Structural Health of a concrete tunnel lining under complex in situ loading

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STRUCTURAL HEALTH OF A CONCRETE TUNNEL LINING UNDER COMPLEX IN SITU LOADING

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Abstract

Supporting structures were designed for a utility tunnel in hard rock which was to be intersected by some road tunnels. With the intersections involving partial overlapping between the ideal cross-sections, a concrete lining and post-tensioned steel cables were added to support the utility tunnel prior to the excavation of the road tunnels. The objectives in this work were to identify the structural behaviour of the system, assess the present state of the supporting structures and suggest an effective monitoring strategy. Preliminary information was collected from the original drawings, technical specifications as well as from site visits. Using on the finite elements software Comsol Multiphysics, suitable models were built to represent the tunnel intersections and to test hypotheses. The significant uncertainties in the problem were addressed by studying limit cases and discussing their results. A measure of validation was gained from comparing cracks in concrete on site with cracking hotspots predicted by the model. It was concluded that the behaviour of the lining is not compatible with that of cable suspended structures and that concrete is subject to relatively high state of stress. Monitoring cable forces was found to be an ineffective strategy for identifying changes in the structural system. Monitoring the evolution of cracks in concrete was found to be the most feasible strategy.
The present work was developed in Stockholm Sweden as result of a double degree programme between Politecnico di Milano and KTH - Royal Institute of Technology. I would like to offer my special thanks to the staff of the Exchange Office of both Politecnico di Milano and KTH who supported me long before my first flight to Sweden.

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Finally, I would like express my gratitude to Prof. Raid Karoumi and the academic staff at KTH for the support I received as well as for the access to PDC - Parallel Data Centre of Sweden I was kindly granted.
Quale simbolo della mia laurea magistrale in Ingegneria Civile presso il Politecnico di Milano ed il KTH di Stoccolma, dedico il presente lavoro ai miei cari nonni: Remo, Bruna, Alfredo e Danila.
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Symbols

- $f_{ck}$: Characteristic concrete compressive strength
- $f_{ctk}$: Characteristic concrete tensile strength
- $E_{ck}$: Characteristic concrete elastic modulus
- $E_{cm}$: Average concrete elastic modulus
- $\rho_c$: Concrete density
- $E_{l,avg}$: Average leca stiffness
- $\rho_{l,avg}$: Average leca density
- $\nu$: Poisson’s ratio
Operating within the field of Structural Health Monitoring (SHM), this work presents the preliminary investigations carried out to ultimately propose a health monitoring plan for a pre-existing structure. As the built environment grows older, monitoring systems become more and more necessary as means to continuously assess the performance of structural elements and act as early warning system for structural deterioration. The case study here considered is that of an underground facility located in Stockholm, Sweden. The facility is a historical utility tunnel in hard rock which had to be reinforced to allow for the subsequent construction of a network of road tunnels. In those segments where the utility and road tunnels intersect one another, the utility tunnel was braced with the addition of a reinforced concrete lining supported by a set of pre-tensioned steel cables anchored in the rock above. The owner of the facility requested that the state of stress of the concrete sections be assessed and that a monitoring strategy be proposed to ensure a constant control over the functionality of the structure.

This master thesis was developed as result of a double degree program between Politecnico di Milano and KTH - Royal Institute of Technology. The work was conducted at the Stockholm office of the consultancy firm Tyréns AB. Numerical models were created in the commercial software COMSOL Multiphysics. Access was kindly granted by KTH to PDC - Center for High Performance Computing of Sweden, where most of the models were run.

1.1 Background

In 1997 construction of the Södra Länken motorway began. Being a part of the Stockholm Ring Road project, planning the creation of a road enclosing central Stockholm, the Södra Länken, or southern link, would follow the southern perimeter of the city from Essingeleden to Värdöleden. When inaugurated, on the 24th of October 2004, the motorway was 6km in length of which 4.5km were underground [1]. This study however does not revolve around this infrastructure project but rather focuses on what was present in the rock before Södra Länken tunnels were excavated.

During the 1960s Century a utility tunnel was excavated in the southern part of Stockholm. Given the good quality of rock present, the tunnel came with no supporting structures and with an approximate section of 2x2 m. In the following years, the tunnel was expanded with the addition of a second branch having a wider cross-section of about 5x4 m, through which a service vehicle could circulate.

When the Södra Länken project was conceived, measures had to be taken to ensure the safety of the utility tunnel as they would meet in several locations; specifically, as schematically shown in
1. **Introduction**

Figure 1.1: Map of the Södra Länken project with underground segments highlighted (from [1])

Figure 1.2, the utility tunnel would partially intersect the motorway tunnel in correspondence to two ramps leading to the surface.

To ensure the continuous functionality of the utility tunnel during both excavation and operation of the road tunnel, a supporting system was designed and constructed. Following a conservative philosophy, well justified by the inherent uncertainties of any problem involving rock mechanics, the supporting system was conceived as combination of a lining of reinforced concrete and a set of post-tensioned steel cables anchored in the rock above the service tunnel. The fundamental idea was to have the system follow the principle of a suspension bridge, where lining and prestressed cables would represent bridge deck and hangers. A standard amount of orthotropic reinforcement was prescribed for the lining assuming the cables to be the main bearing mechanism of the system. Ideally, the support was designed so that the service tunnel would be able to maintain its functionality even in the event of a localised rock collapse of relevant dimensions. Once all supporting structures were installed in the service tunnel, the road tunnels of Södra Länken were excavated.

The utility tunnel meets Södra Länken in several points. As a result, the a supporting structure was required in most locations and its configuration varied depending on how the intersection would take place. In those segments where the road tunnel is located just below the utility tunnel, concrete lining and prestressed cables were employed. In the segments where instead the utility tunnel takes the lower position, no cables were installed.
After the project completion, a monitoring strategy was set up to periodically assess the condition of the supporting structures. In accordance with the assumption of having the concrete lining follow the principles of cable-supported structures, all focus was placed on the steel tendons. The basic reasoning was that if any relevant event were to take place (e.g. degradation, settlements) the cables would react to it. The monitoring plan consisted in periodical inspections where the forces in the cables would be estimated by studying their natural frequencies.

The GK-403 vibrating wire readout box by Geokon, in conjunction with a vibrating wire sensor were employed for this purpose. This instrumentation excites a cable with a spectrum of frequencies and listens to its response to identify the eigenfrequencies. Since all geometrical properties of all cables were recorded after installation together with their prestressing force, the present time cable forces could be estimated and their trend analysed over time. Although the cable anchors on the side of concrete may very well be represented by a pinned connection, the restraint provided by the anchoring length at the opposite end may prove to be much more compliant. If cable forces are estimated for the analytical case of a pinned cable, the validity of any comparison between them and the initial prestress may be reduced. Studying the trend over time however would still allow for possible issues to be flagged. Having run for a decade, the monitoring plan was ultimately dismissed because of a technical malfunction of the instrumentation: after the sensors started picking up significant changes in natural frequencies, measurements were conducted with a different instrumentation and this disproved that any changes were actually be taking place. This event led to a reconsideration over the effectiveness of this monitoring strategy.

In the following list a short summary of the events in time is presented:

1. The utility tunnel is excavated in hard rock (c. 1960).
2. A reinforced concrete lining is cast in place for the service tunnel (1997).
3. Cables are installed and pretensioned.
4. The Södra länken road tunnels are excavated.
5. The Södra länken is inaugurated (2004).
6. A monitoring plan focusing on the steel cables is initiated.
7. The monitoring equipment malfunctions giving false reads and is abandoned.

With the aim of ensuring uninterrupted functionality of the service tunnel for the years to come, the owner of the facility tasked Tyréns AB with the verification the structural stability of the supporting
structures. A monitoring strategy in conjunction with a new valid and effective monitoring system were requested. This thesis was developed in parallel to the project of Tyréns AB. With the aim of reducing the scope of this work, only the two intersection, identified \textit{a priori} as likely to be the most sensitive, were analysed.

### 1.2 Research questions

As briefly outlined in Section 1.1, the task put forth by the owner of the facility was to assess the current state of the underground utility tunnel and to devise a monitoring system capable of identifying any future changes that might occur. In this spirit the following research questions were defined:

- What is a suitable model to capture the structural behaviour with similarity and precision to the physical case?
- What is the stress state in the different elements composing the structural system (concrete, reinforcement, pre-tensioned bars)?
- Which elements are most significant to be monitored?
- What type of monitoring system could be installed?
- What types of degradation might lead to future issues?

These research questions follow the logical progression of the study towards the definition of a monitoring strategy: the first point of order is to construct a model and ensure its robustness and similarity to target; then the model is used to assess where our attention should be focused i.e. where the most stressed points are located or which elements are most susceptible; and finally a monitoring strategy is designed and a monitoring system proposed. From a more practical point of view, the models were used to identify which variables should be monitored and where such variables should be monitored.

### 1.3 Description of the facility

The information presented in this section was acquired from the original project drafted for the reinforcement of the utility tunnel. For security reasons, the original technical drawings could not be published, the following drawings are therefore copies of the original tables where all sensitive details have been removed to avoid disclosing the precise location of the facility.

Figures 1.3 and 1.4 display the plan and elevation views respectively of the utility tunnel. Entailing the Södra Länken a network of numerous tunnels, several crossings take place between it and the utility tunnel. Reinforcing operations were carried out for all tunnel segments where the relative distance between tunnels was deemed insufficient. Out of all the crossings, the cross-sections of the utility and road tunnel physically intersect only in two locations: the scope of this study was reduced to the analysis of these segments alone.

Along the considered part, the utility tunnel divides into two main segments having different planar orientation, slope and cross-section. Historically, the segment where intersection 2 takes place was the first to be excavated: this tunnel has a smaller cross-section and a slope of approximately $-1\degree$ with respect to the horizontal plane. Being more recently excavated, the tunnel segment where intersection 1 is located allows for the circulation of service vehicles and is oriented at an angle of
+5.80° to the horizontal plane. At its highest point, positioned between intersections 1 and 2, the utility tunnel is found at a depth of approximately 35 m from the surface. Identification tags are assigned to relevant elements of the facility for clarity purposes: UT is used as abbreviation for "utility tunnel" while RT for "road tunnel". As a result, in intersection 1, the utility tunnel segment named UT-1 runs below the road tunnel ramp RT-1A and above ramp RT-1B. In intersection 2 the segment UT-2 crosses ramp RT-2.
1. INTRODUCTION

1.3.1 Geometry - Intersection 1

The detailed drawing for the plan view of Intersection 1 is presented in Figure 1.5; here the lateral expansions visible on one side of the utility tunnel were excavated to allow for the manoeuvring of service vehicles. In Figure 1.6 the elevation can be seen together with the reinforced concrete lining and prestressed cables. The approximate cross-section of the utility tunnel at the rock perimeter is proposed in Figure 1.7 while the cross-sections of road tunnels RT-1A and B are visible in Figure 1.8. To account for the method of excavation, human factor and rock spalling, an overbreak was applied to all sections: in UT-1 40 cm of overbreak were added to the sides and 20 cm to the floor (see Figure 1.9); for both road tunnels a uniform overbreak of 40 cm was added as an offset to the ideal cross-section. The overbreak for the utility tunnel was estimated as average of the measurements taken on site at the two ends of the concrete lining, where the distance from the inner concrete wall surface to the rock surface could be measured.

Two different supporting layouts were constructed along this tunnel segment: for simplicity, the reinforced cross-section used at the RT-1A crossing is named UT-1A while the reinforced cross-section for the crossing of RT-1B is named UT-1B. Being RT-1A located above the utility tunnel, only walls and ceiling were strengthened. For the crossing of RT-1B all surfaces were supported by concrete and prestressed cables were installed. Concrete thickness was chosen at 20 cm for the ceiling and 30 cm for walls and floor. The layout of the reinforcement for these segments is presented in Figures 1.10 and 1.11. The concrete lining UT-1B is additionally supported by prestressed cables anchored in the rock above. Details on this additional system are presented in Section 1.3.3.
1.3. Description of the facility

Figure 1.6: Elevation view of Intersection 1

Figure 1.7: Cross-section of UT-1 (rock surface)
1. INTRODUCTION

Figure 1.8: Approximate cross-sections of RT-1A and RT-1B

Figure 1.9: Overbreak of UT-1A and UT-1B
1.3. Description of the facility

Figure 1.10: Concrete lining and reinforcements of UT-1A
(see Figure 1.11 for details on rebars)
1. INTRODUCTION

Figure 1.11: Concrete lining of UT-1B
1.3.2 Geometry - Intersection 2

Following the same order of presentation employed in the previous section, Figures 1.12 and 1.13 present the plan view and the elevation respectively. Figures 1.14 and 1.15 show the ideal cross-sections at the rock surface according to the original technical drawings. For RT-2 a constant overbreak of 40 cm was applied to the ideal cross section; for the service tunnel instead, 30 cm and 20 cm were applied to the walls and floor respectively. Also for this segment of utility tunnel, the overbreak was estimated as average of the simple measurements taken on site. The supporting structure consists here in a reinforced concrete lining having 20 cm of thickness on the ceiling and 30 cm everywhere else. The reinforcement layout can be seen in Figure 1.17. As for UT-1B, prestressed steel cables were also installed to support the concrete lining, details on this are specified in Section 1.3.3.

Intersection 2

![Plan view of Intersection 2](image1)

Figure 1.12: Plan view of Intersection 2

![Elevation view of Intersection 2](image2)

Figure 1.13: Elevation view of Intersection 2
1. **INTRODUCTION**

Figure 1.14: Cross-section of UT-2 (rock surface)

Figure 1.15: Approximate cross-section of RT-2
1.3. Description of the facility

**Figure 1.16: Overbreak of UT-2**

**Figure 1.17: Concrete lining and reinforcements of UT-2**
1.3.3 Prestressed cables

In accordance to the ’belt and braces’ principle, the tunnel segments where cross-sections physically intersect one another were supported by a reinforced concrete lining in combination with an array of prestressed steel cables. The basic principle behind the system is similar to that of a suspended bridge where the load is carried partly by the deck and partly by the cables. Furthermore, the cables have the function of ensuring structural integrity in case the rock surrounding the tunnel were to move or be fractured.

As can be seen in Figures 1.18 and 1.19, cables were installed in a regular pattern with symmetry axis along the tunnel axis. Cables were prescribed to have an inclination of 25° with respect to the vertical axis and a effective cross-sectional area of 450 mm². For each borehole three cables were installed and tensioned individually until an average prestressing load of 1100MPa was reached. Compatibly with the different size of the utility tunnel in intersection 1 and 2, cables were prescribed to have 9m length with 170m spacing for UT-1B and 6m length with 25m spacing for UT-2.

On the rock side, cables were inserted in a ≥100 borehole and submerged in concrete to generate an anchorage length of at least 25m. At the opposite end, the anchorage was performed as show in Figure 1.21: the cable lock was rested on a steel plate of size 150x150x25 mm which was in turn supported by concrete and by a steel cylinder 100mm in diameter and 20mm in thickness.

During the installation phase, length, inclination and prestressing load were recorded for all cables. These information, which were taken into account when modelling, are reported in Appendix B.
1.3. Description of the facility

Figure 1.19: Prestressed cables layout for Intersection 2

Figure 1.20: Front view of the cable anchorage system
Figure 1.21: Detail of cable anchorage
1.4. Common practice

1.3.4 Geology

The tunnels considered in this work are all located just south of Stockholm. As can be seen from Figure 1.22, courtesy of the Swedish Geological Institute, the island of Södermalm hosts a tectonical deformation zone which separates two distinct blocks of bedrock. While north of the fault line the lithology presents granitoid and syenitoid rocks, the southern block is characterised by metagreywacke, mica schist, graphite and/or sulphide-bearing schist, paragneiss, migmatite, quartzite and amphibolite [2].

![Figure 1.22: Bedrock types south of Stockholm (from [2]). Cyan: Metagreywacke, mica schist, graphite- and/or sulphide-bearing schist, paragneiss, migmatite, quartzite, amphibolite. Brown: Granitoid and subordinate syenitoid](image)

According to a technical survey conducted in utility tunnel (see [3]), the rock mass is dominated by finely grained gneiss rocks where elements of medium grained diorite and medium grained foliated granodioritis can be found. The rock was reported to be significantly heterogeneous but with similar properties along the length of the tunnel and hence across the rock mass.

1.4 Common practice

In the country of Sweden, tunnel facilities are regulated by guidelines set by Trafikverket, the traffic administration. Common practice to study tunnels in rock follows three subsequent steps of complexity: if possible, a 2D continuum linear elastic model of the facility is constructed; if no symmetries can be exploited for a 2D model to be built, a 3D linear elastic continuum model is chosen. If the assumption of linear elastic material is insufficient, the choice is then between a discrete finite elements model, or a continuum model with non-linear material formulation (see [4]).
1. INTRODUCTION

Although common practice is able to suggest a starting point, it does not apply to the research questions here posed. The uniqueness of the problem to be investigated makes it fall outside the boundaries of what is covered by common practice in the field of rock science and engineering. No previous studies of sufficiently similar issues could be found in literature.
Methodology

In this chapter, the approaches to investigate the problem and answer the posed research questions are discussed. The methodology here presented highlights which methods are available, which one was chosen and the motivation for doing so. The selected method is then briefly discussed in its main steps of analysis.

2.1 Approaches

When faced with the task of archiving *quantitative* measures for features of interest in the Structural Engineering realm, the Engineer can usually only resolve to experimental testing, analytical studies through theoretically established models, or simulations based on computational methods such as the finite element method.

Case studies are rarely an option in Structural Engineering seen as they are mostly a source of qualitative information and require a relevant case (or set of cases) to be available in literature; this often makes them unfit for design problems. Case studies are also not an acceptable path in the present investigation as they are unlikely to answer the research questions posed in Section 1.2; furthermore, no relevant case study was uncovered in the literature.

For the case described in Chapter 1 and its features of interest, experimental testing can also be ruled out as a viable approach. No destructive tests could be carried out in the facility but even still, any other type of testing would be unable to address all the research questions posed. Experimental testing may be conducted on a scale model of the service tunnel but this would require an extraordinary effort to reproduce all the boundary conditions and interactions. This may ultimately result in excessive uncertainties not making such a choice justifiable.

The use of analytical methods on the other hand, could in theory be able to provide relevant answers to all questions, but it may not be sufficiently accurate in capturing the real behaviour. Neglecting for a moment the complexity that this may entail, which in itself may be sufficient to disregard this path, it should be noted that the problem involves a rather complicated geometry as the service tunnel meets the road tunnel at an angle in both vertical and horizontal planes; the use for example of a simple beam model, would not be able to capture the resulting kind of boundary conditions properly therefore neglecting torsional effects.

The approach chosen to run this study is that of modelling on the basis of the finite element method by creating a model sufficiently similar to the physical reality. Such a model should be able to yield
2. Methodology

various types of data at the necessary level of detail, while still maintaining the flexibility to answer different scenarios of degradation required by the research questions.

When it comes to the modelling of rock behaviour, continuum or discrete models may be used [5]. While discrete finite elements may be used to describe faults in the material, discontinuity lines or even spalling, it is thought that in this case a continuum model would be more suitable: the rock is of good quality without major faults being reported, additionally, the aim is to study the concrete lining rather than the precise behaviour of the rock surrounding it.

2.2 Outline of the method

The interaction between the utility tunnel and the nearby road tunnels was investigated by means of a finite elements model. The geometry of the problem was acquired from technical drawings of the facilities while material parameters were estimated by means of in situ testing and engineering judgement. Having acquired the needed data, simplified 2D models were set up to analyse possible boundary effects and to investigate how the anchoring of the prestressed steel cables on the concrete side could be modelled effectively.

Drawing from the above studies, the 3D finite elements model was constructed and several steps of analysis set up to simulate all changes that the utility tunnel underwent over the years. A separate model was created for each of the intersections between tunnels: the same model could not in fact be used twice as the intersections take place at different angles and involve different cross-sections. In the following list, the basic analysis steps are presented:

1. Initial conditions: solid domain of rock material.
2. Excavate the utility tunnel.
3. Reinforce the utility tunnel by means of a concrete lining.
4. Install prestressed steel cables.
5. Excavate the road tunnels.

Internal validity was ensured by verifying the quality of the FEM model; specifically, a convergence check and a check of the influence volume were conducted. Convergence was verified by analysing the effect of different element types and different mesh sizes in the model; special attention was paid to the areas surrounding the intersections between tunnels. The influence volume was studied to avoid significant boundary effects in the model. In the different steps, especially after the introduction of the road tunnel, it was verified that the stress-state in the rock is not significantly influenced by the boundaries of the model.

Similarity was forced by acquiring the geometries from technical drawings and verifying them in situ. The stress field in the rock was also calibrated to match that experimentally surveyed. The stress state in the rock was experimentally estimated at about 1 km away from the points of interest. Being the rock mass for the most part intact, it was believed that the estimated stress-field would be present before the construction of the road tunnels also in the volume where the intersections take place. This state of stress was emulated in the model at step 1.
2.3 Risks and uncertainties

The present study is not thought to result in any direct source of risk for the users of the road tunnel or for the owner and operator of the utility tunnel. This study is focused on providing understanding rather than producing numerical values.

Risks may arise in the future with the installation of a monitoring system. The installation and data treatment from said monitoring system are however outside the scope of this work.

The main sources of uncertainty are expected to be the material properties, the interaction between the prestressed bars and the rock, the present day state of stress in the rock and the effect of blasting on the rock surrounding the road tunnel. These could however be limited in the final steps through a calibration process aimed at matching results from the model to experimental results. Observational methods and measurement of the deformations in situ could be carried out for this purpose. In Section 3.3 all modelling assumptions are presented with greater detail and discussed. Modelling assumptions were grounded based on different levels of uncertainty.

2.4 Discussion

When discussing the quality of a model emulating reality, the main features to be critically considered are similarity to target, robustness, precision, simplicity, theoretical tractability and transparency.

- Similarity to target was increased by employing technical drawings, in situ measurements and experimentally measured rock stresses. Similarity to target could be additionally improved by measuring the actual stress-field in the rock near the points of interest where the intersections take place. Though this may not be possible because of the concrete lining in between, it could be performed just before or after the reinforced cross-sections. Another way of increasing similarity to target could be to experimentally estimate the stress level in the tension bars supporting the service tunnel which may give information on how much pre-stressing has been lost by the cables over the years. One more means of improving this feature would be to employ the laser scans that were performed for all surfaces along the service tunnel. Although viable as option, this path was not undertaken as it would have increased the complexity of the model without adding a very significant contribution: from surveying the site, the concrete surfaces were found to be regular while the rock, albeit rough because of the excavation techniques used, did not present significant variations in average terms (i.e. no major spalls or mass detachments were observed).

- Robustness with respect to certain features was investigated during the quality assurance of the model by studying different material properties and boundary conditions. It is especially important that the model is robust with respect to the element size and to the boundary conditions to avoid boundary effects.

- Precision was ensured by performing the convergence checks and studies of the mesh properties. Precision was verified with respect to relevant variables such as the stress in the prestressed cables and the displacements of the utility tunnel.

- For the reasons discussed in Section 2.1, the model has a low level of tractability. The size of the models as well as their complexity reduces the level of tractability. The problem could certainly not be solved through hand calculations.
2. **Methodology**

- The model was initially thought to have a degree of transparency as the tunnel supported by prestressed cables was somewhat seen as similar to a suspension bridge. The presence of rock subjected to a non-uniform stress-state did however decrease the overall transparency of the model demonstrating that the structural principle is in fact not that of a cable supported beam.

While internal validity was archived by ensuring the quality of the model, and by following in steps the historical evolution of the structural system, external validity was not ensured in the scope of this work since no experimental measures were conducted. External validity may be ensured by actively installing a monitoring system and calibrating the finite element model against these experimental data for known loading conditions. This step may be undertaken to then install a working monitoring system for the service tunnel, and satisfy the request by the client.
In this Chapter, the method employed to ultimately answer the research questions posed is presented in detail. The chapter is organised to follow the conceptual development of the work conducted, from initial steps to final studies. Overall, this work was aimed at gaining knowledge and insight on the structural conditions of a physical system rather than providing precise numerical values; given the high degree of uncertainty, the problem was only treated in terms of conservative limit conditions.

Following the structure of most engineering problem, the first section deals with the data acquisition and the estimation of the material parameters. In section 3.2 the main preliminary studies are described: these were aimed at identifying the best strategy to model details and acclimatise with the capabilities of the finite elements package used. The 3D model of the intersections to be studied is presented in Section 3.3 along with all the steps taken to build it from ground up. The final sections of the chapter, namely 3.4, 3.5 and 3.6 present the studies of sensitivity performed on the most uncertain parameters of the model. By defining limit conditions in terms of maximum and minimum values of a given parameters, these analyses allowed to partially account for the uncertainties of the problem. With the aim of providing a means of validation for the model, Section 3.7 delineates how the cracking pattern in the concrete lining was analytically studied and then experimentally verified on site.

### 3.1 Material properties

From careful consideration of the technical drawings provided by Tyréns AB, four materials were considered: concrete, leca, steel (regular and high strength) and gneiss rock. Material properties were defined in different ways depending on their nature and uncertainty level. For those materials resulting from a technical prescription, characteristic values were employed. Other properties were estimated.

#### 3.1.1 Concrete

Concrete is the material used to construct the lining supporting and strengthening the utility tunnel. Being a man-made material its properties were given as prescription when the Södra Länken project was developed. Regular concrete was used for shoulders and flooring while shotcrete was employed for the ceiling of the utility tunnel.
Regular concrete

In the original technical specification, concrete is prescribed to be K40 graded according to the Swedish standard Boverkets Handbok om Betongkonstruktioner, BBK 94. As the original project was redacted in 1997, Eurocode classes had not yet been adopted. Before casting the formwork was tested for waterproofing. The maximum aggregate size was 32 mm in a fraction lower than 8%. In Table 3.1 concrete material properties are presented, characteristic values were used without any safety coefficients applied to them.

Table 3.1: Material properties of concrete, see [6]

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_{ck}$</td>
<td>28.5 MPa</td>
</tr>
<tr>
<td>$f_{ck}$</td>
<td>1.95 MPa</td>
</tr>
<tr>
<td>$E_{ck}$</td>
<td>32.0 GPa</td>
</tr>
<tr>
<td>$\rho_c$</td>
<td>2400.0 kg/m$^3$</td>
</tr>
<tr>
<td>$\nu_c$</td>
<td>0.20 -</td>
</tr>
</tbody>
</table>

Shotcrete

Reinforced shotcrete was employed to create the ceiling of the concrete lining. Prescriptions were made in accordance with BBK 79 Chapter 19: Class 1, cement quality $K \geq 40$ and aggregate size between 8 and 16 mm. Experimental tests after casting were also prescribed having the following as requisites:

\[
\begin{align*}
\text{Density} & \geq 2200 \text{ kg/m}^3 \\
\text{Comp. strength} & \geq 40 \text{ MPa}
\end{align*}
\]

Seen as the prescribed shotcrete quality is higher than that of the material used for the rest of the lining, the material properties in Table 3.1 were assumed also for the shotcrete in the roof in the spirit of being conservative.

3.1.2 Leca

As mentioned in Section 1.3, leca was used as an intermediate material separating the rock surface from the outer surface of the concrete lining. This material was prescribed to obviate to the uneven rock surfaces and to generate a regular external boundary for the concrete to be cast subsequently (see Figure 3.1). Leca concrete is a light weight concrete where regular aggregates are substituted by expanded clay pebbles which make for a durable and lightweight aggregate. The main characteristic of expanded clay is low density combined with high strength.

Leca was cast in situ but no precise specifics were given in the technical prescriptions so that its properties had to be entirely estimated. In Figure 3.1 the single clay pebbles are clearly visible as they are not drowned in cement paste and have large voids separating them. This might support the hypothesis that the cast material was in fact of rather low quality with only a minor amount of cement being used which in turn would justify the choice of low strength and stiffness for the material parameters. In contrast, it should be pointed out that the picture (and all the other observations) were made at the ends of the lining where this layer was visible and where the formwork would be located; in these areas the quality of the material might simply be lower because of poor
3.1. Material properties

compaction and absence of vibration during casting. In light of these contradicting observations, the material properties were chosen considering the practical purpose of the Leca layer: that is to provide a regular boundary within which to cast concrete rather than to provide a high quality material for structural purposes. This high uncertainty was however addressed by performing a sensitivity study and considering the structure under two limit conditions: an average quality Leca and a very low quality one (see Section 3.6).

Bogas and Gomes [7] performed a study of the material properties of light-weight concrete including concrete of Leca-type. They were concerned with the definition of strength and stiffness as a function of the components and percentages used in the concrete recipe. To gain representative values of the Leca properties in situ, averages were taken from the data collected (see Table 3.2).

Table 3.2: Average Leca properties from [7]

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_{l,avg}$</td>
<td>20.0</td>
<td>GPa</td>
</tr>
<tr>
<td>$\rho_{l,avg}$</td>
<td>1700.0</td>
<td>kg/m$^3$</td>
</tr>
</tbody>
</table>

In the study, expanded clay was employed in a controlled environment to have batches of good quality light weight concrete. This was thought to be rather in contrast with the function of the leca layer in the utility tunnel; as a result, the average density was employed while engineering judgement applied to the estimation of the elastic modulus. Compatibly with the non-structural purpose of the material, the E-modulus of leca was chosen as $1/10$ of the concrete one to account for the possibly low quality of the material, poor bonding between leca and rock as well as for the presence of a small insulation layer next to the concrete wall.

$$f_{i,red} = f_i \left(0.3 + 0.7 \cdot \frac{\rho}{2400}\right) \quad (3.1)$$
3. Method

The strength parameters of leca were estimated according to BBK 94 where a reduction factor is prescribed to reduce the strength of light-weight concrete as in Eq. (3.1). Strength values were taken from 3.1 and the average leca density was used. In Table 3.3 the final material properties used for the layer of leca are listed.

<table>
<thead>
<tr>
<th>Material properties of leca</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_{lk}$</td>
</tr>
<tr>
<td>$f_{luk}$</td>
</tr>
<tr>
<td>$E_l$</td>
</tr>
<tr>
<td>$\rho_l$</td>
</tr>
</tbody>
</table>

3.1.3 Steel

The strengthened utility tunnel entails the use of three types of steel: regular steel for rebars, steel for rock nails and high strength steel for prestressed cables. The structural purpose of rock nails is to make the rock mass behave as a whole by tying together blocks that were possibly separated before. When modelling, the rock was considered as a unique continuum block and no rock nails were actually modelled. For this reason the material properties of this type of steel are here not presented.

Steel reinforcements were prescribed to be KS500ST with diameter $\Phi 10$ and $\Phi 12$. Concrete cover was designed to be 35 mm towards the formwork and 60 mm towards the rock.

High strength steel was used for the prestressed cables. The ultimate stress is prescribed to be $f_{su} = 1770$ MPa and the ultimate load capacity $F_u = 795$ kN.

3.1.4 Rock

Rocks are often characterised by internal stresses which were accumulated during the geological history of their rock formation. At shallow depths in particular, self-stresses can be several orders of magnitude greater than those induced by gravity which underlines the need for their identification and quantification before moving to any further.

Within the scope of a different project carried out by Tyren's AB, rock stress measurements were performed 1km away from the tunnel sections treated in this work. The stresses were estimated through the LVDT method for hard rock developed by the Finnish company Stress Measurement Company OY, who also conducted the measurements in situ. The method is entirely based on the principle that if a hole is drilled in a solid which is subject to a given stress state, the walls of the opening will deform to allow for the release of all self-stresses locally. The features the use of both experimental testing and numerical modelling and can be summarised in the following steps:

1. Select a section of tunnel where rock is intact and map it with 3D photogrammetry.
2. Drill a pilot hole in five points of the selected tunnel cross-section, normally two from the walls and three on the ceiling as in Figure 3.2. This disposition allows to identify the entirety of the 3D stress state. The pilot holes are 127 mm in diameter to accomodate the measuring probe and need to have at least 350 mm of free length passed the damage zone, if the tunnel was excavated with blasting.
3.1. Material properties

3. Install the LVDT probes in the free length of the pilot holes and overcore with a diameter $\Phi \geq 200 \text{ mm}$ by drilling concentrically to the pilot hole (see Figure 3.3). Overcoring is performed until passed the stress cell. During this procedure, the probe measures the deformation of the surrounding material caused by the release of rock stresses. The measurements are continued for about 2 hours or until the rock has ceased to expand. During the entirety of the process temperature is carefully monitored to account for the heat generated by drilling.

4. Perform uniaxial tests on the cores drilled to excavate the pilot holes and determine the elastic material parameters during the unloading of the specimens. Uniaxial tests are performed having four strain gauges: two in the axial and two in the tangential direction.

5. Solve the inverse problem: model the tunnel segment that was laser-scanned, use the elastic material properties derived at the previous point and for each overcoring, find the values of the stress tensor which best fit the deformations recorded by the LDVT cells.

![Figure 3.2: Location of overcorings (yellow) and installation holes (red) [3]](image)

During the last step of the procedure, an estimate of the stress tensor is obtained for each overcoring performed but none, in practice, matches exactly the estimated rock deformation. The final choice is made through engineering judgement selecting the stress estimate which best matches the most reliable measurement of the rock expansion. In addition procedures such as inspecting the pilot hole for cracks in the rock and ensuring that the LDVT cells are not disturbed by anything other than overcoring, the method features also the estimation of reliability indexes for the measures and statistical analyses of the numerical model.

The average values for the elastic parameters of intact rock are presented in Table 3.4; while the elastic constants were estimated form uniaxial testing, the density was directly measured before testing.
The material parameters of intact rock are very rarely the same as those of the rock mass they are taken from and as such cannot be prescribed to a rock domain in the range of a few hundred meters. Differently from an intact rock specimen, rock masses are heterogeneous and often discontinuous because fractures, cracks and faults; all this makes the integral characteristics of the mass very different from the local ones. The E-modulus of the rock mass was estimated from Equation (3.2) as proposed by Hoek and Diederichs [9]. In it, $E_{rm}$ is the elastic modulus of the rock mass, $E_i$ is the elastic modulus of intact rock, $GSI$ the Geological Strength Index and $D$ the disturbance factor caused by the excavation.

$$E_{rm} = E_i \left( 0.02 + \frac{1 - D/2}{1 + e^{((60+15D-GSI)/11)}} \right)$$ \hspace{1cm} (3.2)

The values of GSI was estimated at $GSI = 80$ which corresponds to "intact or massive rock with few widely spaced discontinuities" and "good surface conditions". The disturbance factor was instead set to $D = 0.8$ which according to Hoek [4] corresponds to "very poor quality blasting in hard rock tunnel"; this choice was justified by a visual inspection of the rock surface and by the historical nature of the utility tunnel. The elastic modulus $E_{rm}$ was finally rounded off to its lower value $E_r$. The elastic material properties employed during the analyses are reported in Table 3.5.

<table>
<thead>
<tr>
<th>Material properties of intact rock</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_{ri}$</td>
</tr>
<tr>
<td>$\nu_{ri}$</td>
</tr>
<tr>
<td>$\rho_{r}$</td>
</tr>
</tbody>
</table>

Figure 3.3: Simplified illustration of the overcoring and pilot hole [8]
Table 3.5: Material properties of the rock mass

| \( E_r \) | 30.0 | GPa |
| \( \nu_r \) | 0.25 |     |
| \( \rho_r \) | 2700.0 | kg/m³ |

The value of \( E_{rm} \) was reduced to increase the safety margins of the analysis. As is discussed in Section 3.3, the main load stressing the concrete lining is in fact caused by the excavation of the road tunnels; reducing the rock stiffness is effectively increasing its compliance and therefore adding pressure to the supporting structures of the utility tunnel.

Table 3.6 displays the \textit{in situ} stresses of the rock, average values have been used. From the stress analysis it was found that the highest principal stresses in compression are laying almost exactly on the horizontal plane. As can be easily noticed, the highest horizontal stress \( \sigma_H \) is one entire order of magnitude greater than the vertical stress \( \sigma_v \), which demonstrates how relevant tensions of geological origin can be. Additionally, the vertical stress compares very well with the formula \( \sigma_v = \rho gh \) which proves it to be mostly caused by the overburden.

The principal stress \( \sigma_H \) was found to be oriented at an angle of about 277° with respect to the tunnel axis of UT-1 and 316° with respect to the axis of UT-2 (the rotation is considered as positive when around the \( z \)-axis of the models presented in Section 3.3). Principal stresses were then translated into \( (\sigma_x, \sigma_y, \sigma_z) \) stresses by means of Mohr’s circles.

Table 3.6: \textit{In situ} principal rock stresses

| \( \sigma_H \) | -14.6 | MPa |
| \( \sigma_h \) | -7.2 | MPa |
| \( \sigma_v \) | -0.9 | MPa |

Since measurements were performed about 1 km away from the tunnel segments in question, it was reasonable to assume that these would not be affected by the excavation of the Södra Länken. These stresses were therefore considered to be a good estimate for the \textit{in situ} stress state of the rock prior to the excavation of the road tunnel.
3. Method

3.2 Preliminary models

Preliminary models were created and analysed to gain knowledge on the capabilities of the software and to identify the most suitable way to model specific details. Results derived from these preliminary models were instrumental in building a suitable 3D model of the utility tunnel in rock.

3.2.1 2D boundary influence

While in structural engineering problems boundaries are generally identified with ease, the fields of geotechnical engineering and rock engineering are often affected by issues connected to boundary effects.

To model what is in reality an infinitely extended domain such as that of a soil, boundaries must be introduced. How far these boundaries are located however depends on the problem at hand and on the size of the area where we are interested in reading the solution. In finite elements, geometric boundaries are effectively an additional restraint because of the conditions they necessarily bear: when an infinite domain is reduced to a discrete one the original solution is to some degree perturbed. Although the solution might ultimately still differ from the original, the aim is to have the boundaries be located sufficiently far away so that their disturbance to the solution in a given subdomain of interest is minimal, or sufficiently low for the purpose of the study.

Figure 3.4: Tunnel opening in preliminary 2D model to test boundary effect

This preliminary model focused on studying the disturbance caused by boundaries to the case of a tunnel excavated in rock. Using Comsol Multiphysics, a rectangular domain with a tunnel opening of dimensions similar to those described in Section 1.3 was modelled (see Figure 3.4). Rock with mechanical properties as in Section 3.1 was used as material and gravity was the sole load introduced. The aim of this analysis was to study the vertical displacement field as a function of the width of the rectangular domain which directly translates to the distance of the boundaries from the area of interest. The rock domain varied from a size of 600x100 m to 40x100 m through eight intermediate steps always having the tunnel opening in the axes origin. Boundary influences were only studied for the horizontal x-direction seen as the rock through which the utility tunnel runs had the first two principal stresses laying on the horizontal plane\(^1\).

Simply supported boundary conditions were applied to all boundaries other than the top one to prevent displacements in the normal direction. The vertical displacement was measured along two

\(^1\) As mentioned in Section 3.1.4 initial stresses in the xy-plane are three orders of magnitude greater than those in the vertical z-direction
3.2. Preliminary models

Figure 3.5: Preliminary 2D model to test boundary effect, width is 600 m

lines, one just above and one just below the tunnel (see Figure 3.5) for the different values of the domain width. The analysis was conducted first having no initial stresses or strains in the rock, and then having an initial stress equal to $\sigma_x = -14.6$ MPa in the horizontal direction; this was done to study the extreme case of the tunnel running orthogonally to the principal stress direction of the rock described in Section 3.1.4.

3.2.2 Modelling of anchor connections

This section presents the preliminary study conducted with the aim of identifying an effective way to model the cable connection. As can be seen from Figure 1.21, the anchoring system employed for the prestressed steel cables is based on steel elements having thickness up to 15 mm. If for the reasons discussed in Section 4.1 the final 3D model is expected to have a domain in the range of $10^2$ m, the small size of the anchor elements combined with the complexity of the system make modelling the actual anchorage a very ineffective strategy. A simple 2D model was therefore built to compare different modelling strategies.

The study model was built starting from a squared 50x50 m rock domain where the concrete lining from UT-1B was introduced\(^2\) together with two prestressed steel cables 8 m long. Simply supported boundary conditions were prescribed to all boundaries of the rock domain. Loads were chosen to be gravity for the rock and a prestressing of 1200 MPa for the cables. Starting from this basic model, four cases were studied:

1. **Detailed model**: the anchoring system is modelled in depth with anchor lodging, anchor plate, steel cylinder, borehole and anchoring length on the rock side. Steel cables are connected through point connection to the steel plate and with embedding constraint\(^3\) along the anchorage length.

2. **Rigid model with borehole**: the anchorage is simplified by having a continuous concrete lining (no cut-out to accommodate the anchor and no opening for the steel cylinder). A rigid connector of flexible type\(^4\) is used to simulate the connection between cable and steel plate. On the side of the rock the cable is embedded along the anchorage length.

3. **Rigid model without borehole**: identical to the previous but no borehole is opened.

4. **Model with point connections**: the cable is anchored to both rock and concrete via a point connection. No borehole is present and the concrete lining is continuous.

As can be seen in Figure 3.20b, the rigid connector for models 2 and 3 was prescribed to the line defining the outer surface of what would be the steel plate in the detailed model. To simplify the modelling procedure, one sole model containing all the needed partitions was initially constructed;

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\(^2\)For simplicity the concrete lining was imported including the overbreak.

\(^3\)The displacement field of one element is assigned as prescribed displacement to the nodes of the other

\(^4\)See Appendix A for details.
starting from this initial template, partitions and domains were then removed to generate the models mentioned above. In Figure 3.20b all the geometrical lines to model the detailed connection are present but no subdomain is removed so that the concrete lining is continuous and concrete material is assigned to all its internal subdomains.

Figure 3.6: Tunnel in rigid model without borehole

Figure 3.7: Detail of cable anchorage with rigid connector highlighted
3.2. Preliminary models

The most effective strategy to model the connection was identified by studying the displacement field of the steel cables; both ends of the cables were analysed to discern the mechanical behaviour of both rock and concrete anchorages. For the cable end attached to the rock the objective was to study whether the simpler point connection could be used instead of an embedding constraint. At the other end of the cable, the focus was on identifying whether a point connection could be used instead of a rigid connector or of the actual steel plate. The relevance of the borehole was also addressed by comparing results from models 2 and 3.
3. Method

3.3 Main models

Having gained relevant information from the preliminary studies conducted, the main models representing the two intersections to be studied were built. The present section highlights which steps were taken to finally answer the initially posed research questions. Compatibly with the problem description, two separate models were built to analyse intersection 1 and 2 respectively.

3.3.1 Assumptions

An array of assumptions were placed to ease the modelling of the physical reality. The main assumptions and simplifications regarding the geometry of the problem were the following:

1. All geometrical measures were taken from the original drawings through hand measures. In the case of drawing inconsistencies (see Section C), engineering judgement was applied.

2. From the planar geometry (see Figure 1.3) it can be noticed that all three road tunnels considered encounter the utility tunnel during a turn: these tunnels are in fact entry and exit ramps which allow users to access the main highway tunnel. In accordance with international guidelines for road engineering, these segments should be described by clothoids.
   
   To simplify the model, all road tunnels were modelled as straight. The angle of intersection was estimated as average relative angle between the two tunnels across the overlapping sections.

3. Compatibly with the in situ conditions, the utility tunnel was reported in the drawings to be locally curved along its path. Most notably, both the concrete lining of intersection 1 and that of intersection 2 showed a small in-plane curvature.
   
   The modelling was simplified by assuming the utility tunnel and its supporting structures to run perfectly straight in plan view.

4. In the elevation view tables the slope of the utility tunnel along the two considered branches is not constant and this fact was confirmed in the first in situ investigation.
   
   When modelling UT-1 and UT-2 a constant slope was employed. The value of these slopes was estimated as average inclination of the concrete linings in the drawings.

5. All cross-sections, be it from the road or utility tunnel were assumed to be constant. It was supposed that the roughness of the rock surface would only have very localised impacts on the behaviour. Similarly, there was no account for local variations in the concrete cross-section seen as these were deemed irrelevant for the overall response.

Adding to those involving the geometry, the following simplifications and assumptions were introduced for the material model. Although all assumptions were applied to the initial models, the validity of assumption no.6 is discussed in Sections 3.6 and 3.7.

6. All materials in the model were assumed to behave according to linear elasticity. This assumption was well justified only for the prestressed steel cables and partly for the rock.

7. The rock mass incorporating the tunnel intersections was assumed to also incorporate the locations were the material properties of the rock were tested. As a result the initial rock stresses experimentally estimated were introduced in the model supposing no changes in orientation or amplitude.
8. Relying on the good quality of the rock and on the absence of major faults being reported, no anisotropic model was used.

Along the modelling process, further simplifications were introduced in the spirit of reducing the complexity and improving the computational time of the model. The most relevant approximations are the following:

9. For both utility and road tunnels, the curved surfaces forming the ceiling were interpolated by means of a parabolic function. The parabolas were further approximated in Comsol by means of eight straight segments (see Figure 3.8).

10. For those concrete lining segments being supported by steel cables (i.e. UT-1B and UT-2), the surface of the walls was left intact without removing volume where cable anchorages would be allocated.

11. The modelling of the prestressed steel cables was performed according to the simplified model Rigid without borehole discussed in Section 3.2.2.

### 3.3.2 Geometry

The main models were built making use of the full potential offered by Comsol for parametrisation. One basic blueprint containing all the needed steps was implemented for a general set of parameters and then manipulated to generate the model of intersection 1 and that of intersection 2. To ease the final procedure of assembly and therefore reduce the number of geometrical operations in the model tree, parts were created for each of the main elements: rock domain, rock surface of the utility tunnel, concrete and leca linings, road tunnels. While cross-sectional parameters depended on the intersection and element considered, the extrusions were all controlled by the parameter defining the width of the rock domain.

For all tunnels, the ceiling was implemented by means of a step-wise function approximating a parabolic shape. According to the procurements set by Trafikverket [10], the ceiling of tunnels is to follow the shape of an ellipsis, but this can for our purposes be well approximated by a parabola.

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5The introduction of a curvilinear ceiling in the geometry would have been meaningful only for a quadratic mesh.
3. Method

Furthermore, no openings were implemented where the cables would be anchored leaving the surface of all walls intact; following the conclusions reached from the preliminary model described in Section 3.2.2, this was identified as a valid way to reduce the complexity of the model.

When extruding the cross-section of concrete linings UT-1A and 1B, an overlap of about 5 cm was added to the first. It is good practice when modelling in Comsol to add overlaps when separate elements are to have their faces coincide with one another and form a continuous solid. This way, after the two elements have been moved to their predefined location by means of translations and rotations, their overlap can be removed by means of a boolean operation⁶.

The single parts were finally imported in the geometry of the model starting with the rock domain. The global reference system was placed at the lower corner of the rock domain with the x-axis in the direction of the utility tunnel, the y-axis normal to it and the z-axis parallel to the vertical. Since for both intersections the study was limited to a small segment of tunnel, it was decided to avoid following the geographical system aligned to the north pole according to which all tables had been drawn. The unreinforced utility tunnel was moved to have its symmetry plane match the xz-plane of symmetry of the rock domain. Its slope was then archived by performing a rotation around the y-direction symmetry axis of the rock domain. The concrete linings were consequently imported: for intersection 1, the common section between UT-1A and 1B was positioned at the abscissa $x = L/2$ where L is the rock domain size in the x-direction; for intersection 2, one end of the concrete lining was moved to $x = L/2$.

Having the utility tunnel in place, the road tunnels were moved into position by considering only their relative distances and angles with respect to the utility tunnel. In Figure 3.9 and example of geometry at this stage can be seen.

Figure 3.9: Geometry of intersection 1 during the assembly

⁶In Comsol, union, intersection and difference are called boolean operations
To perform a multi-step analysis where specific geometrical domains are removed in given steps (e.g. analysis of an excavation), it was decided to have one sole object (excluding the prestressed cables) partitioned into smaller domains. In Comsol Multiphysics the conclusive steps of a geometrical assembly can either be *form union* or *form assembly*. With the first option, the software unites all objects into one and automatically creates domain partitions where overlaps are identified; this option could not be adopted seen as it would have treated the lines representing the steel cables as partition lines. Form assembly on the other hand is the ideal choice for contact problems: it does not carry out automatic partitions, does not combine objects and does not enforce any correlation between objects having coincident faces (no continuity is automatically established). All imported objects were exploited to form intersections, differences and partitions of the rock domain through the boolean operators offered by Comsol. By ultimately choosing to form an assembly, the prestressed cables were preserved and one sole partitioned object resulted from the geometry. During the boolean operations, special attention was paid to the intersection between tunnels: it should in fact be noticed that the layer of leca was partitioned by the road tunnel to allow for its removal during the analysis step simulating the excavation.

![Geometry of intersection 1 after partitioning operations](image)

*Figure 3.10: Geometry of intersection 1 after partitioning operations*

The final step needed to ultimate the geometry was the introduction of all prestressed steel cables and their corresponding anchors. Following the conclusions derived and discussed in Section 3.2.2, it was decided to model the anchorages by means of a rigid connector with flexible formulation and to have it be prescribed to a surface of 150x150 mm representing the anchor plate. Additionally, the modelling of boreholes was avoided as they would have inevitably increased the overall complexity by a significant amount (with a diameter equal to 1/3 of the concrete thickness and a circular cross-section, boreholes would have required a high level of local mesh refinement).
3. Method

As reported in Table B.1, a number of 30 cables are installed in UT-1B and 24 in UT-2. After the installation, the length, orientation and prestressing load of each cable were recorded. To introduce in the geometry all 54 cables and plates an external subroutine was written. Comsol Multiphysics offers a Livelink with Matlab functionality which allows for a Comsol model to be built from scratch and analysed using only the Matlab interface. The following steps were performed to finalise the geometry of the model:

1. Build the basic geometry consisting of rock domain, utility tunnel and rock tunnels using the Comsol interface.

2. Clean the Comsol file and export it as Matlab file.

3. Paste in the Matlab script a subroutine creating a geometrical part with all the prestressed cables as line objects. The geometrical information are taken from a specifically created Excel file.

4. Paste in the Matlab script a subroutine creating a geometrical part hosting all the anchor plates as surfaces. Also here an external file is used to import the geometrical data.

5. Run the model in Matlab and save it as a Comsol file.

6. Open the newly created Comsol file, unite all plates and cables and import them into the geometry. Use the imported plates as tool objects to partition the concrete lining.

7. Use the cables to partition the rock domain and check the box to avoid deleting them after the operation. This partition is needed to avoid incompatibilities in the mesh when cables are rock anchored.

8. Form assembly to ultimate the geometry.

Although as mentioned before Comsol allows for a model to be entirely built using Matlab scripting, doing so proves to be rather cumbersome because of the complex syntax required. In the scope of this work, the most effective way of exploiting the capabilities of Livelink for Matlab was to perform specific changes and operations in the model which would have otherwise required much longer times had the standard interface been used. An example of this are points 4 and 6 in the previous list: although scripting to create geometrical objects is rather straight forward, forming a union of them requires that all object ID numbers be tracked through the geometrical procedure until the union can be established. In conclusion, simplest solution was identified in doing certain operations through scripting and others through the standard interface.

Since cables and anchors were created through an external subroutine, it was decided to employ the geometrical data recorded after their installation. Although the impact of using different inclinations and lengths for the cables compared to using one average inclination and one average length can be debated, the overall implementation time of either strategy was identical and as such the most accurate was chosen.

Figures 3.11 and 3.12 present a detailed look at the steel cables. The anchoring plates are represented by a surface partition of the concrete lining and have the cables reach their centre of gravity. It is furthermore readily noticeable that a certain deviation is present between cables for both inclination and length; these differences are however minor so that the use of one inclination and one length for all steel elements would be well justified.
Finalised geometry of intersection 1

The following pictures present a relevant views of the tunnels involved in intersection 1. All views are taken directly from the model, only the rock surrounding the domain has been hidden from the viewport. All figures are taken from a small models with dimensions 200x100x90 m. It should be noticed that the relative distances between tunnels in the area where the intersection takes place are independent from the size of the domain. Making use of the parametrisation capabilities offered by Comsol, the domain size only affects how much each tunnel prolongs along its axis.
Figure 3.13: Final geometry of intersection 1 with rock domain hidden

Figure 3.14: Plane view of the final geometry of intersection 1 with rock domain hidden
3.3. Main models

Figure 3.15: Elevation view of the final geometry of intersection 1 with rock domain hidden

Finalised geometry of intersection 2

The following pictures display the plane and elevation views of the model for intersection 2. The concrete lining is positioned to have one of its ends in the middle of model which, as clearly visible in Figure 3.17, does not create a balanced model. It should be noticed that although this is very true for a small size model as the one presented, as the model grows in size this effect is reduced to the point where the two ends of the concrete lining are effectively located at a similar distance from the boundaries.

Figure 3.16: Final geometry of intersection 2 with rock domain hidden
3. Method

Figure 3.17: Plane view of the final geometry of intersection 2 with rock domain hidden

Figure 3.18: Elevation view of the final geometry of intersection 2 with rock domain hidden
3.3.3 Analysis steps

As discussed in Section 1.3, rock stresses were available only for the \textit{in situ} conditions prior to the excavation of the tunnels. Accounting for this information while still managing to study the present day state of stress meant simulating the historical succession of events. Rock stresses were prescribed in the first analysis step and then carried through all subsequent steps until the last. The following steps were implemented in Comsol:

1. **Initial conditions**: Rock domain intact with initial rock stresses prescribed.

2. **Excavate UT**: the utility tunnel is excavated from the rock domain. Rock stresses are taken over from the previous step.

3. **Reinforce UT**: a concrete lining is added to the service tunnel. Rock stresses are taken from the previous step.

4. **Install cables**: prestressed steel cables are installed and tensioned to support the concrete lining. Rock stresses are taken from the previous step.

5. **Excavate RT**: All road tunnels are excavated from the rock domain. Rock stresses, stresses in concrete and cable forces are carried over from the previous step.

A schematic view of the analysis steps is visible in Figure 3.19. Here the physics linked to each step are presented together with the variables coupling different physics together.

In Comsol multiphysics, when several analysis steps linked together are present in the same study, the output of the analysis is in incremental terms. Even though in an elastic analysis the software solves each step independently by starting from the undeformed configuration, the output is presented automatically as difference with respect to the previous steps.

\[ (u_4,v_4,w_4) = (U_4, V_4, W_4) - (U_3, V_3, W_3) \]  \hspace{1cm} (3.3)

To solve step 4 in Figure 3.19 for example, Comsol starts from the undeformed model, applies as initial stresses those produced by step 3, computes the resulting displacement field \((U_4, V_4, W_4)\) as if the step were an independent study and finally outputs the incremental displacement field \((u_4, v_4, w_4)\) as presented by Equation (3.3).

**Material sweeps**

Since all geometrical domains (excluding the cables) were constituted partitioning the initial rock domain\(^7\), different materials had to be assigned in different steps to the same domain. The partition representing the concrete lining, for example, had the properties of rock in the first step, was deactivated in the second step and was finally assigned the properties of concrete in all other steps. The need to vary material properties between steps was satisfied by using a material switch option in combination with a material sweep. Comsol offers functionalities to perform parametrical studies for a general parameter, defining a load amplitude or a geometrical quantity for instance, as well as material sweeps where the solution is computed for a given array of materials. Material sweeps were exploited somewhat improperly, outside of their intended purpose, as a way to effectively

\(^7\)See Section 3.3.2 for more information.
3. Method

Initial conditions

Excavate UT

Reinforce UT

Install cables

Excavate RT

![Diagram of analysis steps and related physics and interacting variables](image)

Figure 3.19: Schematic view of the analysis steps, related physics and interacting variables

change material: having created two material switches, one between rock and concrete and the other between rock and leca, a material sweep was added to the analysis steps. Rather than performing sweeps over all materials available in the material switches, a user defined combination of materials was prescribed and in this just one material was specified. Exploiting this *escamotage*, the material sweep only ran rock properties in the first step and only used concrete/leca properties in the other steps.

To simplify even further the implementation of the analysis, two separate studies were introduced: in Study 1 the first and second steps were solved, rock was used as only material; in Study 2, steps from 2 to 5 were solved and only concrete/leca were prescribed to the material sweep. A view of the model tree for the study and materials module is visible in Figure 3.20. This separation was somewhat needed to avoid issues with the material combinations: had rock and concrete been added together to the material sweep in one sole study, the software would have run all steps for both types of material; furthermore, specific measures would also have been needed to ensure that a step running with a certain material would take as input the results from the previous step using the correct material and not another.
3.3.4 Comsol physics

Physics in Comsol

In Comsol, physics define the realm in which a solution is to be sought for a given analysis. In the Structural Mechanics module, some of the offered physics are solid mechanics, shells, beams and trusses. Once a physics has been added and a geometrical domain assigned to it, the software creates an empty solution matrix uniquely identified by an ID number: in practical terms, this means that if both a beam physics and a truss physics are added to one same model, the displacement field of all beams will be \((u_1, v_1, w_1)\) while that of all trusses will be \((u_2, v_2, w_2)\). Physics are themselves independent from one another: to model for example a solid element undergoing temperature variations, a solid physics and a heat transfer physics have to be added to the model; if no coupling is provided, the two physics will behave independently and no stresses will be caused by the thermal gradient. Coupling is provided in Comsol by the Multiphysics option: for the previous example, the multiphysics is going to link the heat transfer analysis to the thermal expansion of the solid and thus to the displacement field of the solid. Although this approach offers great flexibility in solving problems where independent equations are to interact with one another (e.g. heat conduction law, thermal expansion law and continuum mechanics), it can increase the complexity of simple structural mechanics problems. This is exemplified by the procedure needed to create a connection between a beam element and a truss element: two lines are drawn in the geometry, one is assigned to the beam physics, the other to the truss physics; at this stage, even though the two lines coincide geometrically in one point, the analysis cannot run since no connection is established between the two. The connection is created by manually assigning the displacement field of the beam element as prescribed displacement to the truss element. In Equation (3.4), the displacement field of the beam physics is \(U_1(x, y, z)\) while \(U_2(x, y, z)\) represents the displacement field of the truss physics.

\[
U_2(\bar{x}, \bar{y}, \bar{z}) = U_1(\bar{x}, \bar{y}, \bar{z})
\]  

(3.4)
Within the scope of this work two physics were used, namely solid mechanics and truss. While solid mechanics incorporated the rock, concrete and leca domains, the truss physic was needed to model the prestressed steel cables.

**Solid mechanics in the model**

In the options of the solid mechanics physics, the rock domain was restrained by simply supported boundary conditions fixing displacements in the normal direction of all exterior boundaries. Apart from the first analysis step, where the measured rock stresses were prescribed, the output stresses of one step were assigned as initial stresses to the following step. To match the steps of analysis described in Section 3.3.3, five solid mechanics physics were added. The process of rock excavation was simply modelled by deactivating specific domains for each physic; this method allowed to have one sole geometry as opposed to having different geometries for each step. While all domains are active in the physic related to step 1, only the rock domain and road tunnel domains are active in the physic of step 2 (service tunnel, concrete lining and leca layer domains were deactivated and not are considered by this analysis step).

The steel cables anchorage on the side of concrete was realised by means of rigid connectors with flexible formulation. A rigid connector was prescribed to each of the surface partitions visible in Figure 3.12, the centre of rotation was made coincide with the centroid of the plate (where the cable is anchored) and the rotation left free. It is relevant to notice that rigid connectors in Comsol have a double function: they make entities rigid but also generate a uniquely identified displacement field which can be prescribed in a different physic to generate a connection.

To avoid using the standard interface, the model was exported as Matlab script and an external subroutine written. The coded algorithm runs through every plate, prescribes the rigid connector, sets the centre of rotation and creates the cable connection to the plate in the truss physics. The subroutine, which is reported in Annex D, creates rigid connectors and cable connections for both solid mechanics and truss physics related to steps 4 and 5.

**Truss physic in the model**

Since the prestressed cables are present only in steps 4 and 5 (install cables and excavate road tunnel), only two truss physics were added. For step 4, an initial stress equivalent to the prestressing given to the cables was assigned; the output stresses were then assigned as initial stresses to step 5.

For both physics, the rock anchorage was enforced modelling the anchoring length and prescribing it to follow the same displacement of the rock. In Comsol this procedure was carried out by generating a general extrusion from the rock domain, and assigning the extruded displacements to the anchored truss. In Equation (3.5), the expression genext1(_) is the label of the general extrusion from the rock field and \((u, v, w)_4\) are the displacement field of the solid mechanics physic from step 4.

\[
\begin{align*}
  u_x &= \text{genext1}(u_4) \\
  u_y &= \text{genext1}(v_4) \\
  u_z &= \text{genext1}(w_4)
\end{align*}
\]

The cable anchors on the side of concrete were finally created by the subroutine mentioned in the previous subsection: displacements were singularly prescribed to the ends of all cables as reported.
in Equation (3.6). Here, solid4.rig21 indicates to use the displacement field taken from the rigid connector number 21, which is located in the solid mechanics physic related to step 4.

\[
\begin{align*}
    u_x &= \text{solid4.rig21.u} \\
    u_y &= \text{solid4.rig21.v} \\
    u_z &= \text{solid4.rig21.w}
\end{align*}
\]  

(3.6)

When the connections were generated in physic truss2, which is related to the step excavate road tunnel, the displacements were taken from solid5, the solid mechanics physic linked to the same step.

### 3.3.5 Loads

Other than the prestressing of the cables, all analyses were performed considering gravity as the only externally prescribed load. Compatibly with the research questions posed, the main aim was to assess the effects that the excavation of the road tunnels had on the structural integrity of the supporting structures. Within this scope, the major action stressing the concrete reinforcement was related to expansion of the expansion of the rock surrounding it. Although gravity was needed to recreate the experimentally measured *in situ* $\sigma_v$ rock stress, the main excitation came from the principal stresses in the horizontal plane.

Being the tunnels located at a depth of about 35 m from the surface, temperatures remain constant all year round with only minor variations. The same can also be said about the road tunnels which for the considered segments are rather far from their ends. Other load possibilities were identified in live loads on the utility tunnel and temperature loads caused by a fire in the road tunnel. The first was not investigated seen as the considered sections are only subject to periodic inspections but are not involved in any underground operations. The second option, although relevant, was not investigated because of its low relevance with respect to the research questions.

### 3.3.6 Mesh

Meshing was performed in a series of steps aimed at maintaining a good level of user control as well as ensuring compatibility of the mesh. In Comsol Multiphysics meshing is mostly controlled by the software with only limited parameters offered to the user. Common practice in Tunnel Engineering problems is to use solid elements swept meshes which account well for symmetries and reduce the number of degrees of freedom. This approach was deemed unsuitable for the characteristics of this problem for the following reasons:

- There are no symmetries that can be exploited.
- Sweeping from the external surface of utility tunnel is not possible because the rock domain is made hollow by the road tunnels.
- Sweeping across the the thickness of the concrete lining is not possible because of the cable anchors.

It was therefore decided to employ a free tetrahedral mesh for both rock and concrete lining. The use of shell elements for the concrete lining was also investigated; this route was ultimately disregarded partly because of the last reason listed above, and partly because of the very complex state of stress.
found in the concrete lining.

Linear tetrahedral elements were employed for the solid domains while linear truss elements were used to mesh the cables. The meshing sequence was conducted by prescribing 1 truss element for all free lengths, 40 truss elements along the anchoring lengths and 41 equally spaced nodes along the partition in the rock domain created by the anchoring lengths to ensure compatibility. The anchoring plates were meshed by surface elements with a maximum size of 7 cm. Meshing of the concrete lining was performed by assigning to all its surfaces triangular elements with maximum size 20 cm. The rest of the domain was finally meshed automatically by Comsol under the fine element size option for Intersection 1, and extra fine option for Intersection 2.

As a result of the previous prescriptions, the concrete lining was meshed with 2 to 3 elements across the thickness. In general terms, the recommended number of tetrahedral elements across the thickness of a concrete element is 4 or 5 to fully capture the behaviour also in terms of stresses. Employing such a refined mesh was however not possible because of practical constraints: all models took an average of 7 hours to run on PDC, the super-computer located at KTH to which access was kindly granted; the usage of a higher quality mesh would have resulted in unacceptably long computational times. Regardless, the mesh quality was verified performing a convergence study as reported in Section 3.3.7. In all cases, a mesh refinement using Comsol’s automated procedure was carried out to improve the quality of elements by normalising their aspect ratio.

3.3.7 Convergence study

A convergence study was performed with the aim of comparing different element types and mesh sizes. Due to the complexity of the model, the mesh was only modified in general terms by changing the element size parameters offered by the Comsol mesher. Convergence was checked using a 200x200x90 m model of Intersection 2 as benchmark; this intersection was chosen because of its computational lightness. Overall five meshes were compared; in Table 3.7 their properties are listed. More refined meshes were not treated because of computational time restrictions.

<table>
<thead>
<tr>
<th>Mesh ID</th>
<th>Interpolation type</th>
<th>El. size in concrete</th>
<th>Overall el. size</th>
<th>No. of elements</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Linear</td>
<td>0.2 m</td>
<td>extra fine</td>
<td>3.41 · 10^6</td>
</tr>
<tr>
<td>2</td>
<td>Linear</td>
<td>0.2 m</td>
<td>finer</td>
<td>2.86 · 10^6</td>
</tr>
<tr>
<td>3</td>
<td>Linear</td>
<td>0.3 m</td>
<td>extra fine</td>
<td>1.92 · 10^6</td>
</tr>
<tr>
<td>4</td>
<td>Quadratic</td>
<td>0.3 m</td>
<td>finer</td>
<td>1.53 · 10^6</td>
</tr>
<tr>
<td>5</td>
<td>Linear</td>
<td>0.3 m</td>
<td>normal</td>
<td>0.94 · 10^6</td>
</tr>
</tbody>
</table>

The displacement field was selected as reference parameter to check for convergence. Among the five steps of analysis, focus was placed on step 5: in addition to being the most relevant in the study, this step is highly sensitive to poor meshing because of the interaction between different volumes and elements in the model. The displacement component w5 was extracted from the lines defining the edge of the utility tunnel. Even though displacements were considered to be the main target, principal stresses were also investigated in the concrete lining. Given that the convergence order
as function of the element size is lower for stresses than for displacements, the interest was on verifying only that the overall behaviour be correctly captured by different meshes.

Mesh type and distance from the boundaries are similar parameters in the sense that they can potentially lead to three outcomes:

1. Capture the behaviour and the amplitude.
2. Capture the behaviour but not the amplitude.
3. Inability to capture both behaviour and amplitude.

While the first option is the ideal one for any type of analysis, the second can also be accepted in certain conditions: for problems involving high levels of uncertainties for example, results may in practice be rounded off to only one significant digit; this implies that aiming for low levels of deviation from the "correct" solution is meaningless as long as the "correct" behaviour is captured by the model. What is to be entirely avoided is instead to have a model that does not provide precise estimates of the solution and has insufficient accuracy. For models of such geometrical complexity as the ones here treated, poor meshing can result in either option 2 or option 3. The usual consequence from an increase in mesh element size is an increase in stiffness; while it is true that for a simple model this might only cause to underestimate the displacements, varying the stiffness of different elements that are to interact with each other could potentially produce a response that is different in principle from the "correct" one.

![Example of two studies meshes](image)

**Figure 3.21: Example of two studies meshes**

The previous reasoning is clearly exemplified by the plot of Figure 3.23 where the displacement \( w_2 \) from the utility tunnel excavation step is plotted along the tunnel axis. In this case, a very fine mesh was assigned to a segment of the tunnel while a coarse mesh was used for the rest. This difference in element size is clearly demonstrated by the results: the finely meshed part of tunnel proves to be much more compliant than the average trend. In step 2 the utility tunnel is excavated: \( w_2 \) describes the elastic expansion of the rock caused by the removal of material. A finer mesh produces more compliant continuum which leads to more expansion in the rock. Had an accurate check of the mesh properties not been conducted, the output of Figure 3.23 might have led to drastically wrong conclusions of the about the structural behaviour of the tunnel sections considered.
3. Method

Figure 3.22: Detail of the connection between UT-2 and RT-2 (extra fine mesh, 0.2)

Figure 3.23: Vertical displacement along the utility tunnel in step 2 resulting from bad meshing
3.4 Boundary effects - model choice

As briefly discussed for the simplified 2D model of Section 3.2, the distance between areas of interest in a model and boundaries plays a big role in the results produced. Having identified a suitable meshing procedure able to capture the structural behaviour of the supported tunnel segments, focus was moved on selecting a fitting dimension for the rock domain. As for the quality of the mesh, the distance between boundaries have three major outcomes for the solution; the size of the domain was researched with the objective of capturing the modes of deformation of the tunnels and having a good estimate of their amplitudes.

Exploiting the previously parametrised geometry, the analysis steps were solved for several rock domains of different sizes and their outputs compared. Distance from the boundaries was considered in the horizontal as well as vertical directions to minimise boundary effects. Given that both first and second principal rock stresses (in compression) lay on the horizontal plane, priority was given to the horizontal size of the domain. In Tables 3.8 and 3.9 the geometrical properties of the rock domain used in the models are presented. While the same element size was assigned to the concrete lining, different element growths had to be prescribed to avoid an exponential increase in the number of elements.

For all models the displacement field along the utility tunnel was studied; all analysis steps were considered separately. Results from these comparisons can be viewed in Section 4.4. From this procedure, the main models representing Intersection 1 and 2 were designated.

### Table 3.8: Domain sizes analysed for intersection 1

<table>
<thead>
<tr>
<th>Model ID</th>
<th>x-dimension [m]</th>
<th>y-dimension [m]</th>
<th>z-dimension [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>200x100</td>
<td>200</td>
<td>100</td>
<td>90</td>
</tr>
<tr>
<td>200x200</td>
<td>200</td>
<td>200</td>
<td>90</td>
</tr>
<tr>
<td>250x250</td>
<td>250</td>
<td>250</td>
<td>90</td>
</tr>
<tr>
<td>300x300</td>
<td>300</td>
<td>300</td>
<td>90</td>
</tr>
<tr>
<td>200x200x140</td>
<td>200</td>
<td>200</td>
<td>140</td>
</tr>
<tr>
<td>250x250x200</td>
<td>250</td>
<td>250</td>
<td>200</td>
</tr>
</tbody>
</table>

Common practice in Geotechnical Engineering is to employ infinite boundaries. Infinite boundary elements follow a special formulation where the shape functions interpolating the displacement field are modified forcing certain nodes to be located at infinity. This practice allows to reduce the computational cost of most analyses by significantly lowering the size of the soil domain needed. This approach was taken into account but ultimately disregarded because of the inherent differences between a soil and a rock problem. Specifically, the use of infinite boundary elements was impeded by the presence of initial rock stresses as well as by the inclination of the tunnels. Through both 2D and 3D benchmark tests, it was observed that infinite boundary elements have difficulties coupling with initial prescribed stresses. Furthermore, solutions would be distorted because of the model’s geometry: from the principle point of view, infinite boundaries stretch the physical boundaries in the normal direction and this is incompatible with having openings in the geometry that reach the boundary at a relative angle.
3. Method

Table 3.9: Domain sizes analysed for intersection 2

<table>
<thead>
<tr>
<th>Model ID</th>
<th>x-dimension [m]</th>
<th>y-dimension [m]</th>
<th>z-dimension [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>100x100</td>
<td>100</td>
<td>100</td>
<td>90</td>
</tr>
<tr>
<td>200x200</td>
<td>200</td>
<td>200</td>
<td>90</td>
</tr>
<tr>
<td>250x250</td>
<td>250</td>
<td>250</td>
<td>90</td>
</tr>
<tr>
<td>300x300</td>
<td>300</td>
<td>300</td>
<td>90</td>
</tr>
<tr>
<td>200x200x140</td>
<td>200</td>
<td>200</td>
<td>140</td>
</tr>
<tr>
<td>200x200x200</td>
<td>200</td>
<td>200</td>
<td>200</td>
</tr>
<tr>
<td>250x250x140</td>
<td>250</td>
<td>250</td>
<td>140</td>
</tr>
</tbody>
</table>

All results from this study aimed at choosing a suitable model are presented in Section 4.4. Here, the vertical and horizontal displacements resulting from the single steps of analysis are displayed and compared between models. Since the main objective was to have a good quality description of modes of deformation, the stress field was at this stage not considered.

Figure 3.24: Example of two models of intersection 2

(a) Model 100x100
(b) Model 300x300
3.5 Analysis of cable forces

As briefly mentioned in the Introduction, the principle behind the design of the supporting structures of the utility tunnel was that of ‘belt and braces’ where the reinforced concrete lining would be additionally supported by prestressed steel cables. Given that the structural behaviour was thought to be similar to that of cable supported structures, high importance was given to investigating the role of the cables. Operating within the structural field, the following in-depth analyses were performed to fully characterise their behaviour:

- Fitting of cable forces from analysis step 4 to cable prestressing recorded after installation.
- Study on the effect of cable forces over the deformation modes of the concrete lining.
- Study on the effect of changes in concrete stiffness over the cable forces.

For intersection 1, analyses were carried out using the 200x200x140 model with extra fine mesh and 0.2 m element size in the lining; intersection 2 was investigated through a 200x200 model with the same type of mesh. These studies are presented in detail in the following sections. Results from the investigations are displayed in Section 4.6 in their relevant form.

3.5.1 Fitting of cable forces

Having cast the concrete lining, steel cables were installed, anchored and finally post-tensioned. In the field report from these operations, prestressing forces were recorded. While all cables were tensioned to reach a target load of 1100 MPa, in the model this could not simply be prescribed as input through the initial stress option; the target prestressing load is in fact the result of post-tensioning and is therefore equivalent to the resulting cable force from step 4. Differences between input and output stresses are due to the deformation of the cables: as a load is prescribed, the anchors at both ends of a cable displace and consequently lower the state of stress.

To account for this, a parametric study was carried out on the the initial stress assigned to the steel elements. The specific aim was to have all forces resulting from analysis step 4 match the prescribed prestressing of 1100 MPa thus accounting for the deformation of the cables and the displacement of the anchors.

Overall three values of prestressing were investigated: 0, 1100 and 1115 MPa using the model of intersection 2; once conclusions were reached from this, they were tested for intersection 1 as well by only studying the two limit conditions. In older models, a set of cable stresses ranging from 0 to 1130 MPa was subjected to analysis; the same conclusions were ultimately reached which is why only results from the more recent study are presented in Section 4.6.

3.5.2 Effect of cable forces on deformations

In a cable stayed bridge, the stiffness of the supporting cables as well as the prestressing load they carry have significant influence over the deformation of the deck. If a similar behaviour is assumed to characterise also the reinforced concrete sections, the effect of all cable forces is to be investigated.

Using two extreme conditions, the deformation of the concrete lining was studied after the excavation of the road tunnels, when the supporting action from the cables would be most relevant. The vertical displacement $w_5$ along the floor of the concrete lining from step 5 was studied under the following conditions: no prestressing ($\sigma_0 = 0$ MPa), ideal prestressing ($\sigma_0 = 1100$ MPa) and high prestressing ($\sigma_0 = 1115$ MPa). Both intersections were considered in this study to check for
possible differences caused by the different rock stress field.

This study was also performed considering a low stiffness value for the layer of leca surrounding the concrete lining. Since similar results were observed, this study is not presented in the results. The role of leca is discussed in Section 3.6.

3.5.3 Sensitivity of cable forces

Before the instrumentation malfunctioned, the reinforced tunnel segments were monitored by estimating the forces in the steel cables. This monitoring strategy was based on the assumption that had anything varied the stress state in the lining, changes would have been observed in the cable forces. The assumption was furthermore corroborated by the idea that the lining would fall in the class of cable supported structures. In light of the findings discussed in Section 4.6.2 of the results, this last assumption was however disproved.

![Figure 3.25: Region with reduced E-modulus](image)

Given its direct connection to the posed research questions, and having lost corroboration from the assumption of supported bridge-like behaviour, the validity of the assumption had to be tested: while it was observed that variations in cable tension have minor effects on the deformation of the tunnel, a study was set up to identify whether changes in the structural behaviour of the lining would affect the cable forces. Using the final model of intersection 2 (rock domain 200x200 m), one more step was added to the analysis. In the newly created step 6, the elastic modulus of concrete was reduced for all the volume of lining ideally overlapping with the cross-section of the road tunnel; initial rock, concrete and cable stresses were taken from the output of step 5 as schematically presented in Figure 3.26. This step would simulate the variation in stiffness caused for example by extensive cracking across a relevant portion of the lining. To satisfy the objective, effects from this important change were researched in the steel cables.

The material stiffness was reduced for the areas highlighted in Figure 3.25; this region was selected because any type of degradation (or damage) in such areas would have the potential to be critical: other than reducing the functionality of the utility tunnel by restricting circulation in it, damage in these areas might also prove dangerous for the users of the road tunnel located below. Of all the areas where material properties could have been reduced, this was selected because it is the only
3.5. Analysis of cable forces

Table 3.10: Concrete stiffness parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_{ck}$</td>
<td>32.0</td>
<td>GPa</td>
</tr>
<tr>
<td>$E_{c,\text{red}}$</td>
<td>10.0</td>
<td>GPa</td>
</tr>
</tbody>
</table>

In Comsol Multiphysics, the variation in stiffness was implemented by means of an additional parametric study for step 6: the parameter $E_{cc}$ having an initial value of 32.0 GPa was reduced to the value of 10.0 GPa. This reduced stiffness was chosen arbitrarily with the aim of investigating a limit condition and test a hypothesis. Had the aim been to assess numerically a precise scenario, $E_{cc}$ would have been derived through calculations; it should however be noted that differently from a reinforced concrete beam, where the state of stress is usually identifiable and with an estimate of the stiffness loss caused by cracking quantifiable, the lining is here subject to a very complex and 3-dimensional state of stress which would make such path very difficult.

Initial conditions

Excavate UT

Reinforce UT

Install cables

Excavate RT

Reduce $E_c$

Figure 3.26: Schematic view of the analysis steps, related physics and interacting variables
3. Method

3.6 Sensitivity to leca stiffness

Leca is the material used to compensate the rock overbreak in the utility tunnel and to create a smooth surface against which concrete can be cast. Acting as a membrane, the leca layer was highly significant for the problem at hand as it transfers stresses and defines the loading conditions of the concrete lining (for what concerns the walls and floor). This material was however responsible for the introduction of major uncertainties in the problem because, somewhat contradictorily, despite playing a relevant role for the structural behaviour of the lining, it did not itself have structural purposes. Or in short: leca is not a structural material here, but is highly relevant for the structural system. Uncertainties were specifically due to the material properties of leca, and to the state of the interface with rock. Considering that leca did not explicate structural roles, it seemed reasonable to assume that the leca-rock interface would present poor bonding properties and this was then partially accounted for by choosing a low leca stiffness. With the inability to dispose of any better information on the material properties of leca and on the interface leca-rock, studying limit conditions was identified as the only way to gain understanding.

With the objective of proposing an answer to the initial research questions, the effect of leca on the structural problem had to be fully understood. From an epistemological stand point, it was judged that the uncertainties on the role played by this intermediate layer could be partially compensated provided that the models had the ability to produce a good understanding of the overall structural behaviour. With this purpose, the assumption was made that a good model would be one where the leca stiffness only affected the amplitude of the results but not their basic behaviour.

To verify the quality of the models according to the previous statement, a study was devised: using the deformation mode of the supporting structures as main model parameter, results were compared using $E_l = 3 \text{ GPa}$ and $E_l = 3 \text{ MPa}$. The drastic change in leca stiffness, equal to three orders of magnitude, was selected to identify how the lining would respond to a very soft membrane. This comparison was carried out for both intersections 1 and 2.

Having an intermediate layer of concrete-like material separating two elements, uncertainties on the interface properties and material properties may be in simple cases addressed considering the local stress state: supposing an intermediate layer to transfer pure compression, cracking would not take place in the volume and it would not take place in the interfaces thus making it reasonable to employ the characteristic stiffness modulus of the intermediate material. Under a pure state of tension instead, a reasonable choice might be to suppose the intermediate material to be fully cracked and thus model a discontinuous displacement field. Although similar reasoning might be applied to the modelling of simple problems, the intermediate layer of leca is in this case transferring a very complex state of stress. This 3-dimensional state with principal stresses varying their orientation from point to point made such an approach unsuited, which is why, ultimately, a soft leca limit condition was instead analysed.
3.7 Concrete cracking analysis

Having subjected to investigations leca layer and cables, focus was finally placed on the concrete lining itself. In depth studies of all concrete elements were performed to both answer the research questions and to obtain a measure of validation for the model.

When dealing with problems involving high degrees of uncertainty, model validation is a key aspect to ensure that a link with reality be present. In absence of any possibility for validation, reasoning in terms of limit conditions is to be preferred as conservative approach. The supporting structures in the tunnels are clamped by the rock and not subjected to any relevant dynamic load; furthermore, as discussed in Section 4.6, cables are not very responsive to changes taking place in the lining. For the previous reasons, common strategies such as dynamic testing and cable forces measurement were ruled out. The only possibility to compare model predictions with physical reality was found in the non-linear behaviour of concrete by investigating the cracking patterns. Having identified all model-predicted hotspots, an inspection was conducted on site to identify and record concrete cracks. Predictions and recorded data were finally compared to check for similarities and discuss discrepancies. Studying the state of stress inside the reinforced concrete elements was also a highly relevant for the research questions, allowing to both have an understanding of the stress diagram in the lining and to identify suitable areas for monitoring operations.

Codes devoted to concrete design such as Eurocode 2, make possible the estimation of crack spacing and mouth opening given the loading conditions, elastic displacements from a linear model, reinforcement type and reinforcement spacing. Yielding information on the non-linear behaviour starting from a linear model, this approach could not be pursued because of the complexity of the stress conditions. Information from the stress diagrams and stress orientation, in fact, revealed the highly 3-dimensional nature of the stress field found in the lining; any parallelism to a simpler structural case, such as that of a beam under bending, was therefore impossible.

3.7.1 Model predictions

The analysis of the results produced by the main models highlighted a high state of stress across most of the linings both in terms of tensions and compressions. Specifically, the stress diagrams for sp1 and sp3, the first and third principal stresses, were exported for lines running along the walls, floor slab and roof of the lining; stresses were considered for both internal and external walls. Relevance was given to the shape of the diagrams rather than to the stress amplitudes they offered, this was motivated by the awareness of having an insufficiently fine mesh for numerical values to be accurate.

Hotspots for cracking were identified across all concrete surfaces by considering the stress diagrams and contour plots generated; specifically, all areas with high values of sp1 were selected. The stress diagrams however revealed how high values for the first principal stress would in certain areas be coupled to equally high values of sp3 in compression. To account for this highly 3-dimensional stress state, a simple Drucker-Prager criterion was implemented to check which points would fall outside the elastic domain. All areas where concrete was predicted to fall outside the boundaries of the elastic domain were then compared to the hotspots predicted by only studying sp1. The well known Drucker-Prager yield criterion is reported in Equation (3.7), where $I_1$ is the hydrostatic pressure and $J_2$ is the second invariant of the stress deviator. The expressions for coefficients $A$ and $B$ are presented in (3.8) in which the material parameters of Section 3.1.1 have been used.

$$\sqrt{J_2} = A + B \cdot I_1$$ (3.7)
3. Method

\[
\begin{align*}
A &= \frac{2}{\sqrt{3}} \left( \frac{\sigma_c \sigma_t}{\sigma_c + \sigma_t} \right) \\
B &= \frac{1}{\sqrt{3}} \left( \frac{\sigma_t - \sigma_c}{\sigma_c + \sigma_t} \right)
\end{align*}
\]

(3.8)

Observations during the site inspections revealed the majority of cracks to originate from the corners or the openings where the cable anchors are allocated. With the aim of checking for possible stress concentrations in the corners, a new Comsol model was created for intersection 2 where the anchor slots were carved out from the geometry of the concrete lining. Since the addition of such details required a much finer mesh, it was decided to decouple the problem by separating the rock domain from the supporting structures: all rock domains were removed from the model and a displacement field was prescribed to all surfaces of concrete and leca that would have been in contact with rock. The prescribed displacements were taken from the output of the analysis with the full geometry. This modelling simplification allowed to study step 5 (excavate RT) concentrating uniquely on concrete and leca using a highly refined mesh. While it is true that had the main model been run from the start with openings for the cable anchors and a very fine mesh results would have been different (the lining would have been more compliant), the computational time required would have risen dramatically; in addition, it was believed that this simplification would not alter the overall behaviour of deformation of the concrete lining.

Figure 3.27: Geometry of the reduced model with anchor openings

Crack orientations were predicted by assuming that they would be governed by the principal stress in tension sp1. For all internal surfaces of the concrete linings, cracks were predicted to open along
3.8. Non-linear analysis

The orthogonal direction to the direction of sp1.

These studies were conducted pushing the linear-elastic models to the edge of the information they could provide. While a true cracking analysis would call for a non-linear material model, elastic models can still help in suggesting a rough estimate of where cracks are likely to develop. No information is however offered as to how the structure reacts after the first crack has opened.

3.7.2 Site inspection

A site inspection was carried out to research for visible cracks on the concrete surfaces. By visually scanning the inner faces of the concrete lining, cracks were spotted and signalled by means of spray paint for future inspections. On paper crack position, orientation and length were recorded. Due to time restraints, this procedure was only carried out for the concrete lining of intersection 2 which with its smaller size did not require ladders. Focus was especially placed on the concrete walls where cracks could be most easily identified. The rough surface of the ceiling created by shotcrete did not make the spotting of cracks possible. Similarly, cracks could not be observed on the concrete slab of the floor because of the dust and dirt deposited on it.

3.8 Non-linear analysis

With the objective of corroborating the information on possible cracking hotspots gathered from the elastic study of Section 3.7, a non-linear model was created. Using the reduced geometry presented in Figure 3.27, the orthotropic reinforcement was introduced as truss elements embedded in concrete by forcing the displacement field to be the same. Introduced loads were gravity and cable forces; to simplify the analysis, cables were removed from the geometry and a pressure equivalent to the cable forces was applied to the anchor plates. Boundary conditions were added by prescribing a displacement field to all surfaces of leca and concrete that would have been in contact with rock; this displacement field was extracted by the full model of intersection 2.

Non linear material models were added to both concrete and leca using a damage option with crack band. Fracture energy was estimated for both materials according to Model Code 1990. To ease convergence of the problem, an incremental parameter was run from 0 to 2 across two separate fields: for \((0 < Par < 1)\) the pressure on the anchors was increased to its final value and zero displacements were prescribed to all external surfaces; for \((1 < par < 2)\) the displacements of the surfaces ideally in contact with rock were ramped up to their final values.

Unfortunately no results are presented for this study seen as convergence could not be archived despite having tested different mesh types and steps for the displacement-control. It was concluded that this was caused by the complexity of the displacement field being prescribed and by it having been produced by an elastic model. Non-linear problems where brittle behaviour, such as that of concrete in tension, is to be expected are notoriously known to have difficulties with convergence. In this case additionally, the enforced displacements were extracted from an elastic model; with no possibility to create discontinuities, the elastic model for example transferred tensile actions to leca in the normal direction to its surfaces. While in these areas a non-linear model with contact would have predicted the detachment between leca and rock, the elastic model could not. Schematically, the result was that tensile displacements were prescribed to an element with brittle formulation: as soon as the crack appears, the material does not offer additional stiffness and the next incremental step does not reach convergence. Had tensile forces developed within the material, the software would have been able to initiate and propagate cracks, but with tensile displacements prescribed.
3. Method

directly to the boundaries the software could not archive convergence.

For a non-linear model to work, the full model of the intersection, including rock domain, should be solved with a contact formulation between leca and rock. The displacements so computed could then be prescribed in displacement-control to the reduced model. Given the enormous computational times this procedure would require, this was not attempted within this study.
CHAPTER 4

Results and Discussion

4.1 2D boundary influence

In the following graphs the most relevant results from the study conducted for the influence of boundaries are presented. Figures 4.1 and 4.2 display the vertical displacement $v(x)$ as a function of the horizontal coordinate $x$ across the rectangular rock domains, the effect of initial rock stresses is presented in the second graph. In the two figures a line plots the displacement for each of the ten domain widths that were studied. The maximum displacement measured above the tunnel opening is plotted against the width of the rock domain in Figure 4.3. The two cases with and without initial rock stresses are presented side by side.

The results presented are extracted solely from the line running horizontally above the tunnel. Although other lines were studied, these plots are here not presented since the principle behaviour was observed to be the same.

Figure 4.1: Vertical displacements along the line above the tunnel, plot lines for domain widths from 300 to 40 m, no initial rock stresses
4. Results and Discussion

Figure 4.2: Vertical displacements along the line above the tunnel, plot lines for domain widths from 300 to 40 m, with initial rock stresses

Figure 4.3: Maximum displacement above the tunnel as a function of the domain width. Left: no initial rock stresses. Right: with initial rock stresses.

Analysis

The study conducted points out that the main output of the finite element model, the displacement field, is highly affected by the distance between model boundaries. This conclusion is reached for the cases with and without initial rock stresses. When no initial stresses are prescribed, the displacement field \( v(x) \) is affected in terms of amplitude but maintains the same behaviour in
4.2 Cable connection

principle: all lines display two points of inflection, have their absolute minimum in correspondence with the tunnel and their absolute maximum at the boundary.

When initial stresses are assigned to the rock, the solution is more significantly affected by the model size and changes both amplitude and behaviour. Although the point of absolute maximum is located in correspondence to the same abscissa, the location of the minimum varies from line to line: in the bigger models four local minima are identified while in the smaller models only two as results of a smaller number of inflection points.

The higher sensibility of the case with initial stresses is also clearly highlighted by Figure 4.3: while convergence is reached with an approximately 300 m wide domain without initial stresses, convergence is much slower when these stresses are activated.

4.2 Cable connection

For the two models having an embedding constraint on the rock side (Detailed model, Rigid model with and without borehole) the stress field for the cables along the anchoring length is reported in Figure 4.5. The total displacement along the free length for all considered models is presented instead in Figure 4.6. The reference system for abscissas $s_{anch}$ and $s_{free}$ is visible in Figure 4.4.
4. Results and Discussion

Figure 4.5: Total displacement along anchorage length

Figure 4.6: Total displacement along free length
Analysis

The stress field along the anchoring length clearly follows the shape of an exponential decaying function. For the detailed model, the stress reduces of $10^1$ in amplitude after 3 cm from the free surface, and $10^2$ after approximately 20 cm. In terms of displacements, a difference of 1% with respect to the asymptotic value of displacement is reached just after 39 cm of anchoring length. The relative difference falls instead under the 5% threshold after 7 cm.

Considering the free length of the cable, it is clear that the detailed model proves to be the most compliant leading to the highest displacement at both ends. The rigid model with flexible connector type and borehole resembles the behaviour of the detailed model displaying a similar slope for the displacement field. Rigid model without borehole and point connections model do not account for any borehole opening. While these models produce a very similar value for the displacement at the end of the free length, the rigid connector model demonstrates to be more compliant on the side of the concrete anchorage. In comparison to the detailed model, the total displacements at the concrete end are 3% lower for the rigid model with borehole, 5% lower for the rigid model without borehole and 11% lower for the point connections model.

Discussion

Differently from anchorages in cohesive soils such as clay, where most of the anchoring length is contributing to counteract the applied force, only the initial sections of the anchor are here active. The observed behaviour is by all means similar to that of concrete rebars during a pull-out test: during the initial stages, when the model can still be considered to behave elastically, shear stresses are concentrated along a short length from the free surface; as cracking starts to appear in concrete, stresses reduce in amplitude and distribute along a higher length [11]. If the interest were to be directed towards modelling the elastic behaviour of this anchorage, a simple point connection between rock and steel cable would be compatible with the generated stress field in the cable but would concentrate stresses in the rock. Looking at Figure 4.6 it is clear that the third and fourth models underestimate the displacement; since the two employ different connections at the rock end, this is clearly caused especially by the absence of any borehole opening.

At the opposite end, it is concluded that both rigid models provide a good approximation for the behaviour of the cable anchorage. It stands that the absence of any borehole increases the rigidity of the model, but the rigid connector with flexible formulation proves to be much more compliant than the point connection.

In the spirit of balancing accuracy with simplicity, the solution proposed by the third model, rigid connector without borehole, was finally recommended. Although this solution underestimates the displacement at the rock end, it provides a good estimation for the displacement field in the concrete lining while still maintaining a high level of simplicity. Being the focus of this work on the analysis of the concrete lining, it was in fact deemed more significant to have a good estimate of the displacement in this area rather than for the rock anchorage.

Using a point connection in a solid model is generally not recommended because of the high stress concentrations it generates; in this case however there was no interest in studying the behaviour around the rock anchors and these were located sufficiently far away from the lining. For this reason, the strategy proposed by in the third model could be further simplified by replacing the anchoring length with a point connection on the side of the rock.
4.3 Convergence study

Figure 4.7 displays the average shape of \( w_5(x) \), the vertical displacement of step 5, along the lower right corner of the utility tunnel. An average line was presented because results were observed to be very similar between meshes; this is especially visible in Figure 4.8 presenting the peak displacements where all lines are close.

![Graph showing average diagram of \( w_5(x) \) along the lower right corner of UT-2](image)

**Figure 4.7: Average diagram of \( w_5(x) \) along the lower right corner of UT-2 (see Figure 4.15)**

<table>
<thead>
<tr>
<th>Model ID</th>
<th>Max( (w_5) ) [mm]</th>
<th>Ratio [%]</th>
<th>Min( (w_5) ) [mm]</th>
<th>Ratio [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Extra fine - 0.2</td>
<td>0.9942</td>
<td></td>
<td>-0.3311</td>
<td></td>
</tr>
<tr>
<td>Finer - 0.2</td>
<td>0.9925</td>
<td>-0.17</td>
<td>-0.3317</td>
<td>0.19</td>
</tr>
<tr>
<td>Extra fine - 0.3</td>
<td>0.9936</td>
<td>-0.07</td>
<td>-0.3344</td>
<td>0.99</td>
</tr>
<tr>
<td>Finer quad. - 0.3</td>
<td>0.9918</td>
<td>-0.24</td>
<td>-0.3298</td>
<td>-0.40</td>
</tr>
<tr>
<td>Normal - 0.3</td>
<td>0.9773</td>
<td>-1.68</td>
<td>-0.3294</td>
<td>-0.49</td>
</tr>
</tbody>
</table>

**Table 4.1: Maxima and minima of \( w_5 \) with ratios to Extra fine - 0.2**

**Analysis**

All five considered meshes are able to capture the same deformation of the utility tunnel. As visible in Table 4.1, the maxima \( w_5 \) predicted are all rather close while the minima from those meshes employing a 0.3 m element size are further apart from the minima of the 0.2 meshes. While the finer 0.2 mesh proves to be on average 0.18% distant from the reference mesh, the 0.3 extra fine mesh has a more unbalanced behaviour estimating the peak value ten times better than the bottom
4.3. Convergence study

Figure 4.8: Detail of tunnel segment with maximum \( w_5 \) (see Figure 4.7)

one. The normal mesh produces a better estimate of the maximum than of the minimum, it is also
the only mesh going beyond the 1\% of relative distance.

In terms of stresses, the difference in element size is more clearly noticeable. Principal stresses from
meshes 1 and 2 follow a path that is different from that of meshes 3, 4 and 5. Despite presenting
different amplitudes, all meshes capture a similar stress diagram where maxima and minima are
found in the same locations.

Discussion

The displacement field is well captured in terms of behaviour. In considering the numerical values,
the extra fine model with element size 0.2 m was selected as reference; this choice was motivated
by the assumption that that would be the most accurate. The observed trend was to have peak values
approached from below and bottom values reached from above. This is fully compatible with the
general principles of finite elements: coarser meshes are stiffer and as such deform less in both
positive and negative directions. Surprisingly, the quadratic mesh and normal mesh approach the
reference mesh from below for both peak and valleys; although for equal element sizes quadratic
meshes provide better results than linear ones, the element size was in this case considered to be the
dominant parameter. This assumption was corroborated by the stress analysis: the quadratic mesh
provides a stress diagram that is very similar to that of the linear meshes with the same element
size.

A quick look at Figure 4.9 might lead to suppose the problem to be mesh dependent. It should
however be noticed that the overall behaviour is fully captured and that the usage of only 2 elements
across the concrete thickness is insufficient to obtain meaningful stress amplitudes.

For intersection 2 an extra fine mesh with element size 0.2 m was selected: this choice was
motivated by the wish to have the best approximation of displacements as well as of the stresses
while still satisfying the limits on the computational time. Differently from the other mesh with the same element size, this would also allow for a better estimate of the displacement field around the utility tunnel and therefore to conduct wider investigations.

For intersection 1 a finer mesh with 0.2 m in utility tunnel element size was adopted. This choice was motivated by the same reasoning as before, but had to be coupled with the higher computational requirements inherent in the geometry of this intersection.
4.3. Convergence study

Figure 4.10: Detail of principal stress $sp_3$ (see Figure 4.9)
4. RESULTS AND DISCUSSION

4.4 Model choice

In this section, results from intersections 1 and 2 are considered separately. This choice is motivated by the wish to clearly highlight the peculiarities of the two models and allow for their discussion independently.

Figure 4.11: Lower right corner of UT-1 along which results were studied

4.4.1 Intersection 1

Out of all the results studied, only the diagrams of w2 and w5 are here presented for brevity. It was decided to not display horizontal components of displacement since they were found to be much more similar between models; the highest differences were observed in terms of vertical deformation. The first two plots in Figures 4.12 and 4.13 display the results from the four models from 200x100 to 300x300; these are joined by the characteristic of having the same depth for the rock domain: 90 m. In Figure 4.14, the models with a deeper rock domain are compared to model 250x250. For all plots the reference system of model 300x300 was used, the origin of the x-axis of other models was moved to have all concrete linings coincide in the same abscissa. Numerical values and ratios are displayed in Tables 4.2 and 4.3.
4.4. Model choice

![Graph](image)

Figure 4.12: Displacement $w_2$ (excavate UT) along UT-1 near the model boundaries

<table>
<thead>
<tr>
<th>Model ID</th>
<th>$\text{Max}(w_5)$ [mm]</th>
<th>Ratio [%]</th>
<th>$\text{min}(w_5)$ [mm]</th>
<th>Ratio [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>300x300</td>
<td>2.3641</td>
<td>-0.4718</td>
<td>-0.4718</td>
<td>-0.4718</td>
</tr>
<tr>
<td>250x250</td>
<td>2.3420</td>
<td>-0.93</td>
<td>-0.4957</td>
<td>5.07</td>
</tr>
<tr>
<td>200x200</td>
<td>2.2765</td>
<td>-3.70</td>
<td>-0.5459</td>
<td>15.70</td>
</tr>
<tr>
<td>200x100</td>
<td>2.0292</td>
<td>-14.17</td>
<td>-0.5963</td>
<td>26.38</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Model ID</th>
<th>$\text{Max}(w_5)$ [mm]</th>
<th>Ratio [%]</th>
<th>$\text{min}(w_5)$ [mm]</th>
<th>Ratio [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>250x250x200</td>
<td>2.2616</td>
<td>-0.5987</td>
<td>-0.5987</td>
<td>-0.5987</td>
</tr>
<tr>
<td>200x200x140</td>
<td>2.2367</td>
<td>-1.10</td>
<td>-0.6186</td>
<td>3.32</td>
</tr>
<tr>
<td>250x250</td>
<td>2.3420</td>
<td>3.56</td>
<td>-0.4957</td>
<td>-17.20</td>
</tr>
</tbody>
</table>
4. Results and Discussion

Analysis

All models produce similar results when it comes to the mode of deformation of the utility tunnel after its excavation. The general trend that is observed is that as the model gets wider the displacement plot is slightly translated upwards: for the same point, downwards displacement $w_2$ predicted by model 300x300 is smaller than that predicted by model 200x100. The plots of Figure 4.12 present clear boundary effects: displacements $w_2(x)$ reduce sharply as the edges of the model are approached. For step 2, all boundary effects are however relegated to an area of about 15 m from the boundaries and only slightly affect results elsewhere in the model.

The domain size appears to be much more relevant for the deformation resulting from the road tunnel excavation. All models capture the same overall behaviour having maxima and minima at similar abscissas. For those models characterised by the same rock height, the curves are as for $w_2$ translated upwards: model 300x300 presents higher maxima and smaller minima than 200x100. The displacements move towards the same point of convergence as the rock domain is widened, provided that the same height of rock is maintained. When the rock height is increased, the trend changes: 200x200x140 and 250x250x200 provide smaller maxima and higher minima compared to
4.4. Model choice

Figure 4.14: Displacement $w_5$ (excavate RT) along the lower right corner of UT-1

250x250, and provide higher displacements near the boundaries. Overall the model with 140 m of rock height and the one with 200 m provide very similar results.

Discussion

If only the results from Figure 4.12 were to be considered, the final choice would be likely to fall on model 250x250: it captures the mode of deformation, is much lighter to run than 300x300 and still provides fair numerical estimates. It should still be noted however that although convergence appears to be practically reached for the maximum value, it is not reached for the minimum one as a 5% difference from the 300x300 would imply a difference greater than 1% between 300x300 and a possible 350x350 model. Although their study was justified by the fact that the first two principal rock stresses are horizontal, these models force the rock domain to be rather shallow. Since these models have rollers restraining their bottom face, the implicit condition is that for $z < 0$ displacements in the x and y directions would be the same as those recorded in the plane at $z = 0$, without providing however additional stiffness. Somewhat counter intuitively, reducing the height of the rock domain requires its length and depth (x and y directions) to be increased to reach convergence. Although convergence is here reached, even then, if convergence were to be reached, it would be for a problem that is not representing well reality.
This reasoning can be verified by considering the plots of Figure 4.14: here, increasing the height of rock produces results that are distinct from those of the shallow model and convergence is towards a different curve of displacement. Significant to notice is that the displacements of 200x200x140 are closer to those of 250x250x200 than the displacements of 200x200 are to those of 250x250; following the same lines of the previous discussion, this would prove that as the rock height is increased the relevance of the base dimensions diminishes.

Taking into consideration all points mentioned, model 200x200x140 was selected to represent intersection 1 and was therefore used in all further analyses. Deciding to disregard the convergence trend given by model 300x300, importance was given to the fact that despite the relevant difference in domain size with model 250x250x200, results were very similar (less than 5% in difference). This model was identified as the best compromise between computational time and precision. These assumption could be verified by studying a larger model of size 300x300x250 for example, unfortunately this was not attempted because of the computational times this would have required.

### 4.4.2 Intersection 2

For Intersection 2, only the displacement component \( w_5 \) is here presented; the vertical displacement from step 2 was omitted since it presented the same characteristics as that of intersection 1. The output from models 100x100 to 300x300, all having the same 90 m or rock height, are displayed in Figure 4.16. The displacements resulting from the remaining models are presented in Figure 4.17. The detail of the displacements along a small segment is reported to facilitate the reading.

![Figure 4.15: Lower right corner of UT-2 from which results were extracted](image)

**Analysis**

All models, apart from 100x100, prove to capture the mode of deformation of the utility tunnel. Similarly to what was observed for intersection 1, when the domain height is kept constant and base dimensions increased the trend is for the displacement field to move upwards i.e. maxima increase and minima decrease in amplitude. In Figure 4.16 boundary effects are clearly visible in model 100x100: this model does capture deformation mode on the left of the concrete lining \((x < 150 \text{ m})\) but fails to on the other side; here the the model is insufficiently large to account for
4.4. Model choice

Figure 4.16: Displacement $w_5$ (excavate RT) along the lower right corner of UT-2

Table 4.4: Maxima and minima of $w_5$ with ratios to 300x300

<table>
<thead>
<tr>
<th>Model ID</th>
<th>Max($w_5$) [mm]</th>
<th>Ratio [%]</th>
<th>Min($w_5$) [mm]</th>
<th>Ratio [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>300x300</td>
<td>1.0247</td>
<td>-</td>
<td>-0.3148</td>
<td>-</td>
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<tr>
<td>250x250</td>
<td>1.0109</td>
<td>-1.35</td>
<td>-0.3191</td>
<td>1.34</td>
</tr>
<tr>
<td>200x200</td>
<td>0.9925</td>
<td>-3.14</td>
<td>-0.3317</td>
<td>5.36</td>
</tr>
<tr>
<td>100x100</td>
<td>0.8843</td>
<td>-13.70</td>
<td>-0.3827</td>
<td>21.54</td>
</tr>
</tbody>
</table>

the decreasing branch ($x > 200$ m). When the height of the domain is increased, the displacement field starts converging towards a different curve. Convergence proves to be a function of all three domain dimensions, as can be seen comparing 200x200x140 to 250x250x140.
Table 4.5: Maxima and minima of $w_5$ with ratios to 200x200x200

<table>
<thead>
<tr>
<th>Model ID</th>
<th>Max($w_5$) [mm]</th>
<th>Ratio [%]</th>
<th>Min($w_5$) [mm]</th>
<th>Ratio [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>200x200x200</td>
<td>0.9997</td>
<td></td>
<td>-0.3313</td>
<td></td>
</tr>
<tr>
<td>200x200x140</td>
<td>0.9836</td>
<td>-1.6125</td>
<td>-0.3458</td>
<td>4.3846</td>
</tr>
<tr>
<td>200x200</td>
<td>0.9925</td>
<td>-0.7194</td>
<td>-0.3317</td>
<td>0.1176</td>
</tr>
<tr>
<td>250x250x140</td>
<td>0.9911</td>
<td>-0.8548</td>
<td>-0.3411</td>
<td>2.9483</td>
</tr>
</tbody>
</table>

Discussion

Following the same reasoning presented for the first intersection, relevance was given to the depth of the rock domain; for this reason convergence with respect to model 300x300 was deemed meaningless for the accuracy of the models. Model 200x200x200 was selected for reference because of its balance which allows the concrete lining to be located at similar distances from all boundaries.
Comparing other domains to it, it was observed that when the rock height is kept constant at 140 m and the planar size widened\(^1\), accuracy increases. The ideal approach would have been to study a model of dimensions 300x300x200 however this was not possible because of limits on the computational time. Surprisingly, model 200x200 proved to be very close to the reference model with a relative difference under 1% for both maxima and minima. This was interpreted as a good fit between domain height and width providing a balanced model; no other precise explanations for this phenomenon could be found.

Ultimately, model 200x200 was chosen to represent intersection 2. As clearly visible in Figure 4.18, this fell right in between the others making this choice compatible with the aim of identifying a model that would be computationally light and at the same time balanced. Given the high level of uncertainties surrounding the problem, choosing the model in the middle was seen as the cautious answer to the uncertainty of where the displacement curve would converge to.

\(^1\)The comparison is between model 200x200x140 and 250x250x140
4. Results and Discussion

4.5 Results from main models

For reference, the displacement field was extracted along the line highlighted in Figure 4.11 for intersection 1, and along the line highlighted in Figure 4.15 for intersection 2.

4.5.1 Intersection 1

Discussion

All deformations in the presented graphs depends on the deformation of the rock which responds to the creation of new free surfaces. For what concerns the vertical component of displacement, the sign the displacement obtained depends on the relative weight between the action of the initial *in situ* stresses and the gravity load of the removed material. When the utility tunnel is excavated, the lower corner of the newly created tunnel moves downwards (see Figure 4.19; this is because the horizontal rock stresses act as pressure on the walls of the utility tunnel which bend inwards and push downwards the surface of the floor. The lower relevance of gravity compared to rock stresses is also testified by how, in spite of a varying depth (the utility tunnel is inclined at +5.80°), the vertical displacement $w_2(x)$ displays only a slightly inclined trend.

The tensioning of all steel cables produces only minor effects on the deformation of the utility tunnel, displacements are localised around the anchors.

The excavation of the two road tunnels has large effects to the rock mass system causing extensive deformations across the entire elastic continuum. While the first part of concrete lining (UT-1A) is moved downwards, the second part (UT-1B) undergoes a displacement in the opposite direction (see Figure 4.26). Once again the basic principle underlining this response has to do with rock stresses and the geometry of the problem: as discussed for the excavation of the utility tunnel,
Figure 4.20: Displacement $w_4$ (install cables) along the lower right corner of UT-1

Figure 4.21: Displacement $w_5$ (excavate RT) along the lower right corner of UT-1
4. Results and Discussion

Figure 4.22: Displacement v5 (excavate RT) along the lower right corner of UT-1

Figure 4.23: Deformed shape of UT-1A after RT excavation
horizontal rock stresses compress the walls of the road tunnels making their floor bend downwards and their roof bend upwards. As a result of this, the the utility tunnel is located below the road tunnel, it receives a compressive action in the negative z-direction, while when it is located above the opposite happens. The upwards and downwards deformations have rather dissimilar amplitudes also because of geometry: while UT-1A is separated by a few meters of rock from RT-1A, UT-1B
is directly crossing the section of RT-1B. To this factor can also be linked to why the deformation of the two lining segments is different in shape; it should in fact be noticed that the rock mass is a highly redundant system and as such displays a response which is highly dependent on the geometry. Figure 4.25 clearly displays this deformation in the vertical plane.

While all previous steps present low interest for the transverse horizontal displacement $v(x)$, this becomes highly relevant in the final step. The $y$-component of displacement $v_5(x)$ is by far the largest and is responsible for a shearing mode of deformation. The sign of this displacements depends on how the road tunnels are oriented with respect to the utility tunnel: following a hydraulics similitude, the direction of deformation in the horizontal plane can be seen as a fluid entering the boundaries of the model with the direction of the principal rock stress, and running towards the ideal drains that are the road tunnels. Applying this similitude to Figure 3.14, it can be seen how deformations move in the negative $y$-direction before the the intersection with RT-1A, and towards the positive $y$-direction after the crossing of RT-1B. Finally, the graph displays a sharper peak at $x = 120$ m because the utility tunnel is physically crossing the road tunnel cross-section.

Despite the complexity, the mode of deformation of the utility tunnel may be described in average terms as combination between a major shearing mode in the horizontal plane and a set of bending modes in the vertical plane.

Figure 4.26: Total displacement after RT excavation across a cut plane (XZ) of UT-1
4.5. Results from main models

4.5.2 Intersection 2

Figure 4.27: Displacement $w_2$ (excavate UT) along the lower right corner of UT-2

Figure 4.28: Displacement $w_4$ (install cables) along the lower right corner of UT-2
4. RESULTS AND DISCUSSION

Figure 4.29: Displacement $w_5$ (excavate RT) along the lower right corner of UT-2

Figure 4.30: Displacement $v_5$ (excavate RT) along the lower right corner of UT-2
4.5. Results from main models

Figure 4.31: Deformed shape of UT-2 after RT excavation

Figure 4.32: Deformed shape of UT-2 after RT excavation with contour plot of $w_5$
Discussion

The same considerations made for intersection 1 apply to intersection seen as, apart from different geometry and stress orientation the same principles apply.

One relevant difference compared to the other intersection studied is how the displacement field $w_5(x)$ assumes both positive and negative values even though the utility tunnel is located above the road one. As this was explained considering that UT-2 overlaps significantly with the cross-section of RT-2 crossing both roof and part of the side wall: while the part crossing the roof is pushed upwards, the final part of the concrete segment crosses the upper part of the road tunnel wall which is compressed inwards by the rock stresses. Coupling these two different loading conditions, the peculiar shape of $w_5(x)$ can be explained. While the road tunnel in Figure 4.33 caves inwards as mentioned, the deformation across a cut plane for intersection 2 is presented in Figure 4.34.

Compatibly with the hydraulic similitude mentioned in the analysis of intersection 1, the transverse displacements $v_5(x)$ display here the opposite sign: as displayed by Figure 3.17, the road tunnel approaches the utility tunnel from the top left of the domain, as opposed to this happening from the lower left for intersection 1.

In average terms, the mode of deformation can be described as coupling between shearing in the horizontal plane and bending in the vertical plane; this bending may also be interpreted as a relative imposed displacement between the two ends of the lining.

Figure 4.33: Total displacement after RT excavation across a cut plane (XZ) of RT-2
4.5. Results from main models

Figure 4.34: Total displacement after RT excavation across a cut plane (XZ) of UT-2 and RT-2
4.6 Cable forces study

4.6.1 Fitting of cable forces

Figures 4.35 and 4.36 display the stresses in the cables as produced by step 4 (install cables); stresses are produced having on the X-axis the x-coordinate of the cable. Results from the case where no prestressing load is prescribed are not reported: other than small variations, these cable stresses had $10^{-6}$ Pa as order of magnitude. This was considered as a check to verify that step 4 would be working as intended.

![Figure 4.35: Cable stress $\sigma_{\text{post}}$ after post-tensioning, cables on the right wall of UT-2](image)

Analysis

Cable stresses from step 4 were found to depend almost linearly on the prescribed initial stress: as visible in Figures 4.35 and 4.36 which treat the cables of the right and left wall respectively, the diagrams appear identical in shape but are in fact separated by a $\Delta \sigma$ that slightly changes from cable to cable. A certain amount of non-linearity was also observed when comparing results from stress prescriptions: comparing 1100 MPa to 1115 MPa produced in the cables a different stress variation than that from 1115 MPa to 1120 MPa.

It should also be noticed that cables reacted differently to the initial stress as testified by the lines not being straight; the variation is nonetheless relegated to a range of about $\pm 2$ MPa from the average. When the initial stress $\sigma_0 = 1115$ MPa, the average cable stress produced by the cable installation step is 1100.35 MPa for the right cables and 1100.52 MPa for the left ones.

Discussion

As was to be expected, the results underlined a difference between the stress initially prescribed and the cable stress from the step outputs. Overall this difference, representing the pull lost because

---

2 Each dot represents a cable and is positioned at the x-coordinate of the cable it refers to.
3 Each dot represents a cable and is positioned at the x-coordinate of the cable it refers to.
of settlements, was identified at an average of 15 MPa. Since the loss in cable force was observed to be very similar for both intersections, the initial stress of the supporting cables was increased to 1115 MPa in all final models.

The non-linearity in the response to the prescribed initial stresses was motivated with the following arguments: numerical issues and the formulation of the rigid connector of flexible type. Steel cables are modelled by extremely thin elements connected through a hinge constraint to the rock; since it is possible for the stiffness deriving from the truss elements to be much smaller than that coming from the solid elements, the resulting stiffness matrix may be ill-conditioned in certain areas and lead to numerical approximations. Rigid connectors of flexible type, the formulation of which is presented in Appendix A, have the characteristic of making the stiffness matrix non-symmetric. This fact, coupled with the need for stresses to be locally computed around the connector may be source of non-linearities for the problem.

The following reasons were put forth to explain the differences in axial load between cables: cables have different geometries and especially inclinations, rock stresses influence the problem, cable are anchored in different positions of the lining leading to different stiffness in the supports. Regardless, these variations were deemed to be insufficient to be relevant and were not addressed. To put the problem into perspective, while cables already lose less than 2% of stress in average terms, variations between cables are one order of magnitude smaller.

### 4.6.2 Cable forces and deformations

Of the considered cases, results are exclusively presented for the 0 MPa and 1115 MPa limit conditions. The intermediate study is not presented because of its high similarity to the second limit condition mentioned. In Figures 4.37 and 4.38 only results from the second intersection are displayed as they presented a clearer picture of the behaviour to be investigated. Amplitudes for
the maxima and minima of $w_5(x)$, extracted along the corner lines highlighted in Figures 4.11 and 4.15, are reported in Table 4.6 for the first intersection and Table 4.7 for the second.

![Graph showing maxima and minima of $w_5(x)$](image)

**Figure 4.37:** Close-up of displacement $w_5$ from UT-2 with different prestressing loads (reference curve in Figure 4.7)

<table>
<thead>
<tr>
<th>Prestressing</th>
<th>Max($w_5$)</th>
<th>Ratio</th>
<th>Min($w_5$)</th>
<th>Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_0 = 1115$ MPa</td>
<td>2.2369</td>
<td>-0.6186</td>
<td>-0.6189</td>
<td>0.0437</td>
</tr>
<tr>
<td>$\sigma_0 = 0$ MPa</td>
<td>2.2241</td>
<td>-0.5713</td>
<td>-0.6189</td>
<td>0.0437</td>
</tr>
</tbody>
</table>

**Table 4.6:** Intersection 1, maximum and minimum $w_5$

**Analysis**

The comparison between maximum and minimum values of $w_5$ demonstrates how little effect the cables have on the lining. In both intersections, the amplitudes with and without prestressing depart by less than 1%. When looking at the plot of Figure 4.38, which displays the vertical displacement of the floor midline, the same behaviour is produced by the two limit cases and the numerical values...
Table 4.7: Intersection 2, maximum and minimum $w_5$

<table>
<thead>
<tr>
<th>Prestressing $\sigma_0$</th>
<th>Max($w_5$) [mm]</th>
<th>Ratio [%]</th>
<th>min($w_5$) [mm]</th>
<th>Ratio [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma_0 = 1115$ MPa</td>
<td>0.9940</td>
<td>-0.3310</td>
<td></td>
<td></td>
</tr>
<tr>
<td>$\sigma_0 = 0$ MPa</td>
<td>0.9890</td>
<td>-0.5030</td>
<td>-0.3320</td>
<td>0.3021</td>
</tr>
</tbody>
</table>

Figure 4.38: Displacement $w_5$ from the middle line of the UT-2 floor lining

Discussion

Although simple in nature, this analysis was highly relevant to quantify the weight cables bare within the structural system. The first conclusion to be drawn was that, given all the simplifying assumptions used as well as all the modelling decisions taken, the prestressing load of the steel cables does not play a relevant role when it comes to the deformation of the structure. Considering that the relative distance between limit conditions falls below $1\%$, it was concluded that the cables are not close to being a primary source of deformation and thus assume a secondary role in the structural system. This observation was found to be compatible with the purpose with which cables were installed i.e. to ensure structural stability also in case the surrounding mass were to fail locally. Through a simple calculation it was in fact verified that, from the point of vertical equilibrium and
removing all rock from the problem, the vertical cable forces would be able to carry almost six times the weight of the concrete lining. From a practical point of view, given their lower impact, the geometry could have been modelled using only one average length and inclination for all cables; furthermore, accounting for the loss in initial stresses due to settlements could have been avoided. This first conclusion depends directly on the modelling procedure and assumptions it was derived from. Specifically, it could be argued that having displacements be continuous across the rock-leca and leca-concrete boundaries might reduce the validity of said statement. While it is certainly true that using a non-linear model with these interfaces modelled through a friction-based formulation could be closer to reality, it should be considered that the layer of leca was given a relevant over-break and that a very low stiffness was assigned to it; it was believed that a combination of these factors would greatly improve the conservative margin of the analysis.

The second conclusion was that, from the structural point of view, the concrete structures do not behave in any way similarly to a cable suspended structure such as a cable stayed bridge. Had the concrete elements been behaving in such a way, they would have necessarily displayed a much higher sensitivity to the action of the cables. The main responsible for disproving this analogy was found in the loading conditions: the central segment in Figure 4.38 is subject to a positive vertical displacement which is caused by the rock expansion. As the road tunnel is excavated, the rock is given the opportunity to relax its stresses by expanding and thus reducing the size of the opening; when this is coupled with high horizontal stresses, the road tunnel cross-section becomes primarily compressed from the sides and expands in the vertical direction. In those segments where the utility tunnel crosses the road tunnels, the concrete lining is pushed by all directions upwards; the conclusion is that effect of the cables is to pull upwards an element that is already pressed against the rock above. This loading condition is schematically displayed in Figure 4.43.

Observing that the concrete lining deformation displays low sensitivity to the action of the cables was also crucial to raise the question of whether in turn, the cable forces would be sensitive to the deformation of the tunnel; this research question, incredibly important from the point of view of monitoring, is addressed in Section 3.5.3.

4.6.3 Sensitivity of cable forces

Following the same presentation style adopted in all studies concerning cable force, Figures 4.39 and 4.40 display cable stresses before and after the localised reduction of concrete stiffness; the x-coordinate of the cables is used to position dots in the graphs. The total displacement field of the lining in step 6 is displayed in Figure 4.42. The intersection between utility tunnel and road tunnel is showcased in Figure 4.41.

Analysis

Cable stresses from Reduce $E_c$ present minor variations compared to those of step Excavate RT. As a result of the lower concrete stiffness, cable stresses tend to lower in the central part of the lining and slightly increase in the outer parts. The maximum variation for the cables on the right wall is approximately $-4.50$ MPa and $-0.80$ MPa for the left wall. Since the road tunnel meets the utility tunnel at a horizontal angle (see Figure 3.17), cables on the left wall react to the stiffness reduction between $x = 108$ m and $x = 115$ m while those on the right wall react mostly between $x = 112$ m and $x = 123$ m. Cables anchored to the right wall appear to be much more sensitive to

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4Each dot represents a cable and is positioned at the x-coordinate of the cable it refers to.
5Each dot represents a cable and is positioned at the x-coordinate of the cable it refers to.
4.6. Cable forces study

Figure 4.39: Cable stress $\sigma_n$ before and after concrete stiffness reduction, cables on the right wall of UT-2$^4$

Figure 4.40: Cable stress $\sigma_n$ before and after concrete stiffness reduction, cables on the left wall of UT-2$^5$

the introduced change.

Figure 4.42 highlights how deformations take place almost entirely along the boundary where the concrete lining stops being supported by rock. Similar results were observed on the other concrete wall.


**Discussion**

Demonstrating full compatibility between the statements, these results highlight that just like cable forces have low impact on the deformation of the concrete element, a localised change in element properties produces small effects on the cable forces. Once more disproving the assumption of cable-stayed-like behaviour, an explanation for this was retrieved in the loading conditions of the structure: with the rock domain expanding and pushing the lining against the rock above from all sides, increasing the compliance of areas in the floor and even walls has little to no effects on the cable anchors as they are located close to the top. This consideration is schematically presented in Figure 4.43 where the rock is exemplified by the rigid element on top of the cross-section.

Similar conclusions can also be reached considering the displacement field presented in Figure 4.42: despite the reduction in stiffness of such a large portion of concrete, the deformation is located mostly within said area and is highly concentrated along its boundaries. Deformations are not propagating to all the structure because with a thickness of only 30 cm, the lining is very far from
being a stiff element compared to the surrounding rock mass. Additionally, for any deformation to take place in the tunnel, it must first take place in the stiff rock. Deformations are localised around the boundary of this area because there the rock has a free surface: the rock stress field starts from zero at the free surface of the road tunnel and rapidly increases with depth; when the stiffness opposed by concrete is lowered, the rock gains additional possibility to expand close to its free surface. As we move away from the rock surface however, no external loads are applied to the free hanging part of lining which does not deform more.

Among the cables on the right wall, the one located at \( x \approx 118 \, \text{m} \) appeared to be the most sensitive to the introduced variation. Reasons for this can be located in the positioning of its anchor: among all cables, this is the only one to be anchored inside the more compliant part of the wall which makes its anchoring plate settle.

Though it is true that cables were found to react to the lower concrete stiffness, this was observed to hardly translate to any practical application. In contrast with the conceptual finding of this study, the variations recorded in the cables proved to be insufficiently significant to be recorded \emph{in situ}. Considering for example the classical theory of a wire pinned at both ends and subject to a certain tension \( F \), the first eigenfrequency can be estimated as in Equation (4.1), where \( L \) is the cable length and \( M \) is the weight per unit length.

\[
f_1 = \frac{1}{2L} \sqrt{\frac{F}{M}}
\]  

(4.1)
If as for the cables of intersection 2 the numerical values listed in Table 4.8 are used, the first eigenfrequency of the wire is derived as $f_1 = 29.57 \text{ Hz}$. Supposing instead a loss in cable pull of 5 MPa and using the corresponding tensile force $F_2 \approx 493 \text{ kN}$, the first eigenfrequency is computed as $f_2 = 29.51 \text{ MPa}$. In relative terms, a reduction of 0.40% in cable force translates to a variation of 0.20% in frequency. The formula of Equation (4.1) is highly simplified but it helps to shows how if differences in cable force are small, differences in frequency are even smaller.

The main findings and conclusions drawn from this study are listed as follows:

- Cable forces show a low degree of sensitivity to changes in the structural behaviour of the concrete elements (especially floor and walls) which degradation might induce. Relevant variations are recorded only if the stiffness surrounding the cable anchors is reduced.

- Even if high precision instruments were used to measure cable forces, meaningful variations would only be caused by extensive changes in the structural behaviour of the tunnel thus removing the option of capturing initial symptoms of a larger problem to come.

- Monitoring cable forces is not an effective monitoring strategy and is unsuitable to cover the role of early warning system.
4.7 Leca stiffness study

For the two considered values of leca stiffness, the mode of deformation of the concrete lining was identified by extracting results from the three lines highlighted in Figure 4.44, plus an additional line running along the inner surface of the ceiling. Results are only presented for the study of intersection 1. Although intersection 2 was also subject to the same type of analysis, results were not reported since the same conclusions were reached.

![Figure 4.44: Left, centre and right lines for result extraction](image)

4.7.1 Intersection 1

In the following figures, the displacement $w_5(x)$ is presented. Only step 5 (excavate road tunnel) was considered since it is the main cause of deformation for the concrete lining. For the lower left and lower right lines also the horizontal transverse displacement was plotted. A contour plot of the total displacement over the deformed shape of the lining is visible in Figures 4.51 and 4.51 which were directly obtained from Comsol. Similar conclusions were reached considering the other components of displacement.

4.7.2 Analysis

Despite the changes in shape of the displacement curves, a variation by a factor of 3 in the stiffness of leca does not produce any change in order of magnitude for the displacements of the lining.

The vertical displacement along the central line shows a different behaviour depending on the leca stiffness. Some degree of similarity can be observed at $x = 115$ m where both models present a point of inflection. Apart from this detail, the two graphs cannot be correlated by a constant factor. The two peaks observed for the central line are due to the shearing motion the lining is subjected to. While the first peak is caused by the rock expanding in the negative direction of the $y$-axis, and is therefore recorded by the plot in Figure 4.48, the second peak is caused by the rock expanding in the positive direction of the $y$-axis and is recorded in Figure 4.46; this shearing mode is clearly visible in the contour plot of Figure 4.51.
4. RESULTS AND DISCUSSION

\[ w_5(x) \] [m]

\[ \begin{align*}
E_l &= 3 \times 10^3 \text{MPa} \\
E_l &= 3 \text{ MPa}
\end{align*} \]

Figure 4.45: Displacement \( w_5 \) from the middle line of the UT-1B floor lining

\[ w_5(x) \] [m]

\[ \begin{align*}
E_l &= 3 \times 10^3 \text{MPa} \\
E_l &= 3 \text{ MPa}
\end{align*} \]

Figure 4.46: Displacement \( w_5 \) from lower right corner of the UT-1B lining
4.7. Leca stiffness study

Figure 4.47: Displacement $v_5$ from lower right corner of the UT-1B lining

Figure 4.48: Displacement $w_5$ from lower left corner of the UT-1B lining
4. Results and Discussion

Figure 4.49: Displacement $v_5$ from lower left corner of the UT-1B lining

Figure 4.50: Displacement $w_5$ from the roof of the UT-1B lining
Figure 4.51: Deformed shape of the concrete lining with contour plot of total displacement

Figure 4.52: Deformed shape of the concrete lining with contour plot of total displacement
The vertical displacements along the left and right lines is subject to the same considerations. A small degree of resemblance between the stiff and soft leca plots can be identified in terms of inflection points, but there is no proportionality. Different observations can be made regarding the transverse horizontal displacement $v_5(x)$: in Figures 4.47 and 4.49, stiff and soft leca present the same behaviour albeit with different amplitudes. The displacement on the ceiling of the concrete lining displays only minor effects coming from the change in leca stiffness.

![Deformed shape of leca layer](image)

(a) $E_I = 3$ MPa

(b) $E_I = 3$ GPa

Figure 4.53: Deformed shape of leca layer, total displacement (equal magnification factors)

### 4.7.3 Discussion

The expectation before running this study was that the displacement field would simply vary in amplitude while maintaining the same overall shape. This hypothesis was clearly disproved by the results, proving once again the level of complexity of the structural behaviour.

All differences in the mode of deformation are due to the geometry and to the relative stiffness between the elements: when the stiffness of leca is drastically reduced, this intermediate layer is able to incorporate much more deformation. This phenomenon is visible in Figure 4.53: with soft leca, the wall and slab connection is exemplified by a hinge; even though both wall and slab are highly compressed, there is little effect of one over the other. For the case with stiff leca, the connection wall-to-floor is more similar to that of two orthogonal slabs fixed together: while it is true that both wall and floor deform less, the bending of the wall produces an upwards bending
4.7. Leca stiffness study

in the floor which is recorded in the graph of Figure 4.48. This difference in stiffness and thus interaction between wall and floor slab was identified to be the primary reason for the different deformation modes of the lining: given a similar behaviour for the transverse displacement $u_5(x)$, dissimilar responses are obtained for $w_5(x)$.

Considering the limited knowledge available on the material properties and site conditions, dealing in terms of limit cases was thought to be the most conservative approach. The main uncertainties for the behaviour of the intermediate leca layer are its stiffness and the interface properties of the leca-concrete and leca-rock surfaces; with these and an analysis of elastic type, it is arduous to state which behaviour is more representative of the physical reality as this is likely to fall somewhere in between. For example, still considering the corner in Figure 4.53, while it is true that the rock expansion compresses the leca layer in the normal $y$-direction (and there would thus be no reasons to reduce the stiffness modulus to account for cracks etc.), there is also a shearing component across the lower leca-rock surface which would in turn call for a reduction in stiffness.

This study was highly useful in understanding the basic role that this intermediate layer plays for the structural behaviour of the concrete element. Having reached the limit of what a linear elastic study is able to provide, more insight could be archived through a non-linear analysis of the rock-leca surface.
4. Results and Discussion

4.8 Concrete cracking analysis

In this section, the main results from the study conducted on the cracking of concrete are presented. While for intersection 2 both model predictions and data recorded on site are displayed, for the reasons mentioned in Section 3.7 only model predictions are presented for intersection 1. Predicted cracking hotspots were identified in intersection 2 by studying the diagram of the first principal stress and comparing with the Drucker-Prager limit surface for concrete. For intersection 1, only the first principal stress is presented to lower the amount of results to be presented. For the second intersection, the points outside the Drucker-Prager domain are also presented for the case of soft leca discussed in Section 3.6.

4.8.1 Intersection 1

Lining UT-1A

Figures 4.54 and 4.55 display the contour plots for sp1 on the surface of the left and right walls respectively. The upper limit of the contour plots is lowered to 20 MPa to ease the identification of possible hotspots. Even though sp1 was found to be positive across most of the surfaces, the model produced stresses lower than 1 MPa for \( x < 80 \) m making this lining segment of lower interest than segment 1B.

![Surface: First principal stress (N/m²)](image)

Figure 4.54: UT-1A left wall, contour plot sp1
4.8. Concrete cracking analysis

Figure 4.55: UT-1A right wall, contour plot sp1
4. Results and Discussion

Lining UT-1B

For the left wall of lining 1B the contour plot and orientation of the first principal stress sp1 are displayed by Figures 4.56 and 4.57. Figures 4.58 and 4.59 present instead the results for the right wall of the lining. To ease the read, the upper limit of both contour plots has been moved to 20 MPa and the lower limit increased to 5 MPa.

![Contour plot sp1 for UT-1B left wall](image)

Figure 4.56: UT-1B left wall, contour plot sp1

![Direction of sp1 for UT-1B left wall](image)

Figure 4.57: UT-1B left wall, direction of sp1
4.8. Concrete cracking analysis

Figure 4.58: UT-1B right wall, contour plot sp1

Figure 4.59: UT-1B right wall, direction of sp1
4.8.2 Intersection 2

In this section all relevant results for the study of concrete in intersection 2 are reported. For both left and right walls of the concrete lining, the contour plot of the first principal stress, its orientation and the plot of the points outside the elastic domain according to Drucker-Prager are presented. In those plots where the Drucker-Pragher criterion has been used, dark blue represents points falling within the elastic domain while red is used for those falling outside it.

Figure 4.60: UT-2 left wall, contour plot sp1

Figure 4.61: UT-2 left wall, direction of sp1

Figure 4.62: UT-2 left wall, Drucker-Prager contour plot
4.8. Concrete cracking analysis

Figure 4.63: UT-2 right wall, contour plot sp1

Figure 4.64: UT-2 right wall, direction of sp1

Figure 4.65: UT-2 right wall, Drucker-Prager contour plot
Figure 4.66: UT-2 floor slab, contour plot of sp1
Comparison with soft leca solution

As discussed in Section 3.6, the intermediate layer of leca does not have structural purposes but has a strong influence on the structural behaviour of the concrete lining. To account for the high uncertainties surrounding its material properties, limit conditions were presented in Section 4.7 for the deformation induced in the lining. In the following figures, a comparison between the two limit cases is presented in terms of the Drucker-Prager criterion. In Figure 4.67 the comparison is made for the left wall, the right wall is presented in Figure 4.68. The reference system is the same employed in the other results of this section and, as previously mentioned, red is used to identify those points falling outside the elastic domain. For simplicity, results for the soft leca were extracted from the model of Section 3.6; this means that no cable anchors openings were accounted for.

![Figure 4.67: UT-2 left wall, Drucker-Prager for stiff and soft leca](image1)

![Figure 4.68: UT-2 right wall, Drucker-Prager for stiff and soft leca](image2)

Comparison with cracks recorded on site

Figures 4.69 and 4.70 display the comparison between the cracks observed on site and the model prediction using Drucker-Prager. The cracks recorded in situ are drawn drawn in red on top of the model geometry.
4. RESULTS AND DISCUSSION

Figure 4.69: Cracking hotspots predicted by the model and *in situ* cracks, left wall of UT-2

112
Figure 4.70: Cracking hotspots predicted by the model and *in situ* cracks, right wall of UT-2
4. RESULTS AND DISCUSSION

Figure 4.71: Horizontal crack recorded between cable L4 and L7 (left wall)

Figure 4.72: Vertical crack recorded below cable L5 (left wall)
4.8. Concrete cracking analysis

Figure 4.73: Distributed cracking recorded between R10 and R11 (right wall)

4.8.3 Analysis

Lining UT-1A

Of all the considered concrete linings, 1A was found to display the lowest state of stress. While the segment beneath road tunnel UT-1A displays low stresses, these increase close to the lining segment UT-1B. Because of the shearing mode of deformation, stresses concentrate in the lower corner of the left wall and in the top corner of the right wall in correspondence with the connection with lining 1B. Other than these corners, cracking may take place along the height of the walls within about 5 m from lining 1B.

Lining UT-1B

Two major cracking hotspots are predicted by the model on the left wall: the lower corner between \( x = 100 \text{ m} \) and \( x = 107 \text{ m} \) and an area in low part of the wall from \( x = 110 \text{ m} \) to \( x = 120 \text{ m} \). Some cracking may also appear in the middle-to-high part of the left wall at its two ends. Assuming the first principal stress \( sp1 \) to govern the cracking direction, cracks are expected to develop with a small positive inclination\(^6\) to the horizontal plane in the first hotspot, and with a small negative inclination in the second hotspot.

In the right wall three hotspots were identified from the model: one in the lower corner at the end of the wall \((x = 125 \text{ m})\), one at middle height of the wall between \( x = 104 \text{ m} \) and \( x = 115 \text{ m} \) and one on the top corner at \( x = 100 \text{ m} \). Cracks may also develop across the whole wall height near the connection with the 1A lining. Cracks are expected to develop with a small negative inclination\(^7\) with respect to the x-axis in the first hotspot and with a small positive inclination in the other two hotspots.

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\(^6\)Positive angle assumed from x-axis to z-axis for simplicity
\(^7\)Positive angle assumed from x-axis to z-axis for simplicity
4. Results and Discussion

**Lining UT-2**

**Left wall**  Three hotspots were individuated studying the sp1 contour plot of the left wall in intersection 2. The first hotspot is localised along the upper corner at the beginning of the wall and surrounds the top part of the openings allocating the anchors. The second hotspots was identified between $x = 100 \text{ m}$ and $x = 114 \text{ m}$ in the central portion of the wall; stress concentrations were here observed around the corners of the anchor slots. The third area of interest was found in the lower corner of the wall from $x = 123 \text{ m}$ to the end of the lining. Stress concentrations were observed near the corners of several openings hosting the cable anchors. Looking at the direction of sp1, the orientation of the cracks was predicted to be mostly horizontal for all three hotspots.

The contour plot using Drucker-Prager’s criterion of Figure 4.62 highlights one continuous area where points fall outside the elastic domain boundaries. All three hotspots predicted studying the diagram of sp1 are included in this area. The arch shape visible in the central part of the wall is compatible with the line given by the intersection between the lining and the ceiling of the road tunnel.

On site several cracks were identified and recorded. In Figure 4.69 cracks may be collected into three main groups: from cable L2 to cable L4, from L5 to L8 and between L10 and L11. Most of the cracks were observed to develop in the vertical direction starting from either the lower left or lower right corner of the openings allocating the cable anchors; these cracks reached down to the floor slab. Three cracks inclined at around 45°were recorded starting from the top left corners of the openings surrounding cable L6 and ending at the top of the wall. One continuous horizontal crack starting between cables L4 and L5 and ending just before cable L7 was also observed. As visible in Figure 4.71, this horizontal crack was found to develop along a line separating two different colors of concrete; it is possible that this crack developed across a weak plane resulting from the casting of two separate concrete batches. This line, separating two shades of color, was found to continue until the end of the concrete lining without showing any other open crack. Overall, all cracks were found to have minor mouth openings which were estimated to be smaller than 0.1 mm.

Comparing in Figure 4.67 the results using two different values of leca stiffness, some similarities are noticeable. For both cases, a hotspot is predicted in the first segment of the wall although its surface, for soft leca, is smaller. Similarities are also observed for the lower part of the wall near the end of the lining where both models predict cracking. Major differences are visible in the central segment of the wall where the arch-shaped zone predicted by the stiffer model is entirely absent in the other.

**Right wall**  Four hotspots for cracking were identified in the right wall from the study of sp1. The first hotspot was identified near the upper corner of the wall near $x = 129 \text{ m}$ including also the top part of the openings hosting the anchors. The second hotspot was located around $x = 120 \text{ m}$ where stress concentrations were observed near the top right corners of the anchor slots. Another hotspot was found between $x = 110 \text{ m}$ and $x = 112 \text{ m}$ having an inclined shape compatible with the shape of the road tunnel’s roof. The final hotspot was instead found along the lower corner of the utility tunnel from $x = 100 \text{ m}$ to $x = 109 \text{ m}$. According to the orientation of sp1, cracks were expected to propagate horizontally in the first hotspot, at an angle of about -45°starting from the corners in the second hotspot, with a small negative inclination in the third hotspot and horizontally in the fourth area of interest.

Compared to the left wall, the use of Drucker-Prager’s criterion revealed in this case more interesting information. Although all hotspots identified looking at sp1 were observed to fall in the
non-elastic areas predicted by the criterion, the elastic domain excluded many other parts of the surface. Specifically, Drucker-Prager predicted cracking for all the middle to top surface of the wall from \( x = 110 \text{ m} \) to \( x = 129 \text{ m} \), and also predicted that the fourth cracking hotspot would extend up to the anchor openings.

As can be seen in Figure 4.70, several cracks were recorded on site. Three vertical cracks were observed to develop from the lower corners of the anchor openings near cable R2; these cracks reached down to the floor slab. One more vertical crack was located between cables R5 and R6 following the same behaviour. Near the anchor of cable R7, one single horizontal crack propagating for a few centimetres was recorded. Running between cables R8 and R9, two cracks were found to have an arched shape linking the top corners of the anchor openings. Near the end of the concrete lining, a horizontal crack was open between cable R10 and cable R11 stretching from the side of the cable openings. Below the same cables, towards the lower part of the wall, an extensive pattern of small cracks was found. A picture of this is presented in Figure 4.74 where the green pigmentation is due to algae; it should also be noted that had algae not been present, these cracks might not have been spotted because of their small aperture. Similarly to the left wall, all cracks were found to have small mouth openings.

When softer leca is employed the elastic areas according to Drucker-Prager’s criterion increase. As visible in Figure 4.68, the more compliant model still predicts a continuous cracking hotspot along the upper portion of the right wall although to a lower extent. The red area in the central segment of the wall for stiff leca is not captured when soft leca is used. Despite a smaller extension, both stiff and compliant models predict a cracking hotspot in the lower wall along the initial lining segment (smaller values of \( x \)). In the results from the compliant model, the isolated spots in the upper part of the wall are due to the cable anchors: in this model, geometrical openings were not created to accommodate the anchors therefore the rigid connector is fully embedded in the lining; for this reason, cable forces generate tensile stresses in a volume that is not present in the physical reality. These apparent hotspots should be disregarded.

**Floor slab** No cracks could be observed on site because of the dust covering the floor slab. From the study of sp1, two main hotspots for cracking were located: one at \( x = 110 \text{ m} \) and the second one at the ending of the concrete slab. The first hotspot coincides with the intersection between the floor slab and the roof of the road tunnel. Additionally, stress concentration was observed for the initial segment of the floor slab along the connection with the right wall. Because of its very limited size, cracking is here expected to take place only in the very corner.

**4.8.4 Discussion**

**Lining UT-1A and 1B**

For the reasons mentioned in Section 3.7, cracks were researched on site and recorded only for the concrete lining of intersection 2; as such, no discussion on the validation of the intersection 1 model could be had. The scale in the contour plots for linings 1A and 1B was given a lower limit of 5 MPa and an upper limit of 20 MPa to highlight possible hotspots. Had the lower limit been moved down to \( f_{ctk} \) for example, many more areas would have appeared to be subject to tensile stresses above the material capacity. The scale limits were willingly set this way since only minor relevance was given to the numerical values: with an inadequate mesh for a stress analysis, it was thought that only the shape of the stress diagrams could be partially relied upon. Under this assumption, were a finer mesh employed for this analysis, stresses would dramatically reduce in amplitude but still present their peak values in the same areas identified as hotspots.
For both walls, it is relevant to notice that the arch-shaped hotspot follows the perimeter drawn by the roof of the road tunnel when it intersects the concrete lining. Cracks on site were to be recorded, this information could be used for example to adjust the measure of overbreak assigned in the geometry.

**Lining UT-2**

While in non-linear models the solution depends on the previous history of deformation and is updated at every step, the solution of an elastic analysis is unique. The inability of the linear-elastic model used in this study to account for non-linear behaviours was the main limitation. Although relevant information can still be archived from an elastic model, the reader must be aware of the limits this entails. A linear FEM model is able to predict where the first cracks are likely to appear but is incapable to produce information on where the first crack will open let alone what will happen following this event. As soon as a crack opens in a material that is subject to a certain set of forces, energy is dissipated, stresses are released and the entire state of stress in the element readjusts. As a result of this, it is possible that areas where cracking was expected unload while other points experience a stress increase. For the evolution of a cracking scenario to be captured, a non-linear model should be implemented. Given these limitations, the topic of model validation was here considered only in terms of comparing where cracks were expected to develop and where cracks actually developed in the lining.

**Left wall**  The left wall appears to be subject to a rather high state of stress as demonstrated by the number of cracks observed on site. The fact that each vertical crack is substantially separated in space from the following one and all initiate from a corner of the anchor openings would suggest a mechanical cause to be behind them, as opposed to chemical or thermal origin. Additionally, displaying all the same pattern (orientation and length) despite the different locations along the wall, one same mechanism may be behind their formation. The first assumption would be that the cables are responsible for the crack formation: although it is true that additional vertical bars reinforce the area surrounding the anchors, it seems unlikely that this would produce vertical cracks. Even assuming these rebars to transfer the tensile force from the anchors down to the floor slab, several inclined cracks, similar to those obtained in pull-out tests, would be visible.

The reason why the model predicts horizontal cracks to form might be due to the behaviour of the intermediate layer of leca: when a fully linear model is used, the intermediate layer transfers deformations coming from the rock both in the normal and parallel directions to the concrete lining walls. Had leca-rock contact problem been modelled in a non-linear manner, transverse deformations would have induced relative displacements while normal deformations would have been transferred to the lining. If this sort of behaviour was to be demonstrated, the direction of the principal stresses might be very different. A second reason motivating this discrepancy might be that with such a 3-dimensional stress state looking solely at the orientation of the first principal stress is insufficient for predicting cracking phenomena. Interestingly, the second principal stress sp2 was found to be in tension and mostly horizontally oriented; the orientation of the cracks may be governed by both first and second principal stresses.

One case where model and reality were found to agree is that of the horizontal crack running between cables L5 and L7 just above the anchors. As mentioned in the analysis section, this crack was opened along a weak surface possibly created by discontinuous casts of concrete. Given a weak surface, the tendency of cracks is to propagate along the weak plane even when this is at an angle with the principal stress; this statement indicates that this crack, given the same stress state, might develop horizontally even though in a homogeneous material it would develop at an angle to the horizontal plane. The previous reasoning would lower the validity to the assumption
of similar stress orientations between model and reality if it were not for the fact that this weak plane continues until the end of the lining without displaying any other cracks. This fact is highly important seen as it provides a clear element of comparison model-reality: the weak plane is cracked where the models expects cracking while it is not cracked where the models does not predict it to be. Additionally, the crack changes its orientation and aims for the ceiling in the area where the model stresses also change orientation (see Figures 4.61 and 4.69).

The finite elements model was able to capture stress concentrations at the corners of the anchor openings. What the model was only able to capture at times was which of the corners would be responsible for the crack initiation: in certain cases cracks were found in the corner where the model was predicting no stress concentration. Considering how the crack formation process is governed by defects, relevance was not given to this fact.

From the study of the principal stress contour plot in Figure 4.60 information may be archived on the overbreak of the tunnels: the central hotspot in the model appears to be slightly left of the area where cracks develop on site. Although this difference is not visible using Drucker-Prager’s criterion, this might indicate that the chosen overbreaks were too conservative; had the analysis been run with a smaller overbreak, the perimeter of the ceiling from the road tunnel would have been more to the right, possibly giving a better match with reality. Nevertheless it should be noted that all these areas are likely to crack so that it might not be true that the centre of the hotspot should match the centre of the on site crack distribution.

Comparisons between model predictions and physical reality were made in qualitative terms which made any quantification of the model validation rather difficult. Overall, hotspots predicted by the model were found to be in agreement with reality in the sense that no group of cracks was recorded outside a predicted hotspot (see area from cables L8 to L9 in Figure 4.69 for example). For these reasons, the model was qualitatively validated against the physical case.

**Right wall** Most of the points discussed for the left wall were found to apply to the right one as well. The vertical cracks observed on the right wall may have been caused by a similar mechanism.
to that discussed for the opposite wall. Vertical cracks near cable R2 were found to match very well the hotspots in Figure 4.70 predicted by the model; unfortunately no correspondence was found for the crack orientation. As discussed for the other wall, this difference was linked to the insufficiently accurate modelling of the behaviour of the intermediate leca layer.

Differently from the left side, map cracking between cables R9 and R11 was here observed (see Figure 4.74); the causes behind this system of cracks could be mechanical, thermal or chemical. The use of reactive aggregates in a concrete mix is often the cause of distributed cracking; for the present case however, this possibility seems rather unlikely seen as the same aggregates would have been employed for the whole concrete batch and this cracking pattern would have appeared all across the lining. Drying shrinkage is also a mechanism responsible for map cracking of concrete surfaces; in the present case however, it seems unlikely that drying shrinkage would affect only a limited portion of lining. The hypothesis of having mechanical causes behind these cracks is also mildly grounded: although it is true that the finite elements model predicts cracking to develop in the same area, such an erratic distribution seems incompatible with a well defined stress state. It was concluded that further studies should be conducted to clearly identify the origin of these cracks.

Very good correspondence was observed for the cracks between cables R10 and R11 where also the crack orientation matches the prediction. Differently from most other cracks, which were observed to initiate from the corners of the anchor openings, these cracks developed from the sides; in absence of additional information, without specific stress concentrations predicted by the model, these cracks may have been initiated by defects in concrete.

Cracks between cables R8 and R9 were found to match rather well model predictions where stress concentrations could be found around the top corners of the anchor openings. What is unclear is the reasoning behind the specific arch-like shape they displayed: such a symmetric pattern could be expected had an anchor been present also in the opening between cables R8 and R9; this opening is however empty and not subject to any cable forces. Initiating because of stress concentrations, these cracks may then have propagated following a weak plane such as that located in the left wall.

The analysis for the limit values of leca highlighted very different results for cracking hotspots. Although both models predicted cracking hotspots along the top and bottom corners of the right wall, the soft leca model could not account for any of the horizontal cracks and did not foresee hotspots in the central parts of the wall. Comparing results with the data collected on site, the hard leca model was found to give a better fit and be closer to reality.

Overall, similarly to the conclusion reached for the left side, the finite elements model demonstrated several similarities to the physical case so that qualitatively, a certain degree of validation was archived. This statement could be corroborated by refining the material models used for leca and concrete and comparing results.
5.1 Structural behaviour

Drawing directly from the initial research questions, significant conclusions could be archived by this study. Through the analyses performed, a good qualitative understanding was archived for the deformation of the structures supporting the utility tunnel and for the role played by the single elements. Concerning the structural behaviour of the system the following conclusions were reached:

- The loading conditions provided by the expanding rock is highly complex and 3-dimensional. Because of this, no simpler models, such as that of a beam, may be used to correctly represent the deformation of the concrete lining. Although the mode of deformation may be interpreted as coupling between a principal mode in shear and a principal mode in bending, this only stands qualitatively. The heterogeneous pressure field exerted by the rock could not be reproduced on a simplified beam structure.

- For the segments supported by steel cables, the concrete lining should not be seen as cable supported structure as this would lead to erroneous assumptions. The concrete lining is almost entirely clamped by the surrounding gneiss mass and at the same time compressed and pushed upwards against the above rock; because of this, cables do not affect the displacements of the lining and at the same time are not sensitive to local structural changes happening in concrete.

- The intermediate layer of leca, separating rock and concrete has direct influence on the deformation of the lining. Although it was demonstrated that the order of magnitude of the lining displacements is likely accurate, the characteristics of leca have the power to affect the deformed shape of the concrete element.

- The concrete lining is subject to a significant state of stress often surpassing the tensile strength of concrete. This demonstrated by the cracks of mechanical origin identified on site. Most of the cracks were found to be initiated by stress concentrations at the corners of the openings hosting the cable anchors. Even tough site visits focused on identifying cracks for the lining of the second intersection, given the stress amplitudes predicted by the model of intersection 1, numerous cracks are expected also in lining UT-1B.

- Despite using a linear elastic model, a good level of correlation was observed between the concrete cracks recorded on site and the cracking hotspots predicted by the models.
5. Conclusions

Although this study limited itself to the use of linear elastic models, non-linearities could provide higher understanding as well as additional corroboration for the results archived. Specifically, non-linear material models should be introduced for concrete as well as leca with the objective of studying the predicted cracking patterns as well as assessing the effect of leca on the deformation of the lining. For this last purpose, a contact model based on friction could also be implemented for the surfaces between rock and leca.

5.2 Monitoring

Starting from the understanding and knowledge gained on the structural behaviour of the problem, the effectiveness of different monitoring strategies could be quantified. Within the field of monitoring, the following conclusions were reached:

- The concrete lining is not subject to dynamic loads and the level of restraining generated by the rock makes monitoring systems based on dynamic variables unsuitable. Monitoring systems should focus on quasi-static variables such as displacements and strains.

- The post-tensioned steel cables should not be the object of monitoring. Cable forces would provide information only in case significant movements were to take place in the rock mass; even then, this would be in contrast with the purpose of a monitoring system, that is to act as an early-warning system sensitive to small changes and able to predict any failure of rock masses. Monitoring cable forces would likely account solely for the corrosion of the steel tendons caused by the humidity in the environment.

- Monitoring plans should focus on the concrete lining. This operation would be the more facilitated by the presence of already developed cracks signalling possible areas of interest.

With the information provided by the models and the cracks located on site, monitoring efforts should be directed towards these areas; specifically, attention should be placed to the cracks surrounding the free-standing segments of the lining. For the left wall of lining UT-2, the area between cables L5 and L7 in Figure 4.69 should be carefully studied, for the opposite wall instead focus should be placed around cable R6 and between cables R8 and R9 in Figure 4.70. While any damage in the initial and ending portions of the lining would only likely affect the lining, issues in the central segment might lead to dangers for the traffic circulating just below the utility tunnel.

Monitoring could be conducted through visual inspections carried out with a given frequency, or through the installation of instruments. At this stage, the use of visual inspections may be justified by the small width of the cracks as well as by the absence of significant load variations over time. Were instrumentations to be installed, the finite elements model should be improved to account for non-linearities especially in the intermediate layer of leca; a higher definition in the model would allow to identify more accurately the critical areas for the structural integrity of the lining. Instrumentations such as extensometers may be installed across cracks to record their evolution over time; were the state of stress in the lining to change or some mechanism of degradation to take place, it is likely that deformations would take advantage of the weaknesses generated by the already developed cracks and increase their width.

Even though investigations could not be carried out on site for the concrete floor slab and as such no cracks were identified, this structural element should be carefully considered. From the model, the lines along which the ceiling of the road tunnel meets the utility tunnel were found to be characterised by high stress states which would indicate possible cracking. Since it is at this
stage unclear how sensitive the cracks on the walls would be to changes in the floor slab, it is recommended that monitoring efforts take also measures from the floor into account.

5.3 Degradation

In absence of dynamic loads or moving loads on the concrete lining as well as no foreseeable changes in the state of stress of the system, degradation is likely to be the only factor affecting the structural health of the supporting structures. With a long-term optic, degradation mechanisms may reduce the integrity of structural elements and lead to failure for parts of the system. The main degradation mechanisms identified in the scope of this work are the following:

**Cables corrosion** The high levels of relative humidity on site may induce corrosion in the steel cables. From site inspections, corrosion processes were observed to be active on several cables. From the studies conducted, a reduction in the structural performance of cables is expected to only affect the concrete lining in the small vicinity of the anchors; while no major effects are expected on the deformation of the lining, this would lower its ability to resist in case the rock mass were to fail locally. Although this possibility has low probabilities of happening, it would be important to ensure that corrosion does not reach dangerous levels for the cables.

**Rebars corrosion** The concrete lining is reinforced by a low amount of steel rebars forming an orthotropic mesh. Corrosion of rebars may be a major concern for the serviceability of the utility tunnel as well as for the safety of the traffic in the road tunnels. For the conditions on site, corrosion may be induced by leaching, high humidity levels as well as by the process of carbonation which is even more facilitated by the pollutants produced by traffic [12]. The presence of open cracks in a humid environment may open the way for moisture to attack steel elements. The observed leaching phenomena would additionally indicate the presence of small water filtrations through the rock, justified by the tunnel being below the ground water table, which would additionally stimulate corrosive processes in the reinforcements. For those surfaces of concrete crossing the road tunnels and with exposure to the air, the process of carbonation may be accelerated by the pollutants concentrated in the atmosphere from the road traffic; extensive carbonation of the outer surfaces would additionally ease corrosion.

Concerning the road tunnels, corrosion of the reinforcements may lead to spalling of the concrete cover. Given that within the road tunnels the lining is shielded by a thin steel sheet, small concrete pieces are not expected to fall directly into the traffic. Concerning the utility tunnel, extensive corrosion may lead to a reduction in the bearing capacity of the lining with, under extreme conditions, possible global or local failures. Given the clamping exerted by the rock, it is possible for only local portions of the lining to reach a critical condition without stopping the operability of the utility tunnel.
In a 3-dimensional problem rigid connectors define the displacement field of a surface as function of six only kinematic variables: three displacements and three rotations. A rigid connector with flexible formulation is a functionality offered by Comsol Multiphysics where the restraint is applied on static variables rather than kinematic ones. Rigid connectors with flexible formulation allow for the deformation of the surface they are prescribed to but restrain the force field surrounding the constraint to have a linear distribution. When no geometrical non-linearities are accounted for in a problem (small displacements and small strains), the reaction force field around a rigid connector is defined by Equation (A.1).

\[ F_{rs} = F_c + F_d \cdot (X - X_c) \] (A.1)

Here, \( F_{rs} \) is the reaction field around the rigid connector in, \( F_c \) is the reaction force at the centre of rotation of the rigid surface \( X_c \) and \( F_d \) is the force gradient. While \( F_{rs} \) and \( F_c \) have the unit of a force, \( F_d \) is a force per unit length. All vectors have three entries.

Rigid connectors with flexible formulation are an effective way to create connections in Comsol while avoiding the stress concentrations produced by point connections as well as stiffness introduced by purely rigid connectors. This weak formulation of the rigid connector has however the downside of making the stiffness matrix of the finite elements problem non-symmetric which can affect significantly the computational times.
Prestressed cables geometry

Table B.1 reports the experimentally measured lengths and inclination of the prestressed cables for Intersection 1 and Intersection 2. Supposing the tunnel axis to run left to right in Figures 1.18 and 1.19, the cables are numbered backwards when looking in the positive axis direction; uneven numbers refer to the right wall and even numbers to the left.
### B. Prestressed Cables Geometry

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<th>Cable length [m]</th>
<th>Cable inclination [deg]</th>
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<tbody>
<tr>
<td>25</td>
<td>6.36</td>
<td>25.69</td>
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</table>

Average 8.96 27.59

Table B.1: Recorded information on cables inclination and length for UT-1B (left) and UT-2 (right)
Drawing inconsistencies

Dating back to 1997, all drawings were completed at the beginning of the era of modern Computer Aided Design and as such only featured the use of partially-aided computer drawing. As a result some inconsistencies were encountered in the original tables. The most relevant ones are as follow:

• For UT-2, the distance from the top of the wall to the inner face of the concrete layer supporting the roof was written to be 200 mm whereas its measure on paper was of 400 mm. Although in general terms written indications are usually to be followed when in doubt, the measure of 400 mm was observed in several other drawings involving the elevation view of the tunnel. Giving priority to the redundant information, the measure of 400 mm was adopted.

• The width of RT-2 was inconsistent between the plan view and elevation view leading to a full meter difference. Here as well the priority was given to the redundant information: the original drawing of the elevation view presented the cross-section of the road tunnel as it intersects right wall, symmetry axis and left wall of the utility tunnel. These three cross-sections proved consistent among themselves and as such kept as geometrical reference.
Matlab subroutine - create cables and anchors

%%This script generates the tendons in the geometry as well as the anchoring plates. It also generates the partitions needed.

%----------------------------------------GENERATE A PART
create_tendons = 'part50';
pret_cables = 'pret_cables';
model.geom.create(create_tendons, 'Part', 3);
model.geom(create_tendons).label(pret_cables);

create_partition = 'part51';
anch_partitions = 'anch_partitions';
model.geom.create(create_partition, 'Part', 3);
model.geom(create_partition).label(anch_partitions);

%----------------------------------------START AUTOMATED CABLE GEOMETRY GENERATION
%----------------------------------------FILL IN

%xname = 'Cables_211_1';
xrange = 'A2:B25';
b = 2.5;
t_wall = 0.3;
alpha = -1;
    deg
in_sp = 0.40;
    211−1
L = 110;
H = 90;
B = 110;
h = 2.25;
delta_roof = 0.2;

131
data = xlsread(xname, xrange); %Cable length and angle
Ncables = length(data); %Number of cables
l_sp = 2500/1000; %Longitudinal spacing [m]
t_sp = b-2*t_wall+2*0.125*cos(25*pi/180); %Transverse spacing [m]
for i = 1:Ncables
    skip = 50; %Add 50 to the index
    j = length(data)-(i-1); %Read the vector bottom to top
    type_el = 'ls'; %Geometric type
    type_par = 'parel'; %Partition type
    index = num2str(i + skip);
    ID_el = strcat(type_el, index); %ID of the geometric object
    ID_par = strcat(type_par, index); %ID of the partition
    if mod(j,2)==0
        X1 = (l_sp*(i-1)/2)+L/2+in_sp;
        Y1 = (t_sp/2)+B/2;
        Z1 = ((X1-L/2)*sin(alpha*pi/180))+H/2+h-delta_roof-0.125*sin(25*pi/180);
        X2 = X1;
        Y2 = Y1 + (data(j,1))*sin(data(j,2)*pi/180);
        Z2 = Z1 + (data(j,1))*cos(data(j,2)*pi/180);
    else
        X1 = (l_sp*(i-2)/2)+L/2+in_sp;
        Y1 = -(t_sp/2)+B/2;
        Z1 = ((X1-L/2)*sin(alpha*pi/180))+H/2+h-delta_roof-0.125*sin(25*pi/180);
        X2 = X1;
        Y2 = Y1 - (data(j,1))*sin(data(j,2)*pi/180);
        Z2 = Z1 + (data(j,1))*cos(data(j,2)*pi/180);
    end
    %Create the line segments
    model.geom(create_tendons).create(ID_el, 'LineSegment');
    model.geom(create_tendons).feature(ID_el).set('specify1', 'coord');
    model.geom(create_tendons).feature(ID_el).set('coord1', [X1 Y1 Z1]) ;
    model.geom(create_tendons).feature(ID_el).set('specify2', 'coord');
    model.geom(create_tendons).feature(ID_el).set('coord2', [X2 Y2 Z2]) ;
    model.geom(create_tendons).run(ID_el);
    %Partition for the anchorage length
    par_ratio = (data(j,1)-2.5)/data(j,1);
    model.geom(create_tendons).create(ID_par, 'PartitionEdges');
    model.geom(create_tendons).run(ID_el);
    model.geom(create_tendons).feature(ID_par).selection('edge').set(
        ID_el, 1);
    model.geom(create_tendons).run(ID_par);
    model.geom(create_tendons).feature(ID_par).setIndex('param',
        par_ratio, 0);
    model.geom(create_tendons).run(ID_par);
    model.geom(create_tendons).run(ID_par);
    model.geom(create_tendons).run;
end
%------------------------------------------END OF CREATE TENDONS

%------------------------------------------START AUTOMATED ANCHOR PLATES GENERATION

for i = 1:Ncables
    skip = 100;                   %Add 100 to the index
    j = length(data)−(i−1);     %Read the vector bottom to top

    type_el = 'r';               %Geometric type
    type_wp = 'wp';              %Workplane
    type_rot = 'rot';
    type_mov = 'mov';
    index = num2str(i + skip);
    ID_el = strcat(type_el,index);
    ID_wp = strcat(type_wp,index);
    ID_rot = strcat(type_rot,index);
    ID_mov = strcat(type_mov,index);

    %Create the anchor plates
    if mod(j,2)==0
        XI = (Lsp*(i−1)/2)+L/2+in_sp;
        YI = (t_sp/2)+B/2;
        ZI = ((XI−L/2)*sin(alpha*pi/180))+H/2+h−delta_roof−0.125*sin(25*pi/180);
        rot = (90−25); %Inclination of the anch. plates
    else
        XI = (Lsp*(i−2)/2)+L/2+in_sp;
        YI = (−t_sp/2) +B/2;
        ZI = ((XI−L/2)*sin(alpha*pi/180))+H/2+h−delta_roof−0.125*sin(25*pi/180);
        rot = −(90−25); %Inclination of the anch. plates
    end

    model.geom(create_partition).create(ID_wp, 'WorkPlane');
    model.geom(create_partition).feature(ID_wp).set('quickplane', 'zx');
    model.geom(create_partition).feature(ID_wp).set('unite', true);
    model.geom(create_partition).feature(ID_wp).geom.create(ID_el, 'Rectangle');
    model.geom(create_partition).feature(ID_wp).geom.feature(ID_el).set('pos', {'−0.15/2' '−0.15/2'});
D. MATLAB SUBROUTINE - CREATE CABLES AND ANCHORS

model.geom(create_partition).feature(ID_wp).geom.feature(ID_el).set('size', [0.15 0.15]);
model.geom(create_partition).create(ID_rot, 'Rotate');
model.geom(create_partition).feature(ID_rot).setIndex('rot', rot, 0);
model.geom(create_partition).feature(ID_rot).set('axis', [1 0 0]);
model.geom(create_partition).feature(ID_rot).selection('input').set([ID_wp]);
model.geom(create_partition).create(ID_mov, 'Move');
model.geom(create_partition).feature(ID_mov).setIndex('dispX', X1, 0);
model.geom(create_partition).feature(ID_mov).setIndex('dispY', Y1, 0);
model.geom(create_partition).feature(ID_mov).setIndex('dispZ', Z1, 0);
model.geom(create_partition).feature(ID_mov).selection('input').set([ID_rot]);
model.geom(create_partition).run;

%----------------------------------------------END OF CREATE ANCHORS
Matlab subroutine - create connections

```matlab
xname = 'ID_Connections_SMALL';
sheet = 1;
plates = xlsread(xname,1);
points = xlsread(xname,2);
Nplates = length(plates);
Npoints = length(points);
ID_solid = 'solid4';
ID_truss = 'truss';
type_rig = 'rig';
type_disp = 'disp';
type_crb = 'crb';
Name_rig = 'Rigid conn.';
Name_disp = 'Rigid conn.';
skip = 20;
for i = 1:Nplates
    index = num2str(i + skip);
    ID_rig = strcat(type_rig, index);
    label_rig = strcat(Name_rig, num2str(index));
    ID_disp = strcat(type_disp, index);
    ID_crb = strcat(type_crb, '1');
    label Disp = strcat(Name_disp, num2str(index));
```

% MATLAB SUBROUTINE - CREATE CONNECTIONS

37 model.component('comp1').physics(ID_solid).create(ID_rig, 'RigidConnector', 2);
38 model.component('comp1').physics(ID_solid).feature(ID_rig).selection.set([plates(i)]);

% UNCOMMENT TO HAVE FLEXIBLE RIGID CONNECTORS
41 model.component('comp1').physics(ID_solid).feature(ID_rig).set('ConnectionType', 'FlexibleType');
42 model.component('comp1').physics(ID_solid).feature(ID_rig).label(label_rig);
43 model.component('comp1').physics(ID_solid).feature(ID_rig).feature(ID_crb).selection.set([plates(i)]);
44 model.component('comp1').physics(ID_solid).feature(ID_rig).set('CentroidOfRotationType', 'CentroidOfSelectedEntities');

46 reference_u = strcat(ID_solid, '.', ID_rig, '.', 'u');
47 reference_v = strcat(ID_solid, '.', ID_rig, '.', 'v');
48 reference_w = strcat(ID_solid, '.', ID_rig, '.', 'w');
49 model.component('comp1').physics(ID_truss).create(ID_disp, 'Displacement0', 0);
50 model.component('comp1').physics(ID_truss).feature(ID_disp).selection.set([points(i)]);
51 model.component('comp1').physics(ID_truss).feature(ID_disp).set('Direction', [1; 1; 1]);
52 model.component('comp1').physics(ID_truss).feature(ID_disp).set('U0', [reference_u; reference_v; reference_w]);

end

% ____________________________________________________________
% |-----|-----|-----|-----|-----|
% | TO BE PASTED IN THE SECOND TRUSS SECTION |
% |-----|-----|-----|-----|-----|
% ||-----|-----|-----|-----|
% | remember to remove any pre-existing rigid connector or imposed |
% | displacement for the plate-to-truss connections |
% ||-----|-----|-----|-----|
% %__________________________________________________________

xname = 'ID_Connections_SMALL';

pl = xlsread(xname, 1);
points = xlsread(xname, 2);
Np = length(pl); % Number of plates
Np = length(points); % Number of anchor points

ID_solid = 'solid5';
ID_truss = 'truss2';
type_rig = 'rig'; \hspace{1cm} %Rigid connector type
type_disp = 'disp'; \hspace{1cm} %Imposed displ. type
type_crb = 'crb'; \hspace{1cm} %Centre of rotation
Name_rig = 'Rigid conn.';
Name Disp = 'Rigid conn.';
skip = 20;

for i = 1:Nplates

index = num2str(i + skip);
ID_rig = strcat(type_rig, index); \hspace{1cm} %ID of the rigid connector
label_rig = strcat(Name_rig, num2str(index));
ID_disp = strcat(type_disp, index); \hspace{1cm} %ID of the partition
ID_crb = strcat(type_crb, '1'); \hspace{1cm} %ID centre of rotation
label_disp = strcat(Name_disp, num2str(index));

model.component('comp1').physics(ID_solid).create(ID_rig, 'RigidConnector', 2);
model.component('comp1').physics(ID_solid).feature(ID_rig).selection.set([plates(i)]);

% UNCOMMENT TO HAVE FLEXIBLE RIGID CONNECTORS
model.component('comp1').physics(ID_solid).feature(ID_rig).set('ConnectionType', 'FlexibleType');
label_rig = strcat(Name_rig, num2str(index));
model.component('comp1').physics(ID_solid).feature(ID_rig).feature(ID_crb).selection.set([plates(i)]);
model.component('comp1').physics(ID_solid).feature(ID_rig).set('CenterOfRotationType', 'CentroidOfSelectedEntities');

reference_u = strcat(ID_solid, '.', ID_rig, '.', 'u');
reference_v = strcat(ID_solid, '.', ID_rig, '.', 'v');
reference_w = strcat(ID_solid, '.', ID_rig, '.', 'w');
model.component('comp1').physics(ID_truss).create(ID_disp, 'Displacement0', 0);
model.component('comp1').physics(ID_truss).feature(ID_disp).selection.set([points(i)]);
model.component('comp1').physics(ID_truss).feature(ID_disp).set('Direction', [1; 1; 1]);
model.component('comp1').physics(ID_truss).feature(ID_disp).set('U0', [reference_u; reference_v; reference_w]);

end


