abrasive grains might generate higher contact forces CW, due to the higher tool wear, than sands with comparable packing density. Influences on the excavation-specific parameters would be even higher, if parameters will increase the abrasiveness as well as the excavation resistance. Especially regarding to the packing density the double effect can be shown, comparing the contact force CW in compact to very dense gravels (Fig. 8, left and right diagram).

The following soil parameters are defined with regard to clogging [4]: plasticity index, consistency index, liquid limit, plastic limit, water content, mineralogy, percentage of clay. Even the parameters that have no high impact on the excavation resistance, like the clay mineralogy, will influence the excavation-specific components. A higher clogging potential will generate an increased amount of cloggings and therefore decreasing excavation efficiency of the cutting wheel (cf. chapter 2.1). With reference to Fig.7 soft clays with higher clogging potential might generate higher contact forces CW, due to the state of the cutting wheel, than even less critical hard clays. Often described changes of TBM-data while tunnelling from sands to clays in many cases are likely to have been caused by the indirect influence of cloggings occurring in the clays rather than by the excavation of the soil itself.
2.3 Obstacles in the Ground

During shield tunnelling, manmade obstacles can be encountered along the tunnelling profile. Typical obstacles like diaphragm walls or blocks normally are well known before tunnelling, because they often have been installed during construction phase. Other obstacles like old foundations, piles, relics of anchors or sheeting walls as well as lost drilling tubes often occur surprisingly. Because of the high strength contrast between soil and the embedded obstacle in most cases high impacts on the excavation specific components will be generated. The increase of the contact force CW as well as the decrease of the specific penetration can be shown for a hydro shield drive passing the diaphragm walls within loose to compact gravels (Fig. 10).

![Graph](image)

**Figure 10:** Increase of contact force CW and decrease of specific penetration while passing diaphragm walls.

Analyzing the excavation specific components regarding to the properties of the in-situ soils, tunnel sections with obstacles must be avoided, because the data will be distorted. Similar influences can be generated by grouting, because the soils will be cemented. This impact can be seen at the end of the soil section, which was grouted like the mixed face subsoil (Fig. 9). Obstacles, especially if they consist of metal, would increase the wear and therefore have additional influences on the excavation-specific data. This can generate increases of contact forces CW, even if the obstacle was already passed.
3 CONCLUSION

This paper describes many influences on the excavation-specific components of the TBM-data. The data depends on the ground conditions as well as the state of the cutting wheel. The main influences are summarized in Fig. 11, whereby in most cases superposition of the different effects will be realized in practice.

<table>
<thead>
<tr>
<th>excavation-specific influences</th>
<th>ground conditions</th>
<th>obstacles</th>
</tr>
</thead>
<tbody>
<tr>
<td>state of cutting wheel</td>
<td>well-known: diameter, opening ratio, tool assembly</td>
<td>excavation resistance: packing density, consistency, shear strength, proportion of rocks (mixed face, cobbles, boulders)</td>
</tr>
<tr>
<td></td>
<td>changing: grade of wear grade of clogging</td>
<td>abrasiveness: packing density, grain size distribution, equivalent quartz index, proportion of rocks (mixed face, cobbles, boulders), grain roundness, LCPC-index</td>
</tr>
<tr>
<td></td>
<td></td>
<td>clogging potential: plasticity index, consistency index, liquid limit, plastic limit, water content, clay mineralogy, percentage of clay</td>
</tr>
</tbody>
</table>

**Figure 11:** Main excavation-specific influences.

Comparing the data of two hydro shields (both driven in sands and in gravels), with similar penetration rates of 15 to 25 mm/U as the targeted active parameter, seems to be legitimate on the first view. But the extremely different excavation-specific data (Fig. 12), caused by varying cutting wheels and different soil properties indicate the low informative value of such simple analyses.

**Figure 12:** Contact force CW and specific penetration of two hydro shields in sands and gravels.
Even if the TBM-diameters will be considered by scaling the contact force CW to a unit area, the differences between the specific contact forces per unit area of the cutting wheel, caused by different properties of the gravel are still very high (Fig. 13).

**Figure 13:** Contact forces per unit area of the cutting wheel two hydro shields.

As an alternative to the specific contact force per unit area of the cutting wheel, one could argue to calculate the specific contact force per cutting tool. However this again will include a large uncertainty due to the different types, shapes and sizes of tools.

Analyzing the soil-machine-interaction by using the excavation-specific data requires tunnel sections with clear information of the possible influences listed in Fig. 11. It is suggested to compare sections with regard to one single influence (e.g. diameter cutting wheel) while the others should be preferably constant. As this is not easily achieved in practice it is suggested to document the influencing factors as precise as possible. In addition the limited view at the subsoil conditions should be considered. Dealing with soil-machine-interactions, it should be at least standard to identify possible superposing effects. Otherwise the significance of the analysis could be quite low.

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Application of the ANFIS Method for the Prediction of Surface Settlements caused by a Slurry Shield Tunnel Boring Machine

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Abstract

The purpose of the present study is to apply the ANFIS method for predicting the ground surface displacements induced by a slurry shield tunnel boring machine (TBM). Based on the data obtained during the excavation of contract 4 of the subway line B tunnel in Toulouse (France), a Sugeno fuzzy inference system is trained with 90% of the data using an hybrid learning algorithm, and then validated on the remaining data. The performances of the model assessed by the Root Mean Square Error (RMSE) and the coefficient of determination R² indicate a good correlation between predicted and measured displacements; the model is also able to reproduce the observed shapes of vertical displacement troughs (for both settlement and heave).

Keywords: Settlement, shield tunneling, ANFIS, subtractive clustering, TBM operation parameters
1 INTRODUCTION

The prediction of the surface displacements induced by shield tunneling is a complex problem that involves a large number of TBM operation parameters which are strongly interacting; this makes it difficult to model the non-linear relationship between settlement and the parameters using conventional methods such as regression analysis. The technique of neuro-fuzzy systems has become a powerful tool for modeling complex problems with vagueness and uncertainty in the data; it is successfully applied to various geotechnical problems ([1], [3]). In this study, we propose to apply the Adaptive Neuro Fuzzy Inference Systems (ANFIS) method that combines fuzzy inference systems with the technique of neural networks to model the vertical ground surface movements induced by a slurry TBM during the construction of contract 4 of Toulouse subway line B tunnel, according to nine TBM operation parameters and a geometry parameter that represents the position of points with respect to the axis of the tunnel. Two different approaches have been already applied for the determination of correlation between the ground movements and the TBM operation parameters: the non-linear regression [4] and artificial neural network (ANN). The disadvantage of these two approaches is that the non-linear regression is based on the predefined shape of the trough while the ANN method required the introduction of additional points and the modification of the geometry parameter in order to capture the reduced settlements and the heaves observed close to the tunnel axis.

2 ANFIS METHOD

The Adaptive Neuro Fuzzy Inference Systems method developed by Jang [2] incorporates the ANN learning into the development process of fuzzy inference systems (FIS). It is mainly based on fuzzy “if-then” rules of Takagi and Sugeno (TS) type [2], where each rule is defined as a linear combination of membership functions associated to input variables and the final FIS output as the weighted average of each rule’s output. The equivalent architecture of ANFIS is shown in Figure 1. It consists of five layers and each layer is formed by several nodes. Each node performs a particular function and the output signals from nodes in the previous layer will be accepted as the input signals in the present layer and its output will serve as input signals for the next layer. There are two types of nodes: adaptive nodes and fixed nodes which are marked by squares and circles respectively. ANFIS optimizes the FIS parameters from a given input and output data by using an hybrid learning algorithm. It consists of back propagation for the input membership functions parameters (layer 1) and the least square estimation for the output membership function (layer 4).
3 DATA PREPARATION AND MODELLING

The present study is based on the measured data, including ground surface movements and TBM operation parameters, during the construction of the contract 4 of Toulouse subway line B tunnel. A slurry shield machine (7.8 m in diameter) was used to excavate this tunnel (3.7 km in length) in Toulouse molasses, which are overconsolidated and considered homogenous in this study.

A great number of TBM operation parameters were automatically recorded, and the vertical ground movements were monitored by precise levelling of transverse profiles installed every 30 m along the tunnel drive. The collected data were treated by Vanoudheusden [4] and formed a database (485 points) which becomes a useful source for developing predictive models for the ground settlements. In a first analysis, 9 main TBM parameters have been selected by Vanoudheusden [4]: TBM advance rate, hydraulic pressure used for the cutting wheel, confining pressure at the tunnel face, total jack thrust, annular void grouting pressure and volume of grout, time required for the excavation and installation of one tunnel lining ring, overcut and total work required for one ring excavation. The vertical displacements distribution in the transverse direction presented 2 different trends corresponding to continuous zones (Figure 2): “settlements” zones where displacement profiles present a Gaussian shape, “heave in the centre” where settlements were observed at a certain distance from the tunnel axis and reduced settlements or heaves where obtained close to the tunnel axis.
Using a database with 485 values, a neuro fuzzy model was developed for the prediction of vertical ground surface displacement. The input variables are the 9 main TBM parameters and the geometrical parameter \( X/H \), where \( X \) is the horizontal distance from the tunnel axis and \( H \) is the tunnel axis depth. Before training the ANFIS model, the original data is normalized into the range of [0 1]. 90% of the dataset was randomly selected for training the model, and the remaining 10% was used as the validation data. The computation process was implemented using the fuzzy toolbox of Matlab (Version 2010a).

![Figure 2](image_url)

**Figure 2:** Measured ground displacements: a-settlements zone, b-heave in the centre zone

### 4 MODEL RESULTS AND PERFORMANCES

The construction of the ANFIS model starts with generating the initial fuzzy inference system. This step includes a determination of the number of membership functions for each input variable, the shape of membership functions for premise part and the membership functions for consequent part of the rules. In this study, the Gaussian membership functions were selected for the input variables and the linear function for the output variable. Because of the large amount data (10 input variables), the number of fuzzy rules was generated using the subtractive clustering method. For this method, the influential radius \( R \) is a critical parameter for the determination of the optimal number of rules. To achieve minimum Root Mean Square Error (RMSE), different fuzzy models for different values of \( R \) were trained using the hybrid learning algorithm and their performances were evaluated on the testing data. The minimum RMSE was obtained for a radius \( R=0.5 \); this correspond to six fuzzy rules, and each input variable was represented by six Gaussian membership functions.
The performances of the trained model were evaluated using two criteria: the Root Mean Square Error (RMSE) and the coefficient of determination ($R^2$).

**Figure 3:** Predicted versus observed displacement: a-training set, b-validation set

**Figure 4:** Predicted settlement through: a-settlement points, b-heave points

Figure 3 shows a good fit between predicted and observed displacements for both training and validation sets. The performance indices calculated for the training set are RMSE=0.039, $R^2=0.82$ and for the validation set RMSE=0.054, $R^2=0.81$. These results indicate good prediction accuracy and confirm the model capability to predict the ground surface displacements induced by a shield tunnelling. In addition, when
transversal profiles are drawn for different reference points with fixed TBM parameters and by varying X/H, the model is able to reproduce the observed shapes for both settlements and heaves as shown on Figure 4.

5 CONCLUSION

A prediction model for the ground surface displacements induced by slurry shield tunneling was developed using the ANFIS method. Based on the data recorded during the construction of contract 4 of Toulouse subway line B tunnel, the performance indices (RMSE, $R^2$) of the developed model indicate a good correlation between predicted and measured displacements. In addition, the shapes observed for both settlements and heaves are reproduced successfully. The developed fuzzy model is capable of modeling the complex non-linear relation between the displacements and the selected TBM operation parameters. The limitation of the ANFIS modeling in this study is that the relative importance of the nine parameters was not assessed, and the determination of the optimal influential radius for each dimension in the data space is not easy.

REFERENCES


Static Performance Evaluation of a Tunnel built in Very Soft Clay

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Abstract

The static behaviour of a major Metro Line tunnel built in a densely populated city is studied. This tunnel crosses through important layers of very soft clay, which exhibits both low shear strength and high compressibility. The tunnel is 7.75 km long, has a diameter of 10.20 m, and it was built using the earth balance pressure (EBP) technique. The primary lining is comprised by pre-cast reinforced concrete ring sections, 0.40m thick and 1.5 m long. In particular, this paper presents and discusses some of the results gathered from 3D finite differences models developed to simulate the excavation process and the shield advance. The stress-strain-strength relationship was represented with the Cam clay model. The predictive capacity of these models was assessed comparing their results with those obtained from field data. A good agreement was observed between measured and computed responses. With the data compiled from the instrumentation and numerical modelling, insight was gained into the understanding of the static response of this type of structures built in highly compressible clays.

Keywords: Tunnel, instrumentation, soft clay, numerical study, Cam clay.
1 INTRODUCTION

The 12th Metro Line in Mexico City is located towards the south-west part of the valley. It has a total length of 28.4 km, and is comprised by a tunnel 7.75 km long, and 20.6 km of elevated and ground level sections. The tunnel section runs through three geotechnical zones, as established by [2], commonly referred as hills, transition and lake zones. The lake zone is characterized by thick layers of very soft to soft clay, which exhibit high compressibility and low shear strength. The tunnel external diameter of the primary lining is 9.91 m. The primary lining is comprised of reinforced concrete rings, integrated of seven dowels 0.40 m thick and 1.5m long. The maximum ring weight is approximately 65 kN. The shield body tail outside and inside diameters are 10.17 m and 9.99 m respectively. The tail clearance is 0.04 m. The shield body overall length is 11.38 m. The total excavation diameter is 10.20 m, with a maximum overcut of 0.130 m. Ground injections were conducted during the lining placement to fill the voids left between the concrete rings and tunnel walls by the shield over excavation. Specifications indicated that the gap between the lining and the soil should be filled controlling the injection volumes, using low pressures to avoid ground heave, but rapid curing time (i.e two hours in average) grout to reduce settlements.

2 NUMERICAL SIMULATIONS

Soil-tunnel interaction was studied at two critical sections A and B, corresponding to those with the thickest clay strata. Tridimensional finite-difference models were developed with the program FLAC3D (Itasca, 2009). The results obtained with these models were compared with actual measurements. The modified Cam clay model was used to simulate the stress-strain-strength behaviour of the soils found at the studied sites.

2.1 Parameters Determination

Only limited experimental information regarding soil properties at the studied sites was available, and was mostly comprised by conventional undrained unconsolidated triaxial tests, and some one-dimensional consolidation curves. Therefore, it was necessary to resort to available data gathered in a previous research conducted by [4], which carried out series of consolidated undrained tests with pore-pressure measurements to characterize the Cam clay model parameters for the typical materials
found in the so-called Mexico City lake zone, including the upper soft to very soft clay formation, and the lower hard clay typically found after 35m to 40 m of depth, around the city, as well as the deep hard layers, mostly comprised of silty or clayey very dense sands. These authors have established common ranges for M, λ and N modified cam clay parameters. As stated by the modified Cam clay theory, the frictional constant M, is defined as the ratio of the deviatoric stress, q, over the mean effective pressure p\text{cr} at the critical state line, and is obtained commonly from undrained tests with pore-pressure measurement, based on the slope of a best-fitting line of q vs p\text{cr}. The parameters λ and κ are the slopes of the normal consolidation and swelling lines respectively, in an specific volume, v, versus ln(p) plot. In particular, λ was characterized by [4] from isotropically loaded triaxial test. Unfortunately, they did not perform unloading excursions to obtain κ. Thus, in this work, κ was estimated as ¼ of λ. The location of the normal consolidation line in the v - ln (p) plot is defined by the parameter N for a reference pressure p\text{1}. For the numerical simulations presented in this work, mean values for each typical material were deemed appropriated to perform an overall assessment of the tunnel behaviour considering the uncertainty associated with the model properties. These are presented in Table 1, for a reference pressure, p\text{1}, of 1kN/m\text{2}. The idealized soil profiles for sections A and B are presented in figure 1 respectively. The preconsolidation pressure, p\text{c}, determines the initial size of the yield surface, and will correspond to the maximum past mean effective stress. Poisson’s ratio values, ν, used in the analysis are compiled also in Table 1. Thus, the shear moduli vary as a function of the specific volume and the mean stress.

Table 1: Modified Cam-clay parameters used in the analyses

<table>
<thead>
<tr>
<th>Material</th>
<th>Unit weight, γ (kN/m\text{3})</th>
<th>ν</th>
<th>λ</th>
<th>κ</th>
<th>M</th>
<th>N</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very soft to soft clay</td>
<td>11</td>
<td>0.28</td>
<td>2.09</td>
<td>0.7</td>
<td>0.74</td>
<td>16.92</td>
</tr>
<tr>
<td>Medium Clay</td>
<td>13</td>
<td>0.30</td>
<td>0.50</td>
<td>0.17</td>
<td>0.74</td>
<td>7.42</td>
</tr>
<tr>
<td>Very dense clayey sand</td>
<td>15</td>
<td>0.30</td>
<td>0.38</td>
<td>0.13</td>
<td>1.83</td>
<td>4.78</td>
</tr>
</tbody>
</table>
2.2 3D Finite Differences Models

Tridimensional finite differences models were developed using the program FLAC$^{3D}$ [1] to simulate the tunnel excavation process, and primary lining placement. The meshes for sections A and B are comprised of 74,100 three-dimensional zones, as shown in figure 1. The tunnel axis for Sections A and B is found at 12.50 and 16.90 m of depth respectively. Both soil profiles were idealized in three layers, neglecting the manmade fill in the simulation. For section A, the first layer is 19 m thick and corresponds to very soft clay, the second layer is 10.0 m thick and corresponds to medium clay, and the third layer corresponds to very dense clayey sand. For section B, the first layer of 9 m corresponds to very soft clay, the second layer of 15 meters of depth corresponds to medium clay, and the third layer corresponds to very dense clayey sand. The water table in sections A and B is found at 2.0 m and 3.5m of depth respectively. To avoid boundary effects in the computed displacements, these were measured at a control section located 10 m from the mesh edges, which corresponds approximately to one tunnel diameter. The construction process, including the shield advance and lining placement was simulated in stages considering 1.5 meters of excavation each advance (Figure 2), which corresponds to the dowel length. First, equilibrium was reached in the system solving for gravity loads. Second, the measured in-situ pore pressure distribution was applied to the mesh to establish the effective stresses existing prior tunnel excavation, due to the water table drawn down prevailing in the city. Then, 1.5 m of excavation was modelled. The soil was removed and two variable internal pressures were applied to the model: 1) A pressure $P_f(z)$ equivalent to the earth pressure at rest plus the in-situ pore water pressure in the analysed section was applied at the shield face soil, and 2) A pressure $P_s(z)$ equal to the total stress was applied around the tunnel in the portion in contact with the shield, within the over excavation gap. In reality the pressure in the chamber was adjusted along the tunnel segment as a function of depth, and the shield thrust was also acting upon the soil, the evolution of this composed force was not available to be included in the model, thus a simplified $P_f(z)$ was used for the analysis presented herein instead. Then the global equilibrium of the system was found for these load boundary conditions. This procedure was repeated until complete 12.0 m, which corresponds roughly to the shield tail length. Thus, after this initial 12.0 m of excavation were simulated, another 1.5 m were removed and elastic shell elements were placed in the initial excavated 1.5 m, eliminating the pressure $P_s(z)$ in this portion of the soil to simulate the presence of the lining ring. As previously mentioned, specifications
Static Performance Evaluation of an Instrumented Tunnel built in Very Soft Clay

indicated ensuring closing the gap between the liner and the soil by controlling the injection volumes using low pressures to avoid ground heave, but rapid curing time grouting to reduce settlements.

**Figure 1:** Tridimensional finite difference models

**Figure 2:** Simulation of excavation in sections analysed
Therefore, in the model it was considered that the soil rested in the primary lining (i.e. the grout reach its design strength fast enough to transfer the soil load to the concrete ring). The same procedure was repeated until reaching 50 m. The mechanical properties of the concrete rings that comprise the primary lining are: Strength concrete, $f'_c$, of 34,355 kPa, Young modulus, $E_c$, of 26,191,600 kPa, and Poisson ratio, $\nu_c$, of 0.3. The rings were assumed to be monolithic. Thus, the interaction among ring segments was not taken into account in the simulation, and an equivalent reduced Young modulus was used instead. This equivalent modulus was obtained from parametric studies, and is about 80% of the original modulus.

3 NUMERICAL ANALYSES RESULTS

Figures 3 and 4 show a comparison between computed and measured settlements in sections A and B respectively. As can be seen in section A, although the model is able to capture the overall shape of the ground settlements, it over predicts the maximum displacement in about 20%. In this case the model is not able to fully capture the measure response, which exhibits and important ground heave of about 1.6 cm very likely due to an excessive thrust force of the shield, probably combined with over injection in the shield tail in this section. Unfortunately, the evolution of these forces over time was not available to be input in the model. Regarding section 2, the model does a better job reproducing the measure response, both in terms of the general trend as well as the magnitude of the maximum displacement. The horizontal displacements distribution with depth computed and measured at an inclinometer located 10.90 m away of the tunnel axis close to section B, are shown in figure 5. As before, it can be observed a good agreement between computed and measured responses.
Static Performance Evaluation of an Instrumented Tunnel built in Very Soft Clay

Figure 3: Comparison between computed and measured settlements in section A

Figure 4: Comparison between computed and measured settlements in section B

Figure 5: Computed and measured horizontal settlements in section B
4 CONCLUSIONS

Results gathered from the tridimensional finite difference models indicate that the Cam clay model allows capturing the static response of Mexico City clays. A key variable that needs to be properly defined in the model is the evolution of the combined force acting over the tunnel face during excavation, which is composed of the shield thrust and the pressure chamber. If this force is underestimated, the final maximum computed displacements will be overestimated by the model. This effect is even more notorious when dealing with low soil covers. Tridimensional finite difference models could be feed by the data gathered during construction to ensure that this force is properly adjusted in the field and ground movements (heave or settlements) are minimized.

REFERENCES


Face Stability Assessment of Large-diameter Slurry Shields

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Abstract

The development in shield tunnelling is aiming at a rapid increase of the tunnel diameters. Diameters of up to 19.5 m are being examined in detailed planning. In this case the planned tunnel face area of approximately 300 m² is nearly 1.6-times larger than the largest tunnel excavated by a slurry shield until today. For the assessment of the tunnel face stability and face support pressure usually a calculation model based on Horn’s failure mechanism is used. It is assumed, that large shield diameters require a more detailed consideration of boundary conditions than using simplified assumptions made in analytical models. Furthermore, the possibility of encountering a heterogeneous tunnel face large-shield tunnelling is much higher than under regular conditions. Within heterogeneous face conditions, numerical methods are advantageous. Here, the failure mechanism of a heterogeneous face, the global and the local stability of individual soil layers can be investigated close to reality - in contrast to analytical models.

Above all, the interaction between the slurry and the ground plays an important role. In reference to the specific parameters the support pressure can be transferred onto the face in two different ways: (1) via a nearly impermeable membrane at the surface or (2) within a penetration zone of a certain depth in the soil.

The attention will be given to heterogeneous tunnel face conditions considering membrane model of slurry ground interaction for large shield diameters. The options for simplified consideration of the penetration model are discussed. Finally, recommendations for calculating the support pressure for large-slurry shields using by numerical methods are provided.

Keywords: Face stability, slurry shield, large diameter, slurry-ground interaction
1 INTRODUCTION

Slurry shield machines up to a diameter of 19.5 m are proven in detailed planning. Here, the face stability is of particular importance because of the influence of the extensive tunnel face area on the surface.

Until now, it is common practice to use analytical models based on the Horn’s failure mechanism (Horn, 1969) for the assessment of face stability. The Horn’s failure mechanism consists of planar sliding surfaces, which have to be determined before starting the calculation. This determination is required in advance, since the stress conditions on the slip surfaces are not assessed and only global equilibrium condition for the forces on the soil body is formulated. Unfortunately, the Horn’s failure mechanism does not consider polygonal slip surfaces or partial failure mechanism of tunnel face. Especially these factors seem to have an important influence on the stability of extra large tunnel face.

Kasper & Meschke (2006) developed a complex numerical model for assessing the impact of shield tunnelling on the surrounding ground. Bezuijen (2009) investigated numerically the influence of the conditions in the steering gap on the ground during the excavation. Various references are focusing exclusively on the numerical assessment of face stability for homogeneous ground conditions. An extensive numerical parameter study on the face stability of shield machines was carried out by Ruse (2004). These study was conducted exclusively for homogeneous tunnel face conditions and using the Mohr-Coulomb constitutive law. Kirsch (2010) analysed the influence of the basic principles of various constitutive laws during failure. He compared Mohr-Coulomb and Hypoplastic (von Wolffersdorff formulation) constitutive law and registered only negligible difference in the obtained support pressures for the assumed homogeneous face. In contrast to Ruse (2004) the results of Kirsch (2010) show, that the dilatancy of soil has a fundamental influence on the support pressure at failure.

Li et al. (2009) investigated the partial failure of homogeneous tunnel face of extra large shield machines for cohesive soil under undrained conditions. One result of the study is that active partial failure of tunnel face is not decisive for the investigated conditions. Wong et al. (2009) carried out an analysis of heterogeneous tunnel face considering only passive failure (passive resistance of the tunnel face against to high support pressure). The passive failure of tunnel face is unlikely to happen for slurry shield since the break-up of slurry will occur earlier than passive failure of soil body (by lower support pressure).
The demand on numerical investigation of failure mechanism for heterogeneous face conditions of an XXL-slurry shield machine is obvious as the probability of encountering heterogeneous face conditions is growing with increasing shield diameter. Furthermore, the interaction of the slurry and the soil has to be taken into account. The function of the slurry as a pressure balancing fluid in order to stabilize the subsoil is to create a zone of reduced permeability in the ground. In order to fulfill this task, the yield strength $\tau_f$, the unit weight and the stability of the suspension as well as the penetration behaviour in the ground are decisive (Kilchert et al., 1984). According to Walz (1989), the transmission of the hydrostatic pressure gradient by bentonite suspension to the particle structure of the subsoil is independent of time. For this purpose, at the surface or to a certain penetration depth in the subsoil near the surface a zone is formed, whose permeability is less than that of the subsoil. In this zone, the pressure gradient between the suspension and the subsoil, that has to be supported, is converted into effective stress acting on the particle structure of the subsoil. The creation of a zone with reduced permeability can be achieved in two ways. A direct relation exists between the mechanism of the balancing pressure transmission and the penetration behaviour of the bentonite suspension in the subsoil, which has to be supported (Mueller-Kirchenbauer, 1977). On the one hand, the relation of the size of the bentonite particles in the suspension and the size of the pores in soft soil is decisive. On the other hand, the yield strength $\tau_f$ of the bentonite suspension is the determining factor. In order to transfer the hydrostatic pressure gradient to the particle structure of the subsoil in the zone of reduced permeability, two mechanisms are examined: (1) the membrane model and (2) the penetration model.

2 ESSENTIAL ASPECTS FOR NUMERICAL MODELLING OF FACE STABILITY

The aim of numerical face stability analysis is to find the highest support pressure, for which the active failure of the face occurs (collapse). Basically, following methods are available to achieve this state (Kirsch, 2009; Vermeer et. al, 2002): (1) load reduction method, (2) strength reduction method and (3) displacement control method. Due to the fact, that the displacement control method is usually used for verification of laboratory experiments, this method is excluded in this study. For heterogeneous face conditions the load reduction method is advantageous compared to strength reduction method. Since the material parameters of soils are not changed
during the calculation the obtained shape of failure body is close to reality. If the specific support pressure transfer on the face depending on its amount is presumed and the stability factor for such mechanism has to be determined, the strength reduction method is advantageous to apply.

For the numerical modelling a suitable constitutive law for soil has to be chosen. In this study the modelling was carried out with a linear elastic and perfect plastic model based on Mohr-Coulomb failure criterion. Here, this model is evaluated being appropriate for the determination of support pressure during failure (Kirsch, 2010). Furthermore, the non-associated flow rule was assumed in the calculations considering dilatancy angle 0° for all soils.

In general, slurry shield tunneling is executed below a water table. Under certain conditions the presence of a water table can influence the failure mechanism of the tunnel face. Therefore a water table of 5 m above tunnel crown is considered in all models. The excess pore pressure is supposed to be a particular parameter influencing the stress state in a low permeability soil. In slurry shield tunnelling the drained conditions can be assumed, if the soil permeability is higher than then $10^{-7}$ to $10^{-6}$ m/s and the advance rate is lower than 0,1 – 1 m/s (Anagnostou & Kovari, 1996). In general the assumption of drained conditions can be done for sands and gravels (Vermeer, Ruse & Marcher, 2002). Hence, this study is carried out without the influence of excess pore pressure.

The mechanism of face support pressure transfer has to be introduced in the numerical model with respect to the interaction model occurred between slurry and ground. According to the properties of slurry and ground two pressure transfer models for bentonite suspensions as support medium exists (Mueller-Kirchenbauer, 1972). A filter cake as a nearly impermeable membrane is formed, when the bentonite particles are filtered directly at the tunnel face. The penetration zone is formed in front of the face, when slurry is infiltrating the ground up to a certain depth.

The membrane model can be satisfactory considered by direct loading of the nodes on the tunnel face (Vermeer, Ruse & Marcher, 2002). Since the mechanism of support pressure transfer does not depend on the amount of pressure transferred, the load reduction method can be applied without extraordinary calculation time. If the penetration zone is formed in front of the face, the direct loading of tunnel face is no more accurate enough. The usable modeling strategy for considering pressure transfer with penetration zone is described in chapter 5.
3 HETEROGENOUS TUNNEL FACE – MEMBRANE MODELL

In the parametric study the behavior of the heterogeneous face with diameter 19.5 m at failure was assessed. Four soil types were considered in the models (Table 1). The boundary values for the properties of soils were chosen in order to represent a wide span of soil types. The properties of soils for the Mohr-Coulomb constitutive law are presented in Table 1.

Table 1: Properties of soil type considering Mohr-Coulomb constitutive law

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Soil 1</th>
<th>Soil 2</th>
<th>Soil 3</th>
<th>Soil 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>γ (kN/m³)</td>
<td>19</td>
<td>18</td>
<td>21</td>
<td>15,5</td>
</tr>
<tr>
<td>γ' (kN/m³)</td>
<td>9</td>
<td>8</td>
<td>11</td>
<td>5,5</td>
</tr>
<tr>
<td>φ (°)</td>
<td>15</td>
<td>30</td>
<td>37</td>
<td>10</td>
</tr>
<tr>
<td>c (kPa)</td>
<td>5</td>
<td>0</td>
<td>0</td>
<td>5</td>
</tr>
<tr>
<td>ν (-)</td>
<td>0,32</td>
<td>0,3</td>
<td>0,25</td>
<td>0,3</td>
</tr>
<tr>
<td>E (MPa)</td>
<td>14</td>
<td>30</td>
<td>167</td>
<td>1</td>
</tr>
</tbody>
</table>

3.1 Three Layer Face

The behaviour of the tunnel face at failure consisting of three different soils (soils 1, 2 and 3) was investigated (Table 1). The location of the three different soils within the layer formation (Figure 1) was varied in calculation. The overburden height was taken as an additional parameter. The calculations were carried out with overburden height of 10 m, 15 m and 20 m. Furthermore, calculation based on homogenized tunnel face condition was taken as standard reference. The procedure of homogenization was carried out by calculating of weighted average parameters from different soil properties on the heterogeneous face.

Figure 1: Assumed distribution of soil layers on the heterogeneous tunnel face
First of all, the shape of tunnel face failure was investigated. The shape of failure can be described by the help of displacements for three control points located on the tunnel face in the middle of the soil layers (Figure 1: distance of control points from tunnel crown: 1/6 D for layer A, ½ D for layer B, 5/ D for layer C). It was found out, that the shape of failure does not depend on the overburden height for investigated conditions. Just the absolute amount of displacement at the face during the failure differs (higher displacement for higher overburden).

Mentioned results are illustrated for homogenized conditions on the face by the help of load/displacement curves (Figure 2). The highest deformation was observed for point B and the lowest for point C.

Concerning the heterogeneous conditions on the tunnel face the shape of failure diverged considerably within various soil layer arrangements on the face. It was determined, that the most significant influence factor is the location of the soil 1 on the face, which has the lowest shear properties among other soils on the tunnel face. This is influenced by the fact that soil 1 collapsed in all investigated cases in contrast to the other soil types. This was confirmed by the evaluation of load-displacement curves. The results for investigated combinations of heterogeneous face can be summarized as follows (Figure 3):

- Soil 1 in the crown: Partial failure mechanism of the face occurs, wide arching in the overburden and concentration of face deformation in the tunnel crown
- Soil 1 in the centre: Proportion of displacement corresponds with homogenized case of heterogeneous face, highest deformation in the centre of tunnel face, partial failure mechanism and polygonal slip surface may occur
- Soil 1 in the bottom: collapse of the whole tunnel face, polygonal slip surface occurs

**Figure 3:** Displacement of the tunnel face during the failure for different locations of the soil 1; in the crown (soil layer arrangement 123), in the centre (soil layer arrangement 312), in the bottom (soil layer arrangement 231) (from left to right)

Consequently, the obtained support pressures at the failure were evaluated. Table 2 illustrates, that the support pressure was the lowest for homogenized conditions compared to all other investigated cases. The results show that support pressure at failure can be significantly different for various combinations of the investigated heterogeneous face. Nevertheless, for varied overburden heights the obtained pressures diverged only slightly. The variation can be explained with higher influence of soil arching in case of higher overburden. The difference in the face pressure at the tunnel crown up to 8 kPa was obtained between combinations (compared with homogenized conditions even 10 kPa). According to the results, it can be derived that the properties of the layer located in the centre of tunnel face are the most important for the face stability. The lowest support pressure required at failure was observed for combinations when soil 3 was situated in the centre of the tunnel face.

The highest support pressures were calculated for cases, when layer 1 was located in the centre. It is necessary to mention, that for the soil layer arrangement 321, concerning overburden of 15m and 20m, a higher pressure at the failure was obtained than for soil layer arrangement 213 (for 10m overburden were obtained results vice versa). The results generally show that the homogenization procedure can lead to underestimating of influence of the soil 1 on the face stability. In the next step was therefore the influence of soil with weak properties on the face analysed more precisely.
Table 2: Obtained support pressures (kPa) required at failure for investigated cases of heterogeneous and homogenized face conditions

<table>
<thead>
<tr>
<th>Combination of soils (layer a, layer b, layer c)</th>
<th>Overburden</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>10 m</td>
</tr>
<tr>
<td>Homogenized</td>
<td>53.90</td>
</tr>
<tr>
<td>132</td>
<td>56.41</td>
</tr>
<tr>
<td>231</td>
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<td>321</td>
<td>59.88</td>
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<td>312</td>
<td>63.80</td>
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3.2 Tunnel Face with Very Weak Layer

In addition, the aim of investigation was to assess the influence of a very weak soil layer from the view of its shear properties (Table 1: soil 4) on the tunnel face stability. Further purpose was to determine the cases when the homogenization procedure, carried out in the analytical models, is still on the safe side. A variation study was performed in which three locations of the soil 4 on the tunnel face were investigated (Figure 1):

- In the crown of the tunnel face (layer a)
- In the centre of the tunnel face (layer b)
- At the bottom of the tunnel face (layer c)

The rest of the tunnel face consisted from soil 2 in all calculations. The thickness (0.5m, 1m, 3m, 5m and 6.5m) of the weak soil 4 was taken as further parameter in the study. In addition, homogenized conditions were investigated for each case. The homogenized properties of tunnel face were calculated using weighted average method. The obtained face support pressures at the failure are outlined in Figure 4.
For thickness 0.5m of soil 4 the results of obtained support pressure are comparable between all investigated cases. With increasing layer thickness of soil 4 the results become more divergent; nevertheless the results for locations of soil 4 in the crown and bottom were remaining similar for all calculations. Concerning the results for heterogeneous tunnel face only, it is to observe that from the thickness 3m of soil 4 up is the influence of its location on the obtained support pressure decreasing. It could be said that the homogenization procedure is on the unsafe side when the thickness of soil 4 is higher than 0.5m. From a thickness up to 0.5 m the influence of the very weak soil 4 is negligible. The results show that the weak layer can in general endanger the face stability massive when it is located around the centre of tunnel face.

4 CONCLUDING REMARKS

The behaviour of heterogeneous tunnel face was studied numerically on the background of large slurry shields. The properties of the support medium as unit weight and yield strength were taken into account.

Firstly the membrane model of slurry–ground interaction was investigated. It was found out that the location of soil layers with different properties on the tunnel face has a significant influence. The location determines the failure shape and the support pressure during failure. With respect to the results it was derived that the obtained failure shape of tunnel face depends on the location of the weakest layer located on
the tunnel face. It was confirmed that the partial failure of the tunnel face and polygonal slip surface can occur for the large slurry shields. In detailed investigation of the influence of a weak soil it was specified, that the most dangerous location for face stability is at the centre of the tunnel face. The homogenization procedure of different soil properties on the tunnel face, which is usually carried out in analytical models, was on the unsafe side if the thickness of weak soil exceeded 0.5m.

5 OUTLOOK - INVESTIGATION CONSIDERING PENETRATION MODEL

In case of the penetration model, the effective part of the support pressure is transferred on the soil particles by the help of shear stress between infiltrating slurry and grains within the penetration zone. Hydraulic pressure of the slurry, which is equal to the pore pressure in the ground, is transferred directly at the tunnel face. The maximal amount of effective support pressure, which can be transferred within the 1 m penetration depth, is defined by a stagnation gradient of the slurry \( f_{s0} \) (DIN 4126, 2004). The stagnation gradient depends on the characteristic grain size of soil \( d_{10} \) and the yield strength of the slurry \( \tau_f \).

\[
f_{s0} = \frac{3.5 \cdot \tau_f}{d_{10}}
\]

In the numerical model, the hydraulic pressure and the effective pressure of the slurry have to be simulated separately. The nodes directly on the face can be loaded directly by the hydraulic part of the slurry pressure. The nodes in front of the face are to be loaded with proportion of effective support pressure corresponding with node distance from tunnel face and the calculated stagnation gradient. In case of penetration model, the support mechanism is specific for different amount of support pressures, because the penetration depth varies. Hence, the load reduction method can be used for achieving the failure state of tunnel face only with an extraordinary modeling time. Therefore, the strength reduction method is advantageous to be applied for determining the stability factors of the tunnel face.
REFERENCES


Modelling of Foam-Sand-TBM Interaction

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Abstract

The interaction between foam, sand and TBM is investigated, using the results of field tests and model tests. It is shown that the groundwater flow, the compressibility of the foam, the stability of the foam and the two-face flow of the foam and water in the mixture are important phenomena for the foam behaviour, the torque on the cutting wheel and the stability of the foam. Aspects that have to be taken into account in numerical models are discussed.

Keywords: Foam, EPB, sand, pore water, interaction

1 INTRODUCTION

A numerical model is always a simplified version of reality. That is the advantage of a numerical model: we know what aspects are modelled and assume that these aspects are dominant for the phenomena we want to describe. However, it is also the disadvantage of numerical modelling: some aspects are not modelled. The modelling of the interaction between foam, sand and TBM for an EPB shield is rather complicated. Certainly in case the TBM has to drill through saturated sand. Since the air in the foam is highly compressible, pressure fluctuations will cause volume fluctuations in the muck. As soon as the porosity of the muck is lower than the maximum porosity of the sand, the resistance against shearing will increase significantly, see Figure 1. Partly saturated ground water flow will be present in the mixing chamber. Furthermore the stability of the foam is of importance, because this determines how long the foam will exist in the mixing chamber.
The paper shows what mechanisms have been seen in field tests and model tests in the soil in front of the TBM, in the mixing chamber and in the screw conveyer: and will deal with how phenomena observed can be modelled. In the description of the various processes, the paper follows the drilling process. It starts well in front of the TBM in the still undisturbed soil, then processes at the cutting face are described, followed by processes in the mixing chamber and presents briefly the decompression in the screw conveyer.

2 DEFINITIONS

In this paper the following definitions are used:

$FER$: foam expansion ratio, the ratio between the total amount of foam (by volume $Q_F$) and the amount of surfactant solution ($Q_L$) (water and surfactant).

$FER_m$: foam expansion ratio of the mixture, the ratio between the total amount of foam (by volume $Q_F$) and the amount of surfactant solution ($Q_L$) (water and surfactant) + the remaining pore water in the soil.

$FIR$: foam injection ratio, the volume of foam ($Q_F$) divided by the volume of soil removed ($Q_S$). $Q_S$ can be calculated from the advance rate ($v$) and the face area ($A_s$):

$$Q_S = v \cdot A_s.$$

3 IN FRONT OF THE TBM: PORE WATER PRESSURES

The first thing to be noticed from an approaching EPB in saturated sand is a rise in pore water pressure in front of the TBM, see Figure 2. This rise influences the stability of the tunnel face and is therefore of importance when this stability is calculated. The rise is caused by pore water flow from the mixing chamber where an excess pore water pressure is created, as will be explained later. When the pore water inflow at the tunnel face is known, the distribution of the excess pore water pressure in front of the TBM can be easily modelled using groundwater flow computations. It appears that in homogeneous sandy soil conditions the excess pore water flow can be approximated as also shown in Figure 2, although the agreement is less for small
excess pore pressures, probably due to measurement errors [1]. What is meant with relatively permeable or impermeable subsoil, as mentioned in the figure caption, will be explained later.

**Figure 2:** Measured excess pore pressure in front of an EPB shield at the axis (● and ○ different gauges) and approximation (Botlek Rail Tunnel). Left plot relatively impermeable subsoil, right plot, relatively permeable subsoil [6], [1].

## 4 PROCESSES AT THE TUNNEL FACE

### 4.1 Soil Stresses at the Tunnel Face

The foam-sand-water mixture (which will be called: “mixture” in the remaining part of this paper) is created at the tunnel face. Foam is injected in the mixing chamber from the cutting wheel. At some TBMs it is also possible to inject foam from other locations (deeper in the mixing chamber and at the screw conveyer), but most foam will be injected from the cutting wheel in the direct vicinity of the tunnel face. The resulting mixture will be determined by the amount of foam injected, the pore water that comes into the mixing chamber together with the sand that is taken from the tunnel face and the amount of sand itself. Model tests have shown that the air bubbles in the foam hardly penetrate into the soil in front of the tunnel face. Without drilling, for example during ring building, the foam bubbles block the tunnel face and decrease the local permeability at the tunnel face significantly. A decrease from 5*10^-4 m/s for sand without foam to 2.5*10^-6 m/s for a mixture where 83% of the pore water in the sand was replaced by foam [5]. This low permeability at the tunnel face decreases the pore water flow from the mixing chamber into the soil and the air-bubbles pressing to the grains of the subsoil create an effective pressure at the tunnel face.
During drilling there is a different situation. Again the excess pore water pressure will create a groundwater flow from the mixing chamber into the subsoil, but the cutting process will not only take sand from the tunnel face, but sand with pore water. It depends on the permeability of the soil, the radius of the TBM and the drilling speed, which process is dominant. For homogenous soil conditions, as the pore pressure distribution can be approximated by the equation given in Figure 2, the dimensionless parameter:

\[ \alpha = \frac{k\Delta \phi}{v_d R} \]  

(1)

is of importance. In this formula is \( k \) the permeability of the subsoil, \( \Delta \phi \) the difference in piezometric head between the head in the mixing chamber and the original piezometric head in the subsoil, \( v_d \) the drilling speed and \( R \) the radius of the tunnel [4]. When \( \alpha \geq n_s \), with \( n_s \) the porosity of the sand all pore water will be replaced by the foam and there will be an effective pressure at the tunnel face as described above for the situation without drilling. For \( \alpha < n_s \) there will be no effective pressure at the tunnel face and the foam in the mixture will be ‘wetter’ than the original foam because the pore water from the subsoil will enter the mixing chamber. The factor \( \alpha \) was derived from an analytical groundwater flow computation [1], but the influence of \( \alpha \) on the effective stress was also found in model tests [2][5]. The EPB process was simulated in the model set-up shown in Figure 3. Foam is injected through a rotor that cuts the sand and creates a sand water mixture this mixture is removed by a screw conveyer. It can be seen as a miniature EBP shield (diameter is 0.6 m) that tunnels vertically instead of horizontally. The vertical position was chosen because that makes it easier to create the saturated sand model. In this model set up it was possible to regulate the groundwater flow in the sand and thus \( \alpha \). This was done at the bottom of the container by means of an electronically regulated valve that allowed a prescribed flow as a function of the displacement of the rotor, screw conveyer and top plate that moved downward simultaneously. In a test the top plate creates with the container walls between the top plate and the rotor the model mixing chamber.
Figure 4 and Figure 5 show the results of 2 tests with this set up. In Test 202 (Figure 4) there was a 100% replacement of the pore water ($\alpha = n_\text{s}$). In Test 203 (Figure 5) there was 82% replacement ($\alpha = 0.82n_\text{s}$). There is a remarkable difference in the tests results. At the top plate (in the model mixing chamber) the pressure is approximately 100 kPa higher than the atmospheric pressure. This pressure is regulated by the foam injection and the screw conveyer.

In Test 202 after 800s the pressure in the sand is approximately 30 kPa. It can be seen that when the rotor passes one of the pressure gauges there is a pressure jump of around 60 kPa. This means that there is an effective pressure of 60 kpa at the boundary between the soil that is mixed by the rotor and the still undisturbed soil. This can be seen as an effective face pressure. The pressure falls back to atmospheric pressure when the top plate passes the pore pressure gauge. The pressure jump when the rotor passes the pore pressure gauge is completely absent in Test 203, see Figure 5. In this test there is no effective stress on the tunnel face and when this was a ’real’ tunnel face instead of a model test with a horizontal tunnel face, the hydraulic gradient of ground water flow was the only force that stabilized the face.

In the model tests, this difference in effective stresses at the tunnel face resulted, as could be expected, in differences in torque necessary for the rotor to turn it around in the desired number of revolutions, on average the torque was 45% higher in Test 203 compared to Test 202 [5].

4.2 The Foam-Sand-Water Mixture

The foam in the mixture chamber of an EPB tunnelling machine is also influenced by the groundwater flow described above. Again for the conditions that the equation shown in Figure 2 is valid, the relation between the FIR and the original porosity of the soil ($n_\text{s}$) and the porosity of the muck ($n_\text{m}$) can be written as [4]:

$$\text{FIR} \approx \frac{n_\text{s}}{n_\text{m}}$$
The FIR needed to reach a certain porosity of the muck (it will be shown later why a certain minimum porosity is needed in the mixing chamber), depends, apart from the porosities of subsoil and muck also on the dimensionless factor \( \alpha \) defined in Eq (1).

The FER of the mixture (\( FER_{m} \)), the parameter determining how ‘wet’ the foam is can be written as:[4]:

\[
FER_{m} = \frac{n_{s} - \alpha + FIR}{n_{s} - \alpha + FIR / FER}
\]

If \( \alpha \) equals \( n_{s} \) (complete replacement of the pore water), \( FER_{m} = FER \). However, if \( \alpha \) is close to zero (no replacement of the pore water and the mixture in the mixing chamber has to take all the pore water) and for ‘dry’ foam, for example \( FER \) is 20, the \( FER_{m} \) can be approximated as:

\[
FER_{m} = 1 + \frac{FIR}{n_{s}}
\]

Thus for this situation the \( FER_{m} \) is not influenced at all by the original \( FER \) but only by the \( FIR \) and \( n_{s} \). As quantified in [4], the \( FER_{m} \) will be much lower than the original \( FER \), resulting in much ‘wetter’ foam.

\[
FIR = \alpha - 1 + \frac{1 - n_{s}}{1 - n_{m}}
\]

Figure 4: Test 202 pore pressures at various locations in the container. 100% replacement of the pore water with foam. [2]

Figure 5: Test 203 pore pressures at various locations in the container. 82% replacement of the pore water with foam.
5 THE MIXING CHAMBER

5.1 Necessary Porosity in the Mixing Chamber

Figure 1 showed already that the yield stress of the mixture significantly increases as the porosity of the sand in the mixture becomes lower than a certain value. This figure is based on model tests. In those tests the maximum porosity of the sand used was 44.7%. Thus it appears that when the porosity in the mixing chamber has to be higher than the maximum porosity and when the porosity reaches the maximum porosity, the yield stress of the mixture increases. A comparable result was found during the field tests during the construction of the Botlek rail tunnel. In this project it was possible to measure the in-situ porosity in the mixing chamber. It was found that all sand fractions (=1-\(n_m\)) were just below the sand fractions where a grain skeleton with grain stresses could exist (=above the maximum porosity), see Figure 6.

5.2 Pressure Fluctuations in the Mixing Chamber

The pressure in the mixing chamber will be influenced by hydrostatic pressure. For example in a 10 m diameter shield the pressure can be more than 100 kPa higher at the bottom of the mixing chamber compared to the pressure at the top. Assuming that the mixture in the mixing chamber has a homogeneous distribution of sand, air and water, the air will be more compressed in the lower part of the mixing chamber and therefore the porosity will be lower and consequently, according to Figure 1, the yield stress higher.

The pressure distribution is almost perfect hydrostatic in case a slurry shield [7]. However, this is not the case for an EPB, the muck has a higher yield stress than the slurry in a slurry shield and due to that the following processes lead to variations in the hydrostatic pressure distribution: The muck is excavated from the mixing chamber by the screw conveyer. This screw conveyer is located at the lower end of the mixing chamber. This means that the muck has to be transported downwards.
to the yield stress of the muck, this leads to lower pressures around the location where the screw conveyor entered the mixing chamber. Mixing arms at the back of the cutting wheel causes rotation of the muck in the mixing chamber. In the situation that the mixing chamber is not completely filled, this leads to a pressure difference on the left and right side of the mixing chamber, see Figure 7 (From [3]), which show result of measurements of the Botlek Rail tunnel. From this figure it is clear that E5 follows the pore pressure at that location in the mixing chamber, shown with W3, E4 and E6 also follow the pore pressure W3, but only one of them depending on the direction of the revolution of the cutter wheel. With a positive value of the revolutions, E4 is equal to the pore pressure and E6 is equal to the pore pressure when the cutting wheel turns the other way around. The regular spikes on W3 after drilling (cutting wheel is not turning anymore after 1:00:00), are from the cleaning system of this pore pressure gauge.

![Figure 7](image-url)  
Figure 7: Ring 813 S, Botlek Rail Tunnel, Influence of the rotation direction.

The pressure distribution is thus not symmetric and depends on the rotation direction from the cutting head. This is discussed more in detail in [3]. Figure 7 shows that the pressures are higher on that side of the mixing chamber where the soil is moved upwards. Figure 8 plots the gradients that were measured. The density of the muck corresponds to a gradient of less than 18.2 kPa/m (depending on the amount of foam) for hydrostatic conditions. In some cases the gradient is much higher, but only on one side of the cutting wheel. It is assumed that these high gradients correspond with grain
stresses that develop in the mixing chamber. When the drilling stops (after one hour), the gradients become comparable on both sides. Figure 9 shows the pressures measured on all pressure gauges in the mixing chamber at two different times (with different direction of rotation). It shows the influence of the rotation direction, the high gradients that can be present in the lower part of the mixing chamber and the relatively low pressures close to the screw conveyer (E5).

### 5.3 Influence on Muck Properties

In the preceding paragraph it was discussed that the pressure distribution in the mixing chamber is not hydrostatic. However, the pressure will be higher at the bottom of the TBM than at the top. The muck contains air (from the foam) and is therefore compressible. This means that the porosity of the soil in the muck will not be constant, but a function of the local pressure in the mixing chamber. The amount the porosity will change depends on the amount of air in the muck. It was shown in Section 4 that the amount of air in the muck depends on the replacement of the pore water by the foam. For a permeable soil such a replacement was possible and the muck will contain more air and thus be more compressible than muck from a less permeable soil. In case a minimum porosity is necessary to avoid grain stresses at the lower end of the TBM, it can be calculated how the porosity will be as a function of height, see Figure 10. For this calculation two simplifications where necessary: the pressure distribution is taken to be hydrostatic (which, as was shown, is not the case) and the compressibility of the muck ($C_{\text{muck}}$) at a location in the mixing chamber is determined by the percentage of air in the muck using the formula:

$$C_{\text{muck}} = \frac{S}{P_a}$$

(5)

With $S$ the percentage of air in the muck and $P_a$ the pressure in the mixing chamber at that position. Also this relation is an approximation. The compressibility is a bit less due to the influence of surface tension in the foam mixture. In the calculation, it is assumed that a minimum porosity of 0.48 at the bottom of the mixing chamber is necessary to avoid grain stresses. Figure 10 shows that the variation in the porosity over the height of the mixing chamber is considerable and thus, looking at Figure 1, also the variation in strength parameters.
6 SCREW CONVEYER

The interaction between the muck and the screw conveyer and consequences for the pressure distribution are described in [3]. In the screw conveyer the pressure is released from the mixing chamber pressure to atmospheric pressure. This will result in expansion of the foam, higher porosity of the soil and thus a decreasing yield stress from the mixing chamber to the top of the screw conveyer.

7 CONSEQUENCES FOR MODELLING

Various aspects are shown of TBM-foam-soil interaction. Not all these aspects have to be taken into account in a numerical model. What has to be taken into account depends on the aim of the numerical modelling. The groundwater flow in front of the TBM is essential if the stability of the tunnel face is an issue [9]. The pressures and pressure distribution in the mixing chamber may also be of importance for the stability of the tunnel face, but it is possible that, especially when grain stresses are present, the pressures in the mixing chamber differ from the pressures in front of the cutting head. This aspect needs further investigation.
When modelling the processes in the mixing chamber, the compressibility of the mixture and the influence of the porosity on the yield strength of the mixture are prime parameters. Also for these aspects it is necessary to perform additional quantitative research to determine the compressibility of the mixture, and the yield strength for different porosities. The modelling of the processes at the front of an EPB shield, using foam is therefore far more complicated than the modelling of comparable processes for a slurry shield.

Consequently quite some experimental research will be necessary before a reliable numerical modelling of the processes of an EPB shield is possible. This asks for interaction between numerical and experimental modellers, so that the right questions are asked.

8 CONCLUSIONS

The interaction between TBM, foam and soil is investigated, using the results of model tests and field tests for drilling in saturated sand. This has led to the following conclusions:

1) The interaction between the foam and groundwater flow has a significant influence on the properties of the muck. Only in permeable sand (sand with a permeability comparable to the drilling velocity) it will be possible for the foam to replace the pore water. In more usual conditions, the foam will not replace the pore water resulting in relatively ‘wet’ foam in the muck.

2) The properties of the muck depend on the position in the mixing chamber. Since the muck is compressible it is possible that at some locations grain stresses develop where at lower stresses the soil porosity is higher and the muck has only viscous properties and a yield strength.
3) For an EPB shield, the pressure distribution in the mixing chamber is not hydrostatic. The pressure distribution in front of the cutting wheel is not really known.

4) A combination of experimental and numerical research will be needed to acquire the necessary parameters to model the TBM-foam-soil interaction for an EPB shield.

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Two-scale Investigations on the Rheological Properties of Foam and Particle-laden Foams

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Abstract

In Earth-Pressure-Balance (EPB) tunnelling the excavated ground is used as face support medium to prevent surface settlements by controlling ground movements and ground water flow into the excavation chamber. Cohesionless ground does not exhibit suitable conditions for the use as support medium and has to be treated with conditioning agents that are mainly foams. The conditioned soil, i.e. the soil-foam mixture, requires a certain effective viscosity to ensure a homogeneous pressure transfer onto the tunnel face as well as for an effective transport through the excavation chamber. In order to physically understand the rheological properties of the (added) foam and to comprehend its influence on the soil, it is necessary to retrace the process of soil conditioning. Therefore, laboratory investigations were conducted on two different length scales.

In this contribution, the authors present experimental investigations in the context of soil conditioning in EPB tunnelling. Furthermore, experimental testing methods are applied including substantial boundary conditions. First experimental results from the experiments on the mentioned scales will be presented and compared to basic rheological model functions in order to find an effective flow behaviour.

Keywords: EPB tunnelling, soil conditioning, testing, rheology
1 INTRODUCTION

1.1 Soil Conditioning in EPB Tunnelling

The maintenance of face stability in tunnelling is one of the key factors in order to limit surface settlements. Especially in soft ground tunnelling the support of the tunnel face often represents a challenge while advancing at preferably high rates and low costs.

Earth pressure balance (EPB) shield machines currently are the most frequently applied type of tunnel boring machines in soft ground. The tunnel face is supported by using the excavated soil itself. In doing so, time consuming recycling processes of a secondary face support material and associated monetary expenses can be saved. However, the construction ground in its natural state often does not exhibit required conditions for effective face support. Particularly outside the classical application range, which is soil with at least 30% of fines \( d \leq 0.063 \text{ mm} \) [16], a treatment of the excavated soil might be necessary in order to improve the properties of the support medium. The soil is conditioned by injecting additives through the cutting wheel or the bulk head into the excavation chamber. Typically liquid foam is used but also polymers and slurries of fines may be added. In this way, the risk of surface settlements and uncontrolled groundwater inflow is reduced.

The requirements for effective face support comply with the aim of soil conditioning (cf. [2, 17]):

- adequate flow behaviour (effective viscosity) ensuring material flow
- intrinsic permeability \( k \leq 10^{-5} \text{ m/s} \) preventing uncontrolled groundwater inflow
- low inner friction reducing power demand and tool and component wear
- sufficient bulk modulus compensating pressure fluctuations

As the material is artificially generated through the in-situ mixing process, standardised parameters to evaluate the efficiency or the quality of soil conditioning do not exist. In particular, the rheological parameters of the conditioned soil in the excavation chamber are difficult to determine. A widely-used indicative test of the flowability of the support medium is the slump test (see [2, 14, 16]). The slump value, obtained in a standardised slump cone test [4], originally is an indicator used for the evaluation of the concrete’s workability. The slump value cannot provide valid rheological information but it might be used as an index parameter for the workability of conditioned soils. Therefore, an alternative procedure is sketched in the present approach, which could help to determine the effective rheological properties of foam-conditioned soil.
1.2 Pure Foams and Particle-laden Foams- State-of-the-Art

In order to evaluate the different characteristics of the support medium and of the conditioning agents, it is necessary to first define the essential rheological properties and their impact on the quality regarding effective soil conditioning. Furthermore, suitable, most likely standardised experiments have to be specified that enable the determination of parameters describing the quality of foams and soil-foam-mixtures in the context of soil conditioning. Various researchers focused in their experimental work on different aspects of soil conditioning in the framework of EPB tunnelling. Maidl [11] and Budach [2] carried out experimental investigations on soil conditioning, which led to an extensive knowledge about the procedural influences on foams and the interaction between machine, ground and conditioning agent. Therefrom, recommendations for the tunnelling practice could be derived such as in the form of application ranges. Bezuijen et al. combined in situ experiments, laboratory research and theoretical approaches on the interaction between soil mechanics and soil conditioning relating to face stability and material transport. In [1] for example the influence of the soil permeability on the foam is discussed. Further research on soil conditioning is performed at the University of Turin. Besides the investigation of the conditioning process, one aspect of main interest in scientific studies of Peila et al. [13] is the material extraction from the excavation chamber with a screw conveyor. Therefore, a large-scale pressure tank was used simulating the excavation chamber. Additionally, they work on the conditioning behaviour of soils and rock regarding performance optimisation of EPB machines and tunnel face stability [14, 15].

During laboratory-scale investigations, homogeneous rheological experiments, which include flow curve tests, are performed to get rheological behaviour of materials. Several authors have analysed flow properties of foam. Kroezen et al. [10] used a coaxial Brabender viscometer to perform flow curve tests of a foamed aqueous solution, which contains 10 g/l lauryl sulphate and different concentrations of the thickener Solvitone FN. Moreover, Khan et al. [8] determined foam viscosity as a function of shear rate for three different gas volume fractions (0.97, 0.92, and 0.95) by using plate mode of Rheometrics Mechanical Spectrometer.

However, there is not much known about the rheological behaviour of particle-foam mixtures. Cohen-Addad et al. [3] investigated viscoelastic properties of particle laden foam. For amplitude sweep test, they used stable foam (Gillette shaving cream) and solid particles (glass beads) of increasing solid volume from 0 to 39.8%. A detailed summary of the research on soil conditioning as well as of the development of EPB shields can be found in [2, 6].
2 EXPERIMENTAL SCALING

Experiments are carried out on different scales (micro-scale, macro-scale), which are defined by the testing boundary conditions applied (test methods, testing materials). The test methods can be divided into homogeneous and heterogeneous experiments depending on the shear strain rate distribution.

On the micro-scale, homogeneous rheological experiments are performed on foam and particle-foam mixtures using a rotational rheometer (Anton Paar, MCR 301) in order to obtain down scaled results with high resolution and physical significance. The material conditions on the micro-scale are chosen to be well-defined ensuring reproducibility of the tests: i.e. the cell-size distribution of the foam bubbles and the particle shape. In the first level of testing, the experiments are performed on synthetic support medium ingredients being hollow glass-beads ($3M^{TM}$ Glass Bubbles K1) as particles and shaving foam.

On the macro-scale instead, testing is executed referring to more procedural questions. Here, foams as applied in tunnelling practice plus soils are used for the production of soil-foam mixtures. Due to the larger grain-sizes and bubble-sizes an investigation of the more realistic material needs heterogeneous experiments in order to obtain rheological parameters. Therefore, different test methods are used in the investigations, which are able to determine rheological parameters but with different preciseness in the resolution of results. The methods used are: rotational viscometer (Couette system), concrete rheometers, large-scale test stand simulating excavation chamber conditions ("COSMA"), EPB shield machine (machine data analysis). Both the synthetic material consisting of glass beads and shaving foam as well as the realistic material (soil and tunnelling foam) are investigated using this test methods as far as an investigation is feasible. In this way, a transfer of findings shall be established through the scales leading to valuable information about the rheological material behaviour. Additionally, slump tests are performed with soil-foam-mixtures grading the procedural suitability for the use as support medium, like proposed by [2, 14, 16].

3 MICRO-SCALE INVESTIGATIONS

3.1 Characterization of Foam, Particles and Particle-foam Mixtures

In order to reveal the performance of foam as a conditioning agent in its mixture with soil, the particle-size distribution, the rheological behaviour, the water content
Two-scale Investigations on the Rheological Properties of Foam and Particle-laden Foams

etc. must be known. Due to its time stability and easy accessibility, shaving foam (Gillette, liquid foam) seems to be a good choice for investigations of liquid foam as used on EPB shield machines. In order to understand the basic rheology of soil-foam-mixtures occurring in EPB tunneling, mixtures of foam and hollow glass beads ($3M^{TM}$ Glass Bubbles K1) are rheologically investigated. Since the microstructure of the foam is determining the effective rheological properties of the foam [10], the size, shape and distribution of foam bubbles/cells and particles are characterized in detail by laser-granulometry and light microscopy, Figure 1.

![Figure 1: Probability density function of foam bubble size (left) and glass beads diameter (right). Mean value of foam bubbles ($D = 51.8\mu m$) and mean value of particles ($D = 64.5\mu m$) have the same characteristic length scale. Standard deviation of foam bubbles ($\sigma = 11.6\mu m$) and standard deviation of particles ($\sigma = 30.7\mu m$).](image)

In liquid foams without particles, the foam microstructure plays a dominant role and is determining the physical properties like stability. The stability w.r.t breakage of single foam bubbles is largely determined by the effective drainage properties and the rupture of the thin film. The rate of drainage is directly proportional to the bubble diameter. Hence, the foam stability depends also on the bubble size distribution [9]. As it could be observed in Figure 1, the mean bubble diameter $\bar{D}$ is equal to $51.80\mu m$. The distribution is Gaussian. The particle size distribution of the hollow glass beads exhibits a mean diameter close to the mean diameter of foam bubbles. According to the characterized length scales of the two constituents, which are held fixed in the experimental investigations, it is expected, that the effective rheology of particle-foam mixtures depends on the volume fractions of the constituents, $n^\alpha = dv^\alpha/dv$, where $dv^\alpha$ is the volume of the phase $\rho^\alpha$ in the unit cell with volume $dv$. 
3.2 Rheology of Foam and Particle-foam Mixtures

3.2.1 Experimental Procedure

The rheological behaviour of liquid foam is obtained by performing flow curve and oscillation tests (amplitude sweep & frequency sweep). Before performing the rheological tests, the appropriate geometry types (here plate-plate geometry) must be decided in order to get reasonable and reproducible results. During the experimental investigations, slip effects at the foam-metal plate interface occur, which causes the shear stress to decrease and disrupts the continuity of flow. The slip effect can be decreased by employing a defined roughness with asperities having the same characteristic size as the foam bubbles and particles. Therefore, sand paper was glued on the plates, which forms the necessary rough surface. Extra fine sand paper (P320) turned out to be the most appropriate one and, therefore, has been used for all experiments. The grain size diameter of P320 is 36µm.

3.2.2 Experimental Results

Flow Curve Test

The flow curve test (shear stress vs. shear rate) is used to obtain the yield stress of the sample. Although the flow curve test can easily be performed, the determination of the yield stress of foam precisely is difficult. Generally, it could be observed, that a non-Newtonian rheology of Herschel-Bulkley (HB) type [12] is suitable for acquiring the yield stress by a standard curve fitting procedure. HB is a non-linear model (Eq. 1) and its non-linearity stems from the exponent $\alpha$.

$$\tau = \tau_0 + K \cdot \dot{\gamma}^\alpha$$  \hspace{1cm} (1)

where the material parameters $\tau, \tau_0, K, and \dot{\gamma}$ are shear stress, yield stress, constant and shear rate, respectively. Since it was shown in literature [7] that a HB rheology is able to capture the rheology of liquid foam, this non-Newtonian rheology is applied here, too. In Figure 2, the yield stresses were determined for different volume fractions of foam, $n_f$ ($n_f^1 : 0.77$, $n_f^2 : 0.66$, $n_f^3 : 0.55$, $n_f^4 : 0.45$). After exceeding the yield stress, the mixture exhibits a behaviour similar to power law fluids. As the yield stress is hardly captured with high precision in flow curve experiments, we show in Figure 2, that a simpler power law rheology (Ostwald & de Waele) is also able to fit the viscosity-shear rate profiles of particle-foam mixtures.

$$\mu = K \cdot \dot{\gamma}^{\alpha-1}$$ \hspace{1cm} (2)
where \( \mu \) is the effective viscosity of the particle-foam mixture. A power law fluid is able to capture both situations that are denoted by the value of the power law exponent (\( \alpha \)), which can be either bigger or smaller than \( '1' \). If the flow index \( \alpha > 1 \), the fluid is shear thickening (or thixotropic), whereas it is called shear thinning (or pseudo-plastic) if \( \alpha < 1 \).

![Flow Curve](image1)

![Viscosity–Shear rate](image2)

**Figure 2:** Flow curves (lin-lin axis, left) of particle-foam mixtures with different volume fractions of the constituents. Viscosity vs. shear rate (log-log axis, right) for the same composition.

The effective viscosity decreases with increasing shear rate (Figure 2) and, moreover, the power law exponents of mixtures were measured smaller than 1. Consequently, two models provide an equally good fit and show that the flow of particle-foam mixtures is shear thinning (pseudo-plastic).

**Amplitude Sweep (AS) & Frequency Sweep (FS) Tests**

The dynamic method, which includes oscillation tests (AS & FS), is used for determining viscoelastic behaviour of complex fluids. In the frequency sweep test (FS), a harmonically varying shear stress or shear strain is applied to the system. In contrast, an amplitude sweep is applied in the AS test to the sample with varying range of deformation (amplitude) and constant frequency. The storage and loss modulus vs. deformation curves of the liquid foam can be observed in Figure 3.

The decrease in storage modulus \( G' \) can be explained by an increase of the applied shear stress. When the applied shear stress reaches its highest value, disruption of
Figure 3: Amplitude sweep (AS, left) and frequency sweep (FS, right) of pure foam for strains of 0.1% and 0.5%

the microstructure of the foam and/or particle-foam mixture starts to appear. That is the reason why the storage modulus decreases. The amplitude sweep test provided the critical deformation range, observed as the intersection point of linear and nonlinear viscoelastic areas. Another experiment to determine the effective viscoelastic material behavior is the frequency sweep test. As the critical deformation range was found in the amplitude sweep test, the viscoelastic behavior can be observed in the linear range. The frequency sweep test shows that liquid foam has only solid like behavior at 0.1 % and 0.5 % deformation since the storage modulus (G’) is always bigger than loss modulus (G’"). Moreover, below the critical deformation, the elastic modulus (G’) is often nearly independent of frequency, as it would be expected from a structured or solid-like material [5]. Otherwise, the elastic modulus (G’) is a function of frequency causing the material to behave like fluid.

4 MACRO-SCALE INVESTIGATIONS

In the macro-scale investigations, pre-tests have been performed investigating the adaptability of a coaxial cylinder rotational viscometer (Fann Model 35) for soil-foam-mixtures as occurring in tunnelling applications. Usually, this testing device is used for qualitative rheological analysis (inhomogeneous shear stress/strain rate fields) of oils and bentonite slurries. Subsequently, the procedural boundaries and the findings from first testing of foam and particle-foams in the rotational viscometer are presented, particularly regarding its suitability and significance in results for the mentioned materials.
4.1 Test Preparations and Test Conduction

The viscometer was assembled with a cylindrical bob enclosed by an outer cylinder (Figure 4). The rheological system can be considered as Couette geometry with large shear gap \(R_e/R_i = 1.5\). The outer cylinder resembles the shearing device that transfers its constant rotational speed \(\omega\) onto the fluid in the gap. The inner bob then takes over the shear forces, which are translated into degrees of deflection \(\theta\). The viscometer allows six different rotational speeds: 3 rpm, 6 rpm, 100 rpm, 200 rpm, 300 rpm and 600 rpm. Taking into account the geometrical boundaries of the testing device \((R_e = 1.8415 cm, R_i = 1.2276 cm)\) and based on the system assumption of large gap Couette, the rotational speeds as well as the deflection values can be converted into rheological parameters, i.e. the shear rate \(\dot{\gamma}\) (Eq. 3), the shear stress \(\tau\) (Eq. 4) and the viscosity \(\mu\) (Eq. 5), in relation to \(R_i\):

\[
\dot{\gamma}(R_i) = \frac{(1/15)\pi \cdot R_e^2}{(R_e^2 - R_i^2)} \cdot \omega \quad \text{(3)}
\]

\[
\tau(R_i) = \frac{(k \cdot \theta) / 2\pi \cdot L \cdot R_i^2}{R_i^2} \quad \text{(4)}
\]

\[
\mu(R_i) = \frac{\tau(R_i)}{\dot{\gamma}(R_i)} \quad \text{(5)}
\]

where \(k\) is a torsion constant, \(\theta\) the recorded degrees of deflection and \(L\) the height of the inner cylinder.

![Figure 4: Sketch of the system behaviour of the rotational shear viscometer](image)

In the present experimental investigations foams and particle-foam mixtures were used similar to the investigations discussed in the previous chapter. In doing so, a comparison of the results of both scales is more feasible. The volume fractions of foam in the samples were: \(n^{f_1} : 1.00\), \(n^{f_2} : 0.50\). The sample of 350 ml was added into the sample container and placed in the viscometer. Immediately after engaging,
the viscometer the maximum deflexion was recorded. For each test a new sample was prepared in order to maintain undisturbed conditions.

4.2 Test Results

In Figure 5 the shear stress / shear strain rate results of the macro-scale experiments are shown in comparison to flow curve test results with the same materials according to the procedural description in chapter 3. The data shows an increase in shear stress the higher the shear rate, which qualitatively is the same behaviour as on the micro-scale. In comparison, the shear stress of the particle-foam-mixture is higher than of the foam for the macro-scale results, whereas the micro-scale samples show contrarian results. However, the macro-scale results are close to the range of values obtained from the homogeneous experiment on the micro-scale.

![Figure 5](image-url)

**Figure 5:** Comparative presentation of rheometry results (micro- and macro-scale)

CONCLUSION AND OUTLOOK

Laboratory experiments were carried out on liquid foams and particle-foam-mixtures using two test methods with different system assumptions in order to determine rheological parameters. The results obtained in the rotational rheometer under homogeneous shear stress strain rates could be approximated with established non-Newtonian rheological models such as Herschel-Bulkley or power-law. The tested materials exhibit a shear-thinning flow behaviour. Qualitatively similar observations
could be found with a rotational viscometer that provides the possibility to test ma-
terial, which can be faced in EPB tunnelling.
In future investigations, foam and particles will be replaced by foams as used on EPB
shield machines and by soils of different grain-size distributions. And, furthermore,
tests will be executed with same materials but using also other test methods in order
to obtain further results about the rheology of foams and particle-foams on different
scales.
Finally, the findings elaborated shall lead to both a rheological flow description of
foam and particle-foams and to fundamental knowledge about the material behaviour
in the excavation chamber.

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Computational Models for Safety and Security
Application of Computational Fluid Dynamics for Simulating Building Fires in Subway System: Current Situation

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Abstract

Subway systems with many long tunnels and stations with large complicated building structures were built all over the world in the past two decades. Those buildings might have difficulties in complying with fire regulations. Performance-based design has to be used. Fire models based on Computational Fluid Dynamics (CFD) are widely used in the associated fire hazard assessment and system design. However, most of the field models are not properly validated due to the high cost and other difficulties in conducting large-scale fire tests. Consequently, performance-based designs with those models are always challenged by the Authority. Common questions asked in justifying the mistakes made in projects based on CFD simulations for such application will be discussed in this paper.

Keywords: Subway system, tunnels, stations, fire hazards
INTRODUCTION

With the rapid development of economics in the Far East, many big construction projects were found. The new designs have difficulties to comply with the fire codes. Performance-based design (PBD) was then applied to determine fire safety provisions, particularly for underground subway stations in urban areas without much space [1]. However, due to resources limitation, free software of Computational Fluid Dynamics (CFD) [2] was applied in hazard assessment in many construction projects. No full-scale burning tests were carried out to justify the CFD predicted results. Authorities having jurisdictions (AHJ) are now more knowledgeable in fire science and engineering. Many officers are well-trained and possess a master degree in fire engineering. The free CFD software Fire Dynamics Simulator (FDS) is commonly used to study fire-driven fluid flow. It was developed [2] by National Institute of Standards and Technology (NIST) and used in solving practical fire problems. Although consulting engineers believe in CFD-FDS predictions, a very tight inspection scheme was implemented on new project applications based on CFD. Common mistakes made in PBD projects based on CFD as pointed out by different parties including fire research workers, users, fire officers and journal paper reviewers will be discussed in this paper.

Problems [1] identified by fire officers in using CFD in the Far East are:

- Mainly accepted for smoke control design, many doubts on fire simulation such as open kitchen in small residential flats of very tall buildings and wood houses.
- FDS was commonly used, but not yet justified for application in huge space with tall halls such as public transport interchanges.
- Air pressure, turbulence parameters seldom presented.
- Full-scale burning tests on typical scenarios with similar conditions are necessary.
- Hot smoke tests required in atria of irregular shape or taller than 12 m in some places.

Criticisms [3] raised by fire research workers and journal paper reviewers on CFD results are:
• Application to simulate fire needs to watch because fire phenomena are substantially three-dimensional and very unstable in the flow and temperature fields of the buildings concerned.
• Three-dimensionality and instability of the fire-induced flow fields are not fully discussed.
• Ability of CFD to resolve the flow in the turbulent fire plume.
• Ability of CFD to resolve the turbulent exchange flow through the opening.

2 HEAT RELEASE RATE

In fire safety design, the most important parameter is the heat release rate (HRR), which is the single most important variable in characterizing the “flammability” of products and their consequent fire hazard. It gives information on fire size, fire growth rate, available egress time and suppression system impact. The potential for ignition of nearby items, flashover potential in a room, and the amount of water needed to extinguish the fire can be estimated. The evolution of HRR with time becomes the most important input variable [4] which must be estimated properly for fire simulations.

Heat release rate of burning a train should be studied [5] carefully in fire hazard assessment. In general, a small accidental fire was assumed to break out in an empty train. In an electric car, combustibles mainly consist of transformer oil, so it gives a very low heat release rate. Even a low value up to 6.5 MW was originally proposed in the draft version of the new fire safety (FS) code of Hong Kong [6]. The fire safety provisions for a train station – which were designed and derived from a small fire scenario – can only protect against accidental fires when the movable fire load is low, i.e. the absence of large amount of luggage. This assumption should take into account the luggage carried by passengers. In Hong Kong, there are many parallel traders in some stations. Although a 32 kg luggage limit is imposed [7], a fire that breaks out in a train with so much luggage will be very hazardous. The HRR should be increased to 22 MW as proposed in the final version of the FS code [6]. It should be noted that a heavy goods vehicle is believed to emit 100 MW, as specified in NFPA 502-2011 [8].

HRR used to be predicted by CFD without experimental justification. There are even no resources allocated in some places like Hong Kong to compile a database of HRR for local products including train compartments of the railway system [5] as in Japan, China and Korea.
3 FREE OPEN BOUNDARY CONDITION

In applying CFD to building fire hazard assessment, there are always windows and doors open to outside. Bi-directional air flows were observed in experimental studies, with hot gas flowing out and cool air coming into the room. The boundary conditions of the flow parameters, particularly pressure, have to be specified carefully. There are empirical correlations relating the pressure profiles across the vertical openings such as windows or doors, under different room geometries, heat release rates of the fires, and opening sizes. However, such boundary conditions might not give proper specification. A better approach is to extend the computing domains outside, as pointed out years ago by Galea and associates [9].

In FDS simulations, the opening boundary condition was taken as a passive opening to the outside like a door or window. An OPEN boundary was set on the exterior boundary of the computational domain. The outflow of opening boundary condition for the momentum equation was determined by simplifying the pressure in terms of the velocity vector, pressure perturbation and density.

The pressure is set to the ambient pressure by the user, which is defaulted to zero. For inflow, the fluid element on the boundary has been assumed to be accelerating from the state along a streamline. The flow field was calculated by Bernoulli equation. It assumes that pressure is zero infinitely far away. At the boundary between two grids, the pressure boundary condition is similar to that at an external open boundary. But pressure is taken from the adjacent grid where the flow is incoming. Outflow and inflow are separately set on the opening vent. However, it cannot simulate the real free boundary condition with fluid flow on the boundary freely because the position of neutral plane cannot be determined. After extending the model, the original boundary can be seen as an assumed free boundary.

There are different views on specifying free open boundary conditions in applying CFD and these views should be watched carefully. There have been some attempts to solve the problem with free boundary. Markatos and co-workers [10] extended the flow domain to the ‘free boundary’ region outside the doorway when studying the smoke flow in enclosures and obtained results that agreed reasonably with experimental data. Schaelin and co-workers [11] pointed out that extending the computing domains outside was a better approach when simulating plume flow. Galea and Markatos [9] pointed out in their case study on simulating fire development in an aircraft that it is desirable to extend the solution domain outside the fire compartment in order to find physically realistic behaviour in the vicinity of the open
doors. Some pioneering work on fire modelling [9,10] demonstrated that the flow pattern in the vicinity of doorway was entirely different if the free boundary had not been extended sufficiently. However, in applying FDS, Hadjisophocleous and Ko [12] suggested that the impact of the open boundary at the exterior of the computational domain was minor when the boundary had been extended up to 2 m outside a geometry of width 10 m. Therefore, it may not be necessary to extend the computational domain to some distance beyond the opening to obtain good results while using FDS version 4.07. They also pointed out that this situation is rather complicated and should be evaluated for different cases. Note that different results were predicted by different simulation software packages. Further, strongly buoyant flow should be predicted more carefully. It is useful to compare the prediction with the Reynolds Averaged Navier Stokes equation method (RANS). However, this is quite labour intensive to develop a new CFD software and very expensive to purchase commercial CFD license. Earlier studies on thermal plume suggested that results are similar.

4 FUNCTIONAL ANALYSIS

In order to quantify this comparison precisely, functional analysis proposed on zone modeling was applied to evaluate the CFD results [13]. Transient predicted and measured data are expressed as vectors $\vec{P}$ and $\vec{M}$. The Euclidean norm and secant inner product cosine between $\vec{P}$ and $\vec{M}$ are calculated:

$$\text{Norm} = \frac{\|\vec{P} - \vec{M}\|}{\|\vec{P}\|}$$

$$\text{Cosine} = \frac{\langle \vec{P}, \vec{M} \rangle}{\|\vec{P}\| \|\vec{M}\|}$$

Values of norm and cosine are used to compare CFD predicted results with measured data. Values of norm should be 0, and cosine should be close to 1 for good agreement.
5 GRID SIZE VARIATION

Grid size denoted by $\delta x$, $\delta y$ and $\delta z$ is the most important numerical parameter in CFD simulations. Quality of the mesh was assessed by a non-dimensional parameter rather than an absolute mesh cell size. For simulations involving buoyant plumes, a measure of how well the flow field is resolved is given by the non-dimensional expression on $R^*$:

$$R^* = \frac{\max(\delta x, \delta y, \delta z)}{D'}$$

(3)

In large eddy simulation (LES), transient larger eddies are solved and smaller unresolvable eddies are modeled with a time averaged component and a fluctuating perturbation about that average [3]. The fineness of the numerical grid would determine the size of eddies that can be solved. The nominal size of the mesh cell $\delta x$ and the characteristic fire diameter $D^*$ given by fire power $\dot{Q}$, air density $\rho_\infty$, temperature $T_\infty$, specific heat of air $C_p$ and gravitational acceleration $g$ are important in simulating buoyant plumes [10].

$$D' = \left(\frac{\dot{Q}}{\rho_\infty C_p T_\infty \sqrt{g}}\right)^{2/5}$$

(4)

The ratio $D'^*/\delta x$ can be taken as the number of computational cells spanning the characteristic diameter of the fire. A refined grid system can improve the accuracy of results of LES. It was suggested [4] that the value of $D'^*/\delta X$ should be larger than 10 to guarantee a reliable operation of FDS. Zou and Chow [14] got reasonable FDS predictions of temperature and radiation data with $D'^*/\delta X$ of about 14. Study by Hietaniemi et al. [15] on pool fire showed that having at least 20 cells within the diameter of the pool would give predictions agreed with experiments. Validation study given by Hill et al. [16] suggests that $R^*$ should be between 0.06 to 0.25. It is also found that the optimum resolution of a pool fire simulation $R^*$ is around 0.05 by Ma and Quintiere [17], the centerline temperature and velocity in the non-combusting region is also predicted well. However, for the non-combustion flow field prediction, FDS did not give any suggestion about the grid size.
6 EXAMPLE CASES

The most important part is on evacuation study in crowded halls with ASET-RSET approach [18]. It is criticized to be a FLAWED Concept. Evacuation was studied by the timeline analysis with Available Safe Egress Time (ASET) and Required Safe Egress Time (RSET) calculated. The safety margin SM is:

\[ \text{SM} = \text{ASET} - \text{RSET} \]

Problems identified in Southeast Asia in projects on estimating ASET are:

- Predicted by fire models, with very few Validation & Verification works for large halls.
- Small fire scenarios without experimental justification.
- Seldom used big fires.

Tenability limits commonly used (following partly CIBSE Guide E 2010 [18] are:

- Lift safety for occupants and firemen
- Radiative heat flux: 2.5 kWm\(^2\)
- Smoke layer temperature: 120 °C
- Smoke layer interface height: 2.5 m
- Carbon monoxide concentration [CO]: 6000 to 8000 ppm for 5 minutes exposure

Other toxicants, irritants and asphyxiants were not yet specified because there is no fire engineering tool available to predict the chemical species liberated from combustion accurately. There are far too many chemical reactions involved in the burning process [19]. However, toxicity effect of many toxic gases such as hydrogen chloride HCL is very severe. Neglecting them will give problems in estimating ASET. Such problems were even found in the draft new code on building fire safety [6] issued by the Hong Kong authority for consultation in September 2011, and implemented in April 2012. Again, only heat and [CO] are specified. All heat and toxic gases are assumed to be within the stratified smoke layer at high levels. This would not hold for tall atria. These approaches are only applicable for those fuels emitting only carbon monoxide upon burning.

If toxicity of fire gases is included in the tenability limit, ASET is highly reduced.
Problems on RSET are:

- RSET depends on human behaviour.
- To the best of knowledge, no evacuation software with in-depth validation applicable to Hong Kong was reported in advanced journals.
- There are not yet systematic studies on human behaviours in an accidental fire in Hong Kong.
- Not even in many places using evacuation software.
- Criticized to be ‘Robotic Motion’ [20].
- Crowded conditions not included.

RSET will be much longer under crowded conditions [21]. The value of ASET can be plotted against RSET as in Fig. 1. A safe point P can be unsafe point P₁.

![Timeline approach](image)

**Figure 1:** Timeline approach

7 CONCLUSION

Consulting engineers pushed hard to promote the use of CFD, and always believe in the CFD-FDS predictions. However, in-depth verification and validation work is necessary. Heat release rate of a room pool fire was studied by FDS version 5 with different free open boundary conditions. A sensitivity study was carried out to compare predicted results with the experimental results under different grid systems. Predictions with the coarse grid deviated more away from the experimental data. The predictions with a medium grid were found to agree with the experiment well, and
selected for this study. In general, the prediction of HRR by FDS is good. Although the computational domain outside did not give much difference to the predicted HRR, the predicted velocity patterns are very different.

Free opening boundary condition should be evaluated before applying in CFD simulations, especially when the combustion process is included while simulating fires in tall or supertall buildings. Extending the computational domain to a sufficient distance beyond the opening is recommended.

The effects of ventilation factor on HRR and mass flow rate through the door opening were analyzed by FDS. On the five fire scenarios with different ventilation conditions investigated in this paper, air intake rate might be higher to give more oxygen for combustion under higher ventilation factor. Linear correlations can be fitted for the relationship between the intake airflow rate and the ventilation factor, with results very close to those equations derived from simple hydraulic theory. Averaged predicted HRR by the liquid fuel model in FDS did not agree with the experimental results. The burning rate in a room fire is strongly affected by radiative heat feedback which might not be well simulated in the FDS model. This result was incorporated with other studies. It is suggested that liquid fuel model should be applied carefully to engineering application. Further related work on validation and verification of liquid fuel model in FDS should be conducted. As raised by Chen [22], it is difficult to have the whole set of CFD predicted results agreed with experiments. However, macroscopic flow parameters predicted by CFD are very useful [23].

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Comparative Performance of Tunnel Linings under Blast Loading

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Abstract

The present study has compared the performance of different tunnel lining materials under internal blast loading. Three-dimensional finite element analyses have been performed in order to simulate explosions in tunnels using the finite element software Abaqus applying the explicit dynamic procedure. A tunnel in sandy soil has been considered for the analyses. The performance of single layered steel, plain concrete (PC) and steel fibre reinforced concrete (SFRC) linings and sandwich steel-dytherm foam-steel (SDS) and steel-polyurethane foam-steel (SPS) panel linings under blast loading have been examined. The resulting displacements in soil and lining have been recorded and evaluated. For material modelling, Drucker-Prager plasticity for soil, crushable-foam plasticity for foams, Johnson-Cook plasticity for steel and concrete damaged plasticity model for concrete and SFRC have been used. Strain rate dependent material parameters have been used for all lining materials and soil. Internal blast loading of 10 kg TNT has been simulated using the pressure-time curve obtained by a CFD-calculation. Based on these simulations, it has been observed that SDS and SPS sandwich panel linings exhibit much less displacement in the soil under blast loading as compared to the PC and SFRC linings.

\textbf{Keywords:} Blast Loading, Sandwich Foam Panel, Soil, Strain Rate, Tunnel Lining.
1 INTRODUCTION

Today’s civil infrastructure depends largely on underground tunnels. Hence, the increasing terror attacks in the last decade have necessitated the need for blast resistant design of underground structures. Explosion inside an underground structure may lead to the damage of steel or concrete linings and the heavy support systems; the subsequent collapse of surrounding soil or rock-mass may cause loss of lives and properties. Preventive measures should therefore be adapted to protect underground structures from collapse under internal blast loading. In order to safeguard civil engineering facilities under blast induced extreme shock wave load, it is essential to use shock absorbing lining materials in construction.

In the literature, very few studies have been performed to understand the effect of blast loading on underground structures. Subway tunnels under explosive load have been analysed in [1] using finite element (FE) method. However, their analysis did not consider the high strain rate behavior of soils under explosive loading. The effect of blast loading on tunnel considering the high loading rate behavior of soils has been studied in [2]. Their study considered only concrete lining in the tunnel. The use of shock absorbing material as lining of the underground structures has not been investigated so far. The effect of alternative materials with varying density and stiffness in absorbing blast induced shock loading have been investigated in [3] for above-ground structures. They observed that low-density, low stiffness soft polyurethane foam reduces the reflection of blast waves significantly. The performance of steel fibre reinforced concrete (SFRC) under high dynamic load has been investigated in [4]. It was observed that SFRC shows better performance as compared to plain concrete (PC) under high dynamic load.

The objectives of the present study are- (1) to understand the effect of blast loading on an underground tunnel using the finite element (FE) method and (2) to compare the performance of steel, plain concrete (PC), steel fibre reinforced concrete (SFRC) and sandwich steel-dytherm foam-steel (SDS) and steel-polyurethane foam-steel (SPS) panel linings under blast loading. Three dimensional (3D) FE modelling of underground tunnels with different types of tunnel lining materials have been performed using the FE software Abaqus/Explicit [5]. The stresses and displacement at the soil-lining interface have been compared.
2 THREE DIMENSIONAL FINITE ELEMENT MODELLING

Herein, a tunnel of 5 m diameter in sandy soil is considered 2 m below the ground surface. Different tunnel lining materials have been modelled, e.g., steel, plain concrete, steel fibre reinforced concrete and sandwich panel linings, e.g., steel-dytherm-steel and steel-polyurethane-steel. Table 1 presents the analysis cases considered in the present investigation. The thicknesses of lining \((t_L)\) considered herein are 20 mm and 50 mm for steel plates (named as Steel20 and Steel50) and 100 mm and 200 mm for PC (named as PC100, PC200) and SFRC (named as SFRC100 and SFRC200) slabs. The concrete slabs are modelled using M25 grade of concrete (with considered compressive strength of 25 MPa) in the PC and SFRC slabs. The SFRC slabs are assumed to consist of 3% steel fibre by volume as 3% steel fibre by volume exhibits higher strength as compared to 1% and 2% steel fibre by volume [6]. The sandwich panels are constructed of a steel face sheet of 10 mm thickness \((t_{L, fs})\), a foam core with thickness of core \((t_{L, c})\) of 100 mm and 200 mm and a steel back sheet of 10 mm thickness \((t_{L, bs})\). In addition to this, a sandwich panel with steel face sheet of 5 mm thickness, a foam core with thickness of core \((t_{L, c})\) 200 mm and a steel back sheet of 5 mm thickness is also considered. Depending on the thicknesses of the face sheet, core and back sheet, the SDS and SPS sandwich panels are named as SDS10-100-10, SDS10-200-10, SDS5-200-5 and SPS10-100-10, SPS10-200-10, SPS5-200-5.

The curved steel plates are modelled using four node reduced integration shell elements with finite membrane strains (S4R). The curved PC and SFRC slabs are modelled using the three dimensional eight node C3D8R elements. The three

<table>
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<th>Types of Analyses</th>
<th>Plate/Slab Types</th>
<th>Thickness ((t_L)) (mm)</th>
<th>Thickness of core ((t_{L, c})) (mm)</th>
<th>Thickness of back sheet ((t_{L, bs})) (mm)</th>
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<tbody>
<tr>
<td>Steel Types</td>
<td>1. Steel</td>
<td>20, 50</td>
<td>-</td>
<td>-</td>
<td>Steel20, Steel50</td>
</tr>
<tr>
<td></td>
<td>2. Plain Concrete (PC)</td>
<td>100, 200</td>
<td>-</td>
<td>-</td>
<td>PC100, PC200</td>
</tr>
<tr>
<td></td>
<td>3. Steel Fibre Reinforced Concrete (SFRC)</td>
<td>100, 200</td>
<td>-</td>
<td>-</td>
<td>SFRC100, SFRC200</td>
</tr>
<tr>
<td>Sandwich Panel Types</td>
<td>Thickness of face sheet ((t_{L, fs})) (mm)</td>
<td>Thickness of core ((t_{L, c})) (mm)</td>
<td>Thickness of back sheet ((t_{L, bs})) (mm)</td>
<td>SDS10-100-10, SDS10-200-10, SDS5-200-5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>4. Steel-Dytherm-Steel (SDS)</td>
<td>10</td>
<td>100</td>
<td>10</td>
<td>SPS10-100-10, SPS10-200-10, SPS5-200-5</td>
</tr>
<tr>
<td></td>
<td>5. Steel-Polyurethane-Steel (SPS)</td>
<td>5</td>
<td>200</td>
<td>5</td>
<td>-</td>
</tr>
</tbody>
</table>

Table 1: Cases considered for analysis.
A dimensional model of the sandwich panels consist of a curved steel face sheet and a curved steel back sheet, both made up with S4R elements and a curved sandwich core between the face sheet and the back sheet modelled using the C3D8R elements. The interfaces between different layers of the sandwich panels are considered to be in perfect contact with hard contact in the normal direction and frictional contact in the tangential direction. In order to reduce the analysis time, only half of the model has been considered taking advantage of the geometric symmetry. The far field, the bottom boundaries and the two side boundaries have been considered clamped (fixed). For all the analyses, element size of 0.01 m has been decided based on the mesh convergence study. Figure 1(a) shows a typical tunnel mesh considered in the present investigation.

Blast loading gives rise to high strain rates in any material. Hence, strain rate dependent material constitutive models have been used for all the materials in the present investigations. Table 2 summarises the physical properties of all materials, e.g., Young’s modulus, \( E \), Poisson’s ratio, \( \nu \), density, \( \rho \), the material constitutive models used in the present study and the constitutive model parameters. The rate dependent elastic-plastic behaviour of steel is defined by the Johnson-Cook plasticity constitutive model \( [7] \). Model parameters \( A, B, C, m \) and \( n \) are the material constants as described in Table 2. The parameters are taken from \( [8] \). The effects of temperature are ignored in the present analyses.

**Table 2:** Mechanical properties for different materials.

<table>
<thead>
<tr>
<th>No.</th>
<th>Material Description</th>
<th>( \rho ) (kg/m(^3))</th>
<th>( E ) (GPa)</th>
<th>( \nu )</th>
<th>Material Model (Strain Rate Dependent)</th>
<th>Material Model Parameters</th>
<th>Refs.</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Steel</td>
<td>7800</td>
<td>210</td>
<td>0.3</td>
<td>Johnson-Cook</td>
<td>( A = 360 \text{ MPa}, B = 635 \text{ MPa}, n = 0.114, C = 0.075 )</td>
<td>[8]</td>
</tr>
<tr>
<td>2.</td>
<td>Plain concrete (M25)</td>
<td>2643</td>
<td>31</td>
<td>0.2</td>
<td>Concrete Damaged</td>
<td>( \psi = 36^\circ, \sigma_{c,yield} = 2.0 \text{ MPa}, \sigma_{t,yield} = 0.52 \text{ MPa} )</td>
<td>[9]</td>
</tr>
<tr>
<td>3.</td>
<td>Steel Fibre Reinforced Concrete</td>
<td>4583</td>
<td>42</td>
<td>0.2</td>
<td>Concrete Damaged</td>
<td>( \psi = 36^\circ, \sigma_{c,yield} = 4.0 \text{ MPa}, \sigma_{t,yield} = 1.0 \text{ MPa} )</td>
<td>[10], [11]</td>
</tr>
<tr>
<td>4.</td>
<td>Polyurethane</td>
<td>60</td>
<td>0.0075</td>
<td>0</td>
<td>Crushable Foam</td>
<td>( \sigma_{c,yield} = 0.2 \text{ MPa} )</td>
<td>[8], [12]</td>
</tr>
<tr>
<td>5.</td>
<td>Dytherm</td>
<td>100</td>
<td>0.003</td>
<td>0</td>
<td>Crushable Foam</td>
<td>( \sigma_{c,yield} = 0.22 \text{ MPa} )</td>
<td>[8]</td>
</tr>
<tr>
<td>6.</td>
<td>Sand</td>
<td>1800</td>
<td>0.05</td>
<td>0.2</td>
<td>Drucker-Prager</td>
<td>( \sigma_{c,yield} = 0.1 \text{ MPa} )</td>
<td>[14]</td>
</tr>
</tbody>
</table>
The PC slabs are modelled using the strain rate dependent concrete damaged plasticity material model. The SFRC is modelled as an equivalent continuum in the present study using the strain rate dependent concrete damaged plasticity model. Table 2 presents the physical properties, the dilation angle, $\psi$, compression yield strength, $\sigma_c,\text{yield}$ and tensile yield strength, $\sigma_t,\text{yield}$ of PC and SFRC. The strain rate dependent stress-strain curves for concrete are obtained from [9]. The stress-strain curves of the SFRC under compression and tension loading are obtained from [10] and [11]. The dytherm and polyurethane foams are modelled using the strain rate dependent crushable foam constitutive model. The physical properties of the foams and the yield strength in compression are given in Table 2. The stress-strain curves for the dytherm and polyurethane foams are obtained from [8] and [12]. Sand, which is a granular material, has been modelled using the Drucker-Prager material model as an equivalent continuum in the present study, which is extensively used in the literature [13]. Table 2 presents the physical properties of sand. The strain rate dependent stress-strain curves for sand are obtained from the standard literature [14]. Material and numerical damping are not used in any of the dynamic analyses. The strain rate dependence of the foams and sand are included in the model by defining the increase of dynamic yield strength with respect to the static yield strength, i.e. the dynamic increase factor (DIF) with the increase in strain rate. The DIF of different materials calculated from the strain rate dependent stress-strain curves are presented in Table 3.

Table 3: Dynamic increase factor (DIF) for different materials.

<table>
<thead>
<tr>
<th>Sr. No.</th>
<th>Material Description</th>
<th>Strain Rate (( \dot{\varepsilon} )) (sec)</th>
<th>Dynamic Increase Factor (DIF)</th>
<th>References</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Dytherm</td>
<td>950</td>
<td>4</td>
<td>[8], [12]</td>
</tr>
<tr>
<td>2.</td>
<td>Polyurethane</td>
<td>2300</td>
<td>10</td>
<td></td>
</tr>
<tr>
<td>3.</td>
<td>Sand</td>
<td>100, 200</td>
<td>1.37, 3</td>
<td>[14]</td>
</tr>
</tbody>
</table>

Internal blast loading of 10 kg TNT weight (W) placed at 2.5 m distance (R) and 1.16 m/kg$^{1/3}$ scaled distance (Z) has been simulated using the pressure-time history curve obtained by a CFD-calculation. Figure 1(b) shows the blast load profile inside the tunnel. In the enlarged figure in the inset, three peaks are observed in 0.0025 sec. Peak blast overpressure is 2.9 MPa. The blast loading is applied uniformly on the inner surface of the lining.
3 SIMULATION RESULTS AND DISCUSSION

Figures 2(a) to 2(d) show the soil displacement ($\Delta_s$) time history plots for different types of tunnel linings. The displacement values exhibited in soil at the soil-lining interface at the side of the tunnel, as shown in Figure 2(a), are compared. From the figures it is observed that for the tunnels with PC and SFRC slab linings, the maximum displacement is exhibited in soil. The steel plate linings lead to nearly similar displacement in soil as that observed for the SDS and the SPS sandwich panels; the displacement is almost 50% lower than the PC and SFRC slab linings. The SDS and SPS panel linings with different core thicknesses exhibit almost similar displacement in soil which may be attributed to the densification of foams under blast loading.

Figure 2: Time history of radial displacement at the soil-lining interface.
Figure 2 (contd.): Time history of radial displacement at the soil-lining interface.

Figure 3 summarizes the peak displacement in soil for different types of tunnel linings. Among all the linings, PC100 shows the maximum peak displacement followed by SFRC100. Minimum peak displacement is observed for SDS10-200-10 sandwich panel. The SPS sandwich panels exhibit nearly similar peak displacement as that for steel.

CONCLUSIONS

The present study has compared the performance of different tunnel lining materials under internal blast loading. Three-dimensional finite element analyses have been performed in order to simulate explosions in tunnels using the finite element software Abaqus applying the explicit dynamic procedure. A tunnel in sandy soil has been considered for the analyses. The performance of single layered steel, plain concrete and steel fibre reinforced concrete linings and
sandwich steel-dytherm foam-steel and steel-polyurethane foam-steel panel linings under blast loading have been examined. It is observed that the PC and SFRC slabs show the maximum displacement. The steel plates show nearly similar displacement as observed for the SDS and the SPS sandwich panels. Minimum peak displacement is observed for SDS10-200-10 sandwich panel which proves the efficiency of these panels in blast load mitigation.

REFERENCES

Complex Variable Solutions for Ground Movement Induced by Oval-deforming Twin Tunnels

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Abstract

A deforming tunnel may result in movements of the surrounding ground. However, if two tunnels are constructed close together, some degree of interaction between the two tunnels will take place. Complex variable solutions are presented in this paper for determining deformation and stress around twin parallel tunnels in an elastic half-plane. The oval shaped ground deformation patterns are imposed as the boundary condition around each tunnel opening and the interaction effects between the two tunnels are approximated by alternately satisfying the boundary conditions on each of the two tunnels. Comparison among the presented solutions and field results is conducted. It is noticed that the interaction effects between twin tunnels can be approximated and an oval deforming boundary condition around twin tunnels indeed can give a more realistic prediction for ground deformation.

Keywords: Elasticity, Ground movements, Twin tunnels, Deforming pattern
1 INTRODUCTION

Underground construction may result in movements of the surrounding ground. In engineering, it is of considerable interest to establish solutions of displacements and stresses around the tunnel. A full analytical solution for a circular tunnel in an elastic half plane using complex variable analysis has been given by Verruijt [1]. This solution can give the displacements and the stresses throughout the entire half plane, without any approximation by means of an infinite power series in the complex plane, and it is applicable to arbitrary values of Poisson’s ratio. On the other hand, a number of authors have investigated the effects of an oval shaped ground loss pattern to the calculation of tunnelling induced ground deformation by imposing it as a boundary condition at the tunnel opening [2; 3; 4; 5; 6]. It is manifested that the oval shaped ground deformation pattern result in positive effects on predicting tunnelling induced ground deformation. Very limited works have been carried out using analytical solutions to study the ground movement induced by construction of twin tunnels and the possible interaction effects between them. With the boundary condition of a given radial stresses, Kooi and Verruijt [7] make an attempt to investigate twin tunnel interaction by using a relatively simple iteration procedure with the elastic solution given by Timoshenko and Gooder [8]. Yan et al [9] extended this attempt to the complex variable solution for shallow twin parallel tunnels with the uniform radial stress boundary condition. However, the tunnelling induced ground movement is often characterized by the expression of “ground loss”, correlated to the percentage of excavated volume of tunnel per unit length. Hence, radial displacement boundary condition would attract much great interest for the engineers. This paper addresses the problem of the twin-parallel tunnels, constructed close together, in an elastic half plane for the case of ground loss. The problem is proposed to incorporate both the oval shaped ground loss pattern and the interaction effects between the two tunnels into the complex variable solutions suggested by Verruijt [1] using the ‘Schwarz’ alternating method’ introduced by Kooi and Verruijt [7].

2 GROUND LOSS PATTERNS

In general, the ground loss is expressed as a percentage of the ratio of the surface settlement trough volume and the tunnel section volume per unit length. In this paper, the tunnel section is assumed to be undrained conditions and deformations due to time dependent consolidation and creep are neglected. Hence, the ground loss could be expressed as equivalent lost volume with respect to the tunnel cross section. In practice, as pointed out by Rowe and Kack [10] and Loganathan and Poulos [2], the
radial ground movement around the tunnel is not uniform. Taken the ground deformation patterns suggested by Park [4; 5] the following four possible boundary conditions were considered for the twin tunnels in this paper (shown in Figure 1, with polar coordinate shown in Figure 2), by satisfying the prescribed radial displacements at the four points around the tunnel opening and giving expressions as

**Figure 1:** Boundary conditions of prescribed displacement, after Park (2004,2005)

B.C.-1: \( u_r = -u_i = -u_0 \) (1)
B.C.-3: \( u_r = -u_3 \left( 1 + \sin \theta' - 0.5 \cos^2 \theta' \right) \) (2)
B.C.-4: \( u_r = -0.25u_3 \left( 5 + 3 \sin \theta' - 3 \cos^2 \theta' \right) \) (3)
B.C.-5: \( u_r = -0.25u_5 \left( 5 + 3 \sin \theta' - \cos^2 \theta' \right) \) (4)

It should be noted that B.C.-1 is that for uniform radial displacement; B.C.s-3 and -4 are for the ground deformation with a gap [3] \( g = 2u_4 \) at the crown recommended by Wang et al [6]; B.C.-5 is equivalent to the modified oval shaped ground deformation proposed by Loganathan and Poulos [2].

### 3 THE COMPLEX VARIABLE SOLUTIONS

In the complex variable method elaborated by Verruijt [1], the components of the stress and displacement around the circular tunnel can be expressed in terms of two analytical functions \( \varphi(z) \) and \( \psi(z) \) in z-plane (see Figure 2), and the upper boundary of free surface and the tunnel opening can be expressed as

\[
z = \bar{z} : \varphi(z) + z\varphi(z) + \psi(z) = 0 \tag{5}
\]

\[
|z + ih| = r_0 : 2G \left( \mu_\kappa + i\mu_\kappa \right) = \kappa \varphi(z) - z\varphi'(z) - \psi(z) \tag{6}
\]

where \( G \) is shear modulus of the elastic material in the half plane; \( \kappa = 3 - 4\nu \) for plane strain problems, and \( \nu \) is Poisson’s ratio.
The problem is solved through the use of a conformal mapping of the $z$-plane onto a circular ring (region $\gamma$) in the $\zeta$-plane (shown in Figure 2). The circle $|\zeta|=1$ corresponds to the upper boundary of the free surface and the circle $|\zeta|=\alpha$ corresponds to the tunnel boundary. The conformal transformation is written as

$$z = \omega(\zeta) = -i \frac{1 - \alpha^2}{1 + \alpha^2} 1 + \zeta$$

where $\alpha = (h - \sqrt{h^2 - r^2})/h$.

Then the functions $\varphi$ and $\psi$ can be written as Laurent expansion in terms of $\zeta$. Considering $|\zeta| = \alpha \sigma$ and $\sigma = \exp(i\theta)$, Verruijt [1] skillfully introduced that

$$U'(\zeta) = U'(\alpha \sigma) = 2G(1 - \alpha \sigma)(u_x + iu_y) = \sum_{k=-\infty}^{\infty} A_k \sigma^k$$

The problem with uniform radial displacement at the tunnel boundary (BC-1) is solved by Verruijt and the final solution is presented in his paper in 1997 [1]. With the method described by Wang et al [6], the coefficients in equation (8) for the three oval shaped prescribed boundary conditions in this paper can be obtained as

$$A_0 = -(1 + \alpha^2) G u_x i; \quad A_1 = (1.5 + 3\alpha + \alpha^2 - 3\alpha^3 - 0.5\alpha^4) G u_x i$$

B.C.-3:

$$A_k = -0.25 \left( \alpha^2 - 1 \right)^2 \left( 4\alpha + 3 \right) \left( \alpha^2 - 1 \right)^3 (k+1) \alpha^{k-3} G u_x i, \quad (k \geq 2)$$

$$A_k = 0.25 \left( \alpha^2 - 1 \right)^2 \alpha^{k-1} G u_x i, \quad (k \leq 1)$$

B.C.-4:

$$A_k = -0.25(3 + 10\alpha + 3\alpha^2) G u_x i; \quad A_1 = 0.25 \left( 7 + 9\alpha + 6\alpha^2 - 3\alpha^3 - \alpha^4 \right) G u_x i$$

$$A_k = -0.375 \left( \alpha^2 - 1 \right)^2 \left( 2\alpha + 3 \right) + \left( \alpha^2 - 1 \right)^3 (k+1) \alpha^{k-3} G u_x i, \quad (k \geq 2)$$

$$A_k = 0.375 \left( \alpha^2 - 1 \right)^2 \alpha^{k-1} G u_x i, \quad (k \leq 1)$$
Complex Variable Solutions for Twin Tunnels

\[
A_0 = -0.25(1+10\alpha+\alpha^2)Gu_{i}; \quad A_1 = 0.25(9+3\alpha+2\alpha^2-\alpha^3-\alpha^4)Gu_{i} \]

B.C.-5: \[
A_2 = -0.125(\alpha^2-1)^2(2\alpha+3)+(\alpha^2-1)(k+1)k^{k-3}Gu_{i}, \quad \alpha \geq 2
\]
\[
A_2 = 0.125(\alpha^2-1)^2\alpha_{k-1}k^{k-3}Gu_{i}, \quad \alpha \leq -1
\]

4 SOLUTION FOR TWIN TUNNEL INTERACTION

From above analytical method, the additional displacement on the boundary of the second tunnel due to the excavation of the first tunnel can be expressed as

\[
U_{11}^{\alpha\sigma_2} = \left[ k\varphi_{11}(\gamma_i) - \frac{\omega_i(\gamma_i)}{\omega_{11}(\gamma_i)}\varphi_{11}(\gamma_i) - \psi_{11}(\gamma_i) \right] (1-\alpha\sigma_2) = \sum_{k=-\infty}^{\infty} D_k\sigma_2^k
\]

where \( \alpha\sigma_2 \) is the coordinate of the boundary of the second tunnel in \( \zeta_2 \)-plane, \( \zeta_2 = \alpha\sigma_2 \); \( \gamma_i \) is the boundary coordinate of the second tunnel in \( \zeta_1 \)-plane, which comes from \( \alpha\sigma_2 \) after a conformal transformation \( z_2 = \omega_2(\alpha\sigma_2) \), a coordinate transformation \( z_i = z_2 + d \), and an inverse conformal transformation \( \gamma_i = \omega_1^{-1}(z_i) \); \( U_{11}^{\alpha\sigma_2} \) is a complex function in \( \zeta_2 \)-plane with a period of \( 2\pi \), which represents the displacement around the second tunnel induced by the excavation of the first tunnel, depending upon the polar coordinate \( \theta \); \( D_k \) is the coefficient of powers of \( \sigma_2 \). Notice that \( \sigma_2 \) can be expressed in polar coordinate, \( \sigma_2 = \exp(i\theta) \), equation (12) can also be expressed as

\[
U_{11}^{\alpha\sigma_2} = U_{11}^{\alpha\sigma_2}(\theta) = g_1(\theta) + ig_2(\theta)
\]

where \( g_1(\theta) \) and \( g_2(\theta) \) are real functions of the coordinate around the second tunnel. Assuming the function \( g(\theta) \) has a period of \( 2\pi \) and \( n \) values at the interval of \( (0, 2\pi) \), with \( \theta_j = 2\pi j/n, \quad n > 2m; \quad j = 0, \ldots, n-1 \) and \( g(\theta_n) = g(\theta_0) \) . Thus, with proper coefficients \( g(\theta) \) could be approximated by the following trigonometric series

\[
g(\theta) = \frac{1}{2}\delta_0 + \sum_{k=1}^{\infty}(\delta_k \cos k\theta + \eta_k \sin k\theta)
\]

Considering the trigonometric functions form a complete orthogonal system, the \( 2m+1 \) coefficients in equation (14) can be obtained by the equations

\[
\delta_0 = \frac{2}{n-j=0} g(\theta_j); \quad \delta_k = \frac{2}{n-j=0} g(\theta_j) \cos(k\theta_j) = 0; \quad \eta_k = \frac{2}{n-j=0} g(\theta_j) \sin(k\theta_j) = 0
\]

where \( k = 1, 2, \ldots, m \). Then the Equation (13) could be rewritten as
\begin{equation}
U_{11}(\theta) = \frac{1}{2} \delta_0 + \sum_{k=1}^{m} \left( \delta_k \cos k\theta + \eta_k \sin k\theta \right) + i \left[ \frac{1}{2} \delta'_0 + \sum_{k=1}^{m} \left( \delta'_k \cos k\theta + \eta'_k \sin k\theta \right) \right]
\end{equation}

with the Equations(12), (16) and Euler's formula, it is easy to derive

\begin{equation}
U_{11}(\alpha\sigma_2) = \sum_{k=-m}^{m} D_k \sigma_2^k = D_0 + \sum_{k=-m}^{m} \left( D_k e^{i\phi} + D_{-k} e^{-i\phi} \right)
\end{equation}

where

\begin{align*}
D_0 &= \frac{\delta_0 + i\delta'_0}{2}; \\
D_k &= \frac{\left( \delta_k + i\delta'_k \right) - i\left( \eta_k + i\eta'_k \right)}{2}; \\
D_{-k} &= \frac{\left( \delta_k + i\delta'_k \right) + i\left( \eta_k + i\eta'_k \right)}{2}
\end{align*}

Thus, the additional displacement at the boundary of the second tunnel induced by the prescribed displacement at the first tunnel can be approximated. The accuracy of the approximation is dependent on the number \( n \) and \( m \), and the errors can be controlled under arbitrarily small values.

The iterative procedure then is that, this additional displacement plus the prescribed displacement forms the new boundary condition at the opening of the second tunnel. The influence of this new boundary condition would result in additional displacement at the opening of the first tunnel, which, in a contrary manner, would generate new additional displacement around the second opening. This iterative process should be executed until the additional displacements at the boundaries of the two tunnels are close to zero or achieve certain accuracy required. The computation could be accomplished in a relatively simple computer program. In this paper, the values of \( n \) and \( m \) were set to 360 and 100 with the relative approximation error less than \( 10^{-10} \), and the iteration stop when the approximated additional displacements around the two openings are less than \( 10^{-6} \).

5 CASE STUDY

A published case record [11] was selected in this study to assess the applicability of the proposed analytical solutions with different boundary conditions of ground loss around the twin tunnels. The case involved a shield twin tunnels in Bangkok MRTA project with shield skin diameter 6.43\~6.46 m and tunnel lining diameter 6.3 m. The selected section 23-AR-001 is running in stiff clay 22 m below ground surface and with a centre line spacing about 13.5 m. In this paper \( E = 43MPa \) and \( \nu = 0.5 \) as it belongs to the undrained state. The gap parameter \( ( g = 2u_o ) \) is 160 mm considering that the ground loss due to face loss and workmanship can be made up by the positive effects of the tail grouting. In practice, \( u_i << r \) and the area of ground loss in the four possible ground loss patterns should be the same, hence

\begin{equation}
u_i = u_0 = g / 2; \quad u_3 = 2g / 3; \quad u_4 = 4g / 7; \quad u_5 = 4g / 9 \quad (19)
\end{equation}
Complex Variable Solutions for Twin Tunnels

Figure 3: Measured surface settlement and solutions for four ground loss patterns

Figure 3 shows comparisons between the measured surface settlement and the calculated surface settlements with the four boundary conditions for the selected twin tunnels section. The surface settlements under the four boundary conditions resemble to those of observed values. However, the BC-1 and -5 predict wider trough and less maximum settlements than observed while the BC-3 and -4 present almost the same settlement trough as that measured. The boundary condition of BC-4 is the best fit for the measured surface settlements. Hence the proposed complex variable solutions are suggested to be used for making reasonable estimation of ground deformation induced by construction of twin tunnels if prescribed with an oval shaped boundary condition of BC-4 around the tunnel opening.

6 CONCLUSION

This research presents complex variable solutions for ground deformations induced by deforming twin tunnels. The interaction effects can be approximated by alternately satisfying the boundary conditions triggered on each of the two tunnels. Comparison of the solutions and field results show that an oval shaped boundary condition around tunnel opening indeed can improve the prediction of ground deformations, and the BC-4 is recommended for predicting ground deformation induced by construction of twin tunnels.

The proposed analytical elastic solutions may well serve to manifest the characteristics of the resulting fields of stress and strain due to construction of twin tunnels. Although it is realized that elastic medium is a very poor representation of the real behaviour of the soils, it may serve as a preliminary result or a reference for more sophisticated numerical analysis.
REFERENCES


Fluid Grout and the Longitudinal Beam Action of a Tunnel Lining

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Abstract

An overview is given of a series of modelling steps in the quantification of fluid grout loading and consequences for longitudinal beam action of the tunnel lining. The modelling does not involve any advanced numerical modelling, on the contrary the physical relations are as much as possible brought back to their essence and are described by analytical models. The scope of this modelling is from a continuous model that describes the grout pressures around the circumference of the tunnel lining to a 1-D longitudinal beam action model that describes the forces in the tunnel lining as a function of distance. Advises to resolve remaining open questions are listed, as there is the measurement of transverse shear forces between the TBM and the tunnel lining.

In example calculations, the influence is shown of the diameter of the tunnel on the longitudinal bending moments and how the bending moment can be influenced by grout properties and the injection strategy of the tail void grouting.

Keywords: Analytical model, Orlovski, Tail void, Grout, Longitudinal bending moment
1 INTRODUCTION

In the design of bored tunnel projects, calculations have to be conducted regarding structural integrity of the tunnel lining, the excavation process and calculations regarding the TBM. Many of such calculations are conducted in-house at specialist engineering firms and suppliers of tunnel boring machines. Handbooks and guidelines are available: [1], [6], [9] and [10]. Tunnel owners contractors, engineering firms, tunnel builders, research institutes and universities co-operate one many occasion in the monitoring of the construction of tunnels in order to learn more about the building process. Gained knowledge is implemented in follow-up tunnelling projects, and it is believed that this has facilitated the growth of tunnel diameters through the years. Some information is understandably of proprietary character, other information is exchanged at tunnelling conferences and via journal publications.

The authors participated in the monitoring of tunnels from 7 m diameter up to about 15 m diameter. Particularly it was focused on the role of fluid-grout injected in the tail void of the TBM, which is a niche between soil mechanics and structural analysis involved in mechanised shield tunnelling, Figure 1. Especially the important role of controlled injection of fluid-grout has become evident.

Our present contribution focusses on the longitudinal behaviour of the tunnel lining as a tube as it is influenced by up-lift forces from fluid-grout. In the design of tunnelling projects and in back-analysis 3D numerical geomechanical analysis are about routinely conducted by different specialists. Such calculations persistently use a method into which a TBM advances step by step through a stratigraphy of soil and the tunnel lining is piece wise extended. This is rather complicated, and is not necessary for exploratory calculations. A good alternative is to work in a co-ordinate system that advances with the TBM.

Figure 1: Definition sketch

Our present contribution focusses on the longitudinal behaviour of the tunnel lining as a tube as it is influenced by up-lift forces from fluid-grout. In the design of tunnelling projects and in back-analysis 3D numerical geomechanical analysis are about routinely conducted by different specialists. Such calculations persistently use a method into which a TBM advances step by step through a stratigraphy of soil and the tunnel lining is piece wise extended. This is rather complicated, and is not necessary for exploratory calculations. A good alternative is to work in a co-ordinate system that advances with the TBM.
The objective of the present contribution is to show how in the initial phase of tunnel design it is already possible to estimate the gross numbers of transverse shear force and longitudinal bending moment by analytical modelling. A second objective, to satisfy curiosity, is to see what the outcomes are for the largest currently existing tunnel design: Orlovski Neva River Tunnel.

2 THEORY AND MODELS

2.1 Grout Pressures DCgrout

Grout is pressed from the TBM into the tail void, and an associated hydrodynamic pressure field develops governed by the distribution of grout injection rates over the injection ports and frictional properties of the grout. This pressure field adds up to the static pressure distribution associated with the dead weight of the grout. This hydrodynamic pressure may oppose a significant fraction of the static pressure field. This is effective in decreasing the uplift forces on the tunnel lining immediately behind the TBM.

![Figure 2: DCgrout result showing calculated grout pressures [Pa] around the tunnel lining for an unusual a-symmetric injection of grout.](image)

The distribution of grout is calculated by a finite difference model DCgrout [14]. It is by default configured for a maximum of 6 injection ports, but can be extended to a higher number for instance for larger diameter tunnels. It has been applied to the 2nd Heinenoord tunnel [14], in Sophia Rail tunnel [2] and in calculations concerning Groene Hart Tunnel [15]. An example of a calculated distribution of grout pressures
at the rear of the TBM is given in Figure 2. This is an example where it is investigated what the consequences are when a supply pipe becomes blocked. It is not useful to calculate the detailed pressure field associated with two-component chemical grouts. Because of fast hardening, the fluid grout zone is extremely short, and can not have a significant influence on the forces in the lining.

2.2 Beam Action DEABEAM

The authors have created a 1-D analytical beam action model in a co-ordinate system moving with the TBM to quantify the influence of uplift forces from fluid-grout on the just constructed tunnel lining [16]. The acronym of the model is DEABEAM (DEltares Analytical BEam Action Model). The tunnel lining is further loaded by vertical loads exercised by the back-up train, structures inserted in the tube (Figure 3) and mechanical forces from the TBM being transferred by the rear jacks of the TBM onto the first ring of the tunnel lining segments.

![Figure 3: Typical TBM and Back-up train: Groene Hart type](image)

The structure is divided into a section close to the TBM of length \( L \), which is loaded by predetermined fixed loads (TBM loading, weight of the lining, weight of gantry 1 and buoyancy force from the fluid-grout), and a second section further from the TBM that is bedded in the surrounding soil (see Figure 4). This second section is regarded as a half-infinite beam on an elastic foundation in accordance with classic mechanical engineering theory: an elastically-supported beam.
3 DEABEAM CALCULATIONS

3.1 Groene Hart Tunnel (GHT), calculation and validation

DEABEAM calculations for Groene Hart Tunnel are described in [16]. Precise values for loads of back-up train components are derived from the technical specifications provided by the contractor. Stresses in the tunnel lining were monitored, and from these the bending moment curve was determined. Characteristic of the bending moment curve is that it varies with distance, changes sign and a final non-zero value is reached at a distance far behind the TBM, Figure 5.
3.2 Analysis of Shanghai Yangtze River Tunnel (SYRT)

At SYRT there was concern that the transverse shear force in the tunnel lining would become too large, exceeding Coulomb friction between adjacent tunnel rings [8] calculated as 16 MN. The DEABEAM has been applied to the Shanghai Yangtze River Tunnel for a re-analysis by the authors [17]. The result is shown in Figure 6. Here also a non-zero final longitudinal bending moment is calculated, but more importantly, the transverse shear forces in the lining are not as alarmingly high as predicted in [8].

Figure 6: Calculated shear force and longitudinal bending moment for SYRT

3.3 Prediction for St Petersburg Orlovski-Neva River Tunnel

Information on the mono-tube Orlovski Neva River Tunnel is provided in [5], [11], [12] and [13]. The planned depth is 20 to 30 m. The outer diameter of the tunnel lining is 18.65 m. The thickness of the tunnel segments is 0.7 m. The volume weight of the lining elements is 25.25 kN/m$^3$. The class of concrete is not yet established. The length of the mixshield TBM plus back-up train is 82 m. Gantry no. 1 is about 23 m long. Gantry no. 2 is about 15 m long. Gantry no. 3 is about 30 m long. At GHT about 45% of the total weight is attributable to the back-up train. We adhere to the same ratio, and assume that Gantry 1 is about two times heavier than the last gantry of the back-up train. At Orlovski the back-up train is shorter. The technical drawing of Orlovski shows a light wheel arrangement for Gantry 2. So this gantry, which only carries the internal overhead crane, is assumed to be the lightest. Table 1 contains the
weight of main components in calculations for different tunnels. In future calculations the true values can be utilised.

Table 1: Summary of main load components in calculations for tunnel projects

<table>
<thead>
<tr>
<th></th>
<th>GHT</th>
<th>SYRT</th>
<th>Orlovski</th>
</tr>
</thead>
<tbody>
<tr>
<td>TBM diameter</td>
<td>15 m</td>
<td>15.4 m</td>
<td>19.2</td>
</tr>
<tr>
<td>length TBM + back-up train</td>
<td>135 m</td>
<td>120 m</td>
<td>82 m</td>
</tr>
<tr>
<td>Ftotal</td>
<td>35 MN</td>
<td>32 MN*</td>
<td>unknown</td>
</tr>
<tr>
<td>Fshield</td>
<td>Unknown</td>
<td></td>
<td>38 MN†</td>
</tr>
<tr>
<td>Fgantry1</td>
<td>10.5 MN</td>
<td>14.5 MN (est.)</td>
<td>20 MN (est.)</td>
</tr>
<tr>
<td>Fgantry2</td>
<td>bridge</td>
<td>bridge</td>
<td></td>
</tr>
<tr>
<td>Fgantry3</td>
<td>5 MN</td>
<td>7 MN (est.)</td>
<td>10 MN (est.)</td>
</tr>
</tbody>
</table>

The loads of the gantries are transferred via multiple rubber wheel tyre boogies. The loading diagram is shown in Figure 7.

The soil stratigraphy at Orlovski [5] consists of quaternary deposits containing sandy loam, sand, loamy sand, silt loam and silty clay loam (USDA classification). The Young modulus of the soil layers is in the range: $E_{\text{young}} = 10$ to 30 MPa (applied in ring design and in the calculation of ring deformation [5]). The Poisson ratio is in the range 0.3 to 0.4. The spring constant $k$ and the bending stiffness of the tunnel lining are calculated similarly as at GHT and at SYRT. It is assumed that the modulus of elasticity of the concrete lining is 40 GPa, similar to at other tunnels.

In the design of the Orlovski tunnel two-component grout [7] is applied. It is therefore interesting to compare the forces and bending moments that are calculated for two-component grout and those for fly-ash based grout (like utilised at GHT and at SYRT).

* According to Huang, but 23 MN according to www.Herrenknecht.com, read 24/08/2012
† shield only: www.Herrenknecht.com, read 24/08/2012
Table 2: Model input parameters for Orlovski Neva River Tunnel

<table>
<thead>
<tr>
<th>Parameter</th>
<th>value</th>
<th>begin[a] [m]</th>
<th>end [b] [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eq. bending stiffness: EI&lt;sub&gt;eq&lt;/sub&gt;</td>
<td>8.3*10&lt;sup&gt;9&lt;/sup&gt; kNm&lt;sup&gt;2&lt;/sup&gt;</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Spring constant soil: k</td>
<td>4.65*10&lt;sup&gt;4&lt;/sup&gt; kN/m&lt;sup&gt;2&lt;/sup&gt;</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bending moment from TBM</td>
<td>- 65 MNm</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Transverse shear force from TBM</td>
<td>1.5 MN</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vert. grout pressure gradient at TBM</td>
<td>10 &amp; 12 kPa/m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vert. grout pressure gradient at x=0</td>
<td>10 &amp; 9.5 kPa/m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Length lining in TBM: L&lt;sub&gt;i&lt;/sub&gt;</td>
<td>4.0 m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Length fluid grout: L</td>
<td>0.5 &amp; 8 m</td>
<td></td>
<td></td>
</tr>
<tr>
<td>q&lt;sub&gt;water&lt;/sub&gt;</td>
<td>-2840 kN/m</td>
<td>L</td>
<td>1000</td>
</tr>
<tr>
<td>q&lt;sub&gt;lining&lt;/sub&gt;</td>
<td>930 kN/m</td>
<td>-L&lt;sub&gt;i&lt;/sub&gt;</td>
<td>1000</td>
</tr>
<tr>
<td>q&lt;sub&gt;grout&lt;/sub&gt; (0.175 m thick)</td>
<td>103 &amp; 213 kN/m</td>
<td>L</td>
<td>1000</td>
</tr>
<tr>
<td>q&lt;sub&gt;wheel set 1 (estim. load)&lt;/sub&gt;</td>
<td>2170 kN/m</td>
<td>3</td>
<td>9</td>
</tr>
<tr>
<td>q&lt;sub&gt;wheel set 2 (estim. load)&lt;/sub&gt;</td>
<td>1300 kN/m</td>
<td>16</td>
<td>26</td>
</tr>
<tr>
<td>q&lt;sub&gt;wheel set 3 (estim. load)&lt;/sub&gt;</td>
<td>812 kN/m</td>
<td>40</td>
<td>48</td>
</tr>
<tr>
<td>q&lt;sub&gt;wheel set 4 (estim. load)&lt;/sub&gt;</td>
<td>1083 kN/m</td>
<td>64</td>
<td>70</td>
</tr>
</tbody>
</table>

With a two-component grout, basically two low-viscosity chemical fluids are simultaneously injected. The mixed product hardens quickly, and within one hour a strength of 250 kPa can be reached [4]. This material can be injected via one injection point at the crown of the TBM, and the fluid immediately fills the tail void created by the advancing TBM. The fluid has a specific weight not much different than that of water. After injection the fluid will quickly gain plastic properties.

<sup>3</sup> Measured from the rear of the TBM
The input parameters of both calculations are listed in Table 2 (input parameters for two-component grout calculation are underlined). The bending moment and the shear force exerted by the TBM are assumed identical to as measured at GHT. Two of the calculation results for vertical displacement, shear force and the longitudinal bending moment in the lining for both cement (or fly-ash) based grout and two-component grout are shown in Figure 8.

Figure 7: Loading diagram of Orlovski Neva River Tunnel

Figure 8: Calculated shear force and longitudinal bending moment at Orlovski

4 EVALUATION

Despite of the fact that there are still a lot of unknown parameters for Orlovski, exploratory calculations can be conducted already. Weights and weight distribution are important parameters. It is the best to design the back-up train such that the
heaviest loads are as close as possible to the TBM. Considering the significant shorter back-up train length for Orlovski, this seems being pursued. The transverse shear force from the TBM is also important, but its value is unknown. Two-component chemical grout is preferable because it gives smaller shear forces and a smaller bending moment in the lining. Despite the significant advances that have been made in our research, a number of questions are unresolved (not discussed in the present manuscript):
- What determines the different frictional behaviour of cement (Sophia Rail Tunnel) and fly-ash based grouts (GHT) with the concrete lining?
- How to quantify pressure losses in grout supply pipes (the grout pressure sensors are located in the TBM before a 90 degrees bend).
- How does pre-curvature of the tunnel lining develop from the ring building process?
- How to predict the bending stiffness of a segmented tunnel lining?
- And most importantly: a measurement of the transverse shear force between the TBM and the lining.

It would be interesting to see what an integrated 3D numerical simulation of face excavation, TBM, lining and soil gives for vertical transverse shear force between TBM and tunnel lining.

5 CONCLUSIONS

After more than a decade of model development, substantiated by dedicated monitoring of tunnel construction we have arrived at a point where,

- the influence of grout injection on forces in the lining can successfully be calculated by means of analytical models,
- for cement and fly-ash based grouts, the grout pressure distribution can be calculated from rheological properties and distribution of grout injection from the TBM,
- initial design calculations can be conducted with the described models, but the models are also suited for back-analysis, and for independent calculations.

For the 19 m diameter Orlovski tunnel it is concluded, that it is very important to control the vertical pressure gradient from fluid grout immediately behind the TBM. Two-component chemical grout is preferable because it gives smaller forces and smaller bending moments in the lining. The compact design of the back-up train, concentrating the highest vertical loads near the TBM, prevents the longitudinal bending moment in the tunnel lining to become large.
REFERENCES


High Performance Computing in Tunneling and Subsurface Engineering
Automatic Seismic Prediction on Tunnel Boring Machines (TBM)

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Abstract

Safe underground construction requires the most exact knowledge as possible about geological characteristics of the subsurface in the vicinity of the construction. However, exploration ahead of the cutting wheel at Tunnel Boring Machines (TBM) for foresighted building differs from exploration during conventional drill-and-blast building with shotcrete method. The conventional drill-and-blast building with shotcrete method uses special excavation classes, which are a priori calculated during the planning phase for unpredictable situations. Using this method, it is advantageous to have a close look at the tunnel face. Thus, the preparation for the excavation can consider on changing geological characteristics. In comparison, the reaction time on changing geological characteristics using a TBM is restricted. Although, carrying out exploration drillings is possible, but this will slow down the construction process due to the required stop of the cutting wheel. As a result of this, the actual advantage of machine-driven tunnelling decreases. In many cases, a high-resolution exploration of the subsurface from above is not possible due to a high overburden or inaccessibility. For solving this problem, exploration procedures are necessary which can be directly implemented into the TBM process without interrupting the tunnel advance. The
required measuring instruments have to be integrated into the TBM itself. Therefore, the development of TBM-based geophysical exploration systems is mandatory. So far, the development of such exploration systems is still ongoing and improving as they do not yet provide the most sufficient precision of the results. Consider the increasing market share of machine-driven tunnelling, a continuously growing need for sophisticated TBM-based geophysical exploration systems exists. The Herrenknecht AG currently provides two different TBM-based geophysical exploration systems which are used in machine-driven tunnelling:

- **SSP (Sonic Softground Probing)**
  Exploration system for the drilling lane in soft-soil conditions which has already been used successfully in 13 machine-driven tunneling projects.

- **ISIS (Integrated Seismic Imaging System)**
  A new exploration system for the application in hard-rock conditions.

**Keywords:** Tunnel boring machine, geophysical exploration system, detect obstacles, Sonic Softground Probing, SSP, softground, Integrated Seismic Imaging System, ISIS, hard rock
1 SONIC SOFTGROUND PROBING

1.1 SSP Principle and Method of Functioning

The SSP method is a non-destructive seismic exploration method in mechanised tunnelling advance with slurry-supported face designed for softground. It is marketed and further developed by the Herrenknecht AG in conjunction with the Mixshields produced by the company. Compared to all geophysical exploration systems known to the Herrenknecht AG, the SSP system possesses a high degree of integration into a TBM. It is designed especially for the contiguous monitoring of the softground structures ahead of the tunnel face without negatively affecting the excavation process of the TBM. In other words, the SSP method runs parallel to the process of boring.

The transmitting (T) and receiving (Rx) units necessary for seismic exploration are directly integrated into the TBMs cutting wheel (see Figure 1).

![Figure 1: View of the cutting wheel S-326 (City-Tunnel Leipzig). The hardware components within the cutting wheel are marked with T for transmitter and R1, R2, R3, R4 for receivers.](image)

The complicated sequence of data processing, analysing the results, and visualisation was automated as far as possible. It takes place practically in real time (duration roughly 3 to 5h after a measurement cycle is concluded) in the back-up zone of the tunnelling machine.

The measurement method is based on acoustic reflection measurements according to the correlation location principle. Correlation locating methods are especially suitable for use in environments with very high noise levels. Petrophysical contrasts produce reflections when the ground is scanned by sound. The determining physical magnitude here is the seismic impedance (also: acoustic resistance or wave resistance) \( Z \), which results as a product of the density \( r \) of the medium scanned by sound and the seismic (compression wave-) speed \( n \) of the emitted acoustic wave in the affected body of soil:
If the impedance alters abruptly, i.e. a sufficiently large impedance contrast exists on the interface between two media $Z_1$ and $Z_2$ – as for example were to be expected given a clearly defined change of geology – a part of the incident wave is reflected. The strength of this reflection results from the vertical occurrence of the wave on the interface on the basis of the reflection coefficient $a_r$:

$$a_r = \frac{(z_1-z_2)^2}{(z_1+z_2)^2}.$$  \hspace{1cm} (2)

The specially coded acoustic transmission signal is emitted by the SSP transmitter, installed within the cutting wheel, into the support medium of the head face, i.e. the bentonite suspension. The generated wave propagates mostly into the soil ahead of the cutting wheel due to a low contrast of the bentonite suspension to the soil. This source signal is emitted while the drive of the TBM is in progress and as a so-called up-sweep. The up-sweep signal has a duration of 1s with a linearly-increasing frequency range of usually 600 up to 2,400 Hz but also up to 4,800 Hz.

The effectiveness of the SSP system is largely governed in its range and resolution by the physical processes of the spherical propagation of the signal energy. Furthermore, absorption and scattering as well as data coverage, position, and geometry of the reflecting interfaces/bodies, soil heterogeneity and background noises affect the SSP system.

In practical use, exploration ranges of more than 40 m were attained with the resolution capacity varying over this distance and diminishing as the distance increases. Generally only obstacles can be identified whose dimensions are larger than the smallest wave length $\lambda$ ($c =$ wave propagation speed; $f =$ frequency) of the transmission signal; in the most favourable case at close range (up to 20 m ahead of the cutting wheel) objects in excess of roughly 1 m in size can be detected.

1.2 Results from City-tunnel Leipzig

For the City-Tunnel Leipzig, it was possible to detect artificial obstacles (e.g. sealing blocks, drilled pile walls, diaphragm walls of former construction pits, headings for retrieving anchors as well as collector shafts outside the tunnel route) as well as changes in geology (sandstone and quartzite banks).
The following two instructive examples will be provided to show the current performance capability. The result to be interpreted on the spot of the geophysical working process is a 3D image of the reflection elements represented by the backscattered amount of energy. The 3D image is a cuboid ahead of the cutting wheel with a dimension of 10m x 10m x 40m. Orange and green reflection elements are displayed, which correspond to high positive or negative amplitudes of the reflectivity and thus indicate an impedance contrast, as previously described.

In order to present the examples as clearly as possible the corresponding 2D sections were selected from the given 3D cubes and placed on the existing geotechnical cross-sections with the aid of graphical-processing software. In this way a direct reference between the reflector and the object can be established.

The example in Figure 2 indicates the detection of a sealing block in the correct position. Both the front edge area (transition soil to concrete) as well as the rear edge at roughly TM 100630 (transition concrete to air) are outstandingly depicted.

![Figure 2: Detecting front and rear edges of a sealing block.](image)
Construction site reports and machine data indicated that major sandstone intrusions and boulders were extracted mainly between TM 600185 and 600200. The strong reflector in Figure 3 at approximately TM 600186 matches extremely well with the front edge of this encountered change in geology. The red line from TM 600148 to TM 600156 represents the area of data acquired that stem for this cube.

Figure 3: Detected change in geology, in this case a sandstone/quartzite bank.

In other words, the SSP result and in turn the advance warning in this case lay some 25 to 30 m prior to reaching the sandstone bank, taking the processing time into consideration.
1 INTEGRATED SEISMIC IMAGING SYSTEM

Besides SSP, which is designed for contiguous, non-destructive geophysical exploration in softground exclusively, the Integrated Seismic Imaging System (ISIS) has been developed since the year 2000. The ISIS can be seen as an equivalent for application in hard-rock conditions, i.e. on Gripper TBMs. Starting as a project of the GFZ Potsdam, where the general usability of the basic measurement principle (RSSR reflection) for prediction purposes has been proved, ISIS was licensed by the Herrenknecht AG in 2007. Throughout the construction of the Blessberg tunnel, Germany in 2009, the usability of ISIS in karst conditions could be verified. Finally in 2011, ISIS was tested under real jobsite conditions with enhanced Herrenknecht AG hard- and software in Fréjus, France.

2.1 Hardware and Principle of Functioning

The ISIS hardware (see Figure 4) consists of two pneumatically operated impact pulse generators. These impact pulse generators are supplied with pressurized air by two individual maintenance units. Two separate control units serve as interface between the ISIS software and the impact pulse generators. When triggered, the impact pulse generators perform a stroke against the tunnel wall, thereby generating seismic waves. According to the RSSR-principle (see Figure 5) the wave propagation is modelled as follows: starting at the impact source, R-waves (Rayleigh waves/surface waves) propagate along the already excavated tunnel tube. When reaching the tunnel head face, these R-waves are converted into S-waves (shear waves/body waves) and propagate further into the rock ahead of the TBM. On encountering e.g. water-/air-filled cavities or fault zones in the rock mass (i.e. when there is a change of seismic impedance in the medium), the S-waves are partly reflected and run back in direction of the head face. At the head face, the S-waves are converted back into R-waves. Finally, these R-waves are recorded by three sets of three-component geophones attached to the left and right side of the tunnel tube. The corresponding time series are stored on respective wireless data loggers for further processing.

The integration of the recorded time series takes place in the ISIS software. The following equation incorporates the basic model equation of the RSSR principle:

\[ t_{RSSR} = \frac{d_{S,H}}{v_R} + 2 \frac{d_{Ref}}{v_S} + \frac{d_{H,R}}{v_R}, \]

(3)
where \( t_{RSSR} \) denotes the travel time of the RSSR-wave, \( d_{S,H} \) denotes the distance between source and head face, \( d_{Ref} \) denotes the distance between the head face and a potential reflector, and \( d_{H,R} \) denotes the distance between the head face and a receiver. The model seismic velocities \( v_R \) (Rayleigh wave) and \( v_S \) (shear wave) have to be properly chosen according to the current geology. Given the distances \( d_{S,H} \) and \( d_{H,R} \) (by means of simple distance measurements), the desired distance \( d_{Ref} \) can finally be determined.

![ISIS hardware](image)

**Figure 4:** ISIS hardware. (a) Control unit; (b) Data logger; (c) Impact pulse generator; (d) maintenance unit; (e) 3-component geophones

![RSSR wave propagation principle](image)

**Figure 5:** RSSR wave propagation principle
2.2 Current and Future Projects

Presently ISIS is successfully being operated within tunnelling projects in Tel Aviv, Israel (Railway to Jerusalem) and Ecuador (Coca Codo Sinclair Hydropower Project). No major cavities or fault zones have been encountered throughout these projects yet, hence no demonstrative results can be shown up to this point. Future applications of ISIS will be within projects in Pakistan (Neelum-Jhelum Hydropower Project) and Yin Han Ji Wei, China.

REFERENCES


Mesh Adaptation for Coupled Hydro-mechanical Industrial Applications

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Abstract

Numerical simulations using the finite element method need a mesh fine enough to guarantee the accuracy of the solution. However, this mesh usually involves expensive computation. In this paper, we show a methodology based on mesh adaptation which allows to improve the quality and to speed up time consuming computations.

The efficiency of such methodology is proved in the particular case of tunnel boring which is a complex engineering problem with numerous non-linearities. The studied issue is a 3D-computation of a borehole excavation in a porous medium assumed to be fully saturated with water.

Keywords: Poromechanics, mesh adaptation, borehole excavation, finite element, hydro-mechanical coupling, Code_Aster.
1 INTRODUCTION

In France, the reference solution for Intermediate and High Level Long Lived nuclear Waste management is the deep geological disposal. In this context, various simulations of the different phases of tunnel boring are being performed at EDF (major French electric power company). Among them, one important investigation is the prediction of the Excavation Damaged Zone of the porous medium around the drilled tunnels since the stability of the construction and the hydraulic properties of the geological medium should be modified.

Industrial studies involve numerical computations often with complex structures and high quality of the physical modelling. Moreover, most of those simulations are performed with finite element methods which require fine enough meshes to guarantee the accuracy of the solution. However, a uniformly fine mesh implies a more expensive computation. Mesh adaptation offers an effective compromise combining fine mesh in some specific zones with a low computational cost.

The numerical application proposed in this paper is a deep underground excavation of a horizontal borehole in saturated poro-elastoplastic media. The computation has been performed with the finite element software Code_Aster (open source available on www.code-aster.org).

2 DEFINITION AND METHODOLOGY FOR THE BENCHMARK

This section is devoted to the theoretical and the numerical presentation of the model in the restricted case of saturated conditions for the porous media. In the first part, the balance equations of classical poromechanics are described, whereas in the second part, the methodology used with the mesh-adaptation is presented.

In this paper we use the convention of mechanical stresses positive on traction.

2.1 Classical Poromechanics Equations

A porous medium is composed of a solid matrix and a porous (void) space crossed by a fluid. In the particular case of saturated media, the porous space is filled by the fluid. Consequently, the behaviour of such continuum is characterized by the superimposition of a skeleton and a fluid continua (Coussy).
The concept of coupled hydraulical and mechanical phenomena is introduced by the Biot’s principle of the total stresses $\sigma_{ij}$ decomposition

$$\sigma_{ij} = \sigma'_{ij} - b p_w \delta_{ij}$$

(1)

where the effective stresses $\sigma'_{ij}$ are related to the history of the skeleton kinematics through a rheological behaviour, $p_w$ is the water pore pressure and $b$ is the Biot coefficient

$$b = 1 - \frac{K_0}{K_s}$$

(2)

$K_0$ and $K_s$ are the bulk modulus of the solid grains and of the drained porous medium respectively, the latter depending on the elastic skeleton properties

$$K_0 = \frac{E_0}{3(1 - 2v_0)}$$

(3)

The porosity $\phi$ of the skeleton is defined by the ratio of the void space volume to the total volume. In the particular case of saturated media, porosity also characterizes the fluid content of the medium. The variation of this quantity is given by

$$d\phi = (b - \phi) \left( d\varepsilon_v + \frac{p_w}{K_s} \right)$$

(4)

The fluid mass density $\rho_w$ variation is described by the fluid state equation

$$\frac{d\rho_w}{\rho_w} = \frac{dp_w}{K_w}$$

(5)

where $K_w$ is the fluid bulk modulus.
The fluid mass $m_w$ variation from initial configuration depends on the fluid mass density $\rho_w$, the volume change $\varepsilon_V$ and on the porosity $\phi$

$$m_w = \rho_w (1 + \varepsilon_V) \phi - \rho_{w, in} \phi_{in}$$

(6)

where $\rho_{w, in}$ and $\phi_{in}$ are respectively the initial fluid mass density and the porosity of the porous medium.

The water diffusion equation, which characterizes the water flow inside void volumes, is described by the Darcy’s law

$$M_i^w = -\frac{k_{ij}}{\mu_w} \frac{\partial p_w}{\partial x_j}$$

(7)

where $M_i^w$ is the fluid mass flow, $\mu_w$ is the dynamic fluid viscosity and $k_{ij}$ is the intrinsic permeability tensor. This latter is a measure of the easiness to cross the porous medium.

Finally, the balance equations of these superimposition continua should be defined by the balance of the momentum for the mixture (see equation 8) and the fluid mass balance (equation 10) respectively. Taking into account the virtual work principle, these equations read for any kinematically admissible virtual field $(u_i^*, p_w^*)$ in a weak form

$$\int_{\Omega} \sigma_{ij} \varepsilon_{ij}^* dv = \int_{\partial \Omega} p_i u_i^* ds$$

(9)

$$- \int_{\Omega} \frac{dm_w}{dt} p_w^* dv + \int_{\partial \Omega} M_j^m \frac{\partial p_w^*}{\partial x_j} dv = \int_{\partial \Omega} M_j^{w, ext} p_w^* ds$$

(10)

where $p_i$ and $M_j^{w, ext}$ are respectively the prescribed mechanical traction forces and the input fluid mass per unit area.
2.2 Mesh Adaptation

A complete presentation of the mesh adaptation software Homard is done by Nicolas et al on [3, 4]. The software deals with meshes in 2D or in 3D, and is able to refine or unrefine the elements. The refinement process is controlled by an error indicator computed by the physical solver. All the elements identified by a value of this indicator greater than the threshold are split. An example is presented on Figure 1. Then, additional refinements must be performed to produce a conformal mesh, using a specific treatment for the transition between the different levels of refinement. An example is presented on Figure 2.

In order to control the number of elements in the mesh we use a strategy based on the average and the standard deviation of the error indicator as suggested by [5, 6].

![Figure 1: Example of a standard refinement for a triangle](image1)

![Figure 2: Example of a transition refinement for a triangle](image2)
3 NUMERICAL APPLICATION

For our application, we take into account an example based on the GMR Gallery of the underground research laboratory of Bure in the Meuse/Haute Marne region in France. The geometry of the cavity is a horseshoe-shaped (see Figure 3). The length of the cavity is 20 meter (see Figure 4 for the initial mesh) and the rate of excavation is 1 meter per day. We consider in the simulation all the phases of the drilling since these ones should have effective consequences on the behaviour of the porous medium.

![Figure 3: Geometry of the cavity](image)

We also present on Figure 5 the mesh of the structure considered for different steps of the simulation.
Mesh Adaptation for Coupled Hydro-mechanical Industrial Applications

Figure 4: Left: Initial mesh of the 20 meter-long. Right: The final mesh.

Figure 5: Evolution of the mesh for different steps of the simulation.

4 CONCLUSION AND PERSPECTIVES

We show in this work the efficiency of our mesh adaptation methodology to deal with tunnel boring in the context of porous media. Next, we will extend this methodology to take into account more non-linearities as the ones introduced by plasticity to deal with more realistic constitutive models for geomaterials.
REFERENCES


Optimization of Tunnel Profile in Different Ground Conditions Using Genetic Algorithms

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Abstract

The design of a tunnel profile depends mainly on serviceability requirements, ground conditions and construction aspects. The optimal tunnel profile is the one that satisfies all the requirements and conditions with the minimum construction cost. Few researches dealt with the tunnel profile optimization problem. Some of these researches made comparison between certain profiles to pick the best one. Other researches dealt with the profile as a predefined decision and searched for its optimal components (i.e. width, height, curves radii and centres positions). In this research, a genetic algorithm based computer program is used to search for the optimal tunnel profile. A finite element numerical model is employed to represent the ground soil and tunnel profiles. The nodes coordinates for the excavation area outer profile are considered as the design variables in the optimization process. Thus, the program searches in infinite number of profile shapes for the optimal one that satisfies serviceability and stress requirements in different ground conditions. The program produces a quasi-optimal safe profile in reasonable running time. The results do not depend on expectation or experience.

Keywords: Tunnel profile, optimization, genetic algorithms, soil-structure interaction
1 INTRODUCTION

Although tunnel profile design is a critical job, it still depends mainly on designer’s experience. Few trials were performed to make a calculations based expectation for a better profile shape [1], [7]. Other researches went further to automate this process using modern evolutionary algorithms optimization techniques [4]. These researches could not discard the experience based expectation for the profile shape. Thus, they presented an optimization process for certain profiles by getting its optimal parameters (i.e. curves radii and centres positions).

The evolutionary algorithms optimization techniques such as Genetic Algorithms have been developed in the early 1970’s and they have a lot of applications since 1980’s [3]. However few applications, due to the analysis difficulties, were devoted to the geotechnical problems.

In 2010, a Genetic Algorithm was coupled with Finite Elements Analysis to present a new tool that can deal with geotechnical problems. It has been used to find the optimal grouting quantity and quality to strength soil before tunnelling in order to have the minimum surface settlement during construction [2]. In this paper, this tool is used to search for the optimal tunnel profile. This stochastic search does not depend on design experience or expectation.

2 TUNNEL PROFILE OPTIMIZATION PROBLEM

The optimization problem addressed herein is to find out the optimal nodes coordinates to form the optimal profile which has the minimum excavation area and minimum moment in lining. Excavation area expresses the tunnelling cost. Both bending moment and normal force are responsible for lining stress but bending moment is affected largely with profile shape and curvature. Good profiles are expected to induce less moment values. The resulted profile should satisfy the allowable deformation and stress requirements. The objective function here is the excavation area $A$ and moment in shotcrete lining $M$ which is expressed in Eq. (1).

$$\text{Minimize } F = C_1A + C_2M$$ (1)

Where, $C_1$ and $C_2$ are combination factors. These factors reduce the gap between area and moment values to have balanced impact on the optimization process. For example, the excavation area and maximum bending moment in a preliminary circular section are 70 m$^2$, and 40 KN.m respectively. To make balance, area is reduced by 65% ($C_1=0.35$) and moment is reduced by 35% ($C_2=0.65$).
2.1 The Design Variables

The design variables, as shown in Figure 1, are the nodes radian coordinates. Sixteen nodes on the preliminary circular profile are considered as design variables. Each node has a certain moving domain with different 32 possible positions and different distances from the centre. The positions of these nodes form the tunnel profile. Number of nodes, 16, is selected to produce a clear and smooth profile shape. Increasing number of nodes may give smoother profile shape but it will increase the computation time and enlarge search space.

![Figure 1: The design variables](image)

2.2 The Constraints

There are three constraints considered in the current optimization problem:

1) The resulted profile should have the minimum dimensions to satisfy the traffic requirements, as shown in Figure 1.

2) The stress in shotcrete lining should not exceed the lining material strength in. This can be expressed as following:
\[ g_i = \delta_i - \delta_{all} \leq 0 \quad , \quad i = 1, 2, \ldots, n \]  \quad \quad (2)

3) The surface settlement should not exceed the specified limit which can be expressed as following:

\[ g_i = \Delta_i - \Delta_{all} \leq 0 \quad , \quad i = 1, 2, \ldots, n \]  \quad \quad (3)

3 \quad \textbf{STRUCTURAL MODELLING}

In this paper, conventional numerical model with plane-strain analysis is used [5]. Figure 2 shows the finite element mesh used for simulation, the applied boundary condition and soil parameters.

![Finite elements mesh and geotechnical parameters](image)

Figure 2: Finite elements mesh and geotechnical parameters

3.1 \quad \textbf{Geotechnical Parameters}

The tunnel is constructed in a stiff clayey layer at a depth of 14 m with no existence of ground water in the tunnelling vicinity.

3.2 \quad \textbf{Finite Elements Simulation}

For the modelling process in the Finite Element program, FINAL package [6], the soil media is modelled using a six node linearly varying strain triangular finite
elements (L.S.T) and the shotcrete lining by a six node curved boundary beam elements (Beam6). A half-section symmetrical mesh is used in the analysis to reduce computation time. Sufficient mesh depth and width, to model soil infinite body, are used. For boundary condition, vertical and horizontal movements are prevented at the bottom of the model while only the horizontal movements are prevented at both sides. The excavation process is simulated in two steps using stiffness reduction method. In the first step, the stiffness of the soil is reduced by a factor accompanied with stress redistribution at the tunnel zone. In the second step, the excavated part is removed and the temporary lining is installed in its fresh state accompanied with a stress elimination of the excavated soil.

4  COMPUTATION PROCEDURE

The developed steady-state genetic algorithm starts with generating randomly an initial population of ten individuals (group of profiles). Each individual is sent to the FINAL package to check the stress in lining and surface settlement and compare them with the allowable values. Unsafe solutions are replaced with new ones which are also randomly generated. Safe solutions are encoded to binary form to facilitate the application of mating operators. Genetic algorithm’s mating operators are crossover and mutation. Each two solutions are mated together to produce two children solutions. Like their parents, produced solutions have to be checked. Unsafe solutions are replaced and safe solutions pass. All safe parents and children solution are collected in one pool and sorted in an ascending order. The last ten solutions are discarded and the first ten solutions form the parents’ population for the next generation.

4.1  Progression

Processing optimization operators and repeating them through generated population leads to convergence toward global optimum. Difficulty of having optimal or quasi optimal solution increases as convergence rate increases. Looking for the very close better chromosome through random mutation is a time consuming process. In this case, changing mutation to local searching technique is more preferable. The algorithm changes its mutation technique at the final 100 generations of every 1000 generations to avoid having local minima. Figure 3 shows the target function’s value progression through 1000 generation. Progression is fast in the starting generations and getting slower later.
4.2 Resulted Profile

Figure 4 shows the resulted mesh after 1000 generations which consume 5 days running time on a computer with Core2 Duo processor, 2.0 GHz speed and 2GB RAM. Regarding the large searching space \((32^{16} \text{ possible profile shape} = 1.2 \text{ E}24 \text{ solution})\), reaching the optimal solution may need thousands of generations which will also be good when compared to the number of alternatives. To save time, the resulted nodes are interpolated to give the designer an approximate image for the optimal profile.
In order to check the algorithm, two other profiles are produced from the interpolated profile by modifying its dimensions slightly in vertical and horizontal directions as shown in Figure 5. Profile (1) shows a small reduction in moment and normal comparing with the increase in cross sectional area. Profile (2) has a smaller area with a significant increase in moment and normal values. In both cases, the interpolated profile has a smaller target function’s value. Thus, the algorithm most likely works well and moves forward toward the global optima.

Figure 5: Compared profiles

5 CONCLUSIONS

The developed algorithm can find a quasi-optimal profile with safe lining stress and surface settlement. Optimal solution can be reached with nowadays high speed powerful computers. In the algorithm, local search mutation can improve the target function’s value progression and can help to find a better solution. The resulted profile does not depend on expectation or designer experience. This optimization tool can be adapted to work on different soil types and different tunnel depths.
REFERENCES


Simulation of Seismic Wave Propagation for Reconnaissance in Mechanized Tunneling Using the Spectral Element and Nodal Discontinuous Galerkin Method

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Abstract

Seismic reconnaissance is a useful tool for risk prediction and optimization of the tunnel excavation process. In order to understand the seismic wave field in a tunnel environment, numerical simulations using the Spectral Element Method (SEM) and the Nodal Discontinuous Galerkin Method (NDG) are carried out. While both methods are based on the Finite Element Method (FEM), they involve high order polynomials to reconstruct the wave field inside the elements leading to accurate elastodynamic simulations. The SEM is a fast and accurate method widely used in the seismic community. Its biggest drawback, however, is the limitation to hexahedral elements. To construct complex heterogeneous models with a tunnel included, we adapt the NDG to the elastic wave equation. With this method we can build models with tetrahedral elements without the need to invert global matrices. Using both techniques, we perform high resolution massively parallel simulations of seismic waves initialized by a single force acting either on the front face or the side face of the tunnel. The aim is to produce waves that travel mainly in the direction of the tunnel track and to gain a maximum amount of information from the back scattered part of the wave field.

Keywords: Seismic simulation, spectral element method, nodal discontinuous galerkin method
1 INTRODUCTION

For tunnel excavation, it is essential to predict the geological environment and the underlying geotechnical parameters. The soil conditions are influencing the safety of the construction site, the speed of excavation, the wearing of the tools and, together with the supporting measurements, the surface settlements. In order to get information about the ground along the tunnel track, especially during construction, one can use seismic imaging. However, interpreting the data can be very difficult, because measurements are usually taken inside the tunnel. Simulation of a seismic reconnaissance experiment can help to understand the wave field in the domain of a tunnel [3]. Therefore, we perform numerical computations with two different numerical methods, the Spectral Element Method (SEM) and the Nodal Discontinuous Galerkin Method (NDG).

Simulation of seismic waves in realistic full 3D models plays an important role in understanding the earth interior. We often deal with large domains compared to the wavelength, making it necessary to use very efficient and accurate methods. The established techniques reach from Finite Differences (FD) [12] and Spectral or Pseudo Spectral Methods [4] to Boundary Element Methods (BEM) [8]. But these schemes have difficulties in modeling realistic domains, including discontinuities, large numbers of different materials or realistic topography.

To flexibly model the domain of interest, the Finite Element Method (FEM) has been applied to elastic wave simulation [2]. Normally, first or second order elements are used, leading to a fine discretisation of the model in high frequency simulations. This fact makes the method ineffective for large scale parallel simulations because one needs to invert very large matrices to solve the problem. To overcome most of these difficulties, the Spectral Element Method (SEM) was adapted to seismic wave propagation, especially for global and regional seismic settings [9]. When using the SEM for tunnel reconnaissance, problems in building complex models arise due to its limitation to hexahedral elements.

By introducing a modeling technique we call tunnel brick meshing (figure 1) the construction of a tunnel mesh becomes more flexible and enables hexahedral meshing for complex models. Nevertheless there are situations where meshing of certain geometries including sharp edges, fine layering, strong surface topography and tunnels with curvature become very cumbersome. To avoid these difficulties, we developed a simulation based on the Nodal Discontinuous Galerkin Method (NDG). It gives us the ability to use tetrahedral meshes for high order elastic wave simulation. Both methods, the SEM and NDG allow us to perform flexible, fast and accurate simulations of the seismic wave field.
2 METHODS

2.1 Spectral Element Method

The concept of the SEM is very similar to ordinary FEM. A model domain $\Omega$ is subdivided into non-overlapping elements $\Omega_e$ with $e = 1, \ldots, n_e$ where $n_e$ is the total number of elements. The complete domain can now be built of elements so that $\Omega = \bigcup_{e=1}^{n_e} \Omega_e$. The aim is to solve the elastic wave equation [1] given by

$$\rho(\vec{x}) \frac{\partial^2 \vec{u}(\vec{x}, t)}{\partial t^2} = \nabla \cdot \mathbf{T}(\vec{x}, t) + \vec{f}(\vec{x}, t),$$

where $\rho$ is the density and $\vec{u}(\vec{x}, t)$ the displacement field varying in space and time. $\mathbf{T}$ is the stress tensor which can be calculated with Hook's law from the displacement field by

$$\mathbf{T}(\vec{x}, t) = \mathbf{c}(\vec{x}) : \nabla \vec{u}(\vec{x}, t),$$

where $\mathbf{c}$ denotes a fourth-order tensor which describes the elastic properties of the medium.

High accuracy and efficiency is achieved by representing $\vec{u}$ and $\mathbf{T}$ within the elements by Lagrange polynomials of degree $n$. To realize this feature, the element is sampled by special interpolation nodes, the Gauss-Lobatto-Legendre (GLL) points on which the wave field is stored. Using a weak form of the elastic wave equation, the numerical integration is realized on these nodes by the GLL quadrature. These choices lead to a formulation where the global mass matrix of the system becomes diagonal for hexahedral elements in 3D. Typically, a spatial order of $n = 4$ or higher is used for the simulation.

2.2 Nodal Discontinuous Galerkin Method

Since the SEM is limited to hexahedral elements, it can be very time consuming or even impossible to model certain geometries, especially one including a tunnel. In figure (1) we show a special meshing strategy to deal with a tunnel in an hexahedral environment. It might be necessary to build models with very sharp edges, many layers or even a tunnel with a curvature. For these purposes we decided to develop an alternative simulation tool based on the Nodal Discontinuous Galerkin Method (NDG). It allows more flexible modeling and exhibits high order properties similar to those of the SEM. It uses a specially designed point discretisation inside the elements, similar to the GLL points, leading to an accurate reconstruction of the wave field inside the element. [10]
Flexibility is achieved by using numerical fluxes to connect the elements forming the entire model. This gives us the possibility to use tetrahedral elements for the mesh. In contrast to the SEM we have a local representation of the governing equations and we also do not need to invert global matrices. Nevertheless, the calculation of fluxes makes the NDG more expensive than the SEM.

If we rewrite the wave equation in (1) we get the following set of hyperbolic differential equations for the 2D case [7]:

\[
\begin{align*}
\frac{\partial}{\partial t} \sigma_{xx} - (\lambda + 2\mu) \frac{\partial}{\partial x} v - \lambda \frac{\partial}{\partial y} w &= F_1 \\
\frac{\partial}{\partial t} \sigma_{yy} - \lambda \frac{\partial}{\partial x} v - (\lambda + 2\mu) \frac{\partial}{\partial y} w &= F_2 \\
\frac{\partial}{\partial t} \sigma_{xy} - \mu \left( \frac{\partial}{\partial x} w + \frac{\partial}{\partial y} v \right) &= F_3 \\
\rho \frac{\partial}{\partial t} v - \frac{\partial}{\partial x} \sigma_{xx} - \frac{\partial}{\partial y} \sigma_{xy} &= \rho F_4 \\
\rho \frac{\partial}{\partial t} w - \frac{\partial}{\partial x} \sigma_{xy} - \frac{\partial}{\partial y} \sigma_{yy} &= \rho F_5
\end{align*}
\]

with \(\sigma_{ij} = \sigma_{ij}(\vec{x}, t)\) the stress components, \(\lambda = \lambda(\vec{x})\) and \(\mu = \mu(\vec{x})\) the Lamé constants, describing the elastic properties of the medium. \(v = v(\vec{x}, t)\) and \(w = w(\vec{x}, t)\) denote the x- and y-components of the velocity field of the resulting waves and \(\rho = \rho(\vec{x})\) is density. \(F_i = F_i(\vec{x}, t)\) is an external force, which excites the waves for the simulations. This scheme allows us to model complex geometries and run high order simulations, typically of order 4 or higher. In the NDG and SEM, it is sufficient to use at least 5 points per wavelength for a stable and accurate simulation [10]. This leads to an element discretisation of the model, where the element size is in the order of a wavelength.

### 2.3 Tunnel brick modelling

Figure (1) shows a meshing strategy to model a tunnel within an hexahedral mesh using a macro element called tunnel brick. On the outer boundary, we enforce regular elements allowing us to plug the tunnel brick into a more complex model where the only requirement is to have regular elements on the connection surface between the tunnel brick and the rest of the mesh. In this way, an ending tunnel can be incorporated into complex exterior meshes as shown in figure (1) on the right hand side.
Simulation of Seismic Wave Propagation

Figure 1: Tunnel brick meshing.
A tunnel is built with hexahedral elements and the outer surface is constructed with regular elements. This provides the ability to plug the tunnel brick into a bigger model, allowing for more simple tunnel meshing or a flexible change of the tunnel position. The left hand side shows the concept of the tunnel brick, the right hand side shows a complex hexahedral mesh, realized with tunnel brick meshing.

Another advantage is the ability to change the position of the tunnel without having to remesh the complete domain.

3 RESULTS

3.1 Comparison between SEM and NDG
We compare both methods with regard to a tunnel seismic experiment. The software for the simulation with the SEM is an open source tool mainly used for regional earthquake simulations call SPECFEM3D [5]. It can use arbitrary hexahedral meshes and is carefully benchmarked so that we have a reliable reference solution. For the NDG we developed our own software package based on the theory of Hesthaven and Warburton [6]. Both software tools are parallelized using MPI and are suited for high performance computing (HPC).

The meshing of the tunnel is done with the software CUBIT [11], a program that offers meshing using hexahedral or tetrahedral elements. The model dimensions are a width of $x = 40 \text{ m}$, a length of $y = 60 \text{ m}$ and a height of $z = 36 \text{ m}$. A tunnel with $r = 6 \text{ m}$ radius is inserted which ends after $l = 25 \text{ m}$ in the domain. In a distance of $d = 20 \text{ m}$ from the front face of the tunnel a discontinuous material change describes a perturbation of the model.

As source we use a single force perpendicular to the tunnel axis $s = 8 \text{ m}$ away from the front face on the side face of the tunnel. The source time function is a Ricker wavelet with a central frequency of $f_c = 210 \text{ Hz}$ leading to a minimum wavelength of $\lambda \approx 1.5 \text{ m}$ for the slowest waves. Overall, we have 60 receivers with a distance of
\( \Delta x = 1 \, m \), of which 25 are on the side face of the tunnel and the others are continuing the receiver line into the model.

![Figure 1: Meshes for tunnel simulations.](image)

On the left hand side the mesh contains 22084 hexahedral elements and is used for the SEM calculations. The model on the right hand side composed of 83714 elements is used for NDG simulations. The overall dimensions are \( 40 \, m \times 60 \, m \times 36 \, m \), the tunnel has a diameter of \( r = 6 \, m \) and a length of \( l = 25 \, m \). The layer has a distances of \( d = 20 \, m \) from the front face of the tunnel. The resolution of the mesh allows to simulate waves with a frequency of up to \( f_c = 210 \, Hz \) and a spatial resolution of about 1.5 \( m \).

<table>
<thead>
<tr>
<th>Nr.</th>
<th>Color</th>
<th>( v_p [\frac{m}{s}] )</th>
<th>( v_s [\frac{m}{s}] )</th>
<th>( \rho [\frac{kg}{m^3}] )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>green</td>
<td>2000</td>
<td>1500</td>
<td>2000</td>
</tr>
<tr>
<td>2.</td>
<td>yellow</td>
<td>3500</td>
<td>2500</td>
<td>3000</td>
</tr>
</tbody>
</table>

Figure (3) shows synthetic seismograms for the two simulations. Black lines represent the NDG simulation and red lines the SEM simulation. Apparently, there is very good agreement between the two seismogram sections. The section is composed of all 60 records for the different receivers leading to an offset versus time plot for the \( x \)-component of the velocity field. Receiver 1-25 are installed on the tunnel side face and receiver 26-60 are completing this line into the model. One can see the source position, where the signal starts at \( t = 0.005 \, s \). A surface wave marked with \( R \) travels along the surface of the tunnel and is converted into a shear wave \( S \). The discontinuity is located near receiver 45 where a reflection \( S' \) occurs that propagates back to the tunnel and converts to a surface wave \( R' \). Although waves are suppressed by absorbing boundary conditions there are still small artificial reflections left, which
can be identified at AB. At the top of the model, there is a free boundary producing a strong reflection marked as FS. M shows a surface wave which is reflected from the front face.

![Seismic section of the comparison.](image)

**Figure 3:** Seismic section of the comparison.
Section of seismograms for the comparison between SEM in red and NDG in black. There is a very good agreement between both methods. R and R' denotes surface waves, S, S2 and S' shear waves. AB is a reflection from the outer boundary of the model and FS a reflection from the free surface on the top of the model. M shows a reflection from the front face of the tunnel.

## 4 SUMMARY AND OUTLOOK

In this study, we present an optimized strategy for hexahedral meshing of a model containing a tunnel. It allows to build the tunnel in a separate step and to embed it into any surrounding model in a later step. In this way, we can deal with complex models in a flexible manner when using the SEM. Unfortunately, not all geometries can be satisfactorily built with hexahedral elements. To overcome this limitation, we developed a new software based on NDG to simulate high order elastic wave propagation which can handle tetrahedral meshes in an effective manner. Comparison of
both methods shows very good agreement between the results. With these tools in hand, we can model and simulate tunnel reconnaissance with elastic waves in environments exhibiting nearly arbitrary geometries.

In future work, we intend to further analyze seismic wave propagation in the domain of a tunnel, especially in soft soil conditions. For this, it is planned to incorporate attenuation and use inverse modeling techniques to establish a full waveform inversion approach to tunnel reconnaissance.

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Risk and Reliability Analysis
A Two-Step Approach for Reliability Assessment of a Tunnel in Soft Soil

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Abstract

We assess the reliability of a tunnel in Keuper marl with uncertain mechanical properties. The tunnel is constructed by the conventional tunneling method. The limit state function is expressed in terms of a two-dimensional finite element model of the tunnel. Plain strain finite elements are used to represent the soil and the yield surface is modeled with a hardening plasticity soil model. The three-dimensional arching effect is approximated by application of the stress reduction method. In a first step, the reliability analysis is performed by application of the first order reliability method (FORM) and the results are verified by importance sampling. The FORM provides information on the sensitivity of the reliability in terms of the uncertain variables. This information is used in a second step to account for the inherent spatial variability of the ground parameters with the largest influence through a random field modeling. The discretization of the random field leads to a large number of random variables. Therefore, we apply the subset simulation method, which is an adaptive Monte Carlo method known to be especially efficient for such high dimensional problems. The analysis is performed using a reliability tool that is integrated into the SOFiSTiK finite element software package.

Keywords: Reliability analysis, Keuper marl, FORM, sensitivities, subset simulation
1 INTRODUCTION

In tunnel construction, a typical design requirement is the restriction of surface settlements to acceptable values with sufficient reliability. This serviceability condition is particularly important in urban environments, where tunnel induced settlements may have an impact on existing structures. Predictions of the ground displacement can be made with the help of non-linear finite element models. However, there is significant uncertainty involved in the choice of the model parameters. Moreover, the mechanical properties of the soil exhibit an inherent spatial variability. These issues need to be addressed in a proper assessment of the adequacy of the design.

In this paper, we account for the uncertainties in the model parameters and evaluate the probability that the tunnel induced settlements exceed a predefined threshold. In addition, we investigate the influence of the spatial variability of soil parameters on the analysis results.

2 MODEL DESCRIPTION

2.1 Mechanical Model

A conventional driven tunnel with a horse-shoe shaped profile is considered in this study (see figure 1). The problem is modeled in the SOFiSTiK finite element (FE) software package, using plain strain finite elements. The numerical model has a width of 80m and a total height of 26m. In this study, we are interested in surface settlements over the tunnel center line (point A in figure 1). The excavation process is modeled by application of the stress reduction method, which approximately accounts for the three-dimensional arching effect of the stress-distribution.

Three different ground layers are incorporated in the model; the layers are illustrated in figure 1. The cover layer is a man-made fill and has a depth of 5.4m. Heavily weathered soft rock known as Keuper marl forms the second layer. The thickness of this layer is assumed to be 16.8m. We adopt a hardening plasticity soil model [1] to describe the material behavior of the first two layers. This material model allows for a realistic description of the stiffness and hardening behavior of soft soil in settlement analysis. The material properties of the cover layer are as follows: elastic modulus for unloading-reloading: 30MPa, Poisson’s ratio: 0.2, specific weight: 20kN/m³, friction angle: 25°, cohesion: 10kPa, oedometric stiffness modulus: 10MPa, stiffness modulus for primary loading: 10MPa. The exponent in the hardening law is selected as 0.5 for the first and the second layer. The angle of dilatancy is assumed as zero,
corresponding to an non-associated flow rule. The soil parameters of the Keuper marl layer are assumed to be random and their probabilistic description is given in section 2.2. Strong limestone constitutes the bottom layer. The Mohr-Coulomb law is applied for this layer. The material properties are: Young’s modulus: 575MPa, Poisson’s ratio: 0.2, specific weight: 23kN/m³, friction angle: 35°, cohesion: 200kPa. Due to the much larger stiffness of the limestone compared to the stiffness of the overlaying materials, only 3.8m of this layer are modeled.

![Ground layers considered in the model](image)

**Figure 1:** Ground layers considered in the model

The height of the tunnel above the limestone layer is 6.2m. Consequently, the tunnel is located in a depth of 16m below the ground surface. At the intersection of the second and the third layer, the tunnel has a width of 9.16m. In the vicinity of the tunnel the Keuper marl is reinforced with nails. This is modeled by increasing the cohesion in the affected region (see figure 1) by 25kPa. Moreover, it is assumed that the tunnel is located above the groundwater level. The shotcrete lining is modeled using linear beam elements with a normal stiffness of 10.5GN and a flexural rigidity of 26.78MNm².

### 2.2 Stochastic Model

The cover layer and the limestone layer are considered as deterministic in the analysis. Since the cover layer is a man-made fill, we assume that its soil properties are well-known, and the associated uncertainties are small compared to the uncertainties in the material description of the Keuper marl layer and can be neglected. The limestone layer is also modeled as deterministic because - due to its large stiffness - the contribution of this layer to the surface settlements is negligible. The probability dis-
tribution describing the uncertainties in the material parameters of the Keuper marl layer are listed in table 1. We assume that the stiffness modulus for primary loading \( E_{50}^{\text{ref}} \) equals the oedometric stiffness modulus \( E_{\text{oed}}^{\text{ref}} \). We also consider a correlation of 0.7 between the parameters \( E_{\text{oed}}^{\text{ref}} \) and \( E_{\text{ur}} \). The friction angle and the cohesion are assumed to have a negative correlation of -0.5.

In conventionally driven tunnels, there is usually a large uncertainty in the choice of the relaxation factor \( \beta \in [0,1] \) of the stress reduction method [2]. In this study \( \beta \) is modeled as a beta-distributed random variable (see table 1).

**Table 1:** Uncertain parameters of the Keuper marl layer

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Distribution</th>
<th>Mean</th>
<th>C.o.V.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Relaxation factor ( \beta )</td>
<td>Beta(0.0,1.0)</td>
<td>0.5</td>
<td>10%</td>
</tr>
<tr>
<td>Elastic modulus for un-/reloading ( E_{\text{ur}} ) [MPa]</td>
<td>Lognormal</td>
<td>80.0</td>
<td>32%</td>
</tr>
<tr>
<td>Oedometric stiffness modulus ( E_{\text{oed}}^{\text{ref}} ) [MPa]</td>
<td>Lognormal</td>
<td>30.0</td>
<td>32%</td>
</tr>
<tr>
<td>Poisson’s ratio ( \nu )</td>
<td>Beta(0.0,0.5)</td>
<td>0.2</td>
<td>15%</td>
</tr>
<tr>
<td>Friction angle ( \varphi ) [°]</td>
<td>Beta(0.0,45.0)</td>
<td>20.0</td>
<td>15%</td>
</tr>
<tr>
<td>Cohesion ( c ) [kPa]</td>
<td>Lognormal</td>
<td>25.0</td>
<td>30%</td>
</tr>
<tr>
<td>Specific weight ( \gamma ) [kN/m³]</td>
<td>Lognormal</td>
<td>24.0</td>
<td>5%</td>
</tr>
</tbody>
</table>

In the second part of this study, the inherent spatial variability of the parameters \( E_{\text{oed}}^{\text{ref}} \) and \( E_{\text{ur}} \) is taken into account. This is achieved by modeling the two parameters as cross-correlated homogeneous random fields. It is assumed that the spatial variability depends only on the separation in horizontal and vertical direction between two locations, denoted by \( \Delta x \) and \( \Delta y \), respectively. The following exponential autocorrelation function is chosen for both random fields:

\[
\rho(\Delta x, \Delta y) = \exp\left(-\frac{\Delta x}{l_x} - \frac{\Delta y}{l_y}\right)
\]

(1)

where \( l_x \) and \( l_y \) denote the correlation lengths in horizontal and vertical direction, respectively. The cross-correlation coefficient function is:

\[
\rho_{\text{cross}}(\Delta x, \Delta y) = \rho_c \cdot \rho(\Delta x, \Delta y)
\]

(2)

where \( \rho_c \) denotes the correlation of \( E_{\text{oed}}^{\text{ref}} \) and \( E_{\text{ur}} \) at the same location.

The midpoint method [3] is used for the discretization of the random fields. In this study, the stochastic finite element (SFE) mesh is a coarser variant of the deterministic finite element mesh. The SFE mesh consists of 142 deterministic finite element
patches and is illustrated in figure 2, where the patches in the second layer are indicated by areas of different color. In the midpoint method, the random field is assumed to be constant in each SFE and represented by its value at the midpoint of the SFE.

![Figure 2: Stochastic and deterministic finite element mesh](image)

3 RELIABILITY ANALYSIS

3.1 Introduction

In reliability analysis, we compute the probability of failure of a system as:

\[ P_f = \Pr\{g(X) \leq 0\} = \int_{g(x) \leq 0} f_X(x) \, dx \]

where \( g \) is called limit-state function and \( X \) is the \( K \)-dimensional vector of random variables with joint probability density function \( f_X \). Failure of the system occurs if \( g(x) \leq 0 \). In this study the limit state function is defined as:

\[ g(x) = u_{\text{threshold}} - u_A(x) \]

where \( u_A \) denotes the surface settlement in point \( A \) (see figure 2) as computed with the FEM, and \( u_{\text{threshold}} \) is the maximum allowed settlement.

For most reliability methods it is convenient to transform the original space of random variables \( X \) to a space of independent standard normal random variables \( U \). The limit-state function defined in the transformed space is denoted by \( G : U \rightarrow \mathbb{R} \). Consequently, equation 3 can be rewritten as:

\[ P_f = \Pr\{G(U) \leq 0\} = \int_{G(u) \leq 0} \varphi_U(u) \, du \]

where \( \varphi_U \) is the \( K \)-dimensional standard normal joint probability density function.
3.2  FORM
The first order reliability method (FORM) solves the reliability problem formulated in equation 5 approximately by a linearization of the limit-state function \( G(\mathbf{u}) \) at the design point \( \mathbf{u}^* \). The design point is defined as the most probable point of failure. It is obtained by a minimization of \( \sqrt{\mathbf{u}^T \mathbf{u}} \) subjected to \( G(\mathbf{u}) = 0 \). The quality of the approximation depends on how well \( G(\mathbf{u}) \) can be approximated by a linear function. The probability of failure defined in equation 5 is approximated as:

\[
P_f \approx \Phi(-\beta)
\]

where \( \Phi \) denotes the standard normal cumulative distribution function, and \( \beta \) is the FORM reliability index and is defined as \( \beta = \sqrt{\mathbf{u}^*^T \mathbf{u}^*} \). In this work, the standard HL-RF method \([4, 5]\) is applied to solve the optimization problem.

A by-product of the FORM are sensitivities at the design point. In this work the sensitivity is expressed in terms of the influence coefficients \( \alpha_i \). The coefficients \( \alpha_i \) represent the direction cosines along the coordinate axes \( U_i \); they are helpful for estimating the most important uncertain parameters in terms of their influence on the structural reliability \([6]\).

3.3 Importance Sampling
In the standard Monte Carlo simulation, we draw \( N \) samples from \( \mathbf{U} \) and count how often the failure event \( G(\mathbf{U}) \leq 0 \) is observed. An estimate of the probability of failure is obtained by dividing the number of failures by \( N \). This procedure becomes inefficient if the problem to investigate has a small failure probability.

The idea of importance sampling is to draw the samples not from \( \mathbf{U} \), but from a distribution that produces more samples in the failure domain. This distribution is called importance sampling function. In this work, the importance sampling function is chosen as the \( K \)-dimensional normal distribution with unit variance centered around the design point \( \mathbf{u}^* \).

3.4 Subset Simulation
The subset simulation method \([7]\) is an adaptive Monte Carlo method that is efficient for high dimensional problems. In this method, the probability of failure is expressed as a product of larger conditional probabilities.
Let us introduce $M$ intermediate failure events $F_i$, with $1 \leq i \leq M$ and $F_1 \supset F_2 \supset \ldots \supset F_M = F$. The failure events $F_i$ are defined as $F_i = \{ G(u) \leq c_i \}$, where $c_i \in \mathbb{R}$ with $c_1 > \ldots > c_M = 0$. The probability of failure $P_f = \Pr(F_M)$ can be expressed as:

$$P_f = \prod_{i=1}^{M} \Pr(F_i|F_{i-1})$$  \hspace{1cm} (7)

where $F_0$ denotes the certain event, and $\Pr(F_i|F_{i-1})$ is the probability of the event $F_i$ conditioned on the occurrence of the event $F_{i-1}$. The values $c_i$ can be chosen adaptively such that the conditional probabilities $\Pr(F_i|F_{i-1})$, $i < M$ correspond to a given value $p_0$.

Standard Monte Carlo simulation is applied to compute $\Pr(F_1)$. The conditional probabilities $\Pr(F_i|F_{i-1})$ for $2 \leq i \leq M$ are approximated by means of Markov Chain Monte Carlo (MCMC) techniques. In this work, the component-wise Metropolis-Hastings algorithm is used [7]. Moreover, $p_0$ is fixed to 10%.

### 3.5 Two-Step Procedure

To solve the reliability problem we adopt the following two-step procedure. In the first step, the spatial variability of the material parameters is neglected. The stochastic model is described by the seven random variables listed in table 1. The probability of failure is approximated by FORM. Importance sampling is used to verify the results obtained with FORM and to investigate the non-linearity of the limit-state function.

In the second step, the spatial variability of the parameters with the largest influence on the failure probability is considered. This is modeled by a cross-correlated random field. The random field model results in a large number of additional random variables, and, FORM cannot be applied efficiently. Therefore, the subset simulation method is used to perform the reliability analysis.

### 4 RESULTS AND DISCUSSION

#### 4.1 Step 1: Spatial Variability is Neglected

The results of the analysis neglecting the spatial variability of the soil are listed in table 2. $\beta_{\text{FORM}}$ denotes the reliability index obtained with FORM for different threshold values $u_{\text{threshold}}$. $P_{f,\text{FORM}}$ is the associated probability of failure according to equation 6. The number of steps required for convergence of the HL-RF algorithm is given in the column $N_{\text{step,FORM}}$. Importance sampling with 1000 limit-state
function evaluations was used to verify the FORM results. The estimate is listed in column $P_{f, IS}$, the associated coefficient of variation is given in $CV_{IS}$. Comparing the results obtained by FORM with the results from importance sampling, we observe that for all investigated $u_{threshold}$, FORM gives a good approximation of the probability of failure.

Table 2: FORM and importance sampling results

<table>
<thead>
<tr>
<th>$u_{threshold}$ [cm]</th>
<th>$\hat{\beta}_{FORM}$</th>
<th>$P_{f, FORM}$</th>
<th>$N_{Step, FORM}$</th>
<th>$P_{f, IS}$</th>
<th>$CV_{IS}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.12</td>
<td>4.5 $\cdot$ 10^{-1}</td>
<td>2</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>2</td>
<td>2.0</td>
<td>2.1 $\cdot$ 10^{-2}</td>
<td>5</td>
<td>2.0 $\cdot$ 10^{-2}</td>
<td>5.0%</td>
</tr>
<tr>
<td>3</td>
<td>3.3</td>
<td>5.6 $\cdot$ 10^{-4}</td>
<td>7</td>
<td>5.6 $\cdot$ 10^{-4}</td>
<td>6.2%</td>
</tr>
<tr>
<td>4</td>
<td>4.1</td>
<td>1.9 $\cdot$ 10^{-5}</td>
<td>9</td>
<td>2.0 $\cdot$ 10^{-5}</td>
<td>7.0%</td>
</tr>
<tr>
<td>5</td>
<td>4.8</td>
<td>8.8 $\cdot$ 10^{-7}</td>
<td>10</td>
<td>9.8 $\cdot$ 10^{-7}</td>
<td>8.4%</td>
</tr>
</tbody>
</table>

Figure 3 depicts the squared influence coefficients $\alpha^2$ in a pie graph. It is observed that the variable with the largest influence is the oedometric stiffness modulus $E_{ref}^{oed}$. In the next step, we account for the spatial variability of this parameter by a random field modeling. Since $E_{ref}^{oed}$ is strongly correlated with the elastic Young’s modulus $E_{ur}$, the later parameter is also modeled as a random field.

![Squared influence coefficients](image)

**Figure 3:** Squared influence coefficients

### 4.2 Step 2: Spatial Variability is Considered

In this section only the case $u_{threshold} = 3$cm is investigated. As correlation length we choose $l_x = 20$ m and $l_y = 5$ m. The parameters $E_{ref}^{oed}$ and $E_{ur}$ are modeled as cross-correlated random fields, compare section 2.2. With subset simulation using 500 samples per conditioning step we computed a probability of failure of $6.8 \cdot 10^{-5}$. In
a second run with 3500 samples per conditioning step we obtained a probability of failure of $6.1 \cdot 10^{-5}$. Comparing this to the FORM estimate of $5.6 \cdot 10^{-4}$ from table 2, we observe that neglecting spatial variability results in a significantly conservative estimate. This is due to the fact that in the model with spatial variability a local loss of strength becomes possible and a global loss of strength less likely. Among all possible local losses, only a small fraction will lead to failure.

5 SUMMARY

In this paper we accounted for the risk that tunnel induced settlements in soft soil exceed a specified threshold. We used FORM to evaluate the reliability of the problem for the case where spatial variability is neglected. The subset simulation method was used to perform reliability analysis for the case where spatial variability of the most influential ground parameters is taken into account. It was shown that the spatial variability has a significant influence on the computed reliability.

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Concepts for Reliability Analyses in Mechanised Tunnelling – Part 1: Theory

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Abstract

Reliability analysis in tunnelling is of particular relevance, since, in contrast to common engineering tasks, only limited information is available to describe the local geology and corresponding material behaviour. In Part one of this paper, concepts for reliability analyses in mechanised tunnelling are presented. Their applicability is demonstrated in Part two by means of an application example. Uncertainty quantification based on stochastic, interval, fuzzy, and imprecise probability approaches is motivated. A strategy is presented for numerical analyses taking polymorphic uncertainty models into account. Due to the large computational effort of advanced numerical models in mechanised tunnelling, efficient and reliable surrogate models are required. Here, artificial neural networks are used as surrogate models to perform reliability analyses.

Keywords: mechanised tunnelling, surrogate model, artificial neural network, uncertainty, reliability analysis
1 INTRODUCTION

Reliability assessment in underground engineering requires computational models which allow for a realistic description of the relevant physical phenomena and adequate strategies to account for the uncertainty of the underlying parameters. This holds in particular for numerical simulations in tunnelling, where the spatio-temporal variability of parameters associated with the geology and the excavation process needs particular consideration. In general, due to inherently limited information of geotechnical parameters (e.g. related to the geometry of soil layers) and the heterogeneity of soils, leading to a scatter of the material parameters used in constitutive relations of geological materials, purely stochastic reliability concepts cannot be applied in tunnelling analyses. It should be noted, that the uncertainties in geotechnical engineering are associated with variability of parameters, with reduced information represented by a simplified model, or with the lack of knowledge about certain geological boundary conditions. Therefore, reliability concepts in tunnelling should allow to consider both aleatory and epistemic uncertainty.

In mechanised tunnelling, the various interactions of the tunnelling process with the underground and existing surface structures depends on the selected construction technique and the parameters chosen for the steering of the tunnel boring machine (TBM). Reliability analyses in TBM tunnelling may be performed either a priori, in the initial stage of a project, or simultaneously during the construction process. Whereas a priori reliability analyses are important to support decisions related to the tunnel design, reliability analyses performed simultaneously to the construction process may be essential to support decisions, e.g. regarding to optimal steering parameters or regarding to compensation actions to minimize risks of damage in existing buildings.

In Part one of this joint paper, selected models for quantification of uncertain parameters in mechanised tunnelling are presented and their treatment in numerical analyses is described. As a consequence of the large computational effort connected with the application of advanced numerical simulation models, artificial neural network based surrogate models are discussed, too. An example is presented in Part two of this contribution [1].
2 UNCERTAINTY QUANTIFICATION

In engineering practice, the reliability of structures can be estimated by evaluating critical limit states due to structural failure and serviceability. Uncertainties of structural parameters (representing structural actions and describing the structural behaviour) can be considered by several approaches, e.g. deterministic using safety factors, stochastic, interval, fuzzy, or polymorphic approaches. In tunnelling, different limit states and strategies to provide a safety margin against reaching this limit states are in general defined individually for the different components involved (i.e. the soil, the linings, existing buildings etc.). As a consequence of the heterogeneity of geological materials and the limited information available from site investigation, uncertainty models, taking epistemic and aleatory sources of uncertainty into account, see e.g. [2], are valuable tools for the project-specific risk assessment in tunnelling.

2.1 Stochastic Numbers

Aleatory uncertainty of structural parameters, e.g. describing geometry and behaviour of soil layers, can be quantified by stochastic numbers. For each stochastic number, a stochastic model has to be selected, which consists of defining its probability density function (pdf) and cumulative distribution function (cdf). It is possible to consider spatial and time varying uncertainty with stochastic models. However, an adequate data base with a sufficiently large number of samples is required to identify stochastic models and estimate the corresponding parameters.

In tunnelling, stochastic reliability concepts have been recently applied in [3]. To account for spatial variability of geological parameters, random fields can be defined to describe local correlations, see e.g. [3, 4]. In general, assumptions are required to select a correlation function and to define the corresponding correlation length.

In case of time varying uncertainty, structural parameters can be represented as stochastic processes. Typical examples of stochastic processes in mechanised tunnelling are parameters describing the excavation process, e.g. excavation rate depending on the type of soil.

2.2 Intervals and Fuzzy Numbers

If only limited information is available, epistemic uncertainty should be considered within the reliability and risk assessment. Intervals and fuzzy numbers are suitable to quantify epistemic uncertainty. An interval is defined by its lower and upper interval
bounds. Intervals can be used to quantify uncertain geotechnical parameters with rare information. In general, an interval is defined on the basis of expert experience. In geotechnical engineering and tunnelling, intervals are used on a daily basis, e.g. in the context of rock mass classifications such as the rock mass rating (RMR) system [5]. In [6], a comparative study has been performed with stochastic numbers and intervals in a geotechnical analysis.

Fuzzy numbers are suitable, if interval quantified information can be assessed by membership functions. For each realisation, its level of membership to the fuzzy set is described by a number between 0 and 1. A typical application of using fuzzy numbers in mechanised tunnelling is the consideration of imprecise measurements within risk and reliability analyses. Imprecise measurements obtained from monitoring during the tunnelling construction process can be quantified as fuzzy processes. Some approaches for the estimation of membership functions are presented in [7]. It is also possible to obtain membership functions due to time discretisation of data series, see e.g. [8]. In [9], fuzzy models are applied in geotechnical engineering.

2.3 Probability Boxes and Fuzzy Stochastic Numbers

Concepts of imprecise probabilities can be applied to quantify the variability of uncertain parameters in case of limited statistical information, e.g. small sample sizes for estimation of corresponding stochastic models. The probability box (p-box) approach, see e.g. [10], can be used to define imprecise stochastic numbers by its lower and upper bound cdf, i.e. the probability of each realisation is an interval. In general, arbitrary stochastic models can be used for lower and upper bound cdf, including empirical distributions. In [11], p-boxes are used within a random set finite element analysis for reliability assessment of tunnel construction according to the New Austrian Tunnelling Method.

The p-box approach can be generalized to the concept of fuzzy stochastic numbers. In this case, the probability of each realisation is a fuzzy number. Fuzzy stochastic numbers can be described by stochastic models with fuzzy model parameters. In many cases, soil parameters can be described as fuzzy stochastic numbers, see e.g. [6]. If enough information is available to quantify the variability of material parameters, epistemic uncertainty described by fuzziness goes to zero and a stochastic number is obtained in this case.
3 NUMERICAL ANALYSIS WITH UNCERTAIN DATA

The uncertainty models described in Section 2 can be used to quantify uncertain input parameters of tunnelling models with respect to available information. Often, this results in a mix of several (polymorphic) uncertainty models to be handled within reliability assessment. The consideration of polymorphic uncertain input parameters within reliability analyses in mechanised tunnelling requires advanced numerical strategies to compute failure probabilities. It is necessary to combine different stochastic and non-stochastic approaches, e.g. stochastic analysis and interval analysis.

In the proposed modelling approach, a strategy for fuzzy stochastic analyses is presented, see e.g. [6, 12]. It can be applied to compute fuzzy failure probabilities in case of polymorphic uncertain data described by fuzzy stochastic, stochastic, and fuzzy numbers as well as intervals. A numerically efficient way is to discretise the membership functions of all fuzzy input parameters (including the fuzzy parameters of fuzzy stochastic numbers) into $\alpha$-cuts, see Figure 1 exemplified for two fuzzy input parameters $\tilde{x}_1$ and $\tilde{x}_2$.

![Figure 1: Fuzzy stochastic reliability analysis in mechanised tunnelling](image)

For all $\alpha$-cuts, stochastic analyses must be performed to compute failure probabilities, using e.g. Monte Carlo simulation. In each Monte Carlo simulation, samples of (fuzzy) stochastic input parameters of the tunnelling model are used to compute...
the corresponding responses and to evaluate the limit states of selected construction stages \( n \). An optimization problem has to be solved to get the minimal and maximal (i.e. lower and upper bound) failure probabilities of each \( \alpha \)-cut, denoted as \( [n]_{\alpha} P_f \) and \( [n]_{\alpha u} P_f \) in Figure 1. For this task, established methods such as the \( \alpha \)-level optimization according to [13] are available. As a result of the fuzzy stochastic analysis, the membership function of the fuzzy failure probability \( \tilde{P}_f \) is obtained in discretised form. If the fuzzy stochastic analysis is performed for a sequence of construction stages \( n = 1, \ldots, N \), time varying fuzzy failure probabilities \( \tilde{P}_f(t) \) can be computed.

4 NEURAL NETWORK BASED SURROGATE MODELS

Evidently, the large number of realisations required for fuzzy stochastic analyses may soon lead to a prohibitively large computational effort, if complex, large scale numerical models are used for the representation of the tunnel construction process. To make reliability analyses feasible in association with realistic, process oriented computational models for mechanised tunnelling, the underlying deterministic numerical model for TBM driven tunnelling can be substituted by a so-called surrogate model, which only need a small fraction of the computing time compared to the original model. Several approaches for the generation of surrogate models have been developed, see e.g. [14] for an overview. In this paper, we focus on artificial neural networks, see e.g. [15]. In [16], a feed forward neural network (multi-layer perceptron) is used to predict surface settlements due to mechanised tunnelling. The network is trained with data obtained from large scale numerical simulations using a process oriented finite element model for mechanised tunnelling. The free network parameters (weights and bias values) can be adjusted by a number of training strategies. Commonly, backpropagation algorithms are used for feed forward neural networks.

More recently, recurrent neural networks have been proposed, which are particularly suitable for time-dependent processes, see e.g. [17]. In contrast to multi-layer perceptrons, recurrent neural networks are able to learn dependencies between data series without considering time as additional input parameter. They can be applied to capture time-dependent phenomena in data series and predict (extrapolate) further structural responses. Context neurons are used to consider history dependencies. Common training strategies for recurrent neural networks are modified backpropagation algorithms, see e.g. [17], and evolutionary optimization approaches, e.g. based on particle swarm optimization [18].
After training and successful validation with results of numerical simulations, neural networks can be used as numerically efficient surrogate models for a certain, pre-defined space of parameters within reliability analyses of mechanised tunnelling processes.

5 CONCLUSIONS

In this first part of the paper, concepts for reliability assessment in mechanised tunnelling have been presented. The work in this paper is focused on handling polymorphic uncertain data within reliability analyses. A strategy for computing fuzzy failure probabilities in (almost) real time during the construction process has been outlined, which is based on using neural network based surrogate models to substitute a process oriented numerical model for mechanised tunnelling. In Part two of this contribution, the proposed strategy is applied to a prototype case concerned with mechanised tunnelling in a homogeneous soil [1].

Future research will be concerned with combining the presented concepts for reliability assessments in mechanised tunnelling with strategies for computational steering in TBM tunnelling. When available for practical applications, computational steering in mechanised tunnelling considering uncertainty in real time will help to support decisions during tunnel construction processes.

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Concepts for Reliability Analyses in Mechanised Tunnelling – Part 2: Application

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Abstract

The concepts for reliability analyses proposed in Part one of this joint paper are applied to the simulation of a mechanised tunnel excavation problem in this contribution. The complex process-oriented simulation of shield tunnelling is performed by a full-featured finite element model taking ground properties, support and grouting pressures and advancement rates into account. An artificial neural network is trained to map parameters describing ground properties onto displacements of the ground surface. Training data are generated by multiple runs of the finite element model. The trained and validated neural network is used as surrogate model to analyse the tunnelling process with uncertain parameters.

Keywords: mechanised tunnelling, surrogate model, artificial neural network, uncertainty, reliability analysis
1 INTRODUCTION

In recent years, the development in numerical methods and computer technology has enabled the development of numerical models to simulate machine-driven tunnelling processes, see e.g. [1] for an overview. A comprehensive 3D numerical model for shield tunnelling in partially and fully saturated soft soils has been developed in recent years, which takes into account all relevant components of the construction process, see [2, 3] Calculated results of this finite element (FE) model show good agreement with data measured during construction processes.

This FE model is applied as a deterministic forward model to simulate mechanised tunnelling processes as part of the reliability assessment concept, based on fuzzy stochastic analyses as presented in Part one of this paper [4]. However, since using a relatively complex FE model to represent all stages of the construction process inevitably leads to large computational costs, an artificial neural network is used as a numerically efficient surrogate model within fuzzy stochastic analysis. In this numerical example, critical surface settlements are adopted to serve as a limit state. Taking uncertain soil parameters into account, time varying fuzzy failure probabilities are computed.

2 NUMERICAL MODEL

2.1 Numerical Model for TBM Tunnelling Ekate

At the Institute for Structural Mechanics of Ruhr University Bochum, an advanced 3D FE model (Ekate) has been developed for the simulation of shield driven tunnelling in the recent years [2, 5], which considers all relevant components of the construction process [6] and their interactions. Figure 1 illustrates selected components of this simulation model.

The model has been supplemented with an automatic model generator [7] allowing to generate complex 3D FE models for shield tunnelling with minimal effort. The shield machine is modelled as a deformable body with frictional contact to the soil along the shield skin. The contact algorithm is implemented by means of a surface-to-surface contact formulation in the framework of geometrically nonlinear analysis [8]. To accomplish the desired driving path of the shield, an automated steering algorithm is applied. It is supported by truss elements representing the hydraulic thrust jacks. A three-phase model for partially saturated soils to represent heading face support using compressed air is also implemented in this simulation model [9]. The tail void
Figure 1: Components of simulation model for TBM tunnelling: 1) Soil; 2) Shield; 3) Tunnel Lining; 4) Tail void grouting; 5) Heading face support; 6) Contact between shield and soil

gROUT is described as a fully saturated two-phase material considering hydration-dependent material properties of the cementitious grouting material as proposed in [10]. This formulation allows to model infiltration of fluid grout into the surrounding soil.

2.2 Simulation Model

For the purpose of reliability analyses in mechanised tunnelling, a numerical simulation model representing a part of a tunnel constructed by a tunnel boring machine (TBM), has been generated using the Ekate model. The model dimensions of the investigated tunnel section are 48 m long, 85 m wide and 64 m deep in the x, y, z-axis directions respectively, taking into account the symmetry of geometry, material properties and other conditions assigned to the model. The tunnel diameter $D$ is 8.5 m and the overburden equals to $2D$. The chosen monitoring point (the red point in Figure 2) is on the top surface above the tunnel axis. Figure 2 shows the FE mesh of the numerical simulation model. The ground is modelled as a homogenous soil layer without groundwater. The ground model is discretised with 7104 finite elements using 27-node hexahedral elements. The behaviour of the soil is described by an elastoplastic model using a Drucker-Prager yield surface and isotropic hardening. The modulus of elasticity $E$ and the
internal friction angle $\phi$ of the soil are adopted as uncertain parameters in this study. Table 1 contains the assumed range of these two material parameters. All other soil parameters (Poisson ratio $\nu = 0.3$, soil cohesion $c = 100$ kPa, and hardening modulus $H = 8$ MPa) are kept constant for all simulations. In this study, for the sake of simplification, linear elastic behaviour is assumed for the shield, the tunnel lining and the grouting material.

Table 1: Range of investigated parameters

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Min</th>
<th>Max</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modulus of elasticity $E$ [MPa]</td>
<td>10</td>
<td>100</td>
</tr>
<tr>
<td>Internal friction angle $\phi$ [degree]</td>
<td>30</td>
<td>40</td>
</tr>
</tbody>
</table>

At the heading face, a support pressure has been applied with an average pressure of 250 kPa in the axis of the tunnel. Similarly, the gap between tunnel lining and soil is filled by grouting mortar pressurized at 150 kPa.
The simulation of the shield driven tunnel construction is characterized by a step-by-step procedure consisting of individual excavation steps. Each excavation step is divided into different phases: soil excavation, applying the support pressure, moving the shield, applying the grouting pressure and lining installation. The tunnelling process is simulated by excavation steps of 1.5 m length. In this example, a constant duration of $\Delta t = 2.6$ h per excavation step is assumed. In order to compute tunnelling induced settlements at the monitoring point, 22 excavation steps are considered with a total excavation time of $58.6$ h. The investigated part of the tunnelling process is marked in Figure 2. At $t = 24$ h, the heading face is passing the monitoring section. Within the range of the two investigated parameters $E$ and $\phi$ considered as uncertain as shown in Table 1, ten particular values of each parameter are defined. A grid with 100 computation points is generated. For each of the computation points, the tunnelling process within the investigated section is simulated. The vertical displacements of the chosen monitoring point are calculated, stored and continuously updated through the excavation steps. The stored results are collected as training and validating data of a surrogate model.

3 SURROGATE MODEL

A recurrent neural network (RNN) is used to approximate the mapping of the (time invariant) material parameters $E$ and $\phi$ onto the time variant vertical displacement $v(t)$ of the investigated surface position, see Figure 2. The objective within the training phase of the RNN is to establish dependencies between the inputs $E$ and $\phi$ and the corresponding output process $v(t)$. Here, a deterministic version of the RNN approach presented in [11] is applied.

3.1 Neural Network Setup

A RNN with $2 - 8 - 1$ architecture is selected as surrogate model for the deterministic simulation of the tunnel construction within the investigated section, i.e. two input neurons (for the material parameters $E$ and $\phi$), one hidden layer comprising 8 neurons, and one output neuron (for the displacement $v$). The RNN has 9 context neurons to enable history dependent signal computation. In each time step, the displacement is computed based on the time constant inputs $E$ and $\phi$ and network signals of prior time steps.
In the output and context neurons, linear activation functions are used. The signals in the hidden neurons are computed with nonlinear activation functions in the form of the area hyperbolic sine (arsinh). The network parameters are determined by a modified backpropagation algorithm for recurrent neural networks, see e.g. [12]. In contrast to the approaches presented in [11] and [12], deterministic network parameters are computed using deterministic training and validation data.

The available 100 data series are divided randomly into 50 training patterns and 50 validation patterns with 22 time steps. Based on randomly defined initial network parameters, \(10^7\) training and validation runs are performed.

### 3.2 Training and Validation Results

In Figures 3 and 4, four out of 50 training and validation results are presented, respectively. No differences are visible between the desired responses (FE) and the network predictions (RNN). A high approximation quality of the surface displacement processes is obtained for all training and validation patterns using a numerically efficient RNN model. Hence, the RNN provides a suitable surrogate model for the FE model introduced in Section 2. In the next Section, an application of the RNN for reliability analysis is presented.

![Figure 3: FE simulation results and RNN approximations – four out of 50 training data series](image-url)
4 RELIABILITY ANALYSIS

For illustration, a reliability analysis of the mechanised tunnelling process is performed taking uncertain soil parameters into account. The modulus of elasticity $E$ and the internal friction angle $\phi$ of the elastoplastic Drucker-Prager model are defined as fuzzy stochastic and fuzzy numbers, respectively. A fuzzy logistic distribution is used to describe the uncertainty of the parameter $E$. Its fuzzy cdf is

$$\tilde{F}(x) = \frac{1}{1 + e^{-(x-\tilde{a})/\tilde{b}}}$$

with triangular shaped fuzzy parameter $\tilde{a} = \langle 59.7, 60, 60.3 \rangle$ MPa (fuzzy mean value) and deterministic parameter $b = 2$ MPa. The internal friction angle $\phi$ is modelled as fuzzy number with triangular shape

$$\tilde{\phi} = \langle 30, 33, 36 \rangle .$$

In this example, a settlement limit state of 2 cm is defined. For each time step, the fuzzy failure probability is computed using fuzzy stochastic analysis as described in detail in Part one of this contribution [4]. The triangular shaped membership functions of the fuzzy parameters are discretised into four $\alpha$-cuts. The $\alpha$-level optimization presented in [13] is utilized to compute lower and upper bounds of the failure probability at selected stages of the tunnelling process. Within each optimization step, a Monte Carlo simulation is applied for the stochastic analyses. A sample size
of $10^6$ is used in all Monte Carlo simulations. All samples are within the range of the investigated parameters $E$ and $\phi$ defined in Table 1, which allows to use the surrogate model for all computations.

In Figure 5, the computed time varying fuzzy failure probability of the investigated tunnelling process is presented. The plotted trajectories represent interval bounds of the four investigated $\alpha$-cuts. Until 32 h (8 h after the heading face passed the monitoring section), no failure has been observed for all $\alpha$-cuts. Afterwards, an increase of the fuzzy failure probability during the continuing tunnelling process is observed. This is also shown in Figure 6 by means of the membership functions of the fuzzy failure probabilities at selected time points.

**Figure 5:** Time varying fuzzy failure probability of the investigated tunnelling process

**Figure 6:** Membership functions of the fuzzy failure probabilities at selected time points
5 CONCLUSIONS

In this paper, the applicability of advanced reliability concepts in mechanised tunnelling has been demonstrated by means of a prototype reliability analysis of a TBM driven tunnel section within a spatially homogeneous soil. As discussed in Part one of this contribution [4], polymorphic uncertainty models are used to quantify soil material parameters. A process-oriented FE model has been used to simulate the TBM advancement and tunnel construction. In order to perform fuzzy stochastic analyses, instead of using the original FE model, a numerically efficient recurrent neural network (RNN) surrogate model has been trained and validated based upon the FE model. It has been applied to predict the fuzzy failure probability during the tunnelling process, with critical surface settlements adopted as the limit state. The preliminary results documented in this paper have shown, that reliability analysis accompanying the construction of shield driven tunnels in or near to real time is possible, even if complex, advanced computational models for the simulation of the tunnel advance are used. An important step for application of the proposed concept for reliability assessment in tunnelling engineering is the generation of a suitable surrogate model. It has been demonstrated, that recurrent neural network models are excellent candidates to be employed as surrogate models in process-oriented numerical simulations.

The presented fuzzy stochastic analysis of mechanised tunnelling model can be extended to consider also spatial variability of soil parameters by means of uncertain fields. Further extensions may include uncertainties of the duration of excavation steps, considering interactions with construction process models as currently developed in [14]. Additionally, multiple failure scenarios during excavation may be defined and the corresponding limit states can be evaluated using fuzzy stochastic reliability analysis.

ACKNOWLEDGEMENTS

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Reliability Analysis of Unreinforced Tunnel Final Lining

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Abstract

The permanent loads of final lining mainly come from the surrounding geomaterial. These loads have large variation, due to the uncertainties of the geotechnical conditions, the calculation methods and the complex interaction of geomaterial - temporary support - final lining. Yet, in structural design, final lining is analyzed using the same partial factors with “conventional” structures, the permanent loads of which have smaller variation. This paper uses the Point Estimation Method combined with 3D numerical analyses to study the variation of tunnel loads due to the inherent uncertainties of geotechnical parameters and calculation methods for a specific combination of “average geotechnical parameters”. The results show that the variation of the geotechnical conditions alters not only the values, but also the distribution of the loads and the coefficient of variation of the tunnel loads ($V_p$) lies in the lower part of the range proposed in [5]. However, the numerical analyses tend to underestimate the load variation, due to the paradoxical behaviour observed in case of rigid support and small excavation step [3]. In the second part of the paper, reliability analyses are carried out for unreinforced concrete sections showing that the critical parameters for the achieved reliability level of the design are the value of $V_p$, the correlation between the axial force and bending moments and the average load eccentricity.

Keywords: Tunnel loads, final lining, probabilistic analysis, unreinforced concrete
1 INTRODUCTION

Tunnel final lining is designed to undertake all loads in tunnel service life and satisfy the requirements of serviceability (water-proofing, drainage, fire resistance - fire protection) and aesthetics. The most important load is the pressure applied, directly or indirectly (due to long-term deactivation of the temporary support), from the surrounding rock mass. Although, these loads have large variation compared with the permanent loads of conventional structures, final lining is analyzed in structural design using the same partial factors ($\gamma_g$). Firstly in the frame of the present research the coefficient of variation ($V_p$) of tunnel loads has been estimated using two approaches: (a) analytical methods with Monte-Carlo simulation and (b) 3D numerical analyses with Point Estimation Method ([8], [9]). The issue of Monte-Carlo simulation and reinforced final lining has been described in [5]. Therefore this paper focuses on the results of the second approach, which illustrates that the variation of the geotechnical parameters alters significantly not only the values, but also the distribution of the loads. Thence, probabilistic analyses are carried out for unreinforced concrete sections to estimate the design reliability level for the range of $V_p$ that corresponds to tunnel loads.

2 VARIATION OF TUNNEL LOADS

The main factors that influence the uncertainty of the tunnel loads are (a) the limited knowledge of the geotechnical properties, (b) the inherent variation and the spatial distribution of the values of the geotechnical parameters and (c) the calculation methods which are adopted in the analysis. Fortsakis et al. [5] have illustrated that the coefficient of variation of tunnel loads, taking into account only the inherent variation of the geotechnical properties, is $V_p=20\%-50\%$. In the frame of the present paper the Point Estimation Method (PEM) has been applied for a specific combination of “average geotechnical parameters”, since it is not possible to perform an extended parametric approach with 3D analyses due to high computational cost.

The numerical analyses have been carried out using finite elements code ABAQUS, assuming circular tunnel section with $D=8.0$m under $50$m overburden height. The tunnel was excavated in one phase with $1.0$m excavation step and it was supported with a $20$cm thick shotcrete layer ($E_{sh}=25$GPa, $\nu_{sh}=0.20$). The rock mass was simulated with hexahedral, eight-noded, solid elements following the Mohr-Coulomb failure criterion and the support with quadrilateral, four-noded elastic shell elements (Figure 1a).
The uncertainty was assigned to the rock mass Hoek-Brown parameters (GSI, σci, mi) based on the procedure described in [5] and the geostatic stress ratio (Table 1). For every combination of values the equivalent Mohr-Coulomb parameters were determined using the methodology proposed by Hoek et al. [6]. Moreover, in order to highlight the role of the calculation methods, the analyses of PEM have been performed for two different methodologies for the estimation of rock mass modulus ([7], [10]).

Table 1: Distribution and values for the probabilistic parameters of the numerical analyses.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Symbol</th>
<th>Distribution</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Geostatic stress ratio</td>
<td>K</td>
<td>Uniform</td>
<td>minK=0.60, maxK=1.00</td>
</tr>
<tr>
<td>Geological Strength Index</td>
<td>GSI</td>
<td>Normal</td>
<td>μGSI=35, σGSI=7</td>
</tr>
<tr>
<td>Intact rock uniaxial compressive strength</td>
<td>σci</td>
<td>Normal</td>
<td>μσci=12, Vσci=0.25, σσci=3</td>
</tr>
<tr>
<td>Geomaterial constant</td>
<td>mi</td>
<td>Normal</td>
<td>μmi=7, Vmi=0.16, σmi=1.12</td>
</tr>
</tbody>
</table>

![Figure 1:](image)

The probabilistic analyses have shown that the variation of the geotechnical parameters leads to significant differentiation of the load distribution (Figure 1b) and consequently the distribution of the internal forces. The coefficient of variation for the load on the tunnel roof has been calculated 19% - 22% and for the average load 11% - 13%. However, the two different approaches for the estimation of the rock mass modulus...
deformation modulus lead to relative differences for the load on the tunnel roof 12% - 52% and for the average load 12% - 38%. Thus, it is evident that the implementation of empirical methods, which is very common in the design, adds large uncertainty in the calculation procedure, that it is difficult to be quantified in terms of $V_p$. It is noted that the numerical analyses results demonstrate the paradoxical behaviour that has been described in [3]. More specifically in the case of tunnels with rigid support and small excavation step for constant values of deformation modulus and friction angle, increase of cohesion may lead to increase of tunnel loads. This opposite role of the two strength parameters reduces the distance between the “lower” and “higher” combination in PEM and thus reduces the standard deviation of the estimated loads.

3 RELIABILITY ANALYSIS OF UNREINFORCED FINAL LINING

In the reliability analyses of unreinforced concrete sections the coefficient of variation of the internal forces (axial force $N$ and bending moment $M$) was assumed to be equal to $V_p$ (the influence of live, accidental and other permanent loads is disregarded). The probabilistic analyses were performed for uncorrelated ($\rho=0$) and correlated ($\rho=1.0$) internal forces. The actual correlation coefficient ($\rho$) lies between these two cases, due to the non-linearity of the interaction between rock mass - temporary support - final lining and it depends on the specific characteristics of each project, such as geometry, construction sequence, twin tunnel interaction etc. In the first step each section is marginally designed by calculating the value of the section height ($h_{UC}$), since the section width ($b_{UC}$) is taken equal to 1.00m. Thence, the probabilistic calculations follow in order to estimate the corresponding reliability level (reliability index $\beta$ or failure probability $p_f$). The calculations in the case of $\rho=1.0$ are based on an analytical approach via the equations proposed by AFTES [1] and DAUB [4], whereas in the case of $\rho=0$ on Monte-Carlo simulation. The parameters that were adopted in the probabilistic analyses are presented in Table 2. The concrete strength ($f_c$) was assumed to follow normal distribution and the coefficient of variation was set $V_c=10\%$ according to the suggestions of [2] and [11]. The allowable crack depth ($h_{c,max}$) is considered to be a deterministic variable and equal to 50% of the section height (maximum value suggested by [1] and [4]).
Table 2: Parameters for the reliability analyses of the unreinforced concrete section.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Symbol</th>
<th>Range / Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete compressive strength</td>
<td>$f_{ck}$</td>
<td>20, 25, 30MPa</td>
</tr>
<tr>
<td>Final lining load coefficient of variation</td>
<td>$V_p$</td>
<td>10% - 50%</td>
</tr>
<tr>
<td>Mean value of axial force</td>
<td>$m_n$</td>
<td>0.10 - 8.00MN</td>
</tr>
<tr>
<td>Axial force eccentricity</td>
<td>$e/h_{UC}$</td>
<td>0, 0.10, 0.20, 0.30</td>
</tr>
</tbody>
</table>

Figure 2a illustrates the interaction diagram of a specific unreinforced concrete section designed according to the partial factors proposed by the Eurocodes. The results are plotted in terms of normalized axial force $\nu = N / (b_{UC}h_{UC}f_c)$ and normalized bending moment $\mu = M / (b_{UC}h_{UC}^2f_c)$. The allowable crack depth of the UC section, which is $0.50h_{UC}$, corresponds to axial force eccentricity $e/h_{UC} = 0.30$.

Figure 2: (a) Design and probabilistic interaction diagrams of a specific unreinforced concrete section (b) Distribution of reliability index as a function of normalised axial force $\nu_d$ ($b_{UC}=1.00m$, $h_{UC}=0.60m$, $h_{t,max}=0.50h_{UC}$, $f_{ck}=20MPa$, $V_p=0.10$).
It is evident that the scatter of the bending moment resistance in the lower values of axial force is very small, due to the deterministic factor of crack depth. In the upper part of the interaction diagram, this scatter is getting larger, due to the dominant role of concrete strength, which is the only probabilistic parameter. Moreover, Figure 2b shows that the reliability index $\beta$ ($\beta=\text{Erf}^{-1}(1-p_f)$) is maximized in the middle region of the design interaction diagram ($v_d \sim 0.40$). As the normalized axial force increases, each interaction diagram reaches the point where the section is considered marginally cracked ($h_t=0.50h_{UC}$), while the maximum acceptable compressive deformation of concrete is mobilized. This is the point where the curved part of the interaction diagram begins and it is in lower values of $v_d$ for the design interaction diagram, due to the lower value of concrete strength. This rapid change of geometry leads to an increase of the distance between the two diagrams and combined with the small scatter of the resistance moment in the lower part of the diagram results to the maximization of the reliability index in the specific area.

In the case of $\rho=1.0$ the failure probability and consequently the reliability index $\beta$ are calculated through Eq. 1 as a function of the partial factors $\gamma_p$ and $\gamma_c$ and the coefficients of variation $V_c$ and $V_p$ (Figure 3a). The reliability index decreases as $V_p$ increases, but in all cases the calculated values are considered acceptable for all the Consequence Classes defined by Eurocode 0.

$$p_t = \text{Erf} \left\{ - \frac{\gamma_p \cdot \gamma_c \cdot (1 + a \cdot V_p) - (1 - a \cdot V_c)}{(1 - a \cdot V_c) \sqrt{\left(\frac{\gamma_p \cdot \gamma_c \cdot (1 + a \cdot V_p) \cdot V_c}{(1 - a \cdot V_c)}\right)^2 + V_p^2}} \right\}$$

(1)

On the other hand the reliability index in the case of correlated internal forces depends also on the load eccentricity (Figure 3b). Nevertheless, the $\beta$ values are significantly lower compared with the corresponding ones in Figure 3a. If the axial force and bending moment are considered correlated, their values are bound to a specific eccentricity and therefore the potential failure can only be caused due to section overstress. On the contrary if the two parameters are considered uncorrelated the potential failure can also be caused by a small axial force and a relatively high moment, leading to a non-acceptable value of eccentricity.
Reliability Analysis of Unreinforced Tunnel Final Lining

4 CONCLUSIONS

The results of the probabilistic analyses that were performed using Point Estimation Method and 3D numerical analyses have shown that the variation of the geotechnical parameters may lead to significant differentiation of the values and the distribution of tunnel loads. The values of $V_p$ that were estimated for the specific combination of geotechnical conditions is in agreement with the values proposed in [5]. It is also evident that the role of the methodologies which are used in the design is very important, since the two different methods that were used for the estimation of rock mass deformation modulus lead to a differentiation of tunnel load up to 50%.

Regarding the reliability analyses of unreinforced concrete sections the most crucial factors are the variation and the eccentricity of the imposed loads, as well as the correlation between the acting axial force and bending moment. In the case of $\rho=1.0$ the calculated $\beta$ values are generally acceptable for the whole range of parameters that was investigated, whereas in the case of $\rho=0$, the calculated $\beta$ values, especially for high load eccentricity correspond to large probability of failure. Since the load variation and the correlation between the internal forces cannot easily be controlled, the key for the design procedure is the limitation of load eccentricity, since in the region of the marginal value $e/h_{UC}=0.30$ the efficiency of the partial factors in terms of reliability is limited.

Figure 3: Distribution of reliability index $\beta$ as a function of final lining load coefficient of variation $V_p$ (a) for correlated ($\rho=1.0$) internal forces and (b) for uncorrelated ($\rho=0$) internal forces.
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The Computation of Life-cycle Costs for Road Tunnels

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Abstract

During the past 40 years, road tunnels have contributed to make road networks considerably more efficient and safe. Beside the design and the construction of new tunnels, the efforts for the refurbishment of existing tunnels are nowadays continuously increasing. Moreover, limited budgets of public authorities and the participation of private investors in public infrastructure projects require a sustainable estimation of the return on the invested capital. The computation of the life-cycle costs is based on the application of investment appraisal methods and illustrates all monetary flows over the anticipated time period. Since the expenditures for planning and construction are referred to as initial costs, follow-up costs comprise expenses for operation, maintenance and repair. Therefore, the assessment of the life-cycle costs of road tunnels is strongly dependent on the project phase, i.e. the tunnel is either in planning, under construction or in operation. For the preliminary planning stage it is characteristic that alternative design variants exist, but at the same time the level of detail for the planning of each alternative is somewhat low. Consequently, uncertainties arising from both, technical as well as economical input data have to be taken into account.

Keywords: Life-cycle cost analysis, design & operation of road tunnels, uncertainty
1 THE LIFE-CYCLE COST CONCEPT FOR ROAD TUNNELS

1.1 Cost Influences on Tunnel Projects

In public perception, the costs for prestigious, publicly-funded infrastructure projects are usually only associated with the initial costs for design and construction. However, for the owner or the operator, extensive follow-up costs arise from the management of the facility.

If the operation of a tunnel is based on the life-cycle concept, decisive key issues have to be stressed in early project stages. For a road tunnel project, the main key issues are illustrated in figure 1. While some factors either affect initial or follow-up costs, others influence both shares, initial as well as follow-up costs. If the planning of a project is at the very beginning, a substitution of initial by follow-up costs – and vice versa – implies cost savings on a long-term basis. In contrast, if a project is in a sophisticated stage, the set of mutual dependencies is usually limited to operation, repair and maintenance, whereas the remaining dependencies are to be regarded as fix. In figure 1, the shape of arrows indicates whether dependencies exist for projects under planning (straight arrow) or for tunnels in operation (curved arrow).

---

**Structural tunnel design**
- Alignment and gradient
- Overlying strata thickness and undercutting of existing structures
- Construction method and structural design
- Waterproofing and drainage
- Construction materials

**Project-specific conditions**
- Environmental impacts and conservation
- Land acquisition
- Concern of local residents, condemnation

**Financing**
- Public funding
- Public Private Partnership
- Private financing

**Tunnel operation and maintenance**
- Necessary tunnel equipment
- Security concept
- Maintenance intervals
- Theoretical useful lives of components and materials

---

**Figure 1:** Mutual influences on costs for road tunnels as a function of the project stage
1.2 Calculation of the Life-cycle Costs

The most suitable parameter to describe the characteristics of the life-cycle of buildings is the monetary flow. According to ASTM E 917 [1] the sum of all relevant costs associated with owning and operating a building system, are the life-cycle costs; these expenses include costs for acquisition, installation, operation, maintenance, refurbishment and disposal. Equations (1) and (2) express the key indicators of the life-cycle cost approach; these are the total life-cycle costs (TLCC) and the profitability (P) of a specific project:

\[
\text{TLCC}_{A-D} = \sum_{t=t_A}^{t_D} \text{LCC}(t) = \sum_{t=t_A}^{t_B} \text{IC}(t) + \sum_{t=t_C}^{t_D} \text{FC}(t) \quad (1)
\]

\[
\sum_{t=t_A}^{t_D} \text{P}(t) = \sum_{t=t_C}^{t_D} \text{RE}(t) - \sum_{t=t_A}^{t_D} \text{LCC}(t) \quad (2)
\]

where: \( \text{TLCC}_{A-D} \) = Total life-cycle costs covering stages A (Development), B (Construction), C (Operation) and D (Disposal);
\( t \) = time variable; \( \text{LCC} (t) \) = Life-cycle costs;
\( \text{IC} (t) \) = Initial costs; \( \text{FC} (t) \) = Follow-up costs;
\( \text{P} (t) \) = Profit; \( \text{RE} (t) \) = Revenues.

Due to the long-lasting durability of building structures and the unequal distribution of payments along the whole life-cycle, it is reasonable to introduce a factor which takes account of the point in time at which each single cost or revenue arises. Literally speaking, payments with the same nominal amount, which are due – from today’s point of view – at different times possess unequal cash values. For a dynamic life-cycle cost calculation, methods of investment analyses, such as the net present value method, are being applied. When this method is used, all predicted life-cycle payments are being discounted on a common reference time point. The chosen discount rate (i) reflects an appropriate interest rate. Based on Kishk et al. [4], the sum of all discounted payments represents the net present value (NPV), to be calculated from equation (3):
NPV(t) = \sum_{t=t_A}^{t_B} \frac{IC(t)}{(1 + i)^t} + \sum_{t=t_C}^{t_D} \frac{FC(t)}{(1 + i)^t} - \sum_{t=t_C}^{t_D} \frac{RE(t)}{(1 + i)^t}

2 APPLICATION OF THE TUNNEL LIFE-CYCLE COST TOOL

2.1 General Approach
Initially, the project has to be defined by some key-characteristics. These figures include the length of the tunnel, the traffic mode as well as the anticipated duration of the life-cycle. As explained in paragraph 2.1, a complete listing of components has to be established. For each component, basic values like the average expected useful life, the initial costs and a list concerning the follow-up costs has to be completed.
As described in paragraph 2.4, the calculation model also offers the possibility to define maintenance and replacement intervals individually for every component and with different kinds of comprehensiveness. As soon as the useful life for a specific element has been defined, the number of all necessary replacement cycles within the previously specified life-cycle period of the tunnel will be computed.
The entire service life of the tunnel is usually governed by its structural integrity, which is in the range of one century. In contrast, the useful life of a technical component is in compliance with ABBV [2] commonly less than 20 years. The characteristics of technical components obey the principles of the product life-cycle concept as explained by Thewes & Vogt [5].

2.2 Identification of Components
The design and the operation of a road tunnel are characterized by process-specific properties and require the application of various materials and technical components. A road tunnel, which is fitted in compliance with the latest issue of the German guideline for the equipment and the operation of road tunnels (RABT [3]) forms the basis for the subsequent considerations. All materials and components that are to be processed during the construction phase can be divided into two main groups: The first group comprises components and materials for assembling the shell construction of the tunnel (table 1), whereas operational components and technical devices are to be assigned to the second group (table 2).
When all necessary components for a particular tunnel project have been identified, the wear of all elements has to be regarded. Right after installation, materials and components possess a maximum resistance against wear. Under the influence of the tunnel operation, dynamic loading, temperature changes as well as chemical substances from de-icing or combustion engines among others influence the degeneration processes. Depending on the location in the tunnel, identical elements might behave differently and require particular maintenance procedures.

Consequently, all information extracted from tables 2 and 3 combined with its characteristics resulting from wear, have to be analyzed according to their influences on costs.
2.3 Uncertainty and its Influence on Costs

Uncertainty in the a life-cycle cost analysis is influenced by several parameters. Each parameter can at least be assigned to one group:

- Group 1 considers the theoretical service life-span of building materials and components
- Group 2 contains cost estimates for the installation and maintenance of building materials and components
- Group 3 includes the definition of the framework that enables the application of investment appraisals

By applying the life-cycle cost analysis for a tunnel project, uncertainties have to be evaluated. Since the specification of technical failure rates is commonly based on statistical analyses, cost uncertainties consider effects from technical developments or the progress of price indices. Since methods of investment appraisals are being used for the life-cycle cost analysis, the choice of an appropriate interest rate is of major importance.

In [6], Vogt describes a method how to combine a statistical analysis of recorded times of failure for identical components with the choice of an appropriate useful life to be implemented in a life-cycle cost calculation. The chosen statistical approach is based on the Weibull-distribution.

2.4 Calculation Method and Evaluation of Results

The procedure for calculating life-cycle costs is based on the scheme shown in figure 2. The algorithm shown in the flow-chart has to be repeated as often as all necessary components are included in the analysis. All costs to be implemented in the model are only valid for one specific reference date. Usually, the reference date is similar to the beginning of the life-cycle analysis. This date either mirrors the completion of a tunnel after construction or the beginning of a philosophy-change for an existing tunnel. Consequently, the costs are real costs and have to be applied with cost indices in order to consider their time value of money. The resulting time-dependent costs are referred to as nominal costs.
For each component, all nominal costs for installation, maintenance, operation etc. have to be transferred into a time-cost matrix. In the next step, the net present value according to equation (3) is determined and evaluated. A more detailed explanation of the calculation is given by Vogt [6]. By calculating the life-cycle costs of tunnels, the user has the opportunity to examine alternative construction configurations as well as to consider influences resulting from uncertainties. Therefore, the whole life-cycle calculation has to be repeated under variation of variables.

**Figure 2:** Flow-chart for the component-wise calculation of the life-cycle costs
3 CONCLUSIONS

Since public finances are under considerable strain, a more detailed view has to be placed on future tunnel financing. A life-cycle cost model keeps track of all monetary flows. It is based on the cost breakdown structure and uses methods of investment appraisal. The application of the life-cycle cost model enables the user to compare alternative project solutions on a long term basis. Since initial costs and follow-up costs are regarded simultaneously and thoroughly, the described approach is comprehensive and offers a basis for a sustainable financing concept of road tunnels.

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Stochastic Analysis of the Tunnels Using LHS

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Abstract

A rock mass consists of two components: intact rock and discontinuities, each of which has a significant effect on the rock mass strength and deformability. It is also known that intact rocks as natural materials are invariably affected by structural defects, mineralogy, grain size, porosity, degree of weathering and anisotropy. Therefore the engineers are frequently forced to face the question “Which input values should be used for analyses?” The correct answer to such question requires a probabilistic approach. This can be easily achieved using stochastic estimation. This paper is focused on the demonstrating the use of Latin Hypercube Sampling (LHS) to describing stochastic estimation of the rock mass parameters due the variability of their mechanical properties. After the short introduction of the paper the procedure of the founding the appropriate distribution for the measured properties of rock or soil specimens using QC-Expert software is mentioned. In the following part LHS – mean method is described in detailed – with the implementation of the Log – normal distributions to the LHS sampling. At the end of the paper our application of the statistical analysis of the input parameters by LHS with is mentioned with the verification and assessment on the tunnels in the Czech Republic.

Keywords: Latin Hypercube Sampling, stochastic estimation, tunnels design
1 INTRODUCTION

The inherent variability of a rock mass is difficult to model and for this reason engineers very often have to ask the question “What value should be used in analyses?” The answer to such question requires a probabilistic approach in the evaluating the uncertainty in the input parameters in geologic media. In the recent times, specialists have encountered problems in the input parameters derived from uncertainty modelling based on the Fuzzy set theory, Monte Carlo Simulation, Latin Hypercube Sampling (LHS) etc. Engineers prioritize LHS method in the underground design due to it is a procedure with advantages for the qualified statistical evaluation of FE calculation. These methods make significant time saving possible in common statistical methods (Monte Carlo method, estimations of probability moments etc.).

2 RANDOM VARIABLES

Results of numerical modelling of geotechnical problems are very sensitive to input parameters (in the content of the probabilistic analysis we can speak about random variables). The uncertainty associated with the determination of parameters leads to the uncertainty in the determination of the results of the solved problem. For their formulation it is advisable to take into consideration their character of random quantities. A random quantity assumes various values and is described by the probability density distribution. From the practical point of view, it is good takes into consideration only continuous random quantities, which can assume all values from particular interval. And so we must first select a statistical distribution of the input parameters in the engineering practice. Many authors have shown that the rock mass parameters can be well represented by a normal distribution (Sari, 2009). On the other hand we know from practice that this is not general rule (for example fiction angle, joint length, waviness angle have a log-normal distribution - Hamm at al 2006). If a data from laboratory or in-situ tests are known we can easily find the best fitting distribution by application of statistical software. For the statistical processing for the measured properties of rock or soil specimens we used QC-Expert software with probability module (Trilobyte, 2012). This module provides the MLE method (Maximum Likelihood Estimate) for deriving estimations for given data. In general, MLE method selects parameters that produce a distribution that gives the observed data the greatest probability (i.e., parameters for a given statistic that make the known likelihood function a maximum).
3 LATIN HYPERCUBE SAMPLING

To reduce the number of samples required for good accuracy in Monte Carlo simulation have been developed other sampling methods. One of the best is Latin hypercube sampling. The concept of LHS is based on Monte Carlo techniques. Latin hypercube sampling preserves marginal probability distributions for each simulated variables. To fulfilled this aim, Latin hypercube sampling constructs a highly depend joint probability density function for the random variables in the problem, which allows good accuracy in the response parameter using only a small number of samples. LHS is a form of stratified sampling that can be applied to multiple variables. The method commonly used to reduce the number or runs necessary for a Monte Carlo simulation to achieve a reasonably accurate random distribution. A Latin square is a square grid containing sample positions if there is only one sample in each row and each column. A Latin hypercube is the generalisation of this concept - each sample is the only one in each axis – see Figure 1. There are more methods available for the samples selection so we distinguish several kinds of LHS: random, mean and median.

![Latin Hypercube sampling – example](image)

Figure 1: Latin Hypercube sampling – example

The maximum number of combination in LHS can be generally expressed by the following equation:

\[
\prod_{n=0}^{N} (M-n)^{N-1} a
\]

where \( M \) = divisions (sample points); \( N \) = variables.
3.1 Log-normal Distribution in LHS

The reason for this focus on the log-normal distribution in the LHS is that for the describing distribution function of the log-normal distribution we cannot use the standard function (such as in the Gaussian distribution). Probability density function of the log-normal distribution is defined by function:

\[ f(x, \mu, \sigma) = \frac{1}{x\sigma\sqrt{\pi}} e^{\frac{(\ln x - \mu)^2}{2\sigma^2}} \]  

(2)

where: \( x = \) stochastic variable; \( \sigma = \) standard deviation, \( \mu = \) arithmetical average.

Arithmetical average and standard deviation can be expressed using the adequate values of the related normal distribution:

\[ \lambda = \ln(\mu) - \frac{\sigma^2}{2} \quad \zeta = \sqrt{\ln\left[1 + \left(\frac{\sigma}{\mu}\right)^2\right]} \]  

(3)

where: \( x = \) variable; \( \mu = \) aritmetical average; \( \sigma = \) standard deviation; \( \lambda = \) arithmetical average \( \ln(x)\); \( \zeta = \) standard deviation \( \ln(x)\).

Cumulative distribution function can be expressed by error function that is not the elementary function and can be described by Taylor row:

\[ \text{erf}(x) = \frac{2}{\sqrt{\pi}} \sum_{n=0}^{\infty} \frac{(-1)^n x^{2n+1}}{n!(2n+1)} \]  

(4)

where: \( x = \) variable; \( n = \) serial number in the row.

Mathematical relation between input mean, error function and parameters of the normal distribution (\( \mu \) and \( \sigma \)) are describe using following equations:

\[ \sigma = \frac{\ln(\text{error factor})}{1.654} \quad \mu = \ln(\text{input mean}) - 0.5 \cdot \sigma^2 \]  

(5)

where: \( \sigma = \) standard deviation; \( \mu = \) arithmetical average, input arithmetic mean > 0 ; error factor >1.

4 LHS APPLICATION IN THE CZECH REPUBLIC

The majority of works dealing with use of the LHS method was carried out on university premises in the Czech Republic. The statistical analysis of the tunnels in Prague will be described in the following, but we have to mention the other significant use of the LHS. In the tunnel Valík calculation (Hrubešová et al. 2003) was LHS applied to the assessment of the influence of 10 parameters (e.g. the
anisotropy coefficient, dipping of the discontinuities etc.). Vaněčková (2008) uses the LHS method for solving problems of stability of rock slope interspersed by a system of discontinuities. Geotechnical parameters and other properties of discontinuity surfaces can assume for different probability distribution forms. Part of the work by Parák (2008) is the application of the LHS method to the determination of the influence of geotechnical parameters of geological strata, parameters of sprayed concrete and initial conditions on structural forces in a circular tunnel lining.

4.1 Modelling of the Mrázovka Tunnel

The outputs of the Mrázovka tunnel modelling were verified by the Latin Hypercubes method (Barták 2003). The numerical analysis was carried out by means of the PLAXIS program system. The 3D behaviour of the excavation face area, and correct description of the influence on the deformations and the state of the massif were simulated by the common procedure of loading the excavation and lining using the so-called β-method (Fig. 2). The modelled area of the profile was approximately 200 m wide and 110 m high and was divided into eight basic sub-areas according to the types of rock encountered (Fig. 3). The rock mass behaviour was approximated by means of the Mohr-Coulomb model. The input geotechnical parameters of the rock mass \((E_{\text{def}}, \nu, c, \varphi, \gamma)\) were determined on the basis of the engineering-geological investigation results – Tab.1. A comparison of theoretically determined deformations with the values obtained by monitoring was used to verify the applicability of the mathematical model. The results of the statistical study of the West Tunnel Tube (WTT) in profile km 5.160 are shown in Tab. 2. They show that the probability of final settlements being between 71 mm and 213 mm is 95%. The range of settlement without including deformations caused by the excavation of the pilot adit is 65 mm to 198 mm. The predicted range was consistent with the measured value of 194 mm (value does not include effect of pilot adit).
Figure 2: Numerical modelling stages

Figure 3: Geometry of generated mesh in WTT km 5.160

Table 1: Input parameters for the first run

<table>
<thead>
<tr>
<th>Parameter q_{ij}</th>
<th>E_{def} [MPa]</th>
<th>v [-]</th>
<th>γ_c [kN/m^3]</th>
<th>c [kPa]</th>
<th>φ [°]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground 1</td>
<td>4.8</td>
<td>0.30</td>
<td>17.3</td>
<td>11.3</td>
<td>26.0</td>
</tr>
<tr>
<td>Ground 2</td>
<td>13.6</td>
<td>0.30</td>
<td>18.8</td>
<td>5.3</td>
<td>28.0</td>
</tr>
<tr>
<td>Ground 3</td>
<td>24.2</td>
<td>0.35</td>
<td>20.2</td>
<td>14.1</td>
<td>23.5</td>
</tr>
<tr>
<td>Ground 4</td>
<td>79.7</td>
<td>0.31</td>
<td>22.6</td>
<td>22.3</td>
<td>27.5</td>
</tr>
<tr>
<td>Ground 5</td>
<td>171.0</td>
<td>0.29</td>
<td>24.3</td>
<td>45.5</td>
<td>32.5</td>
</tr>
<tr>
<td>Ground 6</td>
<td>271.0</td>
<td>0.26</td>
<td>25.1</td>
<td>91.0</td>
<td>36.5</td>
</tr>
<tr>
<td>Ground 7</td>
<td>421.0</td>
<td>0.20</td>
<td>25.8</td>
<td>91.0</td>
<td>36.5</td>
</tr>
<tr>
<td>Ground 8</td>
<td>24.5</td>
<td>0.33</td>
<td>23.1</td>
<td>13.2</td>
<td>26.5</td>
</tr>
</tbody>
</table>
Table 2: Results of the Mrázovka tunnel calculations (mm)

<table>
<thead>
<tr>
<th>Calculation</th>
<th>Total settlement</th>
<th>Pilot adit settlement</th>
<th>Settlement&lt;sup&gt;1)&lt;/sup&gt;</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Total</td>
<td>Surface</td>
<td>Tunnel</td>
</tr>
<tr>
<td>1</td>
<td>79</td>
<td>107</td>
<td>15</td>
</tr>
<tr>
<td>2</td>
<td>153</td>
<td>197</td>
<td>29</td>
</tr>
<tr>
<td>3</td>
<td>92</td>
<td>125</td>
<td>18</td>
</tr>
<tr>
<td>4</td>
<td>85</td>
<td>111</td>
<td>15</td>
</tr>
<tr>
<td>5</td>
<td>134</td>
<td>171</td>
<td>25</td>
</tr>
<tr>
<td>X (average)</td>
<td>108.60</td>
<td>142.20</td>
<td>20.40</td>
</tr>
<tr>
<td>S (standard deviation)</td>
<td>29.41</td>
<td>35.61</td>
<td>5.64</td>
</tr>
</tbody>
</table>

<sup>1)</sup> without the pilot adit effect

4.2 Modelling of the Špejchar – Pelc-Tyrolka Tunnel

The Špejchar – Pelc-Tyrolka project is 4.320 m long with the length of the tunnelled section being 3.438 m. As well as the tunnel, the project includes underground garages in Letná, four underground Technology Centres and the Trója Bridge. As designed, the excavation will be carried out using the New Austrian Tunnelling Method (NATM). Due to the predicted conditions, a vertical excavation sequence will be adopted in the three-lane tunnel excavation. In the two-lane tunnels, a horizontal excavation sequence is expected. The position of the exploratory drift has been selected to coincide with the complicated sections of the tunnel to provide information to predicted whether the vertical sequence should be applied to the top heading only, or to the entire cross section. Mechanical rock breaking is expected, but in combination with the drill-and-blast when passing through the Quartzites. A transition zone between the Quartzites exists at the foot of the slope falling from the Letná Plain. The gradient parameters mean that the tunnels run are in the vicinity of fully saturated Quaternary sediments. A multicriteria analysis resulted in the selection of pre-excavation grouting. The outputs from the exploratory drift near the transition zone (km 5.900STT) were verified by Latin Hypercube Sampling method.
Jan Pruška and Matouš Hilar

(Louženský 2006). The 2D and 3D numerical analyses were carried out by means of the CESAR - LCPC program system (Fig. 4). The rock mass behaviour was approximated by means of the Mohr-Coulomb model – Tab. 3. The intervals of the input parameters of the rock mass were determined on the basis of the engineering-geological investigation results. The results of statistical study of South Tunnel Tube (STT) at profile km 5.900 showed that final settlements of the tunnel lining will be between 13 mm and 347 mm, with a probability of 95%. The range of predicted surface settlements above excavated tunnels is from 7 mm to 26 mm. The results of the statistical study of the South Tunnel Tube at profile km 5.900 are shown in Tab.4.

**Figure 4:** 3D model of the Špejchar – Pelc-Tyrolka tunnel (detail of the exploratory drift)

**Table 3:** Input parameters according the engineering-geological investigation results

<table>
<thead>
<tr>
<th>Parameter qᵢ</th>
<th>$E_{\text{def}}$ [MPa]</th>
<th>$v$ [-]</th>
<th>$\gamma$ [kN/m$^3$]</th>
<th>c [kPa]</th>
<th>$\phi$ [$^\circ$]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-Recent diluvium</td>
<td>6,0</td>
<td>0,40</td>
<td>19,0</td>
<td>15,0</td>
<td>23,0</td>
</tr>
<tr>
<td>2-Fluvial sediment</td>
<td>25-60</td>
<td>0,33-0,35</td>
<td>20,5-22,0</td>
<td>0,0</td>
<td>34-38</td>
</tr>
<tr>
<td>3-Mouldered quartzite</td>
<td>90-200</td>
<td>0,27-0,30</td>
<td>23,0-24,0</td>
<td>10-20</td>
<td>28-33</td>
</tr>
<tr>
<td>4- Effloresced, quartzite</td>
<td>200-550</td>
<td>0,22-0,27</td>
<td>24,0-25,5</td>
<td>20-60</td>
<td>33-37</td>
</tr>
<tr>
<td>5- Quartzite (0-5m)</td>
<td>550-1000</td>
<td>0,20-0,22</td>
<td>25,5-26,5</td>
<td>80-300</td>
<td>37-40</td>
</tr>
<tr>
<td>6-Quartzite (5-15m)</td>
<td>1000-1600</td>
<td>0,19-0,20</td>
<td>26,5-27,0</td>
<td>300-450</td>
<td>40-42</td>
</tr>
</tbody>
</table>
### Table 4: Results of the Špejchar – Pelc-Tyrolka tunnel calculations - Vertical def. (mm)

<table>
<thead>
<tr>
<th></th>
<th>Total settlement</th>
<th>South Tube</th>
<th>Total settlement</th>
<th>North Tube</th>
<th>Adit settlement</th>
<th>South Tube</th>
<th>Adit settlement</th>
<th>North Tube</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Calcul. surface</td>
<td>tunnel</td>
<td>Calcul. surface</td>
<td>tunnel</td>
<td>Calcul. surface</td>
<td>tunnel</td>
<td>Calcul. surface</td>
<td>tunnel</td>
</tr>
<tr>
<td>1</td>
<td>13,6</td>
<td>22,6</td>
<td>24</td>
<td>24</td>
<td>2,2</td>
<td>4,1</td>
<td>6,5</td>
<td>10,2</td>
</tr>
<tr>
<td>2</td>
<td>17,5</td>
<td>26,9</td>
<td>18</td>
<td>36</td>
<td>2,5</td>
<td>4,7</td>
<td>4,9</td>
<td>11,1</td>
</tr>
<tr>
<td>3</td>
<td>13,1</td>
<td>21,6</td>
<td>16</td>
<td>37</td>
<td>3,4</td>
<td>6,2</td>
<td>5,9</td>
<td>13</td>
</tr>
<tr>
<td>4</td>
<td>12,1</td>
<td>18,4</td>
<td>20,3</td>
<td>32,3</td>
<td>1,68</td>
<td>3,4</td>
<td>6,9</td>
<td>13</td>
</tr>
<tr>
<td>5</td>
<td>25,5</td>
<td>35,8</td>
<td>38</td>
<td>38</td>
<td>2,4</td>
<td>6,6</td>
<td>6,6</td>
<td>13,8</td>
</tr>
<tr>
<td>X</td>
<td>16,4</td>
<td>25,1</td>
<td>23,3</td>
<td>33,5</td>
<td>2,4</td>
<td>5,0</td>
<td>6,2</td>
<td>12,2</td>
</tr>
<tr>
<td>s</td>
<td>4,9</td>
<td>6,0</td>
<td>3,0</td>
<td>5,1</td>
<td>0,6</td>
<td>1,2</td>
<td>0,7</td>
<td>1,3</td>
</tr>
</tbody>
</table>

1) Average  2) Standard deviation

### 4.3 Modelling of the Prague Metro Line A Extension

Prague metro line A Extension is 6134 m long and comprises of four stations, initial works started in April 2010 and the construction will go into service in 2014. All underground stations (except Motol station) are mined using NATM. The running tunnel from Vypich to Motol (two tracks) is excavated using NATM, the other tunnels (single track) is driven by two tunneling machines of EPBS type. Above described LHS method was applied to verify a change in the Petřiny station construction concept. The Petřiny station is the single vault mined station with cross section area of 266 m² and the length of 217 m – Fig. 5. The station excavations sectioning consists of two side-wall drifts and one central core due to very difficult geological conditions. The side-wall drifts have cross section area of 70 m² and are sub-divided into top heading, bench and invert. The numerical analysis was carried out using 3D FEM (Fig. 6) by the MIDAS GTS program (Kožoušek 2010). The results of statistical study for station Petřiny are shown in Table 5. They show that final settlements of the tunnel lining will be with probability 95% between values 10,9 mm and 13,5 mm. The interval of final surface settlement above excavated station is from 5,5 mm 6,9 mm.
Figure 5: Cross-section through Petřiny station

Figure 6: 3D model of the Petřiny station in Midas - GTS

Table 5: Results for Petřiny station - settlement in mm

<table>
<thead>
<tr>
<th>Calculation</th>
<th>Final surface</th>
<th>Final station</th>
<th>Left side-wall surface</th>
<th>Left side-wall tunnel</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>6,5</td>
<td>10,2</td>
<td>2,2</td>
<td>4,1</td>
</tr>
<tr>
<td>2</td>
<td>4,9</td>
<td>11,1</td>
<td>2,5</td>
<td>4,7</td>
</tr>
<tr>
<td>3</td>
<td>5,9</td>
<td>13</td>
<td>3,4</td>
<td>6,2</td>
</tr>
</tbody>
</table>
Table 5: Continuation

<table>
<thead>
<tr>
<th>Calculation</th>
<th>Final surface</th>
<th>Final station</th>
<th>Left side-wall surface</th>
<th>Left side-wall tunnel</th>
</tr>
</thead>
<tbody>
<tr>
<td>4</td>
<td>6,9</td>
<td>13</td>
<td>1,68</td>
<td>3,4</td>
</tr>
<tr>
<td>5</td>
<td>6,6</td>
<td>13,8</td>
<td>2,4</td>
<td>6,6</td>
</tr>
<tr>
<td>average</td>
<td>6,2</td>
<td>12,2</td>
<td>2,4</td>
<td>5,0</td>
</tr>
<tr>
<td>$\bar{X} + 2s$, $\bar{X} - 2s$</td>
<td>5,5 - 6,9</td>
<td>10,9 - 13,5</td>
<td>1,8 - 3,0</td>
<td>3,8 - 6,2</td>
</tr>
</tbody>
</table>

5 CONCLUSIONS

The results of analyses show the effect of the variation of input parameters describing rock mass behaviour on the results of FEM and demonstrate that the differences from this influence can be significant. The differences in the results flow from variations in the geological condition. The works mentioned in the paper assume linearly independent input geotechnics parameters, which is the reason why they did not deal with their correlations. In addition, all of the above mentioned works consider the LHS – Median method type (the mid-point of an interval within the domain of the distribution functions). Further, normal probability distribution of input parameters is usually assumed. However, the importance for determination of the final structure behaviour of at least a basic study of the variation of the input parameters can be clearly seen. The Latin Hypercube Sampling method is a procedure with advantages for the qualified statistical evaluation of FE calculation. These methods make significant time saving possible in common statistical methods (Monte Carlo method, estimations of probability moments etc.).

ACKNOWLEDGEMENTS

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REFERENCES


3D Modeling of the Subsurface Works beneath Rabat Fort, Morocco

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Abstract

The Oudayas tunnel in Rabat, Morocco, lies on an urban dual carriageway designed by the Bouregreg Valley Development Agency. The tunnel comprises a central section built underground and two end sections built as covered trenches. The covered trench section of the tunnel runs beneath part of the Oudayas monuments (17th century). Tunnelling beneath the fort required for the use of very precise construction and recourse to unusual technical methods, i.e. the under-excavation process by transferring twice the load of the building on series of micropiles. A series of 3D non-linear FEM analysis was performed in order to predict the settlements of the building, the loads in each micropile and the bending moments in the beams linking the building and the micropiles. A specific calibration of the embedded pile approach was conducted by comparing the numerical load-displacement curve with the data of full scale in situ tests. A final model was implemented where the whole historical building and the entire excavation process was modeled. Consequently 3D prediction of the soil/structure interaction was obtained to validate the design phases and to adjust the excavation sequences by data cross-comparison with remote and real-time monitoring systems. The final monitored data were compared with the predicted ones.

Keywords: 3D numerical model, soil-structure interaction, embedded pile, construction stages
1 INTRODUCTION: DESIGN OF THE SUBSURFACE WORKS

The construction of Les Oudayas tunnel in Rabat (Morocco) is just one part of a broader project to develop the whole of the Bouregreg valley.

In addition to the typical problems posed by underground excavations in built-up areas with little overburden, the Oudayas tunnel design required complex studies to be undertaken for the following interactions:

- interaction between the structure and adjacent buildings with significant historic and artistic value;
- interaction between the two underground tunnels, which virtually touch over a stretch of some 300 m, and the impact of excavations on historic structures, buildings and highways.

The Oudayas complex comprises two historic buildings, the fort and the library (Figure 1:a). They are built into the walls surrounding the Kasbah, built in the 17th century, and the Les Oudayas Andalusian gardens. The trapezoidal-shaped fort is adjacent to and partly built into the wall itself. At its base, this is some 2.5m thick. The library is a more recent stonework structure, with barrel arches and pillars.

The surveys carried out in the walls showed that their construction had involved the use of a “bag-type” technique, in which miscellaneous infill, which was not cemented and thus highly compressible, was placed between two lateral masonry walls. In some places, the load-bearing point of the walls followed the natural lie of the land. By contrast, the foundation of the library’s pillars consists of distinct bases, made up of stacked layers of calcarenite. All of the historic buildings’ foundations rest directly on the calcarenite substratum and are mostly superficial, sometimes deeper.

Figure 1: a) Batiment Historique (BH); b) geological profile
The measures to ensure the safety of the historic buildings were designed to meet a twofold requirement: firstly, making the wall structure as uniform and monolithic as possible in order to support the differential settlements due to excavations, and secondly respecting the historic and cultural value of these structures, avoiding excessively invasive works. To achieve this, existing cracks were treated by saturating them with lime injections, ensuring the wall as uniform and continuous. The procedure was then completed by erecting a steel support structure inside the building to buttress the walls and ensure adequate rigidity.

Geotechnical surveys conducted on the site of projected works in order to assess the type and thickness of the foundation soil showed that the stratigraphy beneath the Historic Buildings is characterised by the presence of three principal formations (Figure 1:b): sands of various consistencies, from non-cohesive to cemented, with highly variable degrees of alteration including soft calcarenite portions; gravel with silt and clay, sometimes slightly cemented; plastic marls.

![Figure 2](a,b) 3D and in plan views of BH interaction with tunnels; c) U-beam and micropile for temporary supports

Construction of the covered trench beneath the historic buildings was carried out in two phases. For the temporary phase, the structure was supported on micropiles; for the permanent phase, support was provided by the slab of the tunnel roof.

- Temporary support of buildings on micropiles (Figure 3:)
  
  The first phase consisted in transferring loads onto the ground located beneath the levels of the subsequent excavation by means of deep foundation micropiles. Joining the micropiles to the base of the walls involved adding intermediate structural elements capable of taking up the loads and ensuring continuity. U-shaped beams were thus installed along the entire length of the fort walls (Figure 2:c) in two successive stages: firstly, the base of the U-beam was formed by replacing alternate blocks of the wall with reinforced concrete elements. The two
beams forming the sides of the U were then continuously concreted in a second stage. This gave the U-beam lengthwise continuity. For the library pillars, the same U-beam method was used, combined with a kind of foundation baseplate connected to the pillar using steel profiles and to the micropiles by means of a grip system.

- Permanent support of the building on the slab (Figure 3:)
  In the final phase, the structural loads were distributed to a foundation slab which also served as the roof slab of the tunnel. During excavation, the slab rested on three rows of micropiles, one at each edge and one in the centre, protected by retaining walls (Figure 2:b). The purpose of these was to bear horizontal ground loads, while the vertical loads were borne in full by the micropiles. To ensure the structure performed as expected, the head of each micropile was directly integrated into the concreting of the slab, while the top section of the retaining walls was kept separate from the slab.

![Figure 3: a) Phase 1: main construction stages; b) Phase 2: main construction stages;](image)

2 3D NUMERICAL MODELLING

3D numerical models (Figure 4:) were developed by using the finite element code Midas GTS® in order to predict the settlements and the state of stress of the BH and of the new structures. All the excavation phases were simulated and the soil-structure interaction was considered by modeling the whole BH.
2.1 Calibration of Load Transfer on Micropile

The micropiles were modeled by means of embedded pile elements. Their parameters were calibrated referring to in situ load tests, by keeping the geotechnical parameters and the ultimate load constant. The most suitable parameters are chosen to describe the curve of Figure 6:

2.2 Construction Stages

The temporary support of buildings on micropiles (Figure 3:) was modeled by means of the following seven construction phases:
• I area excavation;
• II area excavation, I part slab construction and covering of part of the I area;
• III area excavation, covering of the remaining part of the I area and of the II area;
• IV area excavation, II part slab construction and covering of the III area;
• III part slab construction and covering of the IV part slab;
• IV part slab construction;
• final slab construction.
On the other hand, the permanent support of the building on the slab (Figure 3:) was simulated by five phases:
• casting of the piles of the slab;
• excavation of the I part of the tunnel;
• excavation of the II part of the tunnel;
• excavation of the III part of the tunnel (the last under the slab);
• excavation of the last part of the tunnel.

2.3 Soil-structure Interaction: Induced Displacements
The settlements after some construction stages of the first phase are reported in Figure 7:, while those related to the second phase can be seen in Figure 8:.

*Figure 7:* The buildings settlements after phases 2, 3, 4, 7
It is observed that the maximum settlement after the slab construction, before the micropiles realization, is 4.5 mm. When neglecting the structural modelisation of the building, substituting it with the equivalent load, the maximum settlement resulted 7.3 mm. The nearly double value is due to the lack of the contribution of the stiffness of the structure.

![Figure 8: The buildings settlements after phases 1, 3, 4, 5](image)

The final settlement is 10.6 mm, with an increase of 6.1 mm respect to the last stage of the first phase mainly due to excavation of the tunnel.

### 2.4 Structural Response of Micropiles and Foundation Beams

The maximum axial force (Figure 9:) in the piles (376 kN) arises during the excavation of the first zone. This value is the half of the one that can be obtained without modeling the building.
Figure 9: The axial forces in the micropiles after phases 1, 2, 3, 7

Figure 10: The axial forces in the piles after phases 2 to 5
During the excavation of the tunnel (Figure 10), the self weight of the BH moves towards the new piles of the slab. The maximum load acting on the new piles is about 82 kN.

Furthermore, the stresses in the foundation beam were evaluated by introducing in the solid model a beam element with the same geometry and numerical discretisation of the solid beam, but with a reduced elastic modulus $E^* = 10^6 E_c$, so that the displacements (and consequently the curvature) of the beam were the same as those of the solid foundation, without increasing its stiffness. The results of phase IV are shown in Figure 11, where the maximum and minimum moments can be found as 316 kNm and -520 kNm.

![Figure 11: Bending moments (10^-6 kNm) of the foundation beam in phase IV](image)

### 2.5 Stress Induced in Rabat Fort

The coupled model allowed to evaluate the state of stress and strain in the historical building. In order to evaluate the most critical areas of the structure, an elastic behaviour was considered. In the following (Figure 12–Figure 15), the most significant results in terms of principal tensile stresses are reported for some construction phases.

![Figure 12: Principal tensile stresses σIII after phases I and II](image)
The principal stresses in the building result in not particularly high values. Referring to compressive stresses, they don’t exceed 1.96 MPa (Figure 13:), in particular localised points, such as the corners of the holes of the windows and the joint edges between two walls. The mean value of compressive stresses is about an order of magnitude less than the maximum one. The most stressed elements during the excavation process are the central columns of the library, with a compressive stress of about 1 MPa.

The maximum tensile stresses is 1.73 MPa. Also in this case, this value can be found in singular zones, that can be affected by local effects of the numerical model (Figure 14:). In any case, the onset of cracking associated to the low tensile strength of the
masonry of the fort would induce a stress redistribution in the nearby, without preventing the correct structural behaviour. As in the case of compressive stresses, the mean value of tensile stresses produced by excavation works is about an order of magnitude less than the maximum one.

3 IN SITE WORKS AND MONITORED DATA

Some phase of the in site works are shown in Figure 16.

The monitoring system was designed with the aim of keeping track of the behaviour of surface structures (historic and newer buildings) with special attention paid to soil settlement and building displacement, as well as to soil behaviour round the excavation and within the soil itself in terms of convergence and deformation.

![Temporary work phases](image)

**Figure 16:** Temporary work phases

The monitoring system confirmed the digital analysis forecasts, recording values lower than forecast for all measurements performed.
Figure 17: Monitoring: Changes in settlement around the edge of the historic buildings

Figure 17: shows changes in the settlement measurements recorded along the perimeter of the historic buildings every three months from installation through to completion of the tunnels. The most critical conditions did not relate to settlement due to excavation, but to heave following installation of the jet-grouting columns and, to a lesser degree, the micropile injections. Checking injection parameters (pressure and volume) combined with continuous monitoring during the course of works played a determining role in minimizing this heave, which did not exceed 15mm.

4 CONCLUSIONS

In this work a full couple soil-structure 3D interaction analyses has been conducted. By considering the entire construction stage processes it has been possible to evaluated the effects of complex underground excavation on the historical building before the in-site work; therefore the predictions could be considered in class A. The comparison with monitored data have shown an agreement between numerical and monitored data with a slight overestimation of numerical settlements.
A 3D Finite Element Model for Shield Tunnel Undercrossing the Historical Building in Soft Ground

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Abstract

A three-dimensional finite element simulation model for shield-driven tunnel undercrossing the historical building is presented. The model takes into account the complex tunnel construction process as well as the building above the tunnel, which can reflect the interaction between the soil, the tunnel and the building. The hardening soil small model is used to describe the material behaviours of soft ground, and a new proposed model for lining (district modified stiffness model) is adapted to take the segments and joints of lining into account. For the building, both the weight and the stiffness are considered with an averaging model. The capacity of the model is illustrated through an application to a practical engineering in soft ground in Shanghai. The comparison of the numerical results to the measurements in situ shows good agreements.

Keywords: Underground crossing, deformation, DMSM, finite element method, historical building
1 INTRODUCTION

The construction of the shallow tunnels induces a state of strain in the soil around the excavation that would cause the damaging differential settlements in existing building near or above the tunnel. Therefore, the reciprocal effects of tunnelling-induced settlements and surface buildings are among main concerns in the urban underground engineering. During the last decades, many researches have been carried out to investigate the effects of the tunnelling-induced settlements on the buildings and several analytical, empirical and numerical methods have been presented in the scientific literature (ITA, 2007). However, the interaction between tunnel, the soil and the surface building has been mostly neglected in these studies (Kasper and Meschke, 2004; Meoueh and Shahrour, 2008). Since the interaction between the building and the ground can have a significant effect on the soil movements, the tunnelling-induced settlement trough and the extent of the predicted damage of the existing building, the coupled analysis should be used to realistically predict ground movements as well as the damage of the building.

The empirical and analytical methods proposed in the literature cannot clearly identify the soil movements due to the complex tunnel process. Especially when the building is considered, a two-stage process is adapted to assess the damage of the building. First, the greenfield settlements are approximated through empirical, analytical or numerical methods; second, these displacements are imposed on a building model to obtain an assessment of the expected damage (Mair et al, 1974). The two-stage process neglects the interaction between the building and the ground, causing conservative results. Therefore, only the numerical methods can realistically capture the complex interaction among the tunnel, the ground and the building.

A three-dimensional finite element model in which the tunnel, the soil and a building were all treated in a single analysis was proposed by Burd et al (2000). But the lining was neglected for the computation efficiency and the volume loss was used to control the ground movements, which is clearly a simplification and cannot simulate the interaction between the lining and the soil. For the building, the roof, floors, internal walls and foundation were not included in the model, so clearly these would cause drifts from the particular building. A more detailed three-dimensional finite element simulation model for shield-driven tunnel excavation was proposed by Kasper and Meschke (2004). The model took into account all relevant components of the construction process, but no building was considered and the lining was supposed to be an elastic tube ignoring effects of joints and segmentations. The effects of surface buildings on twin tunnelling-induced ground settlements were investigated by Mirhabibi and Soroush (2012) through a series of two dimensional finite element simulations which ignored the spatial effects and tunnelling process.
The interaction between the building and the tunnelling process is essentially a three-dimensional phenomenon. Therefore, a detailed three-dimensional finite model was presented in this paper to obtain more realistic results. The model took into account the tunnelling process, the building as well as lining joints, which will be described in Section 2. An application to the practical engineering in Metro Line 11 in Shanghai was presented in Section 3 and the comparison of the numerical results to the monitoring data was also presented.

2 3D FINITE ELEMENT MODEL

The model includes the soil, the shield machine, the tunnel lining, the tail void grout and the building as separate components; see Figure 1(b). The shield machine is modelled as a liner elastic and movable body; see Figure 1(a). It is defined by solid element with the equivalent weight and stiffness according to the machine used. The hydraulic jacks are neglected in this model. The construction process is supposed to be ideal for that the interaction between the building and the ground is main consideration in this paper. These hydraulic jacks could be added in a relatively simple way leading to an improved model, but this could be at the expense of additional complexity. A constant distribute load, \( P \), has been applied in the excavation face to simulate the pressure induced in the excavation chamber of the shield machine on the tunnel face. \( P \) is calculated by Equation (1).

\[
P = k_0 (\gamma h + P_1)
\]

where \( k_0 \) is the coefficient of earth pressure at rest, \( \gamma \) is the unit weight (kN/m\(^3\)), \( h \) is the tunnel axis depth (m) and \( P_1 \) is the equivalent building weight (kN/m\(^2\)).

The full fixity is applied to the model base and two side boundaries, while roller supports for the two vertical boundaries at each end of the tunnel. The step-by-step technique is used to simulation the excavation works.

Figure 1: (a) the shield machine model; (b) the tunneling process model.
2.1 Material Model

The soil model and grout model are the critical parameters in obtaining realistic predictions of tunnelling-induced ground movements. According to the field measurements, large parts of the mass around tunnel are in the small strain condition, 0.01%-0.1%. Therefore, the material behaviour of the soft ground was modelled by means of a hardening soil small strain constitutive model (HS-S) (Ben, 2007). HS-S model is a combination of the hardening soil constitutive model (HS) and small-strain constitutive model, which can capture both the soil behaviours in small strain condition and the hardening process. The basic features of the model are illustrated in Figure 2(a). In the model, the grout material characteristics are modelled in a simplified manner by employing a time-dependent Young’s modulus, see Figure 2(b). The largest Young’s modulus is 0.4 GPa. The Poisson ratio is also considered as time-dependent, between 0.17 and 0.45.

The lining and shield machine is modelled as an elastic solid body, using solid elements. For the lining, a new proposed model is used to consider the segmentations and joints effects, which is presented in next section.

2.2 District Modified Stiffness Model (DMSM)

In the literature, the lining was mostly modelled as an elastic tube with a constant stiffness for the whole circle in the 3D soil-tunnel model. The stiffness was supposed to be equal to or a reduction of that of the uniform concrete lining without joints (Named as MSM in this paper, see Figure 3(a)). These would cause a small deformation or a much more compression deformation, respectively. Then a draft from the real ground movements would occur. In reality, the segmentation introduces quite a complex structural behaviour of the lining resulting in local stresses which can be reproduced only by very detailed modelling procedures (Blom, 2002). But the very detailed model is not feasible in the whole 3D model of tunnel and building. Thus a simplified model is needed to obtain more realistic results in 3D model.

![Figure 2: The representation of the yield function in the Principal stress space](image)
According to the St-venenat's principle, the joint just has a significant influence on the lining stiffness around the segmentation not the whole lining. So only a reduction of stiffness around the joints would be more precise, and this method is named district modified stiffness model (DMSM). It uses two parameters which stand for the reduction length of around the joints ($L_{dec}$) and the reduction factor ($\alpha_{dec}$), respectively, see Figure 3(b). EI is the stiffness of the uniform concrete lining without joints.

In this paper, this model was calibrated through the lining experiments and very detailed model, see Figure 3(c and d). Through a series of parametric analyses the reduction length and reduction factor was assumed to be 0.3 m and 0.008, respectively. The comparison of results of experiment, very detailed model, MSM and DMSM were illustrated in the Figure 4. Both the results of DMSM and very detailed model are very similar to these of experiment, while the MSM results have bigger drafts from the experiment results. Therefore, the DMSM is better than MSM in prediction of the deformation of lining, and is adopted in the proposed 3D model to consider the effects of joints and segmentations.

### 2.3 The Historic Building Simulation

A complex building model, which can represent the deformation of the building as well as the interaction of tunnel and building much better, was adapted in this study. According to Pande’s results (1989), the method of averaging was used to simulate the masonry building. In the model, the solid element is used to simulate the facade of the building, the beam element for beams and columns, and the shell element for the roof. The connection between the building and soil used kinematic constraint method in Z_soil software. According to the field test, the elastic module of the four floors is 2.00 GPa, 1.97 GPa, 1.96 GPa and 2.09 GPa, respectively.

---

**Figure 3:** Different model for the lining at the positive bending moment condition: (a) the MSM model; (b) the DMSM model; (c) the real lining model used in the experiments; (d) the very detailed model.
Figure 4: Comparison of the experimental results and the FEM computations by using three different lining models: (a) the negative bending moment condition, and (b) the positive bending moment condition.

3 AN APPLICATION OF THE MODEL TO A PRACTICAL ENGINEERING

Metro line 11 in Shanghai, China, passes through the area where a masonry historic building, Chongsi Building, is located. The detail of project and monitoring results were presented by Ge et al. (2012). A detailed model was established according to the project, see Figure 5(a).

The soil body has a length of 96m, a width of 163m and a height of 54m. The tunnel is characterized by a diameter of 6.3 m and a cover depth of 21 m. The thickness of the lining is 0.35 m and the ring length is 1.2 m. The assumed Young’s modulus is $3.15 \times 10^7$ kN/m$^2$, while the Poisson ratio is 0.2. The weight of shield machine is supposed to be an equivalent uniform load of 182 kN/m$.^3$. The meshes consists of 112661 hexahedral elements, 108344 shell elements and 664 beam elements.

The soil model parameters, calibrated briefly referring to the triaxial consolidation drained shear tests and field data, are showed in Table 1. Note that the shear strain threshold, $\gamma_{0.7}$, is supposed to be seventy percent of the limit shear strain, and the referring stress is 100 kPa while the failure ratio is 0.9 and the exponent is 0.8.

Figure 6(a) shows the settlement induced by tunnel undercrossing the building, and the maximum settlement was 2.14 cm while the maximum upheaval value was 2.98 cm. The comparison of the computed results to the field data in two monitoring section (S1 and S2) is illustrated in Figure 6(b). The computed results are similar to the field data on both values and distributions, through there are a draft between the...
computed values and field data. Note that the simplification of the lining and building would cause the difference between computed and monitor results. However, the proposed 3D model could be used to conduct parametric studies. The investigation of the effects of different factors such as tunnel’s depth and their centre to centre distance, and building stiffness, weight and locations on the surface soil movements and damage assessment of the building is currently under way.

Table 1: Properties of soil used in the HS-S model

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Layer</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Clay-2</td>
</tr>
<tr>
<td>Stiffness modulus (MPa)</td>
<td>4.4</td>
</tr>
<tr>
<td>Young’s modulus for unloading and reloading</td>
<td>19.3</td>
</tr>
<tr>
<td>Tangent stiffness modulus for primary loading</td>
<td>3.4</td>
</tr>
<tr>
<td>The initial Young’s modulus (MPa)</td>
<td>35.4-75.9</td>
</tr>
<tr>
<td>Cohesion (kPa)</td>
<td>10</td>
</tr>
<tr>
<td>Friction angle (°)</td>
<td>25.2</td>
</tr>
<tr>
<td>$\gamma_{0.7}$ (10$^4$)</td>
<td>2.49</td>
</tr>
<tr>
<td>Unit weight (kN/m$^3$)</td>
<td>18.7</td>
</tr>
</tbody>
</table>

Figure 5: (a) the computed settlement at the end of tunnel advance, and (b) comparison of the observations and the FEM computed values

4 CONCLUSIONS

A three-dimensional finite element model for shield tunnel undercrossing a historic building has been developed, in which the soil, the shield machine, the grouting and the tunnel lining (especially the joints) and the building are represented as separate components. The application of the 3D model to a practical project in Shanghai and the comparison of the numerical results to the monitoring results show good agreements, reflecting its good capabilities to investigate the interactions between the soil, the tunnel and the building by systematic sensitivity studies.
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REFERENCES
Deformation Effects of Deep Excavation on the Adjacent Metro Lines and Its Safety Control

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Abstract

The development of metropolis, such as Shanghai, tends toward more and more to the usage of underground space, which inevitably induces deformation and disturbance on the adjacent buildings and city infrastructures, such as metro lines. In this paper, a vast basement pit, 30,000m² in area of plane, 22.8 m in excavation depth, is investigated during its whole processes of construction. A four-storey basement pit is located nearby a large transfer station passed by 4 metro lines. One of the lines, the Line No. 6 passes through the basement in the elevation of the first and second storey. To insure the safety of the metro lines, measures such as ground improvement, supplementary interior retaining wall and deliberate construction sequences, have been taken according to the numerical calculations and in-situ monitors. As a final result, the whole processes of deep excavation were kept in a safe domain and the displacements of the metro lines are controlled in the acceptable range. The rather satisfactory consequence and practical experience obtained in this particular project could be referred to those structures with the similar circumstances.

Keywords: Deep excavation, metro tunnel, deformation effect, substructures, safety control.
1 INTRODUCTION

In the last decades, the investigation of in-situ measurement about the multi-propped excavation in Shanghai had been presented by Wang et al. (2005), Tan and Li (2011), and Ng et al. (2012). Nevertheless, there were few descriptions about the deformation behaviours of tunnel structure adjacent to the excavation. In this paper, a deep excavation adjacent to a large transfer station is investigated. Especially, the Line No. 6 passes through the basement directly in the elevation of the first and second storey. For this project, it is of essential importance during the construction process to control the deformation of the excavation and to guarantee the safety of the near-by running tunnels.

2 SITE CONDITIONS

Shown in Fig. 1, the deep excavation was of a right-triangle shape and supported by the diaphragm walls and the interior multilevel props.

Figure 1: Plan diagram of the excavation of basement adjacent to Metro lines
Two right-angle sides which were parallel to the Zhang-yang Road and Fu-shan Road had lengths of 149 m and 184 m, respectively, which were supported by the reinforced concrete diaphragm walls with a thickness of 1.2 m and a depth of 50 m. Parallel to Century Avenue, the wall thickness and depth have chosen as 1.0 m and 50 m, which is next to the metro transfer station. The pit is divided by the Line 6 into two parts, naming Part A and Part B. In an attempt to diminish the side effect of the large-scale excavation, Part A was further divided into a main zone, denoted as A1, and 7 smaller zones next to the metro lines, denoted as A2 to A8. With the similar procedure, Part B was re-divided into a main zone B1 and 8 smaller zones denoted as B2 to B9.

In relation to the geological condition, Shanghai sits on the alluvial sediment stemming from the Quaternary period and the Shanghai clay therefore consists of mica, oyster shells and organic matter. Prior to excavation, soil profiles at the site were explored by a series of in-situ tests and laboratory observations. There are nine strata within the depth of 90 m under the ground surface, while taking the ground level as the reference datum. The specific results and index, such as the Young’s Modulus, cohesion, friction angle, etc. are shown in the Fig. 2.

![Soil profiles and geotechnical parameters of each stratum](image)

**Figure 2:** Soil profiles and geotechnical parameters of each stratum

### 3 SAFETY CONTROL MEASURES

To reduce the impact and disturbance of deep excavation on the nearby metro lines, measures in technology have been taken, the major ones being stated as follows:


3.1 **Ground Improvement**

Before the excavation, the ground inside the pit was improved by jet-grouting. The improved area was located at the depth of 10.6 m to 24.6 m along the diaphragm walls within a width of 10 m inside the pit of Part A1 and B1. In all range of small zones of the pit, the improved area was located at the depth of 7.6 m to 24.6 m. The cement content is chosen as 20% above the excavation depth, 30% under the excavation depth. The unconfined compressive strength of the improved soils is required to reach 1.5 MPa or above at the twenty-eight days.

3.2 **Supplementary Interior Retaining Wall**

The purpose to set the supplementary interior walls is to divide the vast pit into smaller ones, and increase the stiffness of support system of the pit in plane, and reduce the effects on the existing tunnel by deep excavation. The diaphragm wall between the main zone and the smaller zones has its thickness of 1.0 m and depth of 50 m. Another step is to introduce 29 tied piles applied on each side of the Line NO.6. The tied piles, with the same dimension shown in Figure 3, were casted in-situ along the tunnel with a firm connection at the pile head before the excavation.

3.3 **Construction Sequences**

The numerical analysis shows that the excavation sequence and the support system setting thereafter play an extremely important rule in the safety control of the deformation of the protecting metro lines. Table 1 shows the excavation procedure exactly executed in the project.

<table>
<thead>
<tr>
<th>Stage 1/day</th>
<th>Stage 2 (Part B1)/day</th>
<th>Stage 3 (Part A1)/day</th>
<th>Stage 4 (Small Part)/day</th>
</tr>
</thead>
<tbody>
<tr>
<td>I (a) 50</td>
<td>II (a)/36</td>
<td>III(a)/35</td>
<td>IV(a)/62</td>
</tr>
<tr>
<td></td>
<td>Cast Prop 2</td>
<td>Cast Prop 2</td>
<td>B7, B3</td>
</tr>
<tr>
<td></td>
<td>II (b)/29</td>
<td>III(b)/40</td>
<td>IV(b)/51</td>
</tr>
<tr>
<td></td>
<td>Cast Prop 3</td>
<td>Cast Prop 3</td>
<td>B5, B8, A7</td>
</tr>
<tr>
<td></td>
<td>II (c)/38</td>
<td>III(c)/26</td>
<td>IV(c)/55</td>
</tr>
<tr>
<td></td>
<td>Cast Prop 4</td>
<td>Cast Prop 4</td>
<td>B6, A3, A5</td>
</tr>
<tr>
<td>I (b) 316</td>
<td>II (d)/34</td>
<td>III(d)/29</td>
<td>IV(d)/43</td>
</tr>
<tr>
<td></td>
<td>Cast Prop 5</td>
<td>Cast Prop 5</td>
<td>A8, B4, B2</td>
</tr>
<tr>
<td></td>
<td>II (e)/69</td>
<td>Basement slab</td>
<td>IV(e)/110</td>
</tr>
<tr>
<td></td>
<td>Basement slab</td>
<td>III(e)/36</td>
<td>B9, A4, A2, A6</td>
</tr>
</tbody>
</table>

Table 1: Excavation procedures
Stage 1 served as the preparation for the excavation. The second and third stages concerned about the main zone. Part A and Part B were originally designed to be excavated simultaneously and symmetrically. Nevertheless, since the large-scale excavation might lead to unexpectedly heave of the tunnels due to the soil movement and will deteriorate the safety of the support structure, B1 was excavated firstly. After the basement slab of B1 was finished, then A1 began excavation. After that, smaller zones were excavated.

The diaphragm walls are supported by 5 prop levers in both main zone and the smaller zone. The first prop in both zones was using in-situ cast reinforced concrete at the same time. In the main zone, the rest 4 props were casted in-situ as the excavation proceeded. In the smaller zones, the remainders were steel props.

4 IN-SITU MEASUREMENTS

The horizontal deflection along diaphragm walls and the settlement of tunnel structures of Line 6 were measured and analyzed to show the effect of excavation on the support structures and adjacent tunnels.

4.1 Horizontal Deflection of Diaphragm Walls

Since the Part A and Part B were not excavated symmetrically, it is therefore of importance and interest to compare the deflection characteristics of diaphragm wall in
Part A and that in Part B. Fig. 4 to Fig. 6 display the measured results at CX 21, CX 32 and CX 02 from Panel 1, Panel 2 and Panel 3, respectively, as shown in Fig. 1. As for CX 21, the horizontal deflection increased with the excavation depth due to the nearby excavation of B1. The top deflection tended toward stability and the maximum lateral wall movement moved downward as the excavation proceeded which was located near the surface of excavation. The maximum wall deflection $\delta_{\text{max}}$ is about $0.25\% H_{\text{exc}}$, where $H_{\text{exc}}$ denotes the excavation depth. This coincides well with the existing results from Tan and Li (2011) and Ng et al. (2012). The deflection at CX 32 and CX 02 experienced similar trend. Nevertheless, the top deflection at CX 32 continued to increase with the excavation depth while the deflection in the lower half of CX 02 converged at the depth of 45 m.

4.2 The Deformation of the Tunnel

So as to detect the tunnel deformation, 55 measurement points were installed along each side of the up line and down line of the Line 6. Denoted as SC1 to SC55, the measurement points along the up line were close to the Part A, while those along the
down line, denoted as XC1 to XC55, were close to the Part B. The XC7 to XC34 and the SC7 to SC34 denote the excavation area which the Line 6 runs through. With the Line 6 running through the excavation zones, the tunnels were uplifted as the excavation went deeper, shown in the Figure 7 and Figure 8. It is not difficult to comprehend that the soil surrounding and beneath the tunnel rebounded from the relief of nearby soil gravity. Nevertheless, the tunnel outside the excavation zone suffered a downward trend. Observation in Shanghai (Wang et al. (2005), Tan and Li (2011), Ng et al. (2012)) suggested that the ground surface settlement could be noticed outside the support structure in an excavation.

5 NUMERICAL ANALYSIS

The Z_Soil.PC2010 was applied to analyze the deformation behaviour of this excavation whose constitutive relationships of soils are simulated with the Hardening Soil Small-Strain Model (HSS model) by Zimmermann and Truty (2009). This model is based on the Small-Strain Overlay Model proposed by Benz (2007) and the soil profiles utilized stem from the in-situ tests and laboratory tests. The results of the deflection of diaphragm walls at CX 21 are compared with those of the in-situ monitoring data. With regard to the practical excavation engineering subjected to small strain, great value would be attached to the calculation of excavation deformation which the small-strain stiffness of soil was taken into account rationally.
6 CONCLUSION

This paper introduced a deliberately-chosen construction procedure and the special treatments of the pit excavation adjacent to a metro station and tunnels in Shanghai to guarantee the safety of the construction. The measured results, together with the numerical results, were presented. Conclusions can be drawn as: (a) rational design for excavation can reduce the soil movement and alleviate the adverse impact on the surroundings. (b) A mount of uplift of the existing tunnel structure occurred due to the excavation adjoining the tunnel, and after the completion of exaction, the tunnel experiences a subsidence again. (c) The small-strain stiffness of soil is of great value to implement the calculation of the excavation engineering subjected to small strain.

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Ovalisation of Cast-iron Bolted Tunnels and their Modelling

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Abstract

The behaviour of cast-iron bolted tunnels in London underground railway network was investigated using the 3-D finite element (FE) method. Built upon previous work validated against laboratory tests, a series of numerical simulation on a cast-iron segmental ring was conducted and the calculated results were in line with the field measurement. Results show that the ovalisation of a segmental ring would be induced during tunnel lining construction and during soil loading. Using the verified model, a further parametric study was conducted to examine the effects of soil conditions (stiffness and K₀) and lining construction on the ovalisation of the lining.

Keywords: Cast iron lining, ovalisation, lining construction, soil loading
1 INTRODUCTION

The London Underground Ltd. (LUL) has one of the most extensive network and oldest sections of subway tunnels in the world [1]. To ensure the safe functioning of the system, a significant amount of investigation has been carried out to assess the engineering performance of deep tube tunnels [2]. In particular, the ovalisation of cast iron tunnels derived from circularity measurements has received a lot of attentions from tunnel engineers and researchers [2][3]. Ovalisation (i.e. squatting distortion, diametric distortion) is a typical deformation mode of a circular tunnel curving from a cylindrical form to an elliptical form subjected to external loads [4]. Large ovalisation may increase the bending moments in the segments over their bearing capacity, and cause intolerable rotation at the segmental joints [4]. In typical cast-iron tunnels, the average ovalisation is inspected ranging from 0.5% to 1% by circularity measurements [2]. However, LUL suggested the ovalisation allowance for cast iron linings should be no more than 0.6% [3]. Hence, a comprehensive understanding of the mechanism of the ovalisation in cast-iron linings is needed.

Unlike a continuous ring (e.g. spray concrete lining), the behaviour of a cast iron bolted lining comprises segment bending, joint distortion and etc. Some previous efforts consider the effect of the joint as a reduction ratio to the main segment [5][6][7], while others model a joint as a spring and the main body as a shell or beam [7][8]. However, both methods fail to express the structural behaviour of the joints in precise manner. To evaluate the magnitude of structural behaviour of cast-iron segmental lining, three dimensional finite element analyses were performed in this study.

2 BEHAVIOUR OF CAST IRON SEGMENTAL LINING

2.1 Construction Processes

In London underground railway system, a typical cast-iron tunnel is lined with thousands of segmental rings and each ring generally consists of six segments or more, with a small key segment at the crown. During the tunnel construction, a ring of segments was assembled using bolts under the protection of the shield tail. Hence, the self weight of the cast-iron ring and the construction load was first carried by the inner skin of the shield tail, which induced initial ovalisation. After an additional slice of lining was added, a group of jacks then pushed the shield forward into the ground and left the ring "sitting" on the soil around its invert. In the absence of clear
construction records and drawings, the actual contact area between the ring and the soil is not well understood. Usually several rings were grouted at the same time after the last one build. Back grouting might not entirely fill the annular circumferential shield tail gap around the lining, which might induce further ovalisation [1]. After the shield tail gap was grouted, the lining was subjected to increasing soil load, inducing additional ovalisation. Many deep cast iron tunnels run through a range of depths in London clay with a wide variation in K0 [9], hence the behaviour of tunnels relate to the different strata should be carefully studied. The soil-structure interaction can be expressed as the ratio of soil stiffness over the rigidity of lining stiffness [10][11]. The rigidity of old cast-iron linings can be assumed to be unchanged since recent inspections reveal little evidence of lining distress [1]. On the other hand, as the soil around the lining consolidates, the stiffness of the subgrade usually decreases allowing more soil load to apply on the lining. The increase and redistribution of soil load around the lining may increase deflections of the lining and cause additional tunnel ovalisation [12].

2.2 Finite Element Model

Using ABAQUS, the tunnel segments were modelled as solid elements, while the bolts in the joints were modelled as a series of springs. Each bolt was simplified as nine springs (Figure 1a) to simulate rotational and shear behaviour as shown in Figure 1b&c, respectively; they were calibrated from the experimental data [13][14]. The interactions at the segment-segment contact were explicitly modelled using hard contact elements. A ring model shown in Figure 1d was used for the simulations on the segmental lining subjected to self weight, soil load, etc.

![Figure 1](image)

*Figure 1:* Structural model of a cast iron ring: (a) Illustration of nine springs model (b) Radial spring in rotational load (c) Radial spring in shear load (d) A cast iron ring model
In this study, the subgrade was assumed as a homogeneous soil with earth pressure caused only by self-weight. To conduct analyses on soil-tunnel interaction, the soil around the lining was simplified as a set of whole bedded springs (i.e. tangential springs and radial springs) with constant stiffness \([10][11]\). The tangential spring stiffness \(K_t\) was assumed to be half of the radial spring stiffness \(K_r\) \([11]\). Five cases of different soil spring stiffness values \((K_r = 15, 30, 60, 90 \text{ and } 120 \text{ MPa/m})\) were studied to investigate various interactions between the tunnel and the ground. As a typical underground scenario, the values of parameters mainly used in FE model was based upon the tunnel near Euston Station in Northern Line (Diameter = 3.8 m and Depth = 19m).

2.3 Modelling the Behaviour of Segmental Lining

2.3.1 During tunnel lining construction

In this study, the ovalisation of a tunnel was considered to be induced in two stages: 1) during tunnel lining construction excavation 2) after soil loading. The influence on the ovalisation of the lining (e.g. self weight, contact area) in the first stage are described in this section.

During tunnel lining construction, the contact area between the invert of a ring and its surrounding ground is a critical factor for evaluating the behaviour of a lining. The weight of the tunnel is distributed to a series of soil springs around the lining as shown in Figure 2a. It was assumed that the maximum contact area during tunnel construction was the bottom half of the ring. The less contact area is, the more narrowly soil springs around the invert are distributed. With decreasing soil-tunnel contact area, the available area for the soil spring force becomes more concentrated around the invert. The concentrated spring force increases negative bending moment in the crown and the invert of the lining, while positive moment in the axis of the tunnel (see Figure 2c). Hence, the vertical diameter of lining decreases, while the horizontal diameter increases. The magnitude of ovalisation increases with decreasing contact area as shown in Figure 3.

To consider different ground conditions around the lining, the stiffness of the soil spring was varied according to five typical cases. In most of the cases, the resultant soil spring forces were similar in spite of soil condition. Hence, the ovalisation of the lining only slightly increased with decreasing soil stiffness unless the strata was very soft (e.g. \(K_r \leq 30\text{MPa/m}\)) as shown in Figure 3. In this condition, the lining had a considerable settlement and the redistribution of soil forces increased the bending moment and caused large ovalisation.
During construction

When soil loading is applied

Deformation & bending moment profiles

**Figure 2:** The illustration of soil-tunnel interaction

In the absence of detailed construction records, the magnitude and distribution of construction load (e.g. muck, track bed, grouting) is not well understood and so cannot be carefully modelled. A simply approach to roughly consider such construction loads is to vary the magnitude of the self-weight of the lining as a parametric study. When the self-weight was doubled, the ovalisation of the tunnel approximately doubled as shown in Figure 3.

**2.3.2 During soil loading**

After a ring of tunnel was constructed, the soil pressure was gradually applied around the lining. To evaluate the tunnel ovalisation in different strata, the coefficient of earth pressure $K_0$ and soil stiffness $K_r$ were considered to be two important factors. As a typical scenario, the tunnel was first assumed to have 33.3% contact area with the soil during tunnel lining construction, while it was subjected to soil pressure around the
whole circumferential surface of the lining (i.e. 100% contact area) when the soil load was applied to the tunnel. The coefficient of earth pressure $K_0$ was varied ranging from 1.3 to 0.5 to consider different strata. The decrease in $K_0$ increased the bending moment of the lining, and hence caused the tunnel to have a further ovalisation as shown in Figure 4a. The variation in strata stiffness was considered by different stiffness values of soil spring (i.e. $K_r$ & $K_t$). The softer the soil spring was, the more actual soil pressure was applied on the lining, which caused larger ovalisation of the tunnel correspondingly as shown in Figure 4a.

![Figure 4](image1.png)

(a) the ovalisation subjected to different $K_0$   
(b) the ovalisation with different soil stiffness

**Figure 4**: The ovalisation of cast iron linings when soil loading is applied

If the coefficient of earth pressure $K_0$ is assumed to be constant (e.g. $K_0 = 0.7$), the increasing ovalisation with decreasing stiffness of soil spring can be illustrated in Figure 7b. In this figure, three cases of soil-tunnel contact area during tunnel construction were considered to take account of overall ovalisation of a tunnel. The average ovalisation measured in the field (i.e. 0.5 - 1.0%) locates well within the numerical results.

3 **CONCLUSIONS**

To simulate the ovalisation of cast iron segmental lining observed in London underground tunnels, a series of 3D FE analysis of a tunnel ring is performed. Using the FE model based upon previous work [13]&[14], the ovalisation mechanism of a cast iron ring is studied and the engineering performance of the ring with different soil conditions and lining construction conditions is discussed. The main findings derived from this numerical study are as follows:
1) The ovalisation of an old cast iron tunnel during tunnel lining construction and by soil loading is considered in the numerical model. The calculated results show good agreement with the field observation.

2) During tunnel lining construction, the soil-tunnel contact area dominates the ovalisation of the lining and the moment magnitude. The ovalisation values can be up to 0.4-0.6% when the contact area is 16.7% of the ring.

3) After soil loading is applied, the ovalisation of the tunnel increases when the $K_0$ value is less than one. The magnitude of ovalisation increases with decrease in soil stiffness. In clayey soils, the change in soil stiffness by the long-term consolidation may induce further ovalisation. Further work is needed to study this effect.

ACKNOWLEDGEMENT

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REFERENCES


Structural Analysis of the Load Case Fire for the Segments of the Sluiskiltunnel

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Abstract

For the Sluiskiltunnel in the southeast of the Netherlands extensive numerical structural analysis is required by tender. Full-scale fire testing was carried on a singular segment out with the temperature distribution monitored over time. For structural analyses of the tunnel lining 3D-FE-models of four segmental rings imbedded in the ground are used, that simulate a large number of nonlinearities requiring long computing times. To ease analyses of the load case fire simplified temperature loads based on the decisive actual fire load need to be determined. For this a first surrogate FE-model is used that simulates the impact of the fire load on the structural behaviour of a singular segment using temperature dependent material behaviour. Additional analysis is carried out with parameters for concrete, reinforcement, bedding and normal force in the segment being varied. To a second surrogate shell-model constant and linearly variable temperatures are applied and manually iterated to match the structural behaviour of the surrogate FE-model. From these analyses a single set of surrogate temperature loads is determined for use in all structural analyses for the Sluiskiltunnel.

Keywords: Sluiskiltunnel, structural analysis, load case fire, segmental lining, temperature dependent material behaviour
1 INTRODUCTION

Since the opening of the Westerscheldetunnel in 2003 the accessibility of the Zeeland region in the southwest of the Netherlands through the N62 improved tremendously. The N62 is accessed via the N61 which crosses a swing bridge over the canal Gent-Terneuzen. Each day this bridge is closed for approximately five hours, thus forming a considerable bottleneck for the passing traffic. As a bypass the Sluiskiltunnel is currently being constructed, consisting of two 1.15 km long machine driven tunnel tubes and four cross passages.

The tender demands considerable structural analyses for the tunnel including soil-structure interaction. Therefore analyses are carried out using FEM and a 3D-model which incorporates both tubes of the tunnel and four segmental rings each. Load cases comprise construction phases, final states and accidental situations, among which the load case of fire has to be analysed. Due to the size of the structural model and numerous nonlinearities the computing times amount up to 16 hours. To ease the analysis of the fire loads simplifications need to be introduced.

With the help of two surrogate numerical models and comparative calculations a single set consisting of a constant and a linearly variable temperature change in the segments are determined for use in the 3D-model as surrogate temperature loads.

2 GEOMETRY OF THE SEGMENTAL LINING

The outer diameter of the segmental lining is 11 m and its thickness 45 cm. Each ring has a length of 2 m and consists of seven regular segments plus a key stone of half the regular size. The contact zone in the longitudinal joints has a width of 22 cm and is symmetrical to the segment axis as shown in Figure 1.
3 FIRE TESTING AND TEMPERATURE LOAD

The temperature distribution to be verified in the lining was previously determined over time in a number of full scale fire tests in which singular segments were exposed to the fire temperature curve according to Rijkswaterstaat with temperatures up to 1350 °C (see Figure 2).

4 MATERIAL PARAMETERS

The structural impact of fire is analysed with temperature dependent material behaviour according to EN 1992-1-2 [1]. The change of strength and elasticity parameters for concrete and reinforcement with temperature is displayed in Figure 3. Deviating from the code a constant thermal expansion coefficient of $10^{-5}$ $1/K$ is used.
5  TEMPERATURE INDUCED STRUCTURAL BEHAVIOUR

5.1  Surrogate 2D-FE-model for a Singular Segment

In the surrogate model shown in Figure 4 a singular segment is modelled two-dimensionally with a virtual length of 2 m. Since no noteworthy spalling was observed in fire testing, the modelled segment thickness is 45 cm as in design. 30 rows of finite elements are used for its discretisation. Individual element heights range from 6 mm on the inner side, exposed to the fire, to 25 mm in the middle of the segment. The fine discretization allows for a very good transfer of the fire load to the segment by applying constant temperatures to each row of finite elements respectively, thus approximating the temperature curve. The inner and outer reinforcement is modelled by a row of beams each, their properties equivalent to the amount of reinforcement provided in the lining.

The segment is bedded to the outer side with springs equipped with a tensile failure criterion. These springs are pre-stressed to induce a normal force into the segment that is roughly equivalent to the average normal force in structural analysis. In the contact plane of the longitudinal joints springs with a tensile failure criterion and high stiffness are modelled to simulate the behaviour of the concrete hinges. These springs are also pre-stressed to induce the same normal force as the bedding springs. The behaviour in the ring joints is realistically simulated through the two-dimensional modelling, since bending in the longitudinal direction, resulting from the temperature gradient, is not possible.
5.2 Decisive Point in Time for Fire Load

Pre-calcualtions are executed to determine the decisive temperature distribution. The segment is initially loaded by introducing a normal force of 1800 kN. Subsequently nonlinear temperature distributions, measured in fire testing for various points of time, are specified in the segment’s cross section. As a simplification, and as a first step in understanding the structural reaction of the segment, the material behaviour is assumed to be linear elastic. This approach results in stresses being calculated that exceed the strength of the concrete by far, but also exaggerate the structural behaviour, making the decisive point in time more clearly visible. Based on the analyses, and also consistent with displacements monitored during fire testing, the temperature distribution for 120 min is determined as being decisive.

5.3 Reference Case and Stiffness Iteration

As a reference case the parameters listed in Table 1 are used to analyse the structural reaction of the segment under fire load. The material behaviour is specified for each row of elements depending on the respective temperature. On the safe side the initial elastic modulus $E_{c,0}$ is applied and the tensile strength of the concrete $f_{ctm,0}$ considered. This results in a higher stiffness of the segment and thus the calculation of higher stress resultants.
Table 1: Material parameters for the reference case and the parameter variation

<table>
<thead>
<tr>
<th>parameter variation</th>
<th>$f_{ck}$ [MN/m²]</th>
<th>$f_{ctm}$ [MN/m²]</th>
<th>$E_{c0}$ [MN/m²]</th>
<th>$E_{S, bedding}$ [MN/m²]</th>
<th>reinforcement [cm²/m]</th>
<th>normal force [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>reference case</td>
<td>50</td>
<td>4,1</td>
<td>38751</td>
<td>15</td>
<td>30</td>
<td>-1800</td>
</tr>
<tr>
<td>concrete class</td>
<td>70</td>
<td>4,5</td>
<td>41320</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>tensile strength</td>
<td>-</td>
<td>3</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>reinforcement</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>45</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>bedding</td>
<td>-</td>
<td>-</td>
<td>10</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>pre-stressing</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-1500</td>
</tr>
</tbody>
</table>

Where the calculated stresses exceed the strength parameters, the elastic modulus is locally reduced by manual iteration until the calculated stresses are within the strength limits. In Figure 5 and 6 results for the calculation of the reference case are shown.

![Figure 5: Reference case: spring forces in the longitudinal joint and stresses in the segment](image-url)
5.4 Parameter Variation

To quantify the effect of changed boundary conditions a parameter variation is carried out. Concrete class and tensile strength of the concrete, reinforcement ratio, bedding stiffness, and initial normal force in the segment are varied as denoted in Table 1. The resulting extremal values for normal force $N$, change from the initial to the subsequent normal force $\Delta N$, bending moment $M$, displacement $\delta$, as well as compressive stress $\sigma_c$ and tensile stress $\sigma_{ct}$ in the concrete are listed in Table 2 in comparison to the reference case.

<table>
<thead>
<tr>
<th>parameter variation</th>
<th>$N$ [kN]</th>
<th>$\Delta N$ [kN]</th>
<th>$M$ [kNm]</th>
<th>$\delta$ [mm]</th>
<th>$\sigma_c$ [N/mm$^2$]</th>
<th>$\sigma_{ct}$ [N/mm$^2$]</th>
</tr>
</thead>
<tbody>
<tr>
<td>reference case</td>
<td>-2056</td>
<td>-253</td>
<td>-213,4</td>
<td>18,1</td>
<td>-14,0</td>
<td>4,1</td>
</tr>
<tr>
<td>concrete class</td>
<td>-2056</td>
<td>-253</td>
<td>-213,8</td>
<td>18,1</td>
<td>-14,8</td>
<td>4,5</td>
</tr>
<tr>
<td>tensile strength</td>
<td>-2063</td>
<td>-260</td>
<td>-215,6</td>
<td>18,6</td>
<td>-13,5</td>
<td>3,0</td>
</tr>
<tr>
<td>reinforcement</td>
<td>-2065</td>
<td>-262</td>
<td>-216,8</td>
<td>18,8</td>
<td>-13,3</td>
<td>4,1</td>
</tr>
<tr>
<td>bedding</td>
<td>-1975</td>
<td>-172</td>
<td>-196,5</td>
<td>18,4</td>
<td>-13,8</td>
<td>4,1</td>
</tr>
<tr>
<td>pre-stressing</td>
<td>-1763</td>
<td>-261</td>
<td>-193,1</td>
<td>18,5</td>
<td>-13,4</td>
<td>4,1</td>
</tr>
</tbody>
</table>
A change of concrete parameters or amount of reinforcement results in only small differences compared to the reference case. In contrast the reduction of the bedding stiffness is of greater influence and the stress resultants are smaller than in the reference case. The large reduction of the normal force can clearly be attributed to the softer bedding, which allows the structure to displace slightly more thus relieving the temperature induced constraints. Due to the reduced initial normal force in the segment compared to the reference case also the bedding-springs are pre-stressed less, allowing for slightly larger displacements and thus a reduction of the bending moment.

6 SURROGATE TEMPERATURE ANALYSIS

6.1 Surrogate Shell-Model

To determine the surrogate temperature loads the 3D-model shown in Figure 7 is employed, in which the individual segment is modelled using shell elements of 45 cm thickness. The curvature of the segment is discretized with 77 elements of which 20 rows are used in longitudinal direction. The reinforcement is considered implicitly and equivalent to the amount provided in the lining.

Figure 7: Shell-model for a singular segment for determination of surrogate temperature loads

The segment is bedded to the outside with pre-stressed springs equipped with a tensile failure criterion similar to the 2D-FE-model described in chapter 0. In the longitudinal joints rotational springs with nonlinear material behaviour based on geometry,
Concrete stiffness and normal force simulate the structural behaviour of the concrete hinge. The nodes in the ring joint are fixed in longitudinal direction and provided with very stiff rotational springs to prevent bending in longitudinal direction. The boundary conditions are therefore comparable to the 2D-FE-model used for the temperature analyses.

The material behaviour is non-linear according to EN 1992-1-1 [2], [3], but not temperature dependent, which results in a stiffer structural response when compared to the 2D-FE-Model. However for the shell-model the same structural modelling with shell segments and the same material behaviour as applied in the later 3D-analyses for the tunnels are used.

6.2 Surrogate Temperature Loads

As initial loading a normal force is imposed upon the segment. In the second calculative step a constant and a linearly variable temperature distribution are applied. These are manually iterated until the calculated stress resultants are consistent with those of the 2D-FE-model, thus defining the surrogate temperature loads. Due the larger stiffness of the shell-system the calculated displacements are smaller than those calculated in the temperature dependent 2D-FE-model. This calculation of surrogate temperature loads is executed for the reference case as well as for the variation of parameters for bedding and initial normal force. The results are summarized in Table 3. The variation of parameters for concrete and reinforcement in the FE-model have returned almost identical internal forces as for the reference case and are therefore not investigated with the surrogate shell-model.

Table 3: Summary of results for the determination of surrogate temperature loads

<table>
<thead>
<tr>
<th>parameter variation</th>
<th>N [kN]</th>
<th>ΔN [kN]</th>
<th>M [kNm]</th>
<th>δ [mm]</th>
<th>ΔTN [K]</th>
<th>ΔTM [K]</th>
</tr>
</thead>
<tbody>
<tr>
<td>reference case</td>
<td>2061</td>
<td>260</td>
<td>215,8</td>
<td>13,9</td>
<td>156</td>
<td>167</td>
</tr>
<tr>
<td>bedding</td>
<td>1976</td>
<td>170</td>
<td>196,3</td>
<td>12,9</td>
<td>153</td>
<td>142</td>
</tr>
<tr>
<td>pre-stressing</td>
<td>1758</td>
<td>257</td>
<td>193,2</td>
<td>14,4</td>
<td>155</td>
<td>182</td>
</tr>
</tbody>
</table>
The determined constant surrogate temperature loads $\Delta T_N$ vary only slightly and range from 153 to 156 K. The bandwidth of the linearly variable surrogate load $\Delta T_M$ is much larger and ranges from 142 K to 182 K temperature difference over the thickness of the segment.

In order to approximately cover the determined bandwidth $\Delta T_N = 155$ K as a constant and $\Delta T_M = 180$ K as a linearly variable change of temperature are defined as a single set of surrogate temperature loads for analysis with the 3D-FE-model.

7 SUMMARY AND CONCLUSION

In order to ease analyses for the load case fire, simplified surrogate temperature loads need to be determined for use in the already extensive 3D-FE-analyses for the Sluikiltunnel. In a first surrogate 2D-FE-model the impact of the fire load on the structural behaviour of a singular segment using temperature dependent material behaviour is simulated for a reference case and a variation of parameters. To a second surrogate 3D-shell-model constant and linearly variable temperatures are applied and manually iterated to match the structural behaviour of the surrogate 2D-FE-model. Through these comparative calculations a constant and a linearly variable change of temperatures are determined as a single set of surrogate temperature loads for use in all analyses with the 3D-FE-model.

REFERENCES


Back Analysis of Blow Out in Warsaw Project

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Abstract

Warsaw second metro line is under construction now. The significant problem for TBM tunnelling in urban area, parallel with unpredicted ground conditions are unknown underground objects. This danger elements could be piles, old foundations, deep utilities etc. even if they are over the tunnel lining. Passing through this type of difficulties very often cause influence (excessive settlements or collapse) on existing buildings and surface infrastructure.

The paper presents “blow out” conditions in EPB machine with external diameter 6,3 m in Warsaw project. This situation was caused by caverns on the spot of old building cellars and foundations demolished during the war. The building was located over the tunnels line – now the street run there. Local soil conditions are characterized mainly by sands with water and soft soil.

“Blow out” numerical simulations were carry out by 3D Midas Diana software. The target of analysis was to compare TBM tunnelling numerical process with foam polymers flow in a number of essential back analysis loading cases. The parametric study was restrictively applied to the earth pressure balanced tunnelling technique.

The paper shows also the scheme of monitoring system in the most critical area of foam blow out phenomena.

Keywords: Blow out, old foundations, EPB
1 WARSAW PROJECT

Warsaw has nearly 2 million inhabitants and covers the area of 517 km². In the city now exists only one metro line which doesn’t solve all Warsaw’s severe transport problems. It is necessary to build a network of metro lines. The route of the 2nd line in the east-west direction was established as a result of a transport analysis elaborated on the basis of population distribution in Warsaw and the estimation of passenger flows. The 2nd metro line will be 30.5 km long and will contain of 28 stations. Now the central part of 2nd line is under construction. The central part of the line, 6.3 km long, consist of 7 stations and passes center of the city from east to west crossing the Vistula river – location on Figure 1. It runs under main streets and in many cases close to the historical or monumental buildings.

![Figure 1: II\textsuperscript{nd} metro line central part – C10 station is marked](image)

1.1 History of the City

In the design of the underground structures it is very important to take into account all the external risk factors - especially history of the land if structure is planed in the existing city network. One of the good example of such problems with unpredicted
historical underground residuum is Warsaw C10 IInd metro line station. Figure 2 presents the location of C10 station with holding trucks and tunnels axis in existing urban network.

Warsaw is the city changed a lot during last 70 years. Before IInd World War it was well developed modern city with railway junctions and metro line plans. It had systematic town planning configuration with considerable amount of green areas. Location of the C10 station on the city network from year 1936 shows Figure 3. On the front of the station exist railway siding (gray area on the picture). During the war main part of the city was completely demolished (Figure 4) what took an effect of new town planning. There was a modification in localization of many streets, buildings and squares. Huge amount of buildings where take to pieces (compare Figure 4 and 2).

In the area of C10 station many underground structures were left: residuum of railway siding, house cellars and foundations, old streets utilities. Over the north metro tunnel there were before buildings – today there is a street.

All of this caused a risk for design metro structures, especially for tunnel made by TBM machine. The risk was connected with blow out possibility according to unknown underground empty spaces, open canals, loose soil.

Figure 2: C10 station and holding trucks – foto from year 2012
Figure 3: C10 station and holding trucks – foto from year 1936 – before the war

Figure 4: C10 station and holding trucks – foto from year 1945 – after the war
2 BLOW OUT FENOMENA

Reaching the ultimate limit state in case of tunnel works consists in exceeding the
limit balance state at the excavation face what results in considerable dislocation,
collapsing of ground at the face inwards the tunnel. Soil wedge, which is formed in
this phenomenon may, in extreme cases (shallow tunnels, weak soil, old structures
residuum), reach the surface of the land forming a crater and causing a risk for people
and for infrastructure. Maintaining the balance state at the face is one of the basic
criteria at construction of tunnelling machines.
Correct take-in of pressure in a working chamber which ensures a balance state at the
face depends mainly on the ground-water conditions on a tunnel route. In case of non-
cohesive soil, effective stress for underpin pressure calculations should be accepted
whereas for cohesive soil, size of total stress should also be verified (according to
guidelines [1]). In case of tunnelling below the water-table, drain pressure affecting
the face in stationary flow conditions should be considered in calculations. In design
calculations it is significant to maintain balance of the hewed soil in both directions
i.e. both not to allow the ground to collapse inside a tunnel as well as not to over-
design the support pressure in order not to lead to uncontrolled escape of the
stabilizing medium/foam into the ground. In extreme circumstances it may result in
uncontrolled escape of fluid onto the soil and surface - blow out mechanism.

Figure 5: Foam escaped to the surface in blow-out in the C10 area (rondabout)
However, in case of high permeable soil ($k > 10^{-3}$ m/s - loose sands or gravels) or with unidentified underground structure residuum it is difficult to maintain and at improper suspension pressure selected by a machine operator, too extensive penetration and even uncontrolled fluid escape may occur in extreme situations. Such a phenomenon (blow out) is dangerous for face stability (sudden loss of the support pressure) and in shallow urban tunnels, impermeable soil may result in fluid flow out onto the surface. In the Warsaw example of C10 station and tunnels there occurred several blow out incidents just after starting the TBM (see Figure 5). In spite of old foundations, utilities and debris and also TBM team training just after starting with excavation, the pressure in the chamber caused foam outflow to the surface. In effect there was rinse out of the soil under the street and its local collapse (as shown on Figure 6).

**Figure 6:** Foam escaped to the surface in blow-out in the C10 area (rondabout)

**REFERENCE**

Rock Tunneling in Manhattan - Optimization of Rock Support using Distinct Element Modelling

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Abstract

The MTA Capital Construction’s East Side Access project will provide Long Island Rail Road service to a new terminal being constructed beneath the existing, historic Grand Central Terminal with open access for Long Island commuters to the East Side of Manhattan. This $8.4bn project includes the creation of two large station caverns together with associated facilities beneath Central Manhattan. The project includes running tunnels, wye and crossover caverns, cross-passages as well as public access shafts. Excavation has been performed by a combination of TBMs, road headers as well as drill and blast. During construction, the client and the contractor looked for ways to optimize the excavation, rock support and hence schedule. The redesign was performed in cooperation between the General Engineering Consultant, the contractor and the client. Due to the predominance of the metamorphic Manhattan Schist, with its distinct foliation, occasional shear fractures or highly weathered rock, the Distinct Element Method was used to assist in the design of rock support measures, ground displacement assessment and support optimization. Updated geologic data from field mapping during the tunneling progress, performed by the construction management were used for performing the analyses. This paper focuses on the challenges of the numerical analysis process as these relate to the actual construction stages in the field.

Keywords: Distinct Element Method, UDEC, rock tunnelling, TBM, schist
1 INTRODUCTION

The East Side Access Project will provide MTA Long Island Rail Road (LIRR) customers with train services to Grand Central Terminal on the east side of New York City, in addition to existing services to Penn Station. Trains will diverge from the existing mainline in Queens and pass in new tunnels through the existing 63rd Street tunnel constructed under the East River in the 1970s. In Manhattan, tunnel construction resumes at Second Avenue using two hard rock TBMs. These tunnels diverge into four, and then eight tubes to enter two new station caverns at Grand Central, immediately below Metro-North’s existing terminal. A series of escalator banks and elevator-shafts will carry passengers up to a new LIRR concourse [1].

![Figure 1: East Side Access Project map](image)

With a budget of $8.4 billion, the East Side Access Project is the largest transportation project ever undertaken in New York City. It is managed by MTA Capital Construction and was designed by the General Engineering Consultant (GEC), a tri-venture of Parsons Brinckerhoff, STV and Parsons Transportation Group. All contracts were procured utilizing traditional design/bid/build. The Manhattan excavation contracts started in 2007 and will be completed in 2013. Revenue service for the project is expected to be in 2019.
The Queens contracts consist of four running tunnels with segmental lining excavated by two Slurry TBM, launching and reception structures, and three shafts for emergency access and ventilation. The shallow bored tunnels are located beneath Amtrak's Sunnyside Yard and LIRR's Harold Interlocking. They are connected to the 63rd Street tunnel via open-cut structures and a short Sequential Excavation Method (SEM) tunnel, involving the use of ground freezing below Northern Boulevard. Unlike Manhattan, the Queens contracts are almost entirely excavated in soils.

The Manhattan contracts include rock excavation with Hard Rock TBM for the running tunnels and SEM with the use of blasting and road header for the various caverns and shafts in rock. The Manhattan contracts extend from Second Avenue and East 63rd Street to Park Avenue and East 37th Street. Excavation is adjacent to or underneath a number of prominent New York City buildings, including the Waldorf-Astoria Hotel, the Roosevelt Hotel, the Helmsley Building, the Met Life Building and Grand Central Terminal, as shown in Figure 2. It also passes beneath or close to four New York City Transit (NYCT) subway lines, with a minimum separation of 6 ft (2.0 m) [2].

Figure 2: Grand Central Terminal

2 CONSTRUCTION REVISIONS IN MANHATTAN

The Dragados-Judlau Joint Venture (DJ/JV) was contracted to mine a series of eight 22 ft (6.7 m) diameter running tunnels through the future Grand Central Terminal station prior to enlarging the two station caverns. The proposed four-over-four tunnel configuration strongly encouraged value engineering studies aimed at improving efficiency of TBM re-launch methods at the northern wye caverns [3] and
excavation and support modifications for the crossover caverns. MTA Capital Construction agreed and coordinated the various post-awarded design modifications of DJ/JV and the GEC to facilitate construction and to achieve overall project objectives. All parties cooperated in a coordinated design and construction effort to maintain efficiency.

2.1 Originally planned Excavation Methods in Manhattan

During the early stages of planning, it was thought that drill and blast excavation would be the primary method of construction for the project. However, to minimize project impacts, alternatives to drill and blast were investigated to determine the technical and economic applicability. This was further exacerbated by the events of 9/11 and concern for security and public perception of the use of explosives. This led to the realization that TBM running tunnels would offer significantly better environmental management opportunities [2]. For the various caverns for stations, wyes, crossovers and ventilation it was still envisioned to utilize drill and blast techniques. The initial ground support was designed to prevent failure of rock blocks from the crown and sidewalls of the caverns, tunnels and shafts. The contract documents imposed limitations on vibration, noise and working hours for drilling and blasting at certain locations. The contractor was permitted to choose the means and methods to comply with these restrictions. Blasting in close proximity to existing NYCT and Metro North Railroad (MNR) tunnels, historic and landmark structures, and critical facilities and utilities had special restrictions.

The Manhattan contracts for all underground excavations were initially divided into two contracts. The first contract (CM009) included all the TBM running tunnels with two TBM assembly caverns, three twin wye caverns (GCT3, 4 and 5) and a total of four TBM running tunnels. The design of this contract was facilitated mainly as an excavation contract, relying on cavern enlargement after the initial TBM drives. The subsequent contract (CM019) included basically the cavern enlargement from the four previously mined tunnels through the station, with access shafts, ventilation structures and two crossover caverns (GCT3 and GCT4) and additional three wye caverns (GCT1/2 and GCT4). The initial Manhattan Contract CM009 for the running Manhattan tunnels was awarded to DJ/JV with notice-to-proceed in June 2006. For the running tunnels DJ/JV chose to use two different 22 ft (6.7 m) diameter refurbished TBMs, a Robbins main beam TBM and a SELI double shield TBM.
Figure 3: Station caverns below Grand Central Terminal and CM009 tunnels

Subsequently, Contract CM019 for the station caverns was awarded also to DJ/JV with notice-to-proceed in May 2008. During negotiations for CM019 it was agreed that additional lengths of tunnel would be constructed by the TBM s already utilized for the CM009 contract, thus reducing the amount or rock to be excavated by blasting.

Figure 4: Overview of Manhattan Structures

2.2 Revised Construction Sequence in Manhattan Contracts

A key concept of the revised construction sequence was the maximization of TBM use and substitution of road header excavation for drill-and-blast in non-TBM areas to the maximum possible extent. After studying the interrelationships between the two
contracts, the contractor proposed to drive two additional TBM tunnel through the upper and two additional TBM tunnel through the lower level of the future station caverns prior to cavern enlargement. As a result, a total of eight separate TBM drives were constructed and required quick re-launching with the existing two TBM sets. To facilitate the re-launching of the TBMs it was originally envisioned that part of the wye caverns and the starter tunnels would be excavated using drill and blast techniques. The same applied for the station cavern excavations. However, in the absence of a formal agreement being reached between MTA Capital Construction and MNR, blasting was prohibited within the footprint of Grand Central Terminal. During the TBM mining operations it was recognized that the rock was less massive and weaker along the alignment. Based on this observation, the contractor decided to procure two Sandvik MT-720 roadheaders. The utilization of roadheaders as an alternative to drill and blast excavation gave the contractor more flexibility to excavate the various caverns. The first wye cavern to be excavated with a roadheader was GCT3 EB in the upper level just north of the future station caverns. The originally planned approach to mining the bifurcating tunnels was to re-launch the TBM from the partially excavated GCT3 EB wye cavern to create launch chamber and starter tunnel structures. This slower than anticipated excavation associated with the partial wye cavern development to accommodate the TBM launch chamber, and the subsequent excavation of the starter tunnel, with required pillar and starter tunnel rock reinforcement measures initiated the need for a series of Value Engineering (VE) studies.
Various collaboration sessions to develop alternative re-launch techniques considering the existing site and future operational constraints were conducted in these VE sessions. The number of remaining six TBM re-launches helped justify the additional studies [3]. Through coordination between the GEC, the Construction Manager, and the contractor, under the Construction Phase Services, the so called “concrete plug method” was developed and proved to be a viable way to enlarge caverns of variable cross sections in a shorter period of time than anticipated for the original design. All involved parties realized the economy that could be achieved by reconfiguring structures based on the actual excavation methods that the contractor proposed to use, reducing excavation and concrete volumes. It also was soon realized that substantial time and cost savings might be achieved by having the GEC perform redesign of excavation support systems and final linings, rather than having the work performed by a consultant retained by the contractor, which would have been a more
conventional practice [2]. So, as the concepts evolved for each structure, design of excavation support and final lining was done by the GEC as an Owner Initiated Change. The concrete plug method, as shown in Figure 5 below, involved backing up the TBM and placing a lean backfill in the portion of the recently completed TBM bore to allow re-launching the TBM to begin the adjacent TBM drive. The TBM diverted gradually from the previous alignment onto to new alignments without the need for a pre-excavated cavern.

![Figure 5: TBM in concrete plug and after excavation of backfill](image)

The minimum dimension and strength of the backfill plugs were determined based on the height and force of the gripper pads as well as the stationing where the minimum unreinforced pillar width between adjacent tunnels was achieved [3]. Since the first concrete plug approach proved to be a viable alternative to enlarging wye caverns by roadheader or drill and blast excavation, the concrete plug method was applied for all remaining four wye caverns. By modifying the concept of the concrete plug and combining it with the principles of the required train dynamic clearance envelopes, the enlarged binocular shaped excavation could be specifically located within the limits of the two crossover caverns (GCT3 & 4). The development of the so called M-Caverns allowed for the extension of the bored circular tunnels transitioning into the reduced limits of the crossover caverns, as well as improving air ventilation requirements. The efforts to incorporate the concrete plug and revising the crossover cavern designs minimized the overall construction costs and contributed to construction schedule recovery [2].
3 GEOLOGIC SETTING

The metamorphic rock types underlying Manhattan belong to the New England Upland physiographic unit, locally known as the Manhattan Prong and are tightly folded and metamorphosed. The encountered rock included meta-igneous-volcano-sedimentary rocks, granites, granitic gneisses, granodiorites, granodiorite gneisses, diorites, dacite and hornblende schists. Pegmatite inclusions, variable in dimensions also exist in the metamorphic rock mass. The overburden deposits above the bedrock vary substantially in depth. In the Central Park region, the soil cover is relatively thin and increases southward toward lower Manhattan. The rock mass was classified per three major Rock Classes I, II and III.

**Rock Class I** consists of unweathered to moderately weathered rock with a joint spacing in excess of 5 feet (1.5 m). Rock in this class would be described as massive to moderately jointed using the Terzaghi classification system.

**Rock Class II** consists of rock meeting the general requirements of Rock Class I, but with a joint spacing from 2 feet to 5 feet (0.5-1.5 m), or rock with a joint spacing in excess of 5 feet (1.5 m), but with an observable planar weakness zone. Rock in this class would be described as moderately jointed to moderately blocky and seamy using the Terzaghi classification.

**Rock Class III** consists of moderately weathered to highly weathered rock with a joint spacing of 2 feet (0.5 m). Rock in this class would be described as moderately blocky and seamy to very blocky and seamy using the Terzaghi classification.

Rock joint spatial information, obtained from preliminary investigations and the Construction Manager’s geologists provided continuous mapping of the exposed rock walls to verify rock mass conditions. In the Manhattan area, four major joint sets prevail. The most prominent joint set, Set No. 1, lies parallel to the plane of weakness formed by foliation and strikes N30º to 35ºE with a 70º to 80º SE or 60º to 70º NW dip. Set Nos. 2 and 4 generally strike perpendicular to the foliation jointing with dips in the range of 70º to 80ºSW for Set No. 2 and about 75ºNE for Set No. 4. Set No. 3 appears to run parallel to the foliation, but dips 60º to 70º in a direction opposite to Set No.1 and has been termed its conjugate. Initial geotechnical investigations revealed the existence of shear zones. Additional zones were encountered with the initial TBM tunnels and the information obtained was used along with numerical analysis to verify and optimize initial support.
4 GEOTECHNICAL ANALYSES

A wide range of geotechnical studies were performed for the re-design of the wye and crossover caverns. Due to the revised construction sequence involving the use of concrete plugs in order to re-launch the TBM, the rock support was optimized for the various geometries by considering the allowable rock bolt installation orientation and drilling length limits of the TBM robotic drill, and geometry optimization to avoid conflicts of rock support between each TBM drive.

Figure 6: Concrete plug and GCT4 enlargement with rock support
Furthermore, for each wye or crossover cavern, separate analyses were performed to optimize rock support, at frequent intervals along the alignment due to the continuously variable width of the “binocular” TBM arrangement which deviate from each other, thus affecting the total excavation width. Figure 6 shows an example of the concrete plug at GCT4 with initial rock support for the additional TBM tunnel and the final enlarged excavations that have been validated with UNWEDGE and UDEC analyses. The former consisted of kinematic stability calculations and assessment of maximum sizes of kinetically feasible wedges and rock reinforcement requirements. The latter method is used to assess stress redistribution, excavation induced displacements and loading conditions for the final lining of the lower level structures. Similar sets of analyses were performed to assess initial tunnel support requirements for twin TBM tunnel configurations passing through sensitive shear zones in the rock mass, such as at the GCT5 wye caverns.

4.1 Kinematic Stability

At a preliminary stage kinematic analyses were performed with the software UNWEDGE for all design sections in order to derive a basic initial support design. UNWEDGE permits limit equilibrium analyses of the wedges formed at the sidewalls and crown of the excavation. The wedges are the largest wedges which can be formed for the given geometrical conditions, as shown Figure 7 for the GCT4 crossover. The structural discontinuities, such as joints and foliation, included in the analysis are also assumed to be planar and continuous, which generally gives conservative results. The apex height of 20 ft (6.0 m) was typically applied to scale the wedges down to more realistic sizes. Analysis also considers the sequential excavation, and thus initial support design is performed for two excavation stages, TBM and enlargement.

![Figure 7: Critical Wedges with UNWEDGE at GCT4 crossover cavern](image-url)
The predominant joint set data from mapping during previous excavations was applied. Appropriate joint shear strength parameters were used based on field observations. For the majority of the GCT4 crossover and wye caverns, rock support included typically 15 ft (4.5 m) long Swellex PM24 rock bolts, which was the maximum feasible length that could be installed from within the TBMs.

4.2 Analyses with the Distinct Element Method

Due to the inherent anisotropy, the jointing and the presence of distinct shear zones in the rock mass, the Distinct Element Method (DEM) has been employed for the majority of the numerical analyses. During the revised construction concept, the purpose of this analysis was to verify and modify the preliminary initial support system as derived using UNWEDGE, to assess the performance of the concrete plug under lateral TBM gripper loads, to estimate long term loading conditions for the lower level GCT4 structures and to assess the performance of initial lining in shear zones. The commercial code UDEC (Itasca, 2009) was used. UDEC is a two dimensional plane strain DEM code suited for stress analysis of jointed rock masses. In the DEM, the domain is simulated by a group of “discrete” blocks and joints which are simulated as boundary conditions between the blocks, thus simulating explicitly a fractured rock mass behavior. The independent blocks can be rigid or deformable. For the latter case the blocks are discretized into elements following a finite element or finite difference formulation. In the DEM, the governing differential equations dictate the kinematics of the blocks in the assemblage. The individual blocks can interact following a specific joint normal and shear model while plastic deformation can also occur within the blocks.

The discontinuity features are inherently assumed to strike perpendicular to the model section. This approach appears unrealistic in some cases. However, full representation using a 3D distinct element analysis would demand excessive computing efforts. For more complex underground configurations in other sections of the project the 3-DEC code was used for numerical analyses. It is noted, that based on the true dip and dip direction, the apparent dip angle for a vertical cross section was assumed for the UDEC input. Various models were run at the same station, to bracket the UDEC solutions due to the stochastic variability of each generated joint network. The Convergence-Confinement method was used along with UDEC in order to provide a stress relaxation of the excavation boundary prior to installing any support using Itasca’s FISH programming language. Based on the design and in conjunction with
feedback from the contractor, an initial rock support composed of 15 ft (4.5 m) long Swellex PM 24 bolts installed at 5 ft (1.5 m) spacing and 4 inch (10 cm) minimum wire mesh reinforced shotcrete was selected. Figure 8-left shows a typical UDEC model performed considering the concrete plug and the final excavation stage with initial support at a section with crossover cavern geometry for the lower level. Final lining loads due to deactivation of initial support, and activation of the upper level cavern foundation loads were also performed using UDEC. Two different approaches, one lower-bound using a distinct lining and one upper-bound using a fixed boundary approach, were performed to assess the impact of lining stiffness on the contact loads, as shown in Figure 8-right.

![Figure 8: Typical UDEC model for rock support (left) and modelling of final lining loads on lower level GCT4 wye cavern (right)](image)

However, even for the two most northern wye caverns in a shear zone (GCT 5) the concrete plug approach could be applied due to detailed geotechnical analysis tied into actual ground condition encountered. A comprehensive series of analyses were performed to study the impact of TBM tunnelling through the shear zones of the future GCT5 caverns. The analyses were focused on assessing rock mass displacements during stress relaxation and full TBM passage, but also to study the implications of the TBM gripper pads on the weak pillar zone due to loading and unloading. For this section initial support consisted of steel rib support installed as close as possible to the advancing face. The analyses also considered the performance of a partially filled concrete plug installed on the already mined TBM. Figure 9 shows the final passage of the second TBM and the steel rib support. The second tunnel was mined successfully by slow operation of the TBM, reduced gripper pad pressure and the help of the plug to counteract the grippers.
Figure 9: Shear zone at GCT5 WB wye cavern (left) and plot of DEM model with concrete plug

5 CONCLUSIONS

The multiple TBM drives of the Manhattan Contracts required innovation to help expedite TBM re-launch. Based on a concentrated effort by MTA Capital Construction, GEC and the contractor, a concrete plug scheme was successfully implemented for TBM re-launch and to excavate revised crossover caverns. The geometry and rock support of these caverns could be optimized with the results of numerical modelling and input of actual ground conditions encountered. The project could benefit from design-build workshop collaborations to optimize cavern alignment and rock support for cost and schedule savings. The authors acknowledge the contribution of employees at all levels of MTA CC, Contractor, Construction Management and the GEC.

REFERENCES


Soil Movements associated with Compensation Grouting during Line 9 Excavation in Barcelona: a Case Study

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Abstract

The paper describes a case history -related to the Line 9 excavation in Barcelona- where compensation grouting was initially proposed to minimize the movements of the buildings close to or directly above the tunnel alignment, some of which have piled foundations. Unexpectedly, the ground settlements due to the pre-conditioning stage of the compensation grouting became very significant leading to some light damage to one of the buildings. Eventually, the decision was taken to stop the treatment. Grouting-induced displacements are compared to the movements produced by tunnelling that were very limited. The paper concludes with some considerations concerning the suitability of ground treatment in the context of carefully-executed tunnelling operations.

Keywords: Case history; tunnels; EPB excavation; soil-structure interaction; grouting; field instrumentation
1 INTRODUCTION

A new underground metro line (Line 9) is being built in Barcelona. The line crosses intensively built areas and, therefore, the issue of ground movement control is of paramount importance [4], [6]. One of the methods for controlling ground movements is the performance of compensation grouting, which is widely used as reported in [1], [5], [8], [9], [11], among others. Yet there are relatively few applications of this method to tunnels excavated in cohesionless materials [1], [6], [12], [13] and below piled foundations [1], [2]. This paper is a case study of the compensation grouting that was proposed for the densely built Sant Andreu area in order to protect the buildings close to or directly above the Line 9 tunnel alignment (Fig. 1). Some of those buildings have piled foundations.

![Figure 1: Plan view of the full route of the Line 9.](image)

2 LOCATION AND GROUND CONDITIONS

The Sant Andreu area is located in the Northern part of Barcelona (Fig.1) and the section of Line 9 that crosses it belongs to Contract 4B. In the last 400 m or so, the Contract involved tunnelling underneath a number of buildings, some of which have piled foundations. In view of the proximity of the tunnel to these foundations, compensation grouting was proposed by the contractor as a means of protecting the buildings from possible damage during tunnelling. The tunnel diameter was 12 m and the ground cover, in the Sant Andreu area, varies approximately from 20 to 32 m and
the water table – that is roughly horizontal – is always above the tunnel crown at a depth of about 16-18 m. The cover/diameter ratio (C/D) varies from a minimum of 1.7 to a maximum of 2.7.

The soil sequence in this area consists of a fill, varying in thickness from 1.5 to 8 m (indicated as R in Fig. 2), overlying 25-33 m of alluvial deposits of Quaternary age, which are in turn underlain by siltstones, claystones and fine sands of Pliocene age (Fig. 2). The principal Quaternary alluvial deposits are composed of sandy silts and sands with gravel (Qₜ) and also of gravels in a sandy silty matrix (Qₜ₉). The Standard Penetration Test (SPT) values of both the Qₜ and Qₜ₉ layers are generally in the range 20-35, with occasional much lower values. They suggest that in some areas the Qₜ deposit is a relatively loose sandy silt or silty sand; furthermore, it sometimes has significant clay content (5 to 30%). The Qₜ₉ materials are similar to the Qₜ deposit, except for having a higher gravel content (up to about 64%). In both deposits, layers of silt or clay are present. The granular void ratio in these materials, as defined in [7], [10], [14], generally varies in the range 0.60-0.80 with sporadically lower and higher values (as low as 0.32 and as high as 1.12).

The tunnel is mostly excavated under mixed ground conditions involving the relatively loose Quaternary alluvial deposits and the harder Pliocene material.

Figure 2: Geological profile in the Sant Andreu area.

3 DAMAGE ASSESSMENT AND PROTECTIVE MEASURES

The contractor’s first damage assessment on the tunnel alignment had predicted maximum values of deflection ratios and horizontal tensile strains corresponding, according to [3], to the ‘Moderate’ category of potential damage; therefore protective measures were to be adopted in order to restrict building damage to an acceptable level. Compensation grouting was considered as the best option for settlement mitigation and for protecting sensitive areas. Prior to tunnelling, sleeved grout tubes (tubes à manchette, TAMs) were installed in the ground, from five vertical shafts (Fig. 2). The drilling for the installation of the grout tubes was undertaken from within
the shafts always at a level above the water table. The inclination of the TAMs varied from shaft to shaft (from near-horizontal to a maximum of 36° inclination to the horizontal), depending on the type and depth of the foundations and also on the distance between the shaft and the building to be protected. In all cases, the compensation plane was located in the Quaternary alluvial deposits (Q_r and Q_rg) and it was always ensured that the distance between pile toes and grouting level was greater than or equal to 2.5 m. Quite small settlements were recorded during the installation of the grout tubes, typically only a few millimetres.

4  PRE-CONDITIONING GROUTING STAGE

Initially, pre-conditioning grouting was undertaken in some trial zones using shafts 1’, 2’ and 4 (Fig. 2) and the resulting movements were observed. As soon as the conditioning phase started, the buildings above the compensation plane underwent some settlements. This is consistent with the granular nature of the ground that contained some loose material pockets. Compensation efficiencies not greater than 10% in materials similar to the Quaternary alluvial deposits of the Sant Andreu area (relative densities in the range 40-60%), have been reported [16] indicating probably efficiency loss through soil compaction in some zones. More unexpectedly, however, the downward movement continued with the grout injections at an approximately constant rate (Fig. 3) although, whenever grouting ceased, the settlement rate immediately decelerated and slowly stabilized. It was also noted that there was a practically constant relationship between the volume of grout injected and the observed settlements, without any evidence of a change of tendency as the pre-conditioning grouting progressed. The buildings that settled more were those (both piled and with shallow foundations) closest to the grouting shafts with a maximum measured settlement of about 18 mm (Fig. 3). One of the buildings near shaft 1’ developed some slight damage due to these settlements. The pre-conditioning grouting plan as well as the grout mix were reviewed and modified a number of times. Nevertheless, little or no sign of reduction in the rate of settlement with grout volume was observed, despite significant quantities of grout being injected.

Sub-horizontal TAMs were always located in the Quaternary alluvial deposits (Q_r and Q_rg). It is possible that the alluvial deposits in the Sant Andreu area (i.e., clayey and silty sands with fines content up to about 30%) show a significantly more unstable behaviour than clean sands. As described in [7], those mixed materials generally show rapid stiffness degradation with strain and exhibit pronounced undrained
brittleness and much greater strains at the end of contractant behaviour. Also, some negative skin friction might have developed on the piled foundations during the pre-conditioning stage, leading to some additional settlements as reported by [1] in relation to another case-history. All of those factors may have contributed to the unexpected large settlements recorded. While it seemed probable that injection of more grout would eventually lead to the settlements ceasing and the buildings then starting to rise, there was significant uncertainty as to how much additional settlement (and building damage) would have taken place before this occurred. Therefore, the decision was taken to cease the grouting operations and resume tunnel excavation without undertaking the originally planned compensation grouting treatment.

Figure 3: Evolution of settlement with time during pre-conditioning stage.

5 TUNNELLING

Tunnelling progressed from the vicinity of Shaft 1’ towards the Shaft 4 area (Fig. 2). All observed transverse settlement troughs closely approached Gaussian curves and the measured volume losses were quite small, in the range 0.22-0.25%. Because of the good control of the tunnelling operations, the settlements produced by tunnelling were generally significantly smaller than those caused by the pre-conditioning stage of the compensation grouting, see for instance Figure 4. Subsurface settlement profiles were approximately constant down to the depth of the compensation plane and below it they started decreasing, showing a stiffness increase of the grouted soil, as also described by [15] in relation to another case-history.
No building damage was reported during the tunnelling phase and, with the exception noted earlier, total damage always corresponded to the ‘Negligible’ damage category.

Figure 4: Settlement time evolution at the tunnel centreline at chainage 1840.

6 SUMMARY AND CONCLUSIONS

In the Sant Andreu area of Barcelona Metro Line 9, tunnelling was to be performed mostly through mixed ground conditions and a certain amount of surface subsidence and building settlements were to be expected. Compensation grouting was initially proposed to protect the buildings close to or directly above the tunnel alignment from possible damage. Some of those buildings had piled foundations. As a result of the pre-conditioning stage, ground settlements became quite significant, leading to some slight damage to one of the buildings. A number of factors may have contributed to this phenomenon. Shearing associated with grouting was likely to have caused a densification of the loose alluvial deposits, resulting in ground settlements. The fact that the alluvial deposits consisted mostly of clayey-silty sands, significantly less stable than clean sands [7], may have enhanced the compaction potential of the soil. Also, negative skin friction on the piles may have contributed some additional settlements of the buildings with deep foundations. In view of the uncertainty as to how much more settlement would have to occur before the buildings started to rise with increased grouting, it was decided that no further pre-conditioning grouting would be undertaken.
In fact, the pre-conditioning stage of the compensation grouting treatment was in this case the main source of ground deformation. Subsidence induced by tunnelling itself was very limited and no building damage was reported as a consequence of the excavation. This case history is another instance that illustrates that the use of mitigating measures for building subsidence should be carefully examined in the context of the ground displacements likely to be caused by a well-controlled tunnelling operation.

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Ceneri Base Tunnel: Optimization of the Excavation and Temporary Lining Design to Cope with Time-dependent Mechanical Behaviour

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Abstract

The Ceneri base Tunnel is an underground infrastructure part of the NEAT Gotthard Axis Project that consists of two single-track tunnels approximately 15.4 km long and characterized by covers up to 840 m. The track layout develops in the southern Alps and runs through the Zona del Ceneri tectonic unit (high inclined schistosity gneiss, mica-schists and amphibolites and cataclastic faults zones) and the Zona della Val Colla tectonic unit (sub-horizontal schistosity gneiss and mica-schists and an hectometric band of mainly mylonitic rocks - Linea Val Colla).

The excavation process, in some cases, induced stresses in the rock mass in the failure range, or close to it, which brought forth phenomena of instability of the face, of the crown or high convergences also deferred in time.

In order to prevent these mechanical responses, a protocol for the definition of the optimal parameters of the excavation process and of the temporary lining was defined. These parameters were obtained through numerical analyses calibrated on the basis of the monitoring data, suitable to describe the observed phenomena and to optimize the excavation and lining design process.

Keywords: AlpTransit, Tunnel, Temporary Lining, Face Instability, Failure
1 INTRODUCTION

1.1 The NEAT Project

Corridor 1, one of the main rail freight axes in Europe, runs from Rotterdam to Genoa along the River Rhine through the industrial heart of Europe (Fig. 1). The project is part of the European Commission promotion plan to improve the use of rail freight transport and to enhance sustainable mobility by encouraging the modal shift from road to rail. The centerpiece of Corridor 1 is the NEAT, the Swiss federal project for a faster north-south rail connection across the Alps. This project comprises two major sections, the Gotthard Axis and the Lötschberg Axis (Fig. 1).

Figure 1: Corridor 1 (left) and NEAT Swiss project plan (right).

The Gotthard Axis, besides the Zimmerberg base Tunnel (construction suspended) and the Gotthard base Tunnel, consists of the Ceneri base Tunnel (under Ceneri Mount), in the Swiss Canton Ticino.

1.2 The Ceneri Base Tunnel

The track layout of the Ceneri base Tunnel develops in the southern Alps and runs between Camorino (close to Bellinzona) and Vezia (Lugano). The tunnel consists of two single-track tunnels approximately 15.4 km long, ≈40 m apart and joined by cross passages every ≈325 m. There are no crossovers or multifunction stations.
The Ceneri base Tunnel can be extended to the south from an underground connection arranged in caverns. Excavation of both tubes is performed by the conventional drill and blast method from the south portal of Vezia, the north portal of Vigana and, in consideration of the excavation method, of the geotechnical conditions and in order to speed up the tunnel excavation phase, an additional working interface in Sigirino was built so that the tunnel is excavated simultaneously northwards and southwards from the caverns of this intermediate site (Fig. 2).

Figure 2: Ceneri base Tunnel functional scheme.

1.3 Geomechanical Prognosis

The Ceneri Base Tunnel is situated in the crystalline bedrock of the southern Alps and characterized by covers from \( \approx 10 \) m to \( \approx 840 \) m. The lithostratigraphic units crossed by the tunnel track layout are the Ceneri Zone (in the north, \( \approx 10 \) km long) and the Val Colla Zone (in the south, \( \approx 5 \) km long) (Fig. 3).

Figure 3: Ceneri base Tunnel geomechanical profile.
The Ceneri Zone is primarily formed of gneiss and, secondarily, of basic and ultrabasic rocks which have suffered considerable metamorphism. The whole area is influenced by complex tectonics, the main consequence of which is a high dip angle of the geological structures. The Val Colla Zone includes a series of paragneiss and orthogneiss, combined with basic rocks. In this zone the geological structures follow a sub-horizontal trends. The Val Colla Line fault zone (~600 m thick) is the interface between the two mentioned areas: this band is mainly composed of mylonites characterized by poor geomechanical properties. Moreover, the track layout crosses fault areas caused by fragile alpine tectonics, especially by the insubric phase.

1.4 Main Geotechnical Problems in Excavation

The tunnel excavation was affected by some geotechnical uncertainties and incidents. Problems alike occurred in the south-east tube of the Ceneri base Tunnel due to a fault zone not considered in the prognosis and characterized by a considerable real thickness (~70 m). This deficiency in the geological survey means that the Val Colla Line fault zone was found almost 190 m further north than expected. Furthermore, the particular geometry condition at the cross section between the south-east tube and the cross passage 30-T – due to a prolonged stop in the site operations, which also induced a mechanical properties degradation of the rock mass – generated face instability phenomena (Fig. 4).

Figure 4: Face collapse in south-east tube (left) and geological geometric survey (right).

The geological conditions detected in the beginning sector of the north-west tube (~50 m long), compared to the prognosis that predicted a rock mass mainly
disturbed by oblique fault zones, pointed out a rock mass strongly characterized by small spacing fault zones subparallel to schistosity which determined a poor quality rock mass and a deferred mechanical response. Fig. 5 shows the typical mechanical response of the rock mass to the excavation process in terms of vertical displacements over time.

Figure 5: Ceneri base Tunnel north-west tube monitoring data: vertical displacements over time.

The excavation perimeter displacements (also deferred in time) generated phenomena of temporary lining cracking. Moreover, the anchors plates were bent or broken (Fig. 6).

Figure 6: Ceneri base Tunnel north-west tube: temporary lining cracking and damaged anchors plates.

These problems highlighted the necessity to have an operational tool for the definition of the optimal temporary lining package (TLP) in order to avoid problems of face and crown instability and temporary lining cracking.
2 OPERATIVE PROTOCOL TO OPTIMIZE THE EXCAVATION PROCESS

In order to prevent the geotechnical phenomena described in §1.4, an operative protocol to optimize the excavation process has been defined. The protocol is organized in tasks as shown in the flow chart below (Fig. 7).

Tasks are organised to perform the following operations:

1) B1 – 3D numerical analyses to estimate, for each i-class of the rock mass index $RMR_{corr}$ and for each j-classes of the TLP, the distance $\bar{x}_{ij}$ (from the face) for which the convergence is not affected by excavation effects (see for details §3);

2) B2 – In situ evaluation of the real $RMR_{corr}^{real}$ and of the real tunnel convergence $CC_{ij}^{real}$ (by regular intervals – e.g. 1D).

This task requires a systematic procedure for a monitoring data continuously collection with the on-going of the excavation process;

Figure 7: Flow chart of the operative protocol.
3) B3 – Evaluation of the time dependent effects, by considering the $c_{ij}^{\text{real}}$ in the monitored sections at a distance $x > \bar{x}_{ij}$, via $\alpha_{ij}$ (real deferred convergence rate over time for a specific $RMR_{\text{corr}},i$ and for the adopted $TLP_j$). The monitored convergence rate $\alpha_{ij}$ will be not negligible if greater than $\alpha_{ij}^0$ which represent a watershed for considering time effects;

![Figure 8](image_url)

**Figure 8:** Real deferred convergence rate over time for a specific $RMR_{\text{corr}},i$ and for the adopted $TLP_j$.

4) B4: performing of fitting numerical analyses (e.g. c-φ reduction or adoption of a viscous constitutive law calibrated on monitoring data) to evaluate the optimal $TLP_j$.

Finally the operative protocol can be summarized in a matrix linking $RMR_{\text{corr}},i$, $\alpha_{ij}$ to $TLP_{ij}^{\text{opt}}$ as shown in Tab. 1.

**Table 1:** Correspondence between $RMR_{\text{corr}}, \alpha_v$ and the optimal temporary lining package (TPL).

<table>
<thead>
<tr>
<th>$RMR_{\text{corr}}$</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>81-100</td>
<td>SPV 2</td>
<td>SPV 2</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>61-80</td>
<td>SPV 3</td>
<td>SPV 3</td>
<td>SPV 3</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>41-60</td>
<td>-</td>
<td>SPV 5</td>
<td>SPV 5</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>21-40</td>
<td>-</td>
<td>-</td>
<td>SPV 6</td>
<td>SPV 7</td>
<td>SPV 9</td>
</tr>
<tr>
<td>&lt;21</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>SPV 10</td>
<td>SPV 10</td>
</tr>
</tbody>
</table>
3 FACE COLLAPSE NUMERICAL ANALYSIS

An application example of the operative protocol to the face collapse case discussed in §1.4 is presented. Fig. 9 shows the reconstructed tunnel geometry at the cross section between the south-east tube and the cross passage 30-T.

Figure 9: 3D geometry and mesh of the numerical model.

According to protocol task B1, 3D elasto-purely plastic numerical analyses were performed to evaluate the $\bar{x}$ distance. Fig. 10 shows the displacement contours and different displacement curves.

Figure 10: Displacement contours at different excavation stages and longitudinal tube displacements.
On the basis of the monitoring data (B2 task) and according to B3 protocol task, the \( \alpha_v \) rate was estimated (referred to a \( \alpha^0_v \) value of 2 mm/month).
A fitting analysis (B4 protocol task) was performed by considering the adopted TLP, the real excavation stages and a prolonged stop of the site operations, simulated by a progressive reduction of the strength parameters as shown in Table 1.

**Table 2:** Geomechanical parameters reduction considered for the face collapse analysis.

<table>
<thead>
<tr>
<th>Geomechanical strength parameters</th>
<th>( c ) [kPa]</th>
<th>( \varphi ) [°]</th>
<th>( \psi ) [°]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial parameters</td>
<td>600</td>
<td>30</td>
<td>8</td>
</tr>
<tr>
<td>Int. parameters</td>
<td>300</td>
<td>24</td>
<td>4</td>
</tr>
<tr>
<td>Final parameters</td>
<td>0</td>
<td>18</td>
<td>0</td>
</tr>
</tbody>
</table>

Fig. 11 shows the numerical analyses results at different parameters degradation stages (until the collapse) in terms of deviatoric plastic strains, displacement and deviatoric stress contours. It is interesting to note that the values reported in Tab. 2 represent the maximum and the minimum sets evaluated in the laboratory tests.

**Figure 11:** Deviatoric plastic strains (1-3) and displacement (4-6).
CONCLUSION

In order to prevent instability phenomena in the Ceneri base Tunnel tubes, an operative protocol for the definition of the optimal parameters of the excavation process and of the temporary lining was defined. The protocol parameters were obtained through several numerical analyses calibrated on the basis of the monitoring data. The protocol centerpiece is the matrix that creates a correspondence between the rock mass quality, the rock mass mechanical response nature and the optimal temporary lining package available for the Ceneri base Tunnel.

A protocol application to the face collapse case is shown: the preliminary numerical analyses proved the suitability of the adopted temporary lining package in no-stop excavation conditions. The fitting analyses based on the described monitoring data analysis and geomechanical tests pointed out that an TLP improvement is required if a prolonged excavation stop is considered (e.g. consolidation measures, tube pre-support action).

Figure 12: Rock bolt stresses (7-9) and deviatoric stress contours (10-12).
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