DEEP TUNNELS IN WEAK ROCK-MASSES: ANALYSIS

OF THE GROUND-SHOTCRETE INTERACTION

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Tesi per il conseguimento del titolo

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1. Introduction and Aim of the Work

Shotcrete is the major short-term support element for tunnels excavated using conventional tunneling methods in squeezing ground conditions (i.e., weak rock-masses under high overburden stress). Shotcrete is applied to the tunnel surface immediately after the excavation and forms a continuous and flexible ring, capable of supporting the release of large ground stresses without structural damage. The confining action of shotcrete prevents the decay of the mechanical properties of the ground, thus improving the stability of the tunnel.

Shotcrete is commonly considered the central ingredient of the so-called NATM ("New Austrian Tunneling Method"). NATM tunneling is characterized by the adaptation of the construction parameters, such as driving excavation rate, shell thickness, number of rock bolts, etc., to the observed response of the already excavated part of the tunnel. These adaptations are based on monitoring the deformation of the tunnel (convergence measurements), surface settlements, and extensometer measurements in the surrounding rock mass.

While NATM tunneling is widely and successfully adopted in many different geotechnical conditions, there is still no thorough understanding of the interaction phenomena developing between the ground and the shotcrete lining in tunneling. The design of tunnel support and of tunnel excavation is usually based on empirical knowledge and on rather coarse analytical and numerical analyses. Actually, the analysis of the ground-lining interaction in tunneling is a very challenging task, because:

- tunnel excavation is a three-dimensional problem;
• neither the ground properties nor the geometric dimensions of the different rock layers can be completely explored before starting excavation;

• the final load on the lining and the final convergence of the tunnel strongly depend on the driving parameters. These parameters often change during the tunnel advance and can hardly be modeled exactly in the analytical and numerical analyses.

The shotcrete support adds further complication to the analysis of ground support interaction because it makes the whole problem time-dependent. The mechanical properties of shotcrete change gradually under loading produced by the release of the initial stresses in the ground because of the chemical reaction between cement and water (hydration). The shotcrete becomes stiffer and the strength increases as well as creep deformations develop increasing the lining compliance.

The complexity of the ground-shotcrete interaction in tunneling can not be sufficiently understood by adopting simplified methods of analysis in plane strain conditions.

The aim of this work is to offer a contribution to the understanding of the behavior of tunnels supported by a short-term lining of shotcrete applied near the face. The analysis of the ground-shotcrete interaction has been evaluated for different geotechnical conditions and for different excavation and support schemes by selecting the fundamental parameters that govern the problem.

The study of the ground-shotcrete interaction in tunneling was carried out in two steps. The first step consists in an extensive parametric numerical investigation simulating tunnel excavation and the application of the shotcrete. The second step deals with the analysis of a real case (the Raticosa tunnel) where shotcrete was applied as short-term support.

The thesis is organized in nine Chapters.

The second Chapter of the thesis describes the excavation and support methods adopted for tunneling in squeezing conditions. The diffusion of shotcrete as support element and, more in general, of NATM are evaluated in a wider historical context concerning tunneling progress of last century. Different strategies of conventional tunneling techniques are evaluated and compared and the most recent developments are presented.
Chapter 3 is dedicated to the examination of shotcrete as tunnel support with respect to technological aspects, experimental behavior, and constitutive modeling. Particular attention is devoted to the analysis of the shotcrete properties that most influence the static conditions of the tunnel. Among the shotcrete constitutive models available in the technical literature, the one developed recently at the Vienna University of Technology appears to be the most complete in describing the material behavior (see Hellmich et al. 2001 and references therein). In this model, the mechanical behavior of the shotcrete lining is described by a thermochemomechanical material model. It takes into account aging elasticity, chemical hardening, chemical shrinkage, as well as microcracking and creep in the shotcrete.

In Chapter 4 the current empirical, analytical, and numerical design methods for tunnels supported by shotcrete are described. In professional practice, tunnel design is commonly performed in 2D plane strain conditions by means of analytical methods (e.g., the Convergence-Confinement method) or numerical FE analyses. The main drawback of the plane strain analysis is the estimation of ground stress release at the time of lining installation. This estimation is made on the basis of the mechanical properties of the ground and on the tunnel face distance, disregarding the influence of the already installed lining (Panet and Guenot, 1982). A better estimation of the ground stress release at the time of lining installation is proposed in the New Implicit Method considering different ratios between ground stiffness and the stiffness of the lining (Bernaud and Rousset, 1992). So far, linear-elastic material behavior has been assumed for the lining. Also in this case, the time-dependent behavior of the shotcrete is not taken into account.

The numerical studies carried out in this thesis are reported in Chapters 5 and 6. Axisymmetric conditions are assumed with the axis of symmetry coinciding with the longitudinal axis of the tunnel. Axisymmetric conditions represent a good approximation for the simulation of the excavation of deep tunnels. Tunnel excavation and support installation are simulated by a step-by-step procedure.

In a first group of numerical analyses (Chapter 5) an elastic behavior of shotcrete, with increasing stiffness controlled by means of empirical material functions, is assumed (Boldini and Lembo-Fazio, 2001) (Cosciotti et al., 2001). The performed numerical study investigates the main aspects of the multiple interactions between the hydrating shotcrete, the tunnel
excavation rate, and the stress release in the ground. Finally, a simple strategy to enhance the capability of conventional 2D models is also proposed.

The more realistic constitutive model developed at the Vienna University of Technology is employed in Chapter 6. Results of various studies focusing on the influence of (a) shotcrete characteristics (i.e., stiffness, strength, time-dependent behavior), (b) ground properties, and (c) driving parameters on the deformation and the loading of the lining are presented. Conclusions derived from the obtained numerical results are provided and their potential impact on practical tunneling engineering is discussed (Boldini et al., 2003a).

The last part of the thesis (Chapters 7 and 8) is dedicated to the analysis of a real tunnel where shotcrete was applied as short-term support. The tunnel, called Raticosa tunnel, is one of the several tunnels currently under construction for the new Bologna-Florence high-speed railway line. Because of the squeezing conditions predicted in the preliminary investigation phase, full-face excavation was combined with reinforcement of the tunnel face by means of fiber-glass dowels. The primary lining consisted of a closed ring of shotcrete and steel sets.

In Chapter 7 the in situ measurements of face extrusion and tunnel wall convergence are analyzed and correlated with the single construction stages and specific ground conditions (Boldini et al., 2002) (Boldini et al., 2003b). A tentative explanation of the observed behavior is then proposed on the basis of simplified analytical models.

The excavation of the Raticosa tunnel between chainages 32+150 and 32+300 was simulated with numerical analyses and the results are reported in Chapter 8. The analysis is based on a viscoplastic material model for the soil and on the thermochemomechanical model for shotcrete already applied in the numerical analyses of Chapter 6. Fiber-glass dowels are accounted for in the numerical model by means of a chain consisting of two-node truss elements. The results obtained are compared with the values of extrusion and convergence measured in the investigated sections of the Raticosa tunnel.

Finally, summary and conclusions are reported in Chapter 9.
2. Excavation and Support Methods for Tunnels in Weak Rock-Masses

2.1 Introduction

Large ground deformations characterize the excavation of tunnels in weak rock-masses resulting many times in unacceptable convergence, stability problems and overstressing of the lining systems. These conditions are generally referred to as squeezing conditions and special techniques have to be adopted both during excavation and support installation.

This chapter will describe the excavation and support methods applied in the past and today to overcome stability problems during the construction of tunnels through squeezing ground conditions. Special attention will be devoted to big tunnels with spans larger than 10 m.

Unless specified otherwise, the description will be limited to conventional tunneling methods. Mention in passing will be made of the problematic aspects of excavation using the Tunnel Boring Machine (TBM) in rock masses characterized by squeezing behavior.

2.2 Squeezing Rock Conditions

Following the International Society of Rock Mechanics (ISRM) "squeezing of rock is the time- dependent large deformation which occurs around the tunnel and is essentially associated with creep caused by exceeding a limiting shear stress. Deformation may
terminate during construction or continue over a long time period" (ISRM, 1994). Terzaghi first distinguished between squeezing and swelling behavior in tunnels. He wrote: "Squeezing rock advances into the tunnel without perceptible volume increase. A prerequisite for squeeze is a high percentage of microscopic and submicroscopic particles of micaceous minerals or of clay minerals with a low swelling capacity" (Terzaghi, 1946 in Aydan et al. 1996). Volumetric deformation is due to the dilatancy of the rock-mass during shearing.

An analysis of different case histories has shown that squeezing behavior can occur in rock complexes of phyllitic schists, argillaceous schists, clays, marls, or tectonically altered gneiss (Schubert, 1996).

The magnitude and rate of tunnel deformations related to squeezing depend on several factors such as geological conditions, in situ stress, and groundwater regime as well as the geotechnical properties of the rock-mass and the tunneling technique.

The squeezing potential is mainly related to the $f_c/S$ ratio between the strength of the rock-mass and the in situ stress (Hoek, 2001). Values of the $f_c/S$ ratio less than 0.2 are found to be representative of very severe squeezing conditions.

Prediction of squeezing behavior is not easy. The rock-mass mechanical behavior can be only indirectly identified through correlation criteria with the intact rock properties (see, i.e., Hoek and Brown's criteria). Laboratory tests on intact rock specimens are characterized by many difficulties if complex rock formations are investigated, as in the case of squeezing rocks. The experimental determination of creep behavior is not frequent and no reliable results are in general expected (Panet, 1996).

Empirical and semi-empirical approaches are needed in order to estimate the squeezing potential and the consequent excavation and support requirements. An approximate relationship involving the strain at the tunnel wall, $\varepsilon$, and the ratio $f_c/S$ between the uniaxial strength of the rock-mass and the in situ stress is proposed as an indicator of the severity of squeezing (Hoek and Marinos, 2000) (Figure 2.1).
Different types of squeezing behavior can be observed in relation to the nature of the rock-mass (Aydan et al. 1996). In homogeneous rock-mass, shear deformations develop up to possible complete shear failure (Figure 2.2(a)). Bucking failure is generally observed in metamorphic rocks and thinly bedded ductile sedimentary rocks (Figure 2.2(b)). Thickly bedded sedimentary rocks are instead characterized by sliding along bedding planes and shearing in the intact rock (Figure 2.2(c)).

The orientation of the bedding planes in sedimentary and metamorphic rock-masses is found to highly affect the squeezing potential (Steiner, 1996). Bedding planes parallel to the tunnel may originate convergence of an order of magnitude greater than bedding planes orthogonal to the tunnel axis.

The consequences of the rock-mass squeezing potential are strictly related to the adopted tunneling technique (ISRM, 1994). Immediate support of the excavated tunnel reduces the yielding of the rock-mass around the opening thus limiting the short- and long-term deformations.

Recently, special attention has been paid to the stability of the tunnel face and to the deformation state of the ground core ahead the tunnel face. The control of ground deformations ahead of the tunnel face is found to be instrumental in assuring the whole tunnel stability (Lunardi, 2000). The whole tunnel deformation is governed by the rigidity of the ground core ahead of the tunnel face. Methods for dealing with face stability in squeezing ground have been developed mainly in Europe to deal with tunneling through the Alps (Schubert, 1996).

**2.3 Historical Outline**

Starting from the 19th century tunneling begins to spread in relation to the increasing construction of railway lines. First in England, but very soon elsewhere in Europe, North-America and Australia, tunnels were requested to connect different countries and reduce traveling time. In particular in the mountain region of the Alps, long and very deep routes
posing very difficult geotechnical problems characterized the tunnels between Austria, Switzerland, Germany, Italy, and France. In the same period, also mining production increased demanding new economic systems of rock support (Kovari, 2000). At the beginning of the 20th century a new push came from the necessity to built underground hydroelectric power stations.

Up to the beginning of the 19th century the tunnels were excavated by means of the multiple heading method with the only help of timber as temporary support (Kovari, 2000) (Figure 2.3). The use of timber to support the excavated rock in underground constructions presented many disadvantages, above all the large amount of time required for its installation. The timber structures filled a large part of the tunnel section, sometimes even more than one third of the excavated section, obstructing the space and making the work difficult (Rabczewicz, 1964a). Because of the non-immediate rock support, over-excavation was necessary in such a way as to achieve the necessary clearance of the tunnel section. Many times the temporary timber structures had to be renewed because they would lose their bearing capacity (Rabczewicz, 1964a) (Rabczewicz, 1964b) (Rabczewicz, 1965).

The final lining of the tunnel was constituted by a masonry structure composed of bricks obtained from cut stones or cemented lime-sand, with dry backfilling of irregular stones (Barla, 2001). Backfilling with concrete was adopted only at the beginning of the 20th century and only for the invert (Steiner, 1996). Many times the masonry structures suffered local and generalized decays needing complex and expensive repair actions (Rabczewicz, 1964a).

The first revolution in tunneling techniques was the introduction of the shield by Brunel in 1825 to underpass the Thames River in London. The shield had a rectangular form but some years later (1869) a new cylindrical shield was adopted by Barlow for the second tunnel under the Thames. Cast iron segments were applied in the latter case as final lining.

In the field of the so-called conventional tunneling techniques (as opposed to excavation by tunnel boring machines) timbering continued to be diffusely applied up to the 1940s. In the 19th century and in the early 20th century some new solutions were found and applied to specific underground constructions, but they did not diffuse worldwide (Kovari, 2000). For example, the first "cement-gun" machine was invented in the United States in 1911 and a
patent was assigned in 1913 in Germany for the "the support of roofs and walls in mining without support from below" by means of "bore-holes of sufficient depth that will be drilled into the rock in which rods, tubes or cables made of a load-bearing material, for example steel, will be inserted and fixed at the end in a proper manner or cemented along the whole length" (Stefan et al., 1913 in Kovari, 2000).

The diffusion of rock-bolting and anchoring starts in the United States in the 1940s and in Europe in the 1950s. This technology was applied at the beginning exclusively in mining constructions. Generally, the mine drifts were supported exclusively by rock-bolts, except in squeezing rock conditions where also steel arches were employed (Kovari, 2000).

At the beginning, old railway rails were used as steel supports. During the first decades of the 20th century the technology reached higher and higher quality standards. In 1932 the famous Toussain-Heintzmann (TH) steel sets with frictional sliding connections were presented and became later an important instrument to overcome the problems of squeezing in tunnels under the Alps (Figure 2.4).

During the two decades in the 1940s and 1950s, rock-bolting and shotcrete started to be systematically adopted together also in tunneling and the timber was definitively abandoned as temporary support. Application of a thin layer of shotcrete immediately after the excavation and rock-bolting when necessary opened the way to modern tunneling. Especially the shotcrete increased its importance in the world of underground construction becoming the principal support element of the so-called New Austrian Tunneling Method. Its success began exactly in the 1950s when Senn introduced the first real shotcreting machine capable of producing 3 m³/h of shotcrete.

2.4 The Shotcrete Method and the N.A.T.M.

The combined application of shotcrete, rock-bolts and steel arches characterized the new tunneling method that became widespread in the 1950s in the entire world. The role of shotcrete as tunnel support was decreed as fundamental, so much so that many authors called
this new tunneling technique as "Shotcrete Method" or "Sprayed Concrete Method". In 1948 the Austrian Engineer, Rabcewicz, patented this tunneling technique under the name "New Austrian Tunneling Method (NATM)" that is actually the name with which it is still widely known today (Rabcewicz, 1948).

The main salient feature of the method consists in the application of a semi-rigid lining to the newly excavated surfaces near the tunnel face. The primary lining should be closed as soon as possible by an invert before the rock can experience loosening (Rabcewicz, 1964). The lining can consist of anchors, shotcrete and steel arches or, better, by a combination of these. The lining allows for some deformation in the rock-mass in such a way as to permit the formation of a natural load carrying ring, but avoiding the decay of the strength properties of the ground. "Just in time" is the sentence used to summarize these concepts (Wu and Roony, 2001).

Shotcrete is applied onto the tunnel surface immediately after the excavation and forms a continuous and flexible ring, capable of withstanding the release of large ground stresses without structural damage. The confining action of the shotcrete absorbs the tangential stresses acting in the more superficial rock layer; the shotcrete fills eventually the joints between the rock blocks, and it also occludes the gaps between the steel arches and the tunnel surfaces. If necessary, rock-bolts contribute to increasing the load bearing capacity in the ground.

NATM tunneling is characterized by the adaptation of the construction parameters, such as excavation rate, shell thickness, numbers of rock bolts, etc., to the observed response of the already excavated part of the tunnel. These adaptations are based on monitoring the deformation of the tunnel (convergence measurements), surface settlements, and extensometer measurements in the surrounding rock mass.

Both full-face and sequential excavation methods are possible in the context of NATM, as described in the next Paragraph.
2.5 Conventional Tunneling Techniques for Squeezing Conditions

Several problems can develop if squeezing conditions are encountered during tunnel driving, especially in tunnels characterized by spans bigger than 10 m. The critical factor is the stability of the tunnel face (Hoek, 2001).

Squeezing deformations may produce an unacceptable reduction in the section clearance (with the need for re-profiling, if any), excessive loading on the lining (with possible failure as a consequence), possible water in-flow in the tunnel and pronounced creep behavior of the rock-mass (Schubert et al., 2000).

Different solutions are adopted to overcome the difficulties related to tunneling in squeezing conditions. Conventional tunneling techniques are divided into full-face excavation methods and sequential excavation methods (Figure 2.5). The side-drift method and the top heading and benching down method are two examples of sequential excavation methods that make use of smaller excavation sections to improve tunnel face stability.

In the side drift method, two lateral drifts of the tunnel section are first excavated and supported with shotcrete and steel arches before starting the excavation of the heading and then the bench (Figure 2.5(a)). The shotcrete layer becomes the foundation of the lining applied later to the heading. Tunnel convergence is controlled by placing heavy steel arches in both the side drift and the heading (Kovari and Staus, 1996). No form of advanced mechanization is possible as a result of the small working area of the single drifts resulting in very low advancement rates (Figure 2.6).

The former excavation of the top heading of the tunnel section characterizes the top heading and benching down method (Figure 2.5(b)). The top head generally has a height which is greater than 5 m so as to leave enough space for excavation and support activities (Barla, 2001). If needed, an umbrella of forepoles is installed to protect the top head and a temporary support of shotcrete and steel sets is placed. The steel sets can be eventually founded on micropiles and a shotcrete layer can be placed on the invert. Only for severe squeezing conditions can it be necessary to use fiber-glass dowels for reinforcement. The
excavation of the bench is performed at a distance that depends on the ground conditions, generally at 50÷150 m (Kovari and Staus, 1996). Larger distances allow for the simultaneous excavation of the heading and the bench but can result in excessive tunnel deformation.

Full-face excavation offers the possibility of highly mechanizing the tunnel activities by using larger equipment (Schubert, 2000) (Figure 2.5(c)). To guarantee stability, face reinforcement is necessary followed by the placement of a very heavy temporary lining of shotcrete and steel sets as close as possible to the face. The support ring must be closed as soon as possible by means of an invert of reinforced concrete.

The reinforcing and supporting techniques adopted in sequential and full-face excavation methods are summarized in Figure 2.7 as a function of the expected squeezing potential.

The full-face excavation method, with the early closure of the lining ring, gives rise to less deformation of the tunnel but needs a larger number of reinforcement actions and a heavy support system. It is at higher risk if failure develops, because the whole tunnel section is excavated in a single step. On the contrary, tunnel excavation by means of sequential headings requires more over-excavation with respect to the final clearance of the tunnel because the convergence is higher but the lining is subject to less loading.

Lower costs and shorter working time seem to distinguish the sequential excavation methods as compared to the full-face excavation method, but in general the choice of the tunneling technique is influenced much more by regional preferences (Schubert et al., 2000). For example, In Italy tunnels are excavated prevalently by means of the full-face method, while in the German-speaking countries the sequential headings method prevails. There are no doubts that the success of an underground excavation is often primarily related to the experience of the contractors and of the miners (Hoek, 2000).

The supporting technique adopted for tunneling in heavy squeezing conditions can follow two different approaches: an active or a passive approach (Barla, 2001).

An active approach is followed when the deformation of the rock-mass during the excavation is limited as much as possible. Decompression of the rock is impeded by means of pre-reinforcement of the ground ahead of the advancing tunnel face, and by placing heavy
supports and a large number of rock-bolts immediately after the excavation. As a consequence, the lining is highly loaded.

Another strategy consists in allowing the rock-mass to deform by adopting a passive approach, especially in very poor ground conditions where rock-bolting is ineffective (Hoek, 2000). Deformation of the rock is obtained through the over-excavation of the tunnel or by adopting supporting elements that will yield (Hoek, 2000). For example, the flexibility and ductility of the shotcrete support can be increased by leaving some gaps in the layer (Figure 2.8). Because of possible unsymmetrical deformation of the rock-mass, these gaps are generally equipped with special absorbing elements in order to better control the deformation of the lining (Schubert, 2000) (Figure 2.9). In these conditions the TH steel sets, characterized by frictional sliding joints, have been employed in association with the gaps in the shotcrete lining in several tunnel projects under the Alps (Barla, 2001). Special yielding bolts, which allow a maximum convergence of 18 cm, have also been developed (Figure 2.10).

A last consideration deals with the high water in-flow that can characterize the tunneling in squeezing conditions, especially when faults and more permeable zones are crossed. Coming across high pressure water during the excavation can result in very unsafe circumstances. Probe-holes can be installed ahead of the tunnel face in order to anticipate the hydraulic conditions that will be encountered during tunnel advance.

2.6 The ADECO-RS Approach

ADECO-RS is the acronym of "Analysis of COntrolled DEformation in Rocks and Soils". The control of the ground deformations during tunneling is pursued in the ADECO-RS approach through the increase of the stiffness of the ground ahead of the tunnel face (Lunardi, 2000).

The method emphasizes that ground deformation during tunnel excavation is not only limited to convergence but develops also ahead of the tunnel face. The ground deformations
ahead of the tunnel face is the consequence of two main movements: extrusion (longitudinal displacements of the ground core ahead of the tunnel face) and pre-convergence (convergence of the theoretical profile of the tunnel ahead of the tunnel face).

In the ADECO-RS approach, artificial increase in the rigidity of the ground core ahead of the tunnel face was studied. The solution consists in reinforcing the ground by means of fiber-glass dowels.

The dowels are installed and cemented in bore-holes excavated ahead of the face almost parallel to the tunnel axis. They are removed during tunnel excavation together with the ground owing to their brittle behavior. The initial length of the fiber-glass dowels is approximately 20÷24 m. When the length is reduced to approximately 10÷12 m, a new set of fiber-glass dowels is installed. The reinforcement of the tunnel face is characterized by different length, density, overlap, cross-section area, and geometrical distribution of the dowels depending on ground conditions (Lunardi and Bindi, 2001).

For severe squeezing conditions, the face reinforcement is combined with other systems in order to increase the applied confinement (Figure 2.11). Mechanical precutting allows the tunnel to be excavated under the protection of a shotcrete shell. While the mechanical precutting technique can be employed in cohesive and compact ground, horizontal jet-grouting or shells of improved ground are applied in granular and low-cohesion ground (Lunardi, 2000).

In all ground conditions, full-face excavation is adopted. Full-face excavation guarantees an industrialized tunnel advance (the advance rate may be low but is constant) and high flexibility long all the tunnel track (the same equipment is employed) (Lunardi and Bindi, 2001). The primary lining, consisting of shotcrete and steel arches, must be installed as soon as possible near the tunnel face so as to limit the ground deformation.

The stability of the tunnel face is the central feature of the ADECO-RS approach and is assumed as the fundamental criterion for defining standard tunnel specifications (Lunardi, 2000). Three categories are employed for tunnel classification:
• in category A the tunnel face is stable and only elastic deformations develop in the ground. The tunnel reaches equilibrium conditions even in the absence of supporting systems;

• category B includes tunnels where the face is in short-term stability. If the excavation rate is maintained constant and sustained, the excavation may proceed without face reinforcement;

• in category C the tunnel face is unstable because of the very large amount of plastic deformations in the ground. Reinforcement systems must be adopted during the excavation to guarantee tunnel stability.

On the basis of geological and geotechnical considerations during the design stage, the tunnel is subdivided into uniformly-behaving zones that are classified into three categories A, B, and C. Each zone of the tunnel is analyzed and, depending on the category to which it belongs, a specific reinforcement and supporting system is adopted.

The design assumptions are constantly verified during tunnel construction by monitoring the tunnel response during excavation. The monitoring system is based primarily on extrusion and convergence measurements that allow a rapid and quasi-continuous control of tunnel behavior.

### 2.7 Excavation with TBM

In contrast to conventional tunneling techniques, very little success has been recorded for excavation with TBM in squeezing conditions (Schubert, 2000). Several problems affect machine driving and advanced technological solutions have not yet been developed.

The face instability that generally occurs in squeezing ground conditions is in part avoided thanks to the supporting action of the cutting head (Hoek, 2000). Mild squeezing deformations can be accepted if the exceeding convergence is removed during the cutting
operations. More severe squeezing conditions, on the contrary, result in possible problems of support installation, reduced thrust due to reduced gripper action, difficulty in controlling the direction of the machine up to complete blockage of the TBM (Barla, 2001). TBM blockage is in general related to the high convergence of the tunnel during the excavation but sometimes it is the consequence of sudden in-flows of water and of mud in the proximity of fault zones.

Stops of TBM excavation have to be avoided in squeezing conditions because of the very high probability of machine blockage. Applying the famous empirical expression of Sulem et al. (1987) of tunnel convergence (see Paragraph 2.8), Schubert proposed a relationship between the advance rate of the tunnel and tunnel closure 10 m behind the face (Figure 2.12). The higher the tunnel advance rate, the smaller the probability that the TBM may be trapped in the squeezing rock-mass (2000).

Once the TBM is trapped in the ground the procedures to get it free are very long and costly. Double shielded TBMs can be easily cleared because the opening of the telescopic portion allows to operate at a distance of only 4-5 m from the tunnel face with respect to the 8-9 m of the single shielded TBM (Barla and Pelizza, 2000).

Improvement in tunneling procedures in squeezing conditions is obtained with the help of over-cutting. Over-cutting must be increased from the usual 6-8 cm to 14-20 cm (Barla and Pelizza, 2000). Radial overcutting can be easily achieved with open TBM's (Figure 2.13). In single or double shielded TBMs, overcutting is obtained by shifting the centerline of the cutting head (Barla, 2001). Special TBM's for squeezing conditions have been developed equipping the outer shield with blades that can move independently in both axial and radial directions (Hoek, 2001). Moreover, the thrust of the shielded TBM can be increased in order to assist the machine advance if the lining is able to support the bigger action of the thrust jacks (Barla and Pelizza, 2000).
2.8 Monitoring Systems

The design of a deep tunnel is characterized by many uncertainties related to the partially unknown geological and geotechnical conditions that will be met during the excavation. The uncertainties become even higher if squeezing conditions are expected. Squeezing rock-masses behave in a very complex way and the tunnel excavation and support techniques must be chosen and adapted each time in a specific way (Stille, 2000).

These considerations emphasize the importance of monitoring systems in the context of rock tunnel design. Panet (1996) wrote: "monitoring of convergence during face advance and back analyses of the measurements appear to be the most reliable approach in providing an overall assessment of the behavior of the rock" in accordance with the "observational procedure" defined by Terzaghi (Terzaghi et al., 1967).

The monitoring system has to comply with two main design criteria (Fuoco et al., 2000):

1) control of construction, to verify short-term safety and design requirements;

2) estimation of the rock-mass and support system behavior, to corroborate medium- and long-term safety.

Checking the design requirements during tunnel excavation allows to optimize the excavation and support methods on the basis of the observed tunnel behavior (Stille, 2000).

In order to ensure construction control, data must be collected continuously and frequently throughout the excavation. Generally, monitoring sections are installed every 20÷25 m and data acquisition is carried out every 2÷3 m of excavation. Accordingly, the monitoring and the data acquisition should be an integral part of the production cycle (Fuoco et al., 2000). The aim is achieved by monitoring tunnel convergence or displacements.

The behavior of the rock-mass and the support system need more sophisticated instrumentation in order to be evaluated, as stated in point 2). These types of monitoring sections are commonly installed every 200÷300 m of tunnel advance. Measurements are generally carried out also of displacement, stress and pore-water pressure in the rock-mass and strain, stress and load in the support elements (Fuoco et al. 2000) (Bock, 2000b).
Today observations of tunnel displacements are quasi-uniquely performed by means of geodetic monitoring. Even if the accuracy of convergence measurements is higher, geodetic monitoring of tunnel displacements is preferred because it does not interfere with the excavation and is highly automatic (Bock, 2000a). The monitoring section consists of 5 reflector targets fixed on nails cemented in the rock and equally spaced along the tunnel section. The absolute displacements of each target are surveyed by a total topographic station up to a distance of 100 m (Bock, 2000a).

The observed tunnel displacements are presented graphically as a function of time or of distance from the tunnel face (Figure 2.14). Very often, the displacement rates are also calculated. The non-variability of the displacement rates with time is considered as indirect proof of the achievement of tunnel equilibrium. This empirical criterion does not provide any insight into the state of stress of the support elements. No determination of the safety factor of the tunnel is possible (Staerk et al., 2001).

Recently, an estimation of the stress intensity inside the shotcrete lining was proposed (Staerk et al., 2001, Lackner et al., 2002b). The observed displacements of the tunnel are applied as boundary conditions in a numerical model of the shotcrete lining. The time-dependent behavior of the shotcrete is taken into account in the constitutive law. The ratio between calculated stress and strength of shotcrete allows to estimate the actual safety factor of the support and then of the whole tunnel.

On the basis of observed tunnel displacements, the final tunnel convergence can be estimated by means of the empirical expression proposed by Sulem et al. (1987), which takes into account the effect of both face advance and time-dependent behavior of the rock:

\[
C(x, t) = C_{\infty, x} \left[ 1 - \left( 1 + \frac{x}{X} \right)^{-2} \right] \cdot \left[ 1 + m \left( 1 - \left( 1 + \frac{t}{T} \right)^{-n} \right) \right] \tag{2.1}
\]

with \( x \) standing for the tunnel face distance and \( t \) for the time elapsed since the face passed through the monitoring section. Relationship (1) depends on five parameters: \( C_{\infty, x} \) ("instantaneous closure") and \( X \) (a distance related to the extent of the plastic zone), which control the face effect; \( m, T \) and \( n \) for the time-dependent part. The free parameters can be
determined by a non-linear regression of experimental data. Once the parameters have been estimated, the final convergence of new monitoring sections can be anticipated after a few days of observation.

Another recent research has highlighted that the orientation of the displacement vector with respect to the radial direction of the tunnel may be a sign of rock-mass quality ahead of the tunnel face (Schubert et al., 2000) (Figure 2.15). The approach of a fault zone seems to be connected with an increase in the angle between the displacement vector in the crown and the vertical direction. This circumstance offers the possibility to predict short-term changes in rock-mass stiffness ahead of the tunnel face and consequently to organize the appropriate excavation and support techniques.

Apart from tunnel displacements, the following other measurements are possible:

- deformation measurements of the ground surrounding the tunnel by means of borehole extensometers;
- deformation measurements of the ground ahead of the tunnel face in a direction parallel to the tunnel axis by means of sliding micrometers;
- stress measurements in the ground surrounding the tunnel. Several methods are available (CSIRO, CSIR, hydraulic fracturing, …) but their application is very limited;
- deformation measurements in the support elements (shotcrete, steel arches, rock-bolts and anchors) by means of strain gauges attached to the steel embedded in the shotcrete. To obtain stress values, the stress-strain relationship must be accounted for;
- stress measurements in the support element (shotcrete, steel arches and pre-stressed anchors) by means of total pressure cells. This instrumentation is very sensitive to the installation quality and to other factors;
- pore-water pressure measurements by means of piezometric cells installed in boreholes.
2.9 Figures of Chapter 2

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3. Shotcrete as Tunnel Lining: Technology, Experimental Behavior and Constitutive Modeling

3.1 Introduction

The term "Shotcrete" or "Sprayed Concrete" refers to a "mixture of cement, aggregate, and water projected pneumatically from a nozzle into place to produce a dense homogeneous mass" (EFNARC European Specification, 1999 in Melbye and Dimmock, 2001). Shotcrete, as opposed to normal concrete, is characterized by fast setting characteristics that allow the material to adhere to the ground surface and to resist in place without formwork.

Shotcrete is widely applied in underground constructions, both in civil engineering and mining. As tunnel lining, shotcrete started to be used in the 1930s but it became the principal support element of modern tunneling, in combination with rock-bolts and steel arches, only after the 1950s.

In recent years, significant developments have occurred in shotcrete technology both in mechanical properties and environmentally safe characteristics. In many countries attempts are being made to apply shotcrete as final lining and not only as temporary support.

Unfortunately, little progress has been made in the research field, with regard to the study of ground-shotcrete interaction in tunneling and its consequence on safety.
3.2 Shotcrete as Tunnel Lining

The employment of shotcrete as tunnel lining occurs in a variety of geotechnical conditions. Basically, it is possible to distinguish between two main typical shotcrete applications:

- shotcrete in weak rock conditions;
- shotcrete in hard jointed rock conditions.

In weak rock conditions the shotcrete layer provides real support to the ground by exerting continuous confinement pressure on the excavated surfaces. Frequently, in such geotechnical conditions, rock-bolts are installed in association with the shotcrete. In these cases, the prevalent role of the shotcrete lining is that of protecting the rock surface from loosening (A.F.T.E.S., 1992). When rock-bolting is ineffective because of the poor mechanical properties of the rock-mass, shotcrete can be reinforced by means of a steel wire mesh or steel fibers which enhance its mechanical properties and its support to the ground. Steel arches can be systematically placed in the tunnel to improve the confining action.

The application of shotcrete in hard jointed rock conditions avoids or limits the detachment of single rock blocks from the roof and tunnel wall. Support to the ground is obtained only by rock-bolting.

In some cases the shotcrete is not employed for structural reasons but as a layer to protect the ground from atmospheric agents, in particular moisture (Heuer, 1977). The shotcrete layer produces a sealing effect on the excavated surfaces. In this way, the water content of the ground does not change in time and neither wetting nor drying occur inside the rock-mass. Anyway, if cracks develop in the shotcrete layer its protective effect may be considerably reduced.

The applicability of shotcrete can be drastically prevented in some cases (A.F.T.E.S., 1992). It is useful to remember:

- excessively high water in-flows into the tunnel;
• ground conditions that are not homogenous (i.e. rock blocks in a pelitic matrix) and hence the tunnel surface that is excavated is not sufficiently regular;

• non-cohesive soils that are instable in the short-term;

• very poor ground conditions where the confinement support of the shotcrete is too low.

In most of the above-mentioned situations also rock-bolting becomes problematic.

Some last considerations concern the relationship between shotcrete and tunneling methods (Huer, 1977).

As delineated in Chapter 2, shotcrete is the main support element adopted by the New Austrian Tunneling Method. The combination of excavation cycles and support installation cycles allows the shotcrete to age and develop the necessary strength and stiffness. From an operational point of view, the material that rebounds from the rock surfaces is removed from the working face with the normal spoil of the excavation. An important aspect is represented by the excavation equipment’s ability to withstand dust and rebounded material. Shotcrete is compatible also with drill and blast excavation methods.

On the contrary, shotcrete is not very compatible with mechanized excavation methods using TBM's. The excavation proceeds continuously and the young shotcrete is supposed to support the deforming ground when it still has not hardened properly. In some cases, the excavation rate is so high that there is not enough time for shotcreting. Other negative features are the specific workmanship required for removing the rebounded material and the high sensitivity of the equipment to the latter.

### 3.3 Technology

Shotcrete technology is continuously developing due to the increasingly higher demands on the performance of the material used as tunnel lining and to the stricter environmental requirements. In this paragraph some remarks will be made about all these aspects with
special emphasis on mix design (additives and admixtures), shotcrete reinforcement and production processes.

### 3.3.1 Additives and Admixtures

Several additives and admixtures have been developed to meet specific shotcrete behavior requirements, both in fresh and hardened states. The most important additives and admixtures are the set accelerators, superplasticisers, and microsilica.

Set accelerators play a fundamental role in shotcrete mix design. These additives permit the shotcrete to adhere to the excavated surfaces and to rapidly develop sufficient strength. Some years ago, the most diffused set accelerators were alkaline-based. Alkaline accelerators present many heavy drawbacks (Werthmann, 1995; Jode Kusterle, 1998). Negative consequences on safety and ecological concerns are related to the risk of caustic burns, by contact or air-borne, and to possible ground and water pollution through the dissolution of the alkali compounds. As concerns actual performance, alkaline accelerators cause a strong reduction in final strength (between 30% and 60%) and durability (Figure 3.2).

Alternative solutions to alkaline accelerants are the recently developed alkali-free additives and micro-cements.

Alkali-free accelerators increase working safety and have a reduced impact on the environment. Moreover, the loss of final strength is very limited.

Micro-cements are characterized by surfaces that are three to five times bigger than normal cements. This characteristic provokes an increased rate of hydration thanks to the larger contact surface with the water molecules.

Further increase in strength can be obtained with special additives called superplasticisers. Superplasticisers allow to reduce the water/cement ratio without diminishing the slump of the shotcrete wet-mix. The last generation of water reducers (the so-called hyperplasticisers) decreased the water/cement ratio to 0.38. Many positive consequences in terms of quality are
related to water reduction like the increase in early and final strength, greater durability, lower rebound, and fast setting.

These benefits and others more can be obtained with the addition of microsilica to the shotcrete mix. Microsilica are a sub-product of the silica and of the manufacture of iron-silica alloy. The high specific surface confers strong pozzolana characteristics to the microsilica. The addition of microsilica in proportions between 5% and 15% to the shotcrete mix contributes to a more homogeneous distribution of the hydration products throughout the shotcrete layer (Melbye and Dimmock, 2001). The primary effect is the reduction of permeability with positive repercussions on sulphate resistance and freeze-thaw durability. Other advantages are: better pumpability, increased cohesiveness of the fresh material, higher early and final strength, improved bonding between the shotcrete layers and between the shotcrete and the fibers, and reduced rebound (Melbye, 1990; Herfurthe e Nilsen, 1990).

### 3.3.2 Reinforced Shotcrete

Shotcrete, as concrete, is a brittle material and is characterized by very limited tensile strength. To overcome these drawbacks, it can be coupled with steel reinforcement.

Steel wire-meshes fixed at the tunnel wall were commonly used in the past and are still used today for very poor ground conditions where loosening of the surfaces does not guarantee the adhesion of shotcrete (Hoek et al., 1995). Starting from the 1970s an alternative solution became common which consisted in small steel fibers added directly to the shotcrete mix.

The reinforcement of shotcrete produces a moderate increase in peak strength and a significant increment in residual bearing capacity (Wood, 1990; Franzen, 1992; Hoek et al., 1995). Figure 3.3 shows the results of a flexural test carried out on a shotcrete slab with different fiber contents where the benefits of the reinforcement are clearly visible.

Another positive consequence of the reinforcement is the reduction of cracks during the shrinkage of the material. Crack reduction significantly decreases the total permeability of the lining thus limiting the attacks by chemical and physical agents.
Reinforcement of the shotcrete by means of steel fibers, instead of a steel wire-mesh, results in practical and economic advantages (Malmberg, 1990; Melbye, 1990; Vanderwalle, 1990). The steel fibers are added to the shotcrete mix as normal aggregates. On the contrary, the installation of the wire-mesh requires special workmanship and time scheduling. If the excavation surfaces are irregular, the shotcrete material must fill the voids behind the wire-mesh in order to cover the reinforcement properly (see Figure 3.4). In contrast, steel-fiber-reinforced shotcrete takes the shape of the excavated surface reducing the amount of material required to achieve the design thickness (Morgan, 1990).

In general the steel fibers are homogeneously distributed throughout the shotcrete layer if microsilica and superplasticisers are added to the wet-mix. On the other hand, the position of the wire-mesh in the shotcrete layer is almost casual and the steel reinforcement is not always located where tensile stresses develop (Franzen, 1992). In these cases, the contribution of the wire-mesh to the post-crack increase of the bearing capacity is reduced.

Different types of steel fibers are available, with length varying between 18 and 40 cm. The EE type (Enlarged Ends) is highly workable with the normal equipment, while the longer Dramix type requires special expertise but is characterized by better performance (Kompen, 1990) (Fig. 3.5). Particular glues, soluble in water, are employed in order to compact the fibers until they are added to the shotcrete mix (Ellison, 1990).

Recently, structural synthetic fibers have been developed with the aim of avoiding the corrosive phenomena that occur in the steel fibers (Melby and Dimmock, 2001). Moreover, synthetic fibers do not provoke skin lacerations and they reduce wear of the spraying equipment. In some cases, they give the shotcrete better residual bearing capacities compared with steel fibers.

### 3.3.3 Wet-mix and Dry-Mix Processes

The shotcrete can be applied by means of two methods: the dry-mix process and the wet-mix process.
In the dry-mix process, cement and aggregates are mixed preventively in a site-based plant or pre-batched and dried in silos or bags. The dry-mix is fed into the dry-mix shotcrete pump and transported to the nozzle by compressed air. Water is added to the dry-mix only at the nozzle (Figure 3.6). The operator governs the amount of water added at the nozzle. Admixtures are eventually introduced in the form of powder into the dry-mix or as a liquid with the water at the nozzle.

Cement, aggregates and water are mixed together in a way similar to conventional concrete in the wet-mix process. The wet-mix is fed into the pump and delivered to the nozzle by the action of pressure (Figure 3.7). Compressed air is introduced into the transportation system just below the nozzle in order to speed-up the wet-mix.

Both methods present advantages and disadvantages, even if for the dry-mix process the latter prevail (Hoek et al., 1995; Morgan, 1990; Melbye, 1990; Melbye 1994).

The dry-mix process is characterized by high flexibility because:

- the dry-mix can be stored even for a long period of time;
- the equipment is very manageable and adaptable to small working spaces.

The main drawbacks of the dry-mix process are:

- very high production of dust during the application of the shotcrete;
- possible caustic burns if no alkali-free admixtures are employed;
- unknown and variable water/cement ratio related to the modality of the introduction of water at the nozzle;
- possible pre-hydration of the cement if water comes into contact with the dry-mix;
- percentage of rebound varying between 30% and 50% due to the high air content in the mix. If fiber-reinforced shotcrete is applied, the rebound of the fibers reaches 70%;
- low production (maximum 12 m³/h).
On the contrary, the wet-mix process is characterized by:

- low dust production during spraying of the shotcrete;
- low rebound of the material from the excavated surfaces (typically between 5% and 10%);
- known value of the water/cement ratio. The water/cement ratio is actually reduced down to 0.37 with very high benefits on the performance of the shotcrete (final strength can reach values of 60 MPa and the shotcrete may assume the function of permanent lining);
- high production (typically 20 m³/h).

Negative aspects are instead related to the:

- limited workability of the wet-mix;
- cleaning costs for the equipment.

Until the 1990s the prevailing method of shotcrete application was the dry-mix process. Due to the heavy drawbacks of the dry-mix process, today the wet-mix process is preferred worldwide, especially in tunneling or in large underground constructions.

Only in some countries, like Austria and Germany, the dry-mix process is still adopted thanks to recently introduced improvements. For example, the employment of highly reactive cement has reduced the need for accelerators. Faster setting can be obtained by limiting the amount of gypsum in the dry-mix or by using micro-cements characterized by very high specific surface that reacts with water during hydration (Ruffert, 1995). Moreover, dust production in the dry-mix process can be limited through the introduction of water at 5–8 m before the nozzle (Aschber et al., 1995).

In Table 3.1, the typical mix design for sprayed shotcrete using the dry-mix process and the wet-mix process can be compared (Wood, 1992).
Table 3.1: Typical shotcrete mix design (after Wood, 1992).

<table>
<thead>
<tr>
<th>COMPONENTS</th>
<th>DRY-MIX</th>
<th>WET-MIX</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(kg/m³)</td>
<td>(%) dry materials</td>
</tr>
<tr>
<td>Cement</td>
<td>420</td>
<td>19.0</td>
</tr>
<tr>
<td>Silica fume additive</td>
<td>50</td>
<td>2.2</td>
</tr>
<tr>
<td>Blended aggregate</td>
<td>1670</td>
<td>75.5</td>
</tr>
<tr>
<td>Steel fibers</td>
<td>60</td>
<td>2.7</td>
</tr>
<tr>
<td>Accelerator</td>
<td>13</td>
<td>0.6</td>
</tr>
<tr>
<td>Superplasticizer</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Water reducer</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Air entraining admixture</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Water</td>
<td>controlled at the nozzle</td>
<td>180</td>
</tr>
<tr>
<td>TOTAL</td>
<td>2213</td>
<td>100</td>
</tr>
</tbody>
</table>

3.4 Experimental Behavior

Different structural performances are required of shotcrete depending on ground conditions.

In weak rock conditions, shotcrete is prevalently loaded at compression. Knowledge of how compressive strength evolves in time is the central topic of experimental investigation and the basis of tunnel design. The increment of Young's modulus in time is another key-factor as well, but experimental difficulties have limited its determination to very few cases.

In hard-jointed rock conditions, the most important characteristics of shotcrete are flexural strength and adherence to rock surfaces.
In accordance with the topic of the thesis, only the experimental behavior of shotcrete in compressive stress conditions will be described in detail.

During his PhD, Sezaki performed several uniaxial and triaxial compression tests on shotcrete specimens characterized by different curing times (Sezaki, 1989 in Aydan et al., 1992). The cube-shaped specimens were prepared by spraying the shotcrete as in tunnel practice. Shotcrete aging results in an increase in both strength and stiffness (Figure 3.8). Deformation at failure generally ranges between 0.6 and 4%, where the lower values correspond to higher values of compressive strength (Figure 3.9). At the same curing time, larger confining pressure increases peak strength and determines a more ductile post-peak behavior, similarly to what happens for geomaterials (Figure 3.10).

The characteristic of the shotcrete lining whereby it behaves as a ductile material during the first hours after its application is fundamental in allowing the ground to deform without undergoing structural damage.

Many empirical relationships have been proposed for describing the growth of Young’s modulus in time. All the expressions are characterized by the following exponential law:

\[ E = a_1 \cdot E_\infty \cdot \exp(c_1 / t^{b_1}) \]  

(3.1)

where \( E \) is Young’s modulus at time \( t \) while \( E_\infty \) is the final Young modulus at 28 days. By expressing Young’s modulus in GPa and time in days, the empirical coefficients are equal to \( a_1=1.062, \ b_1=0.6, \) and \( c_1=-0.446 \) and Young’s modulus at 28 days \( E_\infty=24 \) GPa. These values refer to laboratory tests carried out by Huber and Fischmaller (in Chang, 1994) (Figure 3.11).

The addition of steel fibers to the mix does not appreciably influence the stress-strain behavior of the shotcrete in pre-peak conditions (Remakrishnan, 1981 in Chang, 1994; Torstein, 1986 in Chang, 1994).

Much more experimental data are available for compressive strength even if the possibility of obtaining cylindrical cores is limited to material with compressive strength higher than 10 MPa. Therefore indirect methods have been developed. The Austrian Concrete Society
(1999) standardized the test methods for the compressive strength measurement of young shotcrete (Figure 3.12).

A law similar to the one employed for the growth of Young's modulus characterizes the growth of the compressive strength of shotcrete:

\[ f_c = a_2 \cdot f_{c,\infty} \cdot \exp(c_2 / t^{b_2}) \]  

with \( f_c \) standing for the compressive strength in MPa at time \( t \) (expressed in days), and \( f_{c,\infty} \) for the final compressive strength at complete hydration. Chang (1994) collected and interpolated several experimental results obtaining \( a_2 = 1.105 \), \( b_2 = 0.7 \), and \( c_2 = 0.743 \). The final compressive strength usually varies between 30 and 55 MPa (Figure 3.13).

Also for the strength, as for the deformability properties, the values do not seem to be influenced by the addition of steel fibers (Little, 1985) (Figure 3.14).

Young’s modulus and the compressive strength of shotcrete at different curing times are strongly correlated. This fact allows to calculate the Young modulus of shotcrete on the basis of the value assumed by the compressive strength. A simple expression proposed by Kuwajima (1991, in Einstein et al., 1991) is:

\[ E = \frac{d_1 \cdot f_c^{f_1}}{1 + d_2 \cdot f_c^{(f_1 + f_2)}} \]  

where \( d_1 = 1306 \), \( d_2 = 7194 \), \( f_1 = 1.92 \), and \( f_2 = 0.363 \).

Research studies have highlighted that creep plays a fundamental role in shotcrete behavior, especially in the first 10 hours after shooting (Einstein et al. 1991). Unfortunately, as for Young's modulus, very limited experimental data are available. The viscous behavior of shotcrete is investigated by evaluating the deformation of a sample subjected to a constant load and by comparing this value with the shrinkage deformation occurring in an unloaded sample (Byfors, 1980 in Einstein et al., 1991; Nelville, 1981 in Einstein et al., 1991).
Experimental results show that the rate of viscous deformation is more than linearly related to the increment of deviatoric stress, especially for shotcrete younger than 1 day (Pottler, 1990) (Figure 3.15).

Nevertheless, for curing times longer than 10 hours, a linear relationship between the rate of deformation and the stress is sufficiently approximated, and a Kelvin body in series with a spring can be representative of the shotcrete behavior (Kuwajima, 1991 in Einstein et al., 1991) (Figure 3.16). Denoting the spring rigidity with $E$ and the Kelvin body rigidity and viscosity with $E_t$ and $\eta$, respectively, on the basis of experimental results, the following values are proposed: $E=15$ GPa, $1/E_t=0.03$ GPa$^{-1}$, and $E_t/\eta=0.003$ min$^{-1}$ (Figure 3.17).

### 3.5 Constitutive Modeling

In most cases, a linear elastic model for shotcrete is assumed for the analysis of the ground-shotcrete interaction in tunneling. The complex nature of shotcrete behavior (non-linearity of the stress-strain relationship, hardening, creep deformation) is taken into account by adopting a Young's modulus characteristic of the average behavior of the material in compression.

For usual tunnel driving rates, no significant error is made if the damper in the rheological model of Kuwajima in Figure 3.16 is disregarded (Einstein et al., 1991). The equivalent elastic modulus in this case is equal to $(E_E/(E+E_t)=10$ GPa. This value of the equivalent elastic modulus is proposed by (Einstein et al., 1991) for shallow tunnels.

Pottler (1990) performed several numerical analyses in which he varied Young's modulus of the ground, the in situ stress, the tunnel excavation rate, the stress release factor at the moment of shotcrete installation (for details see the Chapter 4), the Young’s modulus of the shotcrete at 28 days, the empirical parameters employed in the evolution law of shotcrete stiffness (Equation 3.1), the excavation length, the thickness of the shotcrete lining, and the temperature of the rock. The shotcrete behavior was simulated by a visco-elastic law. By plotting, in a stress-strain diagram, the maximum values of the compressive stress
experimented by the shotcrete layer in all the numerical analyses and the corresponding
deformations, all the points were found to be almost aligned along a single line (Figure 3.18). The slope of this line in the stress-strain plane gives an equivalent elastic modulus for
the shotcrete of 7 GPa.

More advanced constitutive models are available in the literature. Swoboda and Moussa
(1992) proposed a constitutive model for shotcrete which accounts for the non-linearity of
the stress-strain relationship, failure both in tensile and compressive stress-paths, and
strength increase in time. The stress-strain relationship depends on the uniaxial compressive
strength of the material and then on time. Because of the hardening of the material, the
stress-strain relationship is expressed in an incremental form: the updated stress-strain
relationship is translated in the stress-strain plane in order to take into account the already
developed deformation (Figure 3.19). Viscous deformations are disregarded.

The constitutive models reported in Aydan et al. (1992) and Koprik and Mang (1996) are
elasto-plastic models where the strain hardening is related both to plastic deformations and to
the ageing of the shotcrete (i.e., the strength is supposed to increase with time even if no
plastic deformations occur). Also Young's modulus is assumed to increase with time.

A thermochemomechanical model was recently developed at the Vienna University of
Technology. The effect of the hydration of shotcrete on strength, stiffness, and chemical
shrinkage is considered. Moreover, microcracking as well as creep are accounted for.

Since this shotcrete model has been extensively applied in the numerical simulation
described in the next Chapters, a special paragraph is devoted to its characteristics.

### 3.5.1 Shotcrete Model of the Vienna University of Technology

Since shotcrete is applied to the newly excavated surfaces of the tunnel, already at early
ages, i.e., during the chemical cement-water reaction (hydration), it is loaded mechanically
by the inward moving rock mass. For the simulation of shotcrete under such loading
conditions, a thermochemomechanical material model was developed at the Vienna University of Technology (Hellmich et al., 2001); (Sercombe et al., 2000). It is formulated in the framework of thermodynamics of chemically reactive porous media (Coussy, 1995); (Ulm and Coussy, 1995); (Ulm and Coussy, 1996).

The hydration process is described by the degree of hydration which is defined by the mass of formed hydrates, \( m \), related to the total mass of formed hydrates at the end of the hydration process, \( m_\infty \):

\[
\xi = \frac{m}{m_\infty}, \quad \text{with} \quad 0 \leq \xi \leq 1.
\]  

The thermo-activated nature of the hydration process is accounted for by an Arrhenius-type law for \( \xi \), reading (Ulm and Coussy, 1996):

\[
\dot{\xi} = \tilde{A}(\xi) \exp\left( -\frac{E_a}{RT} \right),
\]  

where \( \tilde{A}(\xi) \) represents the normalized chemical affinity which is the driving force of the hydration process. \( E_a \) is the activation energy, \( R \) is the universal gas constant, with \( E_a/R=4000 \text{ K} \). \( T \) is the temperature in K.

Microcracking of the shotcrete is described in the context of multisurface chemoplasticity (Hellmich et al., 1999a); (Lackner et al., 2001). The space of the admissible states of stress \( C_E \) is given by:

\[
\sigma \in C_E \iff f_\alpha = f_\alpha[\sigma, \xi] \leq 0
\]  

In the employed material model, the compressive failure of shotcrete is controlled by the Drucker-Prager criterion, whereas cracking under tensile loading is described by the tension-cut-off criterion. The respective yield functions read (Figure 3.20):

\[
f_{DP}[\sigma, \xi_{DP}] \leq 0 \quad f_{TC}[\sigma, \xi_{TC}] \leq 0
\]
In contrast to the standard plasticity theory, the hardening forces $\zeta_{DP}$ and $\zeta_{TC}$ depend on both the hardening variables $\chi_{DP}$ and $\chi_{TC}$ and the degree of hydration $\xi$ (chemical hardening). For the Drucker-Prager criterion, strain-hardening from $\zeta_{DP}=\omega \cdot f_c(\xi)$ to $\zeta_{DP}=f_c(\xi)$ is considered as (see Figure 3.20(b)):

$$
\zeta_{DP}(\chi_{DP}, \xi) = \begin{cases}
\omega \cdot f_c(\xi) + (1 - \omega) \cdot f_c(\xi) \left[ 1 - \frac{(\chi_{DP} - \chi_{DP}^2)}{\chi_{DP}} \right] & \text{for } \chi_{DP} \leq \chi_{DP}^C, \\
f_c(\xi) & \text{for } \chi_{DP} > \chi_{DP}^C,
\end{cases}
$$

(3.8)

where $\omega$ is the ratio between the elastic limit under uniaxial compressive loading, $f_y$, and the uniaxial compressive strength of shotcrete, $f_c$. It is set equal to 0.25. The parameter $\chi_{DP}$ is related to strain at peak stress under uniaxial compressive loading, $\varepsilon^u$: $\chi_{DP} = \varepsilon^u - f_c(\xi = 1)/E(\xi = 1)$. For the tension-cut-off criterion, pure chemical hardening is considered, i.e., $\zeta_{TC}(\chi_{TC}, \xi) = \zeta_{TC}(\xi)$.

The creep properties of shotcrete are of fundamental importance for the interaction between the shotcrete shell and the surrounding rock mass. Plots of creep-strain rates obtained from creep tests reveal the existence of two distinct creep mechanisms (Ulm, 1998):

- One of them is related to the hydration process. It is caused by the diffusion of water into the capillary space. The respective strains are referred to as viscous strains $\varepsilon^v$. The evolution of $\varepsilon^v$ is given as (Ulm, 1998):

$$
\tau_w(\xi) \dot{\varepsilon}^v = J^v(\xi)G : \sigma - \varepsilon^v \quad \text{with} \quad G = E(\xi)C^{-1}(\xi)
$$

(3.9)

where $J^v(\xi)$ represents the asymptotic value of viscous compliance. $\tau_w$ is the characteristic time of the viscous-creep process.

- The second creep mechanism originates from dislocation-like processes in the nanopores of the hydrates. The respective strains are referred to as flow strains $\varepsilon^f$. Their evolution is given by a linear law of the Maxwell-type, reading:
\[ \dot{\varepsilon}^f = \frac{1}{\eta_f} \mathbf{G} : \mathbf{\sigma} \]  

with \( \eta_f \) standing for the viscosity governing the kinetics of the aforementioned dislocation-like process. It can be written as \( 1/\eta_f = c \cdot S \cdot \left( -2U/R (1/T - 1/\bar{T}) \right) \), where \( U/R=2700 \text{ K} \) reflects the thermally activated nature of long-term creep (Bazant, 1988). \( \bar{T} \) is a reference temperature. \( c=1 \text{ MPa}^{-2}\text{s}^{-1} \) is a dimensional constant (Sercombe et al., 2000). The rate of the microprestressed force \( S \) is linked to the rate of viscous slip \( \gamma \) via (Sercombe et al., 2000)

\[ \dot{S} = -H \dot{\gamma} \]  

where \( H \) denotes a constant material parameter. The evolution law for the viscous slip \( \gamma \) is given as (Ulm, 1998)

\[ \dot{\gamma} = cS^2 \left[ -\frac{U}{R} \left( \frac{1}{T} - \frac{1}{\bar{T}} \right) \right]. \]  

In contrast to the first creep mechanism, no characteristic time can be identified for the flow-creep process.

During hydration of shotcrete, new hydrates are formed in a microstress-free state (Bažant, 1979). This is reflected by an incremental stress-strain law, reading (Sercombe et al., 2000):

\[ \Delta \sigma = \mathbf{C}(\xi) : \left[ \Delta \mathbf{e}^\sigma - \Delta \mathbf{e}^p - \Delta \mathbf{e}^\gamma - \Delta \mathbf{e}^f - \Delta \mathbf{e}^s - \Delta \mathbf{e}^T \right], \]

where \( \Delta \mathbf{e}^\sigma \) and \( \Delta \mathbf{e}^i \) represent the increment of the plastic and chemical-shrinkage strain tensor. \( \Delta \mathbf{e}^T = 1\alpha \Delta T \), where \( \alpha \) represents the thermal dilatancy coefficient, is the increment of the thermal strain tensor. The chemical-shrinkage strains are related to the degree of hydration by \( \mathbf{e}^\chi = 1\beta(\xi) \), where \( \beta \) is the chemical dilation coefficient.

In general, the different processes, i.e., thermal, chemical and mechanical processes, depend on each other. Such dependencies are referred to as couplings. E.g., the interaction between the hydration process and the deformations is denoted as chemomechanical coupling. The
interaction between the deformations and the temperature is a thermomechanical coupling. Finally, the interaction between the hydration and the temperature is referred to as thermochemical coupling (see Figure 3.21).

On the basis of experimental evidence, some couplings have minor influences on the behavior of shotcrete (see Figure 3.22). E.g., the chemomechanical coupling resolves in a one-way coupling, with the hydration process influencing the mechanical state of the material through shrinkage strains, aging elasticity and chemical hardening. On the contrary, mechanical deformations have low influence on the hydration process. In the same way, the thermomechanical couplings can be reduced to a one-way coupling, with temperature changes resulting in temperature strains.

Based on the one-way couplings depicted in Figure 6.22, both thermal and chemical processes are not influenced by mechanical deformations. This permits to split the numerical analysis into two parts: a thermochemical analysis for determination of the temperature field and the field of the degree of hydration for the subsequent chemomechanical analysis.

For isothermal conditions the incremental stress-strain law given in Equation (3.13) becomes:

\[
\Delta \sigma = C(\xi); \left[ \Delta \varepsilon - \Delta \varepsilon^p - \Delta \varepsilon^v - \Delta \varepsilon^f - \Delta \varepsilon^s \right],
\tag{3.14}
\]

The material parameters for the shotcrete used in the following refer to the "shotcrete cement" of Lafarge (Hellmich et al., 1999b). The mixture characteristic of this shotcrete and the material parameters are listed in Table 3.2. In addition, parameters for intrinsic material functions for the compressive strength \( f_c \), Young's modulus \( E \), the viscous compliance \( J_\infty \), the chemical affinity \( \tilde{A} \), and the chemical dilation coefficient \( \beta \) are given.

More details on the constitutive model of shotcrete and on the experimental determination of the parameters can be found in Hellmich (1999) and references therein.

In order to illustrate the performance of the material model for shotcrete, a creep test in isothermal conditions is analyzed numerically. A shotcrete cube of 1 m × 1 m × 1 m is loaded uniaxially with 10 MPa at an age of 8 h. The load is applied for a time span of 292 h.
At an age of 300 h, the specimen is unloaded. Figure 3.23 shows the strain histories in the direction of loading obtained for different degrees of complexity of the material model. Consideration of the aging of the shotcrete (thin, solid line) results in an inelastic strain commonly referred to as aging strain (Meschke, 1996) (Cervera et al., 1998). This strain becomes evident after unloading. Aging strains are a consequence of the increase in stiffness from the time instant of loading to the time instant of unloading. Consideration of chemical shrinkage (thick, dashed line) results in an increase in the deformation starting at a time instant of approximately 30 hours. This time instant refers to the onset of chemical shrinkage characterized by $\beta=0$. According to the parameters given in Table 2, $\beta$ equals zero at $\xi=-a_v/b_v=0.43$. Microcracking (thin, dashed line) occurs only at the time instant of loading at $t=8$ h. Consideration of viscous-creep (thick, dash-dotted line) results in an increase in deformation by a factor of almost two. In accordance with the final value of the characteristic time for viscous-creep, $\tau_{w,\infty}$, of 24 hours (see Table 3.2), the main evolution of the viscous strain is observed during the first 50 hours after application of the load. Flow-creep strain, on the other hand, does not have an asymptotic behavior. It develops until the specimen is unloaded (thin, dash-dotted line).
<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>cement content</td>
<td>380 [kg/m³]</td>
</tr>
<tr>
<td>aggregate/cement ratio</td>
<td>4.79 [-]</td>
</tr>
<tr>
<td>water/cement ratio</td>
<td>0.60 [-]</td>
</tr>
<tr>
<td>compressive strength</td>
<td>39.6 [MPa]</td>
</tr>
<tr>
<td>( f_c(\xi) = f_{c,\infty} \frac{\xi - \xi_0}{1 - \xi_0} )</td>
<td>( \xi_0 [-] 0.01 )</td>
</tr>
<tr>
<td>biaxial compressive strength factor ( f_b/f_c [-] )</td>
<td>1.16 [-]</td>
</tr>
<tr>
<td>Young's modulus</td>
<td>40800 [MPa]</td>
</tr>
<tr>
<td>Poisson's ratio</td>
<td>0.2 [-]</td>
</tr>
<tr>
<td>characteristic time ( \tau_w(\xi) = \xi \cdot \tau_{w,\infty} )</td>
<td>24 [h]</td>
</tr>
<tr>
<td>viscous compliance ( J_{\infty}^\nu = J_{\infty}^{\nu,0}(1 - \xi) )</td>
<td>127·10⁻⁶ [MPa]</td>
</tr>
<tr>
<td>softening modulus for flow creep ( H )</td>
<td>1/7·10⁻⁸ [MPa]</td>
</tr>
<tr>
<td>reference temperature ( T )</td>
<td>20 [°C]</td>
</tr>
<tr>
<td>chemical affinity</td>
<td>7.313 [1/s]</td>
</tr>
<tr>
<td>( \tilde{\Lambda}(\xi) = a_A \frac{1 - \exp(-b_A \xi)}{1 + c_A \xi} )</td>
<td>10.46 [-]</td>
</tr>
<tr>
<td>( c_A [-] )</td>
<td>169.3</td>
</tr>
<tr>
<td>( d_A [-] )</td>
<td>4.37</td>
</tr>
<tr>
<td>chemical dilation coefficient ( \beta(\xi) = a_x + b_x \xi )</td>
<td>-0.405·10⁻³</td>
</tr>
<tr>
<td>( b_x [-] )</td>
<td>0.943·10⁻³</td>
</tr>
</tbody>
</table>

Table 3.2: Mixture characteristic of shotcrete and material parameters of Lafarge 'shotcrete cement'.
3.6 Figures of Chapter 3

Figure 3.1: Cross-section and composition of a single permanent shotcrete lining (Kusterle and Lukas, 1990).

Figure 3.2: Effects of different accelerator dosages on the development of shotcrete strength (Kusterle and Lukas, 1990).
Figure 3.3: Load deflection curves for unreinforced and steel fiber reinforced shotcrete slabs (Kompen, 1990).

Figure 3.4: Comparison between mesh reinforced and steel fiber reinforced shotcrete application (Morgen, 1990).
Figure 3.5: EE-fiber (left) and Dramix fiber (right) (Kompen, 1990).

Figure 3.6: Sketch of a typical dry mix shotcrete system (after Mahr et al., 1975 in Hoek et al., 1985).
Figure 3.7: Sketch of a typical wet mix shotcrete machine (after Mahr et al., 1975 in Hoek et al., 1985).

Figure 3.8: Results of uniaxial compression tests at different curing times (Sezaki, 1989 in Aydan et al., 1992).
Figure 3.9: Correlation between strain at failure and uniaxial compressive strength of shotcrete (Sezaki et al., in Swoboda and Moussa, 1992).

Figure 3.10: Results of triaxial compression tests: influence of the confining pressure and curing time (Sezaki, 1989 in Aydan et al., 1992).
Figure 3.11: Growth of the Young's modulus of shotcrete in time (based on data from Huber, 1991 and Fischmaller, 1992 in Chang, 1994).

Figure 3.12: Test methods for the measurement of compressive strength of young shotcrete (Austrian Concrete Society, 1990).
Figure 3.13: Growth of compressive strength of shotcrete in time (based on data from literature in Chang, 1994).

Figure 3.14: Results of compression tests on unreinforced and steel fiber reinforced shotcrete (Little, 1985).
Figure 3.15: relationship between deformation rate and deviatoric stress at different curing times of shotcrete (Pöttler, 1990).

Figure 3.16: Connection between elastic spring and Kelvin body in the rheological model proposed by Kuwajima (1991 in Einstein et al., 1991).
Figure 3.17: Results of creep tests (Kuwajima, 1991 in Einstein et al., 1991).

Figure 3.18: End values of stress and strain in the shotcrete layer obtained in the parametric numerical analyses by Pöttler (1990).
Figure 3.19: Constitutive model proposed for shotcrete based on history of deformation (Swoboda and Moussa, 1992).

Figure 3.20: Multi-surface plasticity model for soil and shotcrete: (a) yield surfaces in the \( \left( \frac{l_1}{\sqrt{3}} \right) - \left( \frac{\sqrt{2J_2}}{\sqrt{3}} \right) \)-space and (b) strain-hardening of Drucker-Prager criterion considered in the shotcrete model (\( f_c \): uniaxial compressive strength; \( f_y \): elastic limit under uniaxial compressive loading)
Figure 3.21: Material model for shotcrete: possible couplings between thermal, chemical, and mechanical processes.

Figure 3.22: Material model for shotcrete: relevant couplings.
Figure 3.23: Significance of thermochemomechanical material law for shotcrete: strain for uniaxial loading at different stages of constitutive modeling.
4. Review of Available Design Methods

4.1 Introduction

As pointed out in Chapter 2, shotcrete is the main support element for tunnels driven by conventional excavation methods, especially when squeezing conditions are encountered. In fact, the green shotcrete layer forms a continuous and flexible ring near the tunnel face, which can tolerate large strains without undergoing structural damage.

Even if this construction method is adopted nowadays (Kovari et al., 2000; Hoek, 2001), the available design methods are not completely satisfactory: tunnel design is usually performed empirically, on the basis of the experience of the contractors and consultants involved in the project.

On the other hand, the application of numerical modeling to the prediction of tunnel behavior is still a challenging task because of the following issues:

- modeling excavation advance and support installation near the face is essentially a 3D problem. Nevertheless 2D analyses in plane strain conditions are still more popular because 3D modeling is time-consuming and requires more experience in interpreting results;

- the final load on the lining and the final convergence of the tunnel strongly depend on the details of the construction process, which often changes during the tunnel advance and can hardly be modeled in the numerical analyses;
• ground conditions along the route of the tunnel can only be approximately predicted before the beginning of the excavation,

• the state of stress of the shotcrete lining is significantly influenced by the progressive aging of the material and by the possible decrease in the long-term strength of the disturbed rock mass around the excavation.

The preceding issues highlight the need to find a compromise between rough design methods and detailed numerical simulations so as to provide tunneling practice with a suitable predictive tool.

In the following the current empirical, analytical and numerical design methods for tunnels supported by shotcrete will be described.

\section*{4.2 Empirical Design Methods}

Based on a wide number of case histories, many authors have developed empirical guidelines concerning the appropriate tunnel support in different rock-mass conditions (Hoek et al., 1995).

The first time shotcrete appeared in the classification schemes of rock-mass quality and support estimation was in 1972. Wickham et al. related shotcrete thickness as well as rock-bolt spacing to a quality index denoted as RSR (Rock Structure Rating) (Figure 4.1). The RSR system of classification is based on geological, geometric, and hydraulic features in addition to joint conditions and quality considerations.

Because the RSR classification was developed for small tunnels, it is not widely adopted. Actually, the most well-known classification systems are the RMR (Rock Mass Rating) proposed by Bieniawski (1976, 1989) and the Q index (Tunneling Quality Index) suggested by Barton et al. (1974).
Six parameters are involved in the RMR classification based on the 1989 version: the uniaxial compressive strength of the rock material, the RQD (Rock Quality Designation), the spacing of discontinuities, the condition of discontinuities, the groundwater conditions, and the orientation of discontinuities. Depending on the value assumed by the RMR index in the $0\div100$ range, an appropriate excavation and support method should be adopted. Bieniawski (1989) drew up the guidelines reported in Table 4.1 for a tunnel characterized by a 10 m span and a horseshoe shape, excavated using the drill and blast method in a rock-mass with in situ stress lower than 25 MPa. Since the 1973 guidelines have never been submitted to thorough revision, only wire mesh is indicated as shotcrete reinforcement.

The Q system is mainly based on joint characteristics, namely the RQD, joint set number, joint roughness number, joint alteration number, and joint water reduction factor (Barton et al., 1974). Unlike the RMR classification, the influence of the in situ stress is considered as well through the stress reduction factor. The Q index may vary over a wide range of values (between 0.001 and 1000). In order to apply the Q index classification for support requirements, Barton et al. (1974) introduced another parameter denoted as $D_e$ (Equivalent Dimension). The $D_e$ is defined as the ratio between either the excavation span, the diameter, or the height of the tunnel, and a quantity called ESR (Excavation Support Ratio) that refers to the purpose of the underground construction and to the corresponding required degree of security. A list of the ESR values proposed by Barton et al. (1974) is reported in Table 4.2. Figure 4.2 shows the proposed support categories as a function of the Q index and of parameter $D_e$ (Grimstad and Barton, 1993 in Hoek et al., 1995). This figure was revisited in order to take into account the diffusion of shotcrete reinforcement by means of steel fibers.

On the basis of the three classification systems described above and on their own experience, Hoek et al. (1995) summarized the support requirements and the relative shotcrete applications in underground mining constructions (Table 4.3).
4.3 The Convergence-Confinement Method

In professional practice, tunnel design is commonly performed in 2D plane strain conditions by means of analytical methods or numerical FE analyses. As originally proposed by the Convergence-Confinement Method (CC) (Lombardi 1973; A.F.T.E.S., 1983), the three-dimensional problem of tunnel excavation can be analyzed with an equivalent plane strain problem where the 3D conditions are simulated through a radial pressure applied at the tunnel wall (Figure 4.3).

Given the hypotheses of circular tunnel and hydrostatic state of stress of the CC Method, the internal pressure is uniformly applied. This pressure is calculated in such a way that the corresponding convergence of the tunnel must coincide with the convergence of the unlined tunnel in 3D conditions. The internal pressure $P_i$ can be expressed as a fraction of the hydrostatic in situ stress $S$ as follows:

$$P_i(x) = [1 - \lambda(z)] \cdot S$$  \hspace{1cm} (4.1)

where $z$ is the distance from the tunnel face and $\lambda$ is a factor called ground stress release factor. $\lambda=0$ refers to the ground ahead of the tunnel face in undisturbed conditions, while $\lambda=1$ applies to unsupported tunnel sections where face influence is extinguished.

The relationship between the internal pressure $P_i$ and the convergence of the tunnel wall $u$ is the so-called convergence curve (curve CV in Figure 4.4). For an elastic medium it reads:

$$\frac{u(z)}{R} = \frac{1 + \nu}{E} \left( S - P_i(z) \right)$$  \hspace{1cm} (4.2)

where $R$ is the tunnel radius, $E$ Young's modulus and $\nu$ Poisson's ratio.

The ground-lining interaction is evaluated in the CC Method considering the behavior of the ground, through the convergence curve, separately from the performance of the lining, given by the confinement curve. The latter expresses the relationship between the pressure acting at the extrados of the lining $q$ and the convergence it experiences (curve CF in Figure 4.4):
with $K_s$ standing for the stiffness of the lining and $u(z_0)$ for the tunnel convergence at the moment of lining installation. In fact, the installation of the lining takes place when the tunnel convergence is already partially developed.

The ground-lining system is said to reach equilibrium conditions when the convergence curve and the confinement curve intersect and $P_i = q$.

The main uncertainty of the CC approach is the evaluation of the convergence of the tunnel $u(z_0)$, or, analogously, of the equivalent internal pressure $P_i$ supporting the tunnel wall at the time of lining installation.

Panet and Guenot (1982) simulated the excavation of an unlined tunnel by means of numerical analyses in axisymmetric conditions. They proposed the following expression for tunnel convergence in an elastic medium as a function of distance from the tunnel face $z$:

$$u(z) = u_f + (u_\infty - u_f) \cdot a^0(z)$$

(4.4)

where $u_f \cong 0.27 \cdot u_\infty$ is the convergence at the tunnel face, $u_\infty$ is the final convergence of the unlined tunnel at great distances from the face, and $a^0$ is the so-called shape function. It results:

$$a^0(z) = 1 - \left[ \frac{0.84 \cdot R}{z + 0.84 \cdot R} \right]^2$$

(4.5)

The evaluation of the final convergence $u_\infty$ can be obtained by putting in Equation (4.2) $P_i = 0$, hence obtaining:

$$u_\infty = \frac{(1 + \nu) \cdot S}{E} \cdot R$$

(4.6)

The expression of tunnel convergence in an elasto-plastic medium can be obtained by replacing the tunnel radius $R$ with the plastic radius $R_p$ in Equation 4.5 (Panet and Guenot, 1982).
Corbetta et al. (1991) proposed the so-called principle of similitude to calculate tunnel convergence in an elasto-plastic material. Through a geometric transformation, defined by the homothetic ratio $\chi$, of the tunnel convergence of an elastic medium, it is possible to compute the expression of convergence in an elasto-plastic medium. The homothetic ratio $\chi$ is defined as the ratio between the final convergence of an elasto-plastic and an elastic medium, respectively. Tunnel convergence in an elasto-plastic medium is then equivalent to the convergence of a tunnel with radius $\chi R$ in an elastic medium (Figure 4.5).

In the CC Method tunnel convergence at the time of lining installation $u(z_0)$ is obtained by rewriting Equation (4.4) for $z = z_0$, where $z_0$ is the distance from the tunnel face where the lining is placed:

$$ u(z_0) = u_f + (u_{x_0} - u_f) \cdot a^0(z_0) $$

(4.7)

### 4.4 The New Implicit Method

A careful analysis of the classical Convergence-Confinement Method has shown that the critical point of the method is the procedure used to evaluate the convergence of the tunnel when the lining is installed. More recent numerical results highlight how tunnel convergence at the time of lining installation is a function of both distance from the tunnel face $z_0$ and stiffness of the lining (Kielbassa & Duddeck 1991, Bernaud & Rousset 1992). Actually, installation of the lining modifies tunnel convergence even in the unsupported tunnel area between the lining and the face. The higher the lining stiffness, the lower the tunnel convergence with respect to the value calculated for an unsupported tunnel (Figure 4.6). Overestimation of tunnel convergence $u(z_0)$ at the time of lining installation leads to an underestimation of the final load on the lining, thus giving rise to unsafe tunnel design.

Bernaud and Rousset (1992) elaborated a simplified method, called New Implicit Method (NMI), to obtain more reliable predictions of tunnel equilibrium conditions on the basis of
three-dimensional numerical results. They substantially redefined the expression of tunnel convergence at the time of installation of the lining (Equation 4.7) as follows:

\[ u(z_0) = u_f + (u_{eq} - u_f) \cdot a^s(z_0) \quad (4.8) \]

where \( u_{eq} \) is the equilibrium convergence of the lined tunnel and \( a^s \) is a modified shape function depending on the lining stiffness which is related to the original shape function in the following way:

\[ a^s(z) = a^0(\alpha \cdot z) \quad (4.9) \]

The lining parameter \( \alpha \) is a function of (Figure 4.7):

\[ \alpha = \alpha \left( \frac{K_s}{E} \right) \quad (4.10) \]

Convergence at the time of lining installation depends also on the equilibrium convergence of the tunnel \( u_{eq} \); this is the reason why the method is called implicit.

If the tunnel is excavated in an elasto-plastic medium, the parameter \( \alpha \) is modified in order to include the influence of the plastic radius:

\[ \alpha^s = \alpha \left( \frac{R}{R_p} \right) \quad (4.11) \]

The Convergence-Confinement method and the New Implicit methods refer to linings for which a time-independent behavior is assumed. For shotcrete linings, an equivalent Young's modulus should be employed, as described in Paragraph 3.5.
4.5 2D Numerical Analysis

Numerical analyses allow to make simulations of non-circular tunnel sections and of non-hydrostatic in situ stresses. Moreover, also the time-dependent characteristics of shotcrete linings can be taken into account.

Analogously to the Convergence-Confinement Method, two-dimensional analyses are performed in plane strain conditions. The three-dimensional conditions can be accounted for through two different strategies:

1. by the fictitious pressure method;
2. by the stiffness reduction method.

The approach of the fictitious pressure method consists in the application of an internal pressure $P_i$ simulating the 3D conditions of the tunnel. This internal pressure is released by a factor $\lambda$ before the activation of the lining. The value of $\lambda$ can be calculated through the Equations described in Paragraphs 4.2 and 4.3. Depending on the distance from the face $z_0$ at which the lining is installed, Equations 4.4 or 4.8 provide the corresponding tunnel convergence $u(z_0)$. The value of the associated internal pressure $P_i(z_0)$ can be estimated through the convergence curve detailed in Equation 4.2 (see Figure 4.8). Finally, the value of factor $\lambda(z_0)$ is achieved by applying Equation 4.1.

In the stiffness reduction method, the preliminary convergence of the tunnel before installing the lining is attained by reducing ground stiffness inside the tunnel perimeter. Young's modulus of the rock-mass is reduced by a factor $\beta$ as follows:

$$E^* = \beta \cdot E$$ (4.12)

As for $\lambda$, factor $\beta$ may range between 0 and 1 depending on construction factors.

Usually the first strategy is preferred, because no clear indications are available on the values assumed by factor $\beta$. 
Comprehensive parametric analyses in plane strain conditions of the ground-shotcrete interaction were performed by Hellmich (1999) and Hellmich et al. (2001) who applied the shotcrete model developed at the Vienna University of Technology (see Paragraph 3.5.1).

### 4.6 3D Numerical Analysis

A general criticism can be addressed to all plane models, including 2D FE models for the reason that the effects of the face and face advance can be represented only by some artifices and therefore the ground-support interaction can be evaluated only by a crude approximation. The question becomes of paramount importance in the case of immediate shotcreting of the tunnel wall because of the time-dependent hardening of the shotcrete during the excavation advance. The most advanced shotcreting constitutive models have been applied so far only in plane strain numerical analyses.

Three-dimensional analyses allow a more realistic evaluation of the ground-lining interaction. No fictitious systems are necessary to represent the three-dimensional state of stress and deformation near the tunnel face. The excavation and the support sequences can be simulated in detail and it is possible to evaluate the influence of the tunnel excavation rate on tunnel equilibrium conditions. No simplifications of tunnel geometry and in situ stress are necessary.

The simulation of tunnel excavation can follow two different approaches:

1. the step-by-step procedure (see, i.e., Hanafy and Emery, 1980);

2. the steady-state method (Corbetta et al., 1991).

In the step-by-step procedure, tunnel excavation is reproduced by deactivating the corresponding ground elements while the support installation is simulated by changing the properties of the elements located at the lining. After a certain number of steps, equilibrium conditions are reached, i.e., the state of stress and deformation in both the ground and the
lining are the same with increasing face distances. The dimensions of the numerical model should be determined in order to avoid the influence of the boundary conditions on numerical results in the investigated area of the tunnel. The mesh should be finer in the area of influence of the tunnel face and can be coarser at the periphery of the numerical model. The calculation times are generally very high because of the number of nodes necessary in the numerical model.

The second approach, denoted as the steady-state method, can be applied to the study of tunnels characterized by constant excavation rate, section, overburden, in situ stress, excavation procedures, and rock-mass properties. The finite element mesh is supposed to advance as shown in Figure 4.9. Element \( e \) at time-step \( t \) takes the place that the element \( e+1 \) had at time \( t-1 \). The updating of the stress vector and deformation is given by:

\[
Z_e^t = Z_e^{t-1} + \Delta Z_e^t, \quad e=1,...(n-1) \tag{4.13}
\]

In this way, time-integration is transformed into space-integration for all the finite elements. The global stiffness matrix remains constant throughout the calculation, considerably reducing computational time.

Kielbassa and Duddeck (1991) performed three-dimensional numerical analyses by adopting the steady state procedure. In Figure 4.10, total and incremental values of convergence and hoop force in the lining are plotted as a function of distance from the tunnel face. The maximum increment of convergence develops in the unsupported tunnel length after every excavation step. As a consequence, the maximum hoop force in the lining is found in the area closest to the tunnel face. A saw-toothed shape, due to the procedure employed for the excavation simulation, characterizes the convergence and hoop force profiles. After 9 excavation advancements no increment of convergence or hoop force is recorded. Figure 4.11 shows the vertical stress in the ground obtained in the same numerical analysis. Tunnel excavation causes a stress redistribution in the ground. The stress release is not uniform in the unsupported ground area because a bearing arch develops between tunnel face and lining.
<table>
<thead>
<tr>
<th>Rock mass class</th>
<th>Excavation</th>
<th>Rock bolts (20 mm diameter, fully grouted)</th>
<th>Shotcrete</th>
<th>Steel sets</th>
</tr>
</thead>
<tbody>
<tr>
<td>I - Very good rock RMR: 81-100</td>
<td>Full face, 3 m advance</td>
<td>Generally no support required except spot bolting</td>
<td></td>
<td></td>
</tr>
<tr>
<td>II - Good rock RMR: 61-80</td>
<td>Full face, 1 - 1.5 m advance. Complete support 20 m from the face</td>
<td>Locally, bolts in crown 3 m long, spaced 2.5 m with occasional wire mesh</td>
<td>50 mm in crown where required</td>
<td>None</td>
</tr>
<tr>
<td>III - Fair rock RMR: 41-60</td>
<td>Top heading and bench, 1.5 - 3 m advance in top heading. Commence support after each blast. Complete support 10 m from the face</td>
<td>Systematic bolts 4 m long, spaced 1.5 - 2 m in crown and walls with wire mesh in crown</td>
<td>50 - 100 mm in crown and 30 mm in sides</td>
<td>None</td>
</tr>
<tr>
<td>IV - Poor rock RMR: 21-40</td>
<td>Top heading and bench, 1 - 1.5 m advance in top heading. Install support concurrently with excavation, 10 m from the face</td>
<td>Systematic bolts 4 - 5 m long, spaced 1 - 1.5 m in crown and walls with wire mesh</td>
<td>100 - 150 mm in crown and 100 mm in sides</td>
<td>Light to medium ribs spaced 1.5 m where required</td>
</tr>
<tr>
<td>V - Very poor rock RMR&lt;20</td>
<td>Multiple drifts, 0.5 - 1.5 m advance in top heading. Install support concurrently with excavation. Shotcrete as soon as possible after blasting</td>
<td>Systematic bolts 5 - 6 m long, spaced 1 - 1.5 m in crown and walls with wire mesh. Bolt invert</td>
<td>150 - 200 mm in crown, 150 mm in sides, and 50 mm on face</td>
<td>Medium to heavy ribs spaced 0.75 m with steel lagging and forepoling if required. Close invert</td>
</tr>
</tbody>
</table>

Table 4.1: Guidelines for excavation and support of 10 m span rock tunnels in accordance with the RMR system (After Bieniawski, 1989).
Table 4.2: List of ESR values proposed by Barton et al. (1974).

<table>
<thead>
<tr>
<th>Excavation category</th>
<th>ESR</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>3 - 5</td>
</tr>
<tr>
<td>B</td>
<td>1.6</td>
</tr>
<tr>
<td>C</td>
<td>1.3</td>
</tr>
<tr>
<td>D</td>
<td>1.0</td>
</tr>
<tr>
<td>E</td>
<td>0.8</td>
</tr>
</tbody>
</table>

Table 4.3: Summary of recommended shotcrete applications in underground mining, for different rock mass conditions (Hoek et al., 1985).

<table>
<thead>
<tr>
<th>Rock mass description</th>
<th>Rock mass behavior</th>
<th>Support requirements</th>
<th>Shotcrete application</th>
</tr>
</thead>
<tbody>
<tr>
<td>Massive metamorphic or igneous rock. Low stress conditions.</td>
<td>No splalling, slabbing or failure.</td>
<td>None.</td>
<td>None.</td>
</tr>
<tr>
<td>Massive sedimentary rock. Low stress conditions.</td>
<td>Surfaces of some shales, siltstones, or claystones may slake as a result of moisture content change.</td>
<td>Sealing surface to prevent slaking.</td>
<td>Apply 25 mm thickness of plain shotcrete to permanent surfaces as soon as possible after excavation. Repair shotcrete damage due to blasting.</td>
</tr>
<tr>
<td>Massive rock with single wide fault or shear zone.</td>
<td>Fault gouge may be weak and erodible and may cause stability problems in adjacent jointed rock.</td>
<td>Provision of support and surface sealing in vicinity of weak fault of shear zone.</td>
<td>Remove weak material to a depth equal to width of fault or shear zone and grout rebar into adjacent sound rock. Weldmesh can be used if required to provide temporary rock-fall support. Fill void with plain shotcrete. Extend steel fibre reinforced shotcrete laterally for at least width of gouge zone. Apply 50 mm shotcrete over weldmesh anchored behind bolt faceplates, or apply 50 mm of steel fibre reinforced shotcrete on rock and install rockbolts with faceplates; then apply second 25 mm shotcrete layer. Extend shotcrete application down sidewalls where required.</td>
</tr>
<tr>
<td>Massive metamorphic or igneous rock. High stress conditions.</td>
<td>Surface slabbing, spalling and possible rock-burst damage.</td>
<td>Retention of broken rock and control of rock mass dilation.</td>
<td></td>
</tr>
<tr>
<td>Rock mass description</td>
<td>Rock mass behavior</td>
<td>Support requirements</td>
<td>Shotcrete application</td>
</tr>
<tr>
<td>-----------------------</td>
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</tr>
<tr>
<td>Massive sedimentary rock. High stress conditions.</td>
<td>Surface slabbing, spalling and possible squeezing in shales and soft rocks.</td>
<td>Retention of broken rock and control of squeezing.</td>
<td>Apply 75 mm layer of fibre reinforced shotcrete directly on clean rock. Rockbolts or dowels are also needed for additional support.</td>
</tr>
<tr>
<td>Metamorphic or igneous rock with a few widely spaced joints. Low stress conditions.</td>
<td>Potential for wedges or blocks to fall or slide due to gravity loading. Bedding plane exposures may deteriorate in time. Combined structural and stress controlled failures around opening boundary.</td>
<td>Provision of support in addition to that available from rockbolts and cables. Sealing of weak bedding plane exposures. Retention of broken rock and control of rock mass dilation.</td>
<td>Apply 50 mm of steel fibre reinforced shotcrete on rock surface on which discontinuity traces are exposed, with particular attention to bedding plane traces.</td>
</tr>
<tr>
<td>Sedimentary rock with a few widely spaced bedding planes and joints. Low stress conditions.</td>
<td>Potential for wedges or blocks to fall or slide due to gravity loading. Bedding plane exposures may deteriorate in time.</td>
<td>Provision of support in addition to that available from rockbolts and cables. Sealing of weak bedding plane exposures.</td>
<td>Apply 75 mm plain shotcrete over weldmesh anchored behind bolt faceplates or apply 75 mm of steel fibre reinforced shotcrete on rock, install rockbolts with faceplates and then apply second 25 mm shotcrete layer. Thicker shotcrete layers may be required at high stress concentrations.</td>
</tr>
<tr>
<td>Jointed metamorphic or igneous rock. High stress conditions.</td>
<td>Slabbing, spalling and possibly squeezing.</td>
<td>Control or rock mass failure and squeezing.</td>
<td>Apply 75 mm of steel fibre reinforced shotcrete to clean rock surfaces as soon as possible, install rockbolts, with faceplates, though shotcrete, apply second 75 mm shotcrete layer.</td>
</tr>
<tr>
<td>Bedded and jointed weak sedimentary rock. High stress conditions.</td>
<td>Ravelling of small wedges and blocks defined by intersecting joints.</td>
<td>Prevention of progressive ravelling.</td>
<td>Apply 50 mm of steel fibre reinforced shotcrete on clean rock surface in roof of excavation. Rockbolts or dowels may be needed for additional support for large blocks.</td>
</tr>
<tr>
<td>Highly jointed metamorphic or igneous rock. Low stress conditions.</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rock mass description</td>
<td>Rock mass behavior</td>
<td>Support requirements</td>
<td>Shotcrete application</td>
</tr>
<tr>
<td>----------------------------------------------------------------</td>
<td>-------------------------------------------------------------------------</td>
<td>-----------------------------------------------------------</td>
<td>----------------------------------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>Highly jointed and bedded sedimentary rock. Low stress conditions.</td>
<td>Bed separation in wide span excavations and ravelling of bedding traces in inclined faces.</td>
<td>Control of bed separation and ravelling.</td>
<td>Rockbolts and dowels required to control bed separation. Apply 75 mm of fibre reinforced shotcrete to bedding plane traces before bolting.</td>
</tr>
<tr>
<td>Heavily jointed igneous or metamorphic rock, conglomerates or cemented rockfill. High stress conditions.</td>
<td>Squeezing and &quot;plastic&quot; flow of rock mass around opening.</td>
<td>Control of rock mass failure and dilation.</td>
<td>Apply 100 mm of steel fibre reinforced shotcrete as soon as possible and install rock-bolts, with face-plates, trough shotcrete. Apply additional 50 mm of shotcrete if required. Extend support down sidewalls if necessary.</td>
</tr>
<tr>
<td>Heavily jointed sedimentary rock with clay coated surfaces. High stress conditions.</td>
<td>Squeezing and &quot;plastic&quot; flow of rock mass around opening. Clay rich rocks may swell.</td>
<td>Control of rock mass failure and dilation.</td>
<td>Apply 50 mm of steel fibre reinforced shotcrete as soon as possible, install lattice girders or light steel sets, with invert struts where required, then more steel fibre reinforced shotcrete to cover sets or girders. Forepoling or spiling may be required to stabilize face ahead of excavation. Gaps may be left in final shotcrete to allow for movement resulting from squeezing or swelling. Gap should be closed once opening is stable.</td>
</tr>
<tr>
<td>Mild rockburst conditions in massive rock subjected to high stress conditions.</td>
<td>Spalling, slabbing and mild rockbursts.</td>
<td>Retention of broken rock and control failure propagation.</td>
<td>Apply 50 to 100 mm of shotcrete over mesh or cable lacing which is firmly attached to the rock surface by means of yielding rockbolts or cablebolts.</td>
</tr>
</tbody>
</table>
4.7 Figures of Chapter 4

Figure 4.1: Correlation between RSR index and support requirements for a 7.3 m diameter circular tunnel (Wickham et al., 1972 in Hoek et al., 1995).

Figure 4.2: Estimated supports categories according to Q index (the number in parenthesis in the Figure corresponds to: (1) unsupported, (2) spot bolting, (3) systematic bolting, (4) systematic bolting with 40-100 mm unreinforced shotcrete, (5) fiber reinforced shotcrete 50-90 mm and bolting, (6) fiber reinforced shotcrete 90-120 mm and bolting, (7) fiber reinforced shotcrete 120-150 mm and bolting, (8) fiber reinforced shotcrete >150 mm with reinforced ribs of shotcrete and bolting, (9) cast concrete lining (Barton et al., 1974).
Figure 4.3: Simulation of 3D conditions by means of radial internal pressure $P_i$ (Panet and Guenot, 1982).

Figure 4.4: The Convergence Confinement Method (CV= convergence curve; CF= confinement curve) (Lombardi, 1973).
Figure 4.5: Definition of the tunnel convergence in an elasto-plastic medium through the principle of similitude (Corbetta et al., 1991).
Figure 4.6: Curves of tunnel convergence for different lining stiffness in an elastic rock-mass (Bernaud and Rousset, 1992).

Figure 4.7: Values of the lining parameter $\alpha$ for different values of the ratio $K_s/E$ and of the stability number $N_s=2\cdot S/\sigma_c$ ($\sigma_c$ is the uniaxial compression strength of the ground) (Bernaud and Rousset, 1992).
4.19

Figure 4.8: Application of the fictitious pressure method in 2D numerical analyses.

\[ \lambda(z_0) = 1 - \frac{P(z_0)}{S} \]

\[ P_i = (1 - \lambda(z_0)) S n \]

Figure 4.9: Numerical simulation of the excavation advance by an advancing finite-element mesh (steady state method) (Kielbassa and Duddeck, 1991).

Figure 4.9: Numerical simulation of the excavation advance by an advancing finite-element mesh (steady state method) (Kielbassa and Duddeck, 1991).
Figure 4.10: Calculated total and incremental values of converge and hoop force in the lining by 3D numerical analyses (Kielbassa and Duddeck, 1991).

Figure 4.11: Distribution of vertical stress in the ground in consequence of tunnel excavation (Kielbassa and Duddeck, 1991).
5. Numerical Investigation of Tunnels Supported by Shotcrete

5.1 Introduction

Ground-support interaction in the vicinity of a tunnel face is a typical three-dimensional problem. Nevertheless, the design of tunnel support is usually based on simplified plane strain analyses. The results of such analyses strongly depend on the choice of ground stress release at the time of lining installation.

This chapter focuses on tunnels supported by shotcrete whose mechanical behavior adds further complication to the analysis of ground-support interaction because it makes the whole problem time-dependent.

The results of an extensive parametric study based on 3D axisymmetric models are presented, taking into account the effect of increasing stiffness of shotcrete in time (Boldini and Lembo-Fazio, 2001) (Cosciotti et al., 2001). An elastic constitutive law characterized by time-dependent stiffness is employed for the shotcrete.

The work presented in this chapter was aimed to:

1. carry out and analyze extensive parametric analyses whereby the main features of complex tunnel behavior can be predicted on the basis of the values of a set of non-dimensional parameters;

2. propose a simple strategy to enhance the capability of conventional 2D models in accordance with the results of the parametric study.
5.2 Definition of the Significant Parameters Governing the Problem

The ground-lining interaction is governed by several factors such as the mechanical behavior of the ground, the mechanical behavior of the lining and the constructive schedule of the tunnel.

Even for such complex conditions, the whole problem can be analyzed with reference to some simple non-dimensional parameters.

For a tunnel in an elastoplastic medium subjected to an isotropic state of stress $S$ and characterized by the material constants $E$, $\nu$, $c$, $\phi$ and $\psi$, the convergence of the tunnel $u$ with respect to the tunnel radius $R$, depends on the following non-dimensional parameters (Anagnostou and Kovari, 1993):

\[
\frac{u}{R} = F\left(\frac{E}{S}, \frac{c}{S}, \frac{q}{S}, \nu, \phi, \psi\right)
\] (5.1)

where $q$ is the internal pressure applied to the tunnel wall. If the internal pressure $q$ is supplied by the lining, the relationship reported in Equation (5.1) assumes the following aspect:

\[
\frac{u}{R} = F\left(\frac{E}{S}, \frac{c}{S}, \frac{K_s}{E}, \frac{z_0}{R}, \nu, \phi, \psi\right)
\] (5.2)

where $K_s$ is the stiffness of the lining, and $z_0$ is the distance from the face at which the lining is installed. The stiffness of thin linings (see also Chapter 4 for further details) is defined as:

\[
K_s = E \cdot \frac{e}{R}
\] (5.3)

with $E$ standing for the Young's modulus, and $e$ for the thickness.

For the problem under examination (analysis of ground-shotcrete interaction in tunneling), time-dependent phenomena should also be considered. In this case, the most important time-
dependent phenomena are: the hydration process of the shotcrete, the time-dependent behavior of the ground (e.g., consolidation, creep), the tunnel excavation process, and the construction schedule (e.g., different time spans for excavation, shotcreting, etc.).

For each phenomenon a characteristic time can be conventionally defined.

The characteristic time $\tau_{\text{hyd}}$ for the hydration process of the shotcrete depends on the adopted constitutive model. In this chapter an elastic constitutive law with time-dependent stiffness is assumed for the shotcrete. The corresponding characteristic time is defined as the time required by the shotcrete to reach 50% of its final stiffness. Generally, it is much easier to measure the strength of the shotcrete than its stiffness, especially during the early hydration of the material. In this study, since an elastic material model is employed for the shotcrete, strength increase is not considered at all. Nevertheless, it is still possible to obtain the value of the corresponding strength by means of empirical formulae that are available in the literature (see Paragraph 3.4).

Following a rock-mechanics approach, the time-dependent behavior of the rock-mass is described by viscoplasticity. The characteristic time of the viscous process in the ground is denoted as $\tau$. Since ground viscosity is not considered in the numerical analyses presented in this chapter, the corresponding evolution law will be described only in Chapter 6.

The characteristic time of the excavation process, $\tau_{1R}$, is defined in the following as the time required for the excavation and shotcreting of a tunnel length equivalent to one tunnel radius. Every value of $\tau_{1R}$ corresponds to a different value of the tunnel excavation rate $v$:

$$\tau_{1R} = \frac{R}{v} \quad (5.4)$$

Analogously, the characteristic time $\tau_{\text{sh}}$ for shotcreting is defined as the time required for the shotcreting of a tunnel length equivalent to one tunnel radius. Given a constant tunnel excavation rate, the characteristic time $\tau_{\text{sh}}$ for shotcreting is a fraction of $\tau_{1R}$. 
In conclusion the relevant time-dependent phenomena of the ground-shotcrete interaction in tunneling can be summarized in the following non-dimensional parameters: \( \tau_{1R}/\tau_{hyd} \), \( \tau/\tau_{hyd} \), and \( \tau_{sh}/\tau_{1R} \).

### 5.3 Numerical Analyses

Axisymmetric conditions are assumed in the numerical analyses presented in this Chapter. The axis of symmetry coincides with the longitudinal axis of the tunnel.

The assumption of axisymmetry restricts the numerical simulation to the following cases: isotropic state of stress, constant lithostatic state of stress, circular tunnel section, simultaneous excavation of the whole cross-section (i.e., full-face excavation), and circular lining.

Axisymmetric conditions represent a good approximation for the simulation of the excavation of deep tunnels. Actually, deep tunnels are characterized by a negligible stress increase between the top and the bottom of the tunnel in consequence of the gravity load, with respect to the mean in situ stress. In addition, a relatively high ratio between the stiffness of the rock-mass, E, and the flexural stiffness of the lining, \((E \cdot e^3)/R^3\), is found and results in negligible bending moments in the lining (i.e., the effect of lithostatic stress anisotropy on the loading of the lining is low) according to Einstein and Schwartz (1979).

The next paragraphs describe the constitutive models employed for the ground and the shotcrete and go into details of the structural model and finite difference discretization assumed in the numerical investigation. The parametric study, performed in order to evaluate the ground-shotcrete interaction in tunneling, is then described.
5.3.1 Constitutive Model for the Ground

The ground is modeled by means of a linear elasto-plastic law, characterized by a Mohr-Coulomb strength criterion. Different values of the mechanical properties of the ground are employed in the numerical analyses in order to investigate their influence on the interaction with the shotcrete lining.

5.3.2 Constitutive Model for the Shotcrete

The shotcrete is characterized by an elastic constitutive law with time-dependent stiffness. Increases in the elastic modulus as a function of time have been expressed by the empirical relationship (Chang 1994):

\[
E(t) = c_1 \cdot E_\infty \cdot e^{c_2/t^c}
\]

(5.5)

where \(c_1\), \(c_2\), \(c_3\) are material constants equal to 1.062, -0.446 and 0.6, respectively; the Poisson ratio is assumed to be time-independent (\(\nu_s = 0.3\)). The elastic modulus \(E\) is expressed in GPa, and time \(t\) in days. The elastic modulus \(E_\infty\) at complete hydration is set equal to 24 GPa.

Time does not appear directly in the material model: the elastic modulus of the shotcrete is updated during the analysis as a function of the distance from the tunnel face. The updating is evaluated at each increment of excavation in relation to the selected tunnel excavation rate.

5.3.3 Structural Model and Finite Difference Discretization

The axisymmetric problem of a circular tunnel driven through a medium with isotropic in situ stress \(S\) is considered. In all the numerical analyses the isotropic in situ state of stress \(S\)
is set equal to 1.5 MPa. The tunnel radius $R$ is assumed to be equal to 5 m while the thickness of the shotcrete lining $e$ equals 20 cm.

The dimensions of the structural model are shown in Figure 5.1. A local system of coordinates is introduced. Coordinate $z$ refers to the longitudinal direction parallel to the tunnel axis, whereas coordinate $r$ refers to the radial direction. The final face position at the end of the simulated excavation process is located at the center of the model, i.e. at $z=0$. Tunnel advancement is simulated for a tunnel length of $10R$: this distance was reputed sufficient in order to reach steady state conditions and avoid any influence of the boundary conditions applied to the model on the calculated solution.

Consideration of axisymmetric conditions implies that the initial state of stress is isotropic. In the structural model, the initial state of stress is introduced by applying a constant pressure $p_0$ at the top boundary of the model, with $p_0 = 1.5$ MPa.

The Finite Difference discretization consists of 9538 nodes. Near the tunnel face the mesh is refined in order to provide better approximations of rather large stress and strain gradients in this zone (Figure 5.2).

### 5.3.4 Excavation Scheme

The construction process was modeled by applying a step-wise procedure, as illustrated in Figure 5.3. During the excavation step the ground elements are changed into cavity elements. Application of the shotcrete is simulated by changing the appropriate cavity elements into shotcrete elements. It follows that perfect bonding is assumed between the ground elements and the shotcrete elements. The excavation step and the shotcrete installation step are repeated iteratively until the final face position at $z=0$ m is reached.

The length of the excavation step is set equal to 1 m. Hence, the length of the unsupported part of the tunnel equals 1 m as well. Consequently, also the application of the shotcrete lining is performed every 1 meter; the shotcrete lining is applied onto the unsupported tunnel wall up to the tunnel face.
Chapter 5

5.7

The time assigned to complete one meter of tunnel is divided into two equal parts: the first part is devoted to the simulation of the excavation, whereas the second part is dedicated to the simulation of the shotcrete application.

5.3.5 Performed Analyses

The numerical analyses performed in this study are devoted to the analysis of the ground-shotcrete interaction. Assuming the shotcrete behavior to be uniquely defined (see Paragraph 5.3.2), the interaction between shotcrete and lining is evaluated considering different ground mechanical parameters and different tunnel excavation rates. In particular:

1. the effect of different elastic behavior of the ground is explored by assuming different values of the elastic modulus $E$;

2. the influence of the strength of the ground is taken into account by considering different values of ground cohesion $c$, while the friction angle $\varphi$ is set constant. Actually, cohesion is a parameter that is supposed to vary much more than the friction angle for different rock-masses.

3. three tunnel excavation rates are simulated in the numerical analyses adapting the stiffness of the shotcrete lining to the face distance. Two limit tunnel excavation rates are also considered in the numerical analyses: the first, equal to zero, is equivalent to assuming a lining of constant stiffness; the second, equal to infinity, corresponds to the case of an unsupported tunnel.

It follows that each numerical analysis is marked by the values assumed by the three non-dimensional parameters $E/S$ or $K_s/E$, $c/S$ and $\tau_{1R}/\tau_{hyd}$.

The influence of friction angle $\varphi$, Poisson's ratio $\nu$, dilatancy angle $\psi$ of the ground is not considered in this study, nor is that of $z_0/R$, i.e., the shotcrete is applied always after 1 meter of excavation.
Because of the time-dependent behavior of shotcrete, the stiffness of the lining $K_s$ appearing in the ratio $K_s/E$ is conventionally calculated by referring to the final value of the stiffness assumed by the shotcrete at twenty-eight days, $E_\infty$ (see equation 5.3).

Table 5.1 summarizes the values adopted by the parameters and the corresponding non-dimensional parameters in the numerical investigation.

The values adopted for the parameters were selected in order to explore a wide range of typical tunnel conditions. Particular attention has been devoted to the investigation of difficult geotechnical conditions in relation to tunneling where the immediate application of shotcrete plays an important role.

### 5.4 Numerical Results

Figure 5.4 shows the radial displacement of the tunnel wall $u_r$ as a function of the tunnel face distance $z$ for different values of the non-dimensional parameter $E/S$. The ground is assumed to behave elastically ($c/S>0.7$).

In order to compare the different curves, the radial displacement $u_r$ is normalized with respect to the maximum value of the radial displacement $u_{r,\text{max}}$, obtained in an unlined tunnel excavated in a rock-mass characterized by the same $E/S$ ratio. The value $u_{r,\text{max}}$ of the unlined tunnel is calculated as the average value of the tunnel radial displacement between a distance from the face of $z=39$ m and $z=40$ m. Trivially, the values of the normalized radial displacement $u_r/u_{r,\text{max}}$ for an unsupported tunnel are identical for different values of $E/S$. On the contrary, tunnels supported by shotcrete are characterized by different normalized values of the radial displacement $u_r/u_{r,\text{max}}$ depending on the ratio $E/S$. The lower the ratio $E/S$, i.e. the higher the ratio $K_s/E$, the more the normalized radial displacement $u_r/u_{r,\text{max}}$ decreases with respect to that of the unsupported tunnel, i.e. the more the lining is effective in reducing the tunnel convergence.
The upper part of Figure 5.4, shows the numerical results obtained for a tunnel excavation rate equal to zero ($\tau_{1R}/\tau_{hyd} = \infty$), i.e. for a lining of constant stiffness. In contrast, the curves on the lower part of Figure 5.4 refer to a tunnel excavation rate of 8 m/d ($\tau_{1R}/\tau_{hyd} = 0.67$). In this latter case, the curves of the supported tunnel are closer to the curve of the unsupported tunnel than the curves obtained for a tunnel excavation rate equal to zero. Actually, the higher the tunnel excavation rate, the lower the stiffness of the shotcrete at equal tunnel face distances and, as a consequence, the lower the support offered by the lining to the deforming ground.

Figure 5.5 illustrates the corresponding results obtained in an elasto-plastic ground with $c/S=0.87$. It is evident that the radial displacement $u_r$ of the unsupported tunnel does not reach an equilibrium value even for $z/R=8$. On the contrary, tunnels supported by the shotcrete lining reach equilibrium conditions very soon. With respect to the case of an elastic ground, the radial displacement $u_r$ of the supported tunnel is much lower than the corresponding displacement of an unsupported tunnel. The curves of $u_r/u_{r,max}$ are closer to each other for different values of $K_s/E$ as well as for different tunnel excavation rates.

A saw-toothed profile characterizes the curves shown in Figures 5.4 and 5.5. Each saw-tooth refers to a one-meter excavation step. This shape is related to the non-uniform deformation of the ground in proximity of the face. If the lining is placed near the tunnel face, it is possible to observe the formation of a bearing arch in the ground after the excavation, spanning from the face to the first lining segment. Figure 5.6 shows a typical distribution of radial stresses in the ground in the area close to the tunnel face obtained for $K_s/E=10.7$, 60, $c/S= 0.087$, and $\tau_{1R}/\tau_{hyd}= 1.17$.

As a consequence of the non-uniform state of stress and deformation in the ground, the stress within each shotcrete segment, i.e. the shotcrete placed during the same time interval, is far from being uniform as well. Figure 5.7 shows the corresponding axial forces, $n_\phi$ and $n_z$, and bending moments, $M_\phi$ and $M_z$, in the circumferential and longitudinal directions, respectively, acting in the eight shotcrete segments closest to the tunnel face. The longitudinal force $n_z$ is one order smaller than the circumferential force $n_\phi$. The latter reaches the maximum value at the extremity of the segment closest to the tunnel face. This maximum
value is about 1.5 times greater than the average value of the circumferential force $n_\phi$, calculated for the whole segment.

In the following, only the average value of the forces acting in each shotcrete segment will be considered. The radial load applied to the lining by the ground is then calculated as:

$$q = n_\phi \frac{e}{R}$$  \hspace{1cm} (5.6)

allowing a comparison with the analytical solutions proposed by the Convergence-Confinement method.

Figure 5.8 shows the equilibrium load on the lining $q_{eq}$ normalized by the in situ stress $S$, obtained for different values of the non-dimensional parameters $E/S$ (or $K_s/E$) and $c/S$. The equilibrium conditions for the tunnel are reached when the tunnel advance no longer influences the state of stress and deformation both in the ground and in the lining. $q_{eq}$ is calculated on the shotcrete segment whose distance from the tunnel face is 6 times the tunnel radius $R$. Here, the distance from the border of the numerical model is 4 times the tunnel radius. This latter distance is considered adequate in order to avoid the influence of boundary conditions on the numerical solution.

The upper and the lower parts of Figure 5.8 show the results corresponding, respectively, to a tunnel excavation rate equal to zero ($\tau_{1R}/\tau_{hyd}=\infty$) and 8 m/d ($\tau_{1R}/\tau_{hyd}=0.67$).

For the same value of the non-dimensional parameter $c/S$, the equilibrium load $q_{eq}$ on the lining is obviously greater for lower values of the ground stiffness $E$.

Less intuitive is the influence of the parameter $c/S$ on the numerical results. As observed also by Corbetta and Nguyen-Minh (1992), a reduction of ground cohesion can produce a decrement of $q_{eq}$ if the ground stiffness is low. This is the case, for example, of the numerical analyses characterized by $E/S=60$ (i.e., $K_s/E=10.7$).

Also the simpler Convergence-Confinement method can be applied to support these results. In Figure 5.9 the convergence curves for $c/S=0.7$ (elastic ground) and $c/S=0.175$, where $E/S=60$ (i.e., $K_s/E=10.7$), are shown. The Confinement curves are determined by adopting
an elastic modulus of the lining equal to $E_x=24$ GPa (exactly equal to the value employed in the numerical analysis with $\tau_{1R}/\tau_{hyd}=\infty$). The convergence of the tunnel at the time of the installation of the lining is calculated in accordance with Equations 4.4 and 4.5: a reduction in ground cohesion results in an increment of the convergence of the tunnel at the time of lining installation. It is evident, by observing the figure, that in this case the reduction in ground cohesion produces a reduction in the equilibrium load on the lining $q_{eq}$.

The effect of the tunnel excavation rate observed in the numerical analyses is also shown in Figure 5.8. The equilibrium load on the lining $q_{eq}$ is found to be lower for a tunnel excavation rate of 8 m/d ($\tau_{1R}/\tau_{hyd}=0.67$) with respect to the case of a tunnel excavation rate equal to zero ($\tau_{1R}/\tau_{hyd}=\infty$), at a parity of $E/S$ (or $K_s/E$) and $c/S$. In fact, the higher the tunnel excavation rate, the lower the stiffness of the shotcrete at the same tunnel face distance, and hence the lower the support it can supply.

### 5.5 Proposal for a Simplified Design Method

As originally proposed by the Convergence-Confinement method (Lombardi, 1973), the effect of the excavation advancement can be simulated by a progressive reduction of the internal pressure $q$ applied at the tunnel wall from the initial litostatic pressure $S$, controlled by the ground stress release factor $\lambda$:

$$q = (1 - \lambda) \cdot S \quad (5.7)$$

The ground stress release factor $\lambda$ at the time of lining installation depends on the distance $z$ from the face to the section where the lining is installed (Panet and Guenot, 1982), but also on the relative stiffness between the ground and the lining (Kielbassa and Duddeck, 1992; Bernaud and Rousset, 1992).

Following the conventional Convergence-Confinement method the dependence of the ground stress release factor $\lambda$ on the relative ground-lining stiffness (between the ground and
the lining) is usually disregarded, and $\lambda$ is assumed to be only a function of the face distance $z$ and of ground behavior, according to the well-known Panet and Guenot expression (see Equations 4.4 and 4.5).

The results of 3D numerical analyses can be used to improve the accuracy of the conventional Convergence-Confinement method.

The approach proposed herein consists in taking advantage of 3D numerical results to back-calculate a relation factor $\lambda$, which accounts for the influence of the relative stiffness between ground and lining. Moreover, because of the time-dependent behavior of the shotcrete lining, the effect of the tunnel excavation rate is also incorporated into the back-calculated ground stress release factor $\lambda$. The back-calculation is performed in the $(q,u)$ plane starting from the point on the convergence curve given by the equilibrium load on the lining $q_{eq}$ obtained in the 3D numerical analysis (Figure 5.10). Moving back along a confinement curve characterized by a constant stiffness, set conventionally for all the cases equal to the final elastic modulus of the shotcrete $E_\infty$, it is easy to determine the corresponding value of the convergence at the time of lining installation and then the consequent relation factor $\lambda$.

Figure 5.11 shows the back-calculated values of the ground stress release factor $\lambda$ as a function of the non-dimensional parameter $E/S$ (or $K_s/E$) and of different values of $c/S$ and of $\tau_{1R}/\tau_{hyd}$. For a given ratio $c/S$, the ground stress release factor $\lambda$ increases for higher tunnel excavation rates $v$ and for higher values of the ratio $E/S$ (i.e., for lower values of the ratio $K_s/E$). The higher the ground stress release factor, the lower the value of the equilibrium load on the lining $q_{eq}$.

The $\lambda$ curves for the different tunnel excavation rates are well-differentiated for the elastic ground, while they are very close to each other for $c/S=0.044$: the influence of the tunnel excavation rate on the tunnel equilibrium conditions decreases when also the strength characteristics of the ground decrease.

For sake of clarity, the corresponding values of the ground stress release factor $\lambda_{Panet}$ obtained on the basis of the Guenot and Panet procedure are also represented in Figure 5.11. These values do not depend on the elastic modulus of the ground. They only increase with decreasing cohesion of the ground, i.e. with increasing values of the plastic radius of the
tunnel (see Equation 4.5 and relative remarks). Because in the numerical analyses the lining is installed instantaneously after every excavation step of 1 m, the factors \( \lambda_{\text{Panet}} \) were calculated by assuming an average distance of the lining installation from the tunnel face of \( z_0 = 0.5 \) m.

Large differences come from the observation of Figure 5.11 between the back-calculated \( \lambda \) and \( \lambda_{\text{Panet}} \).

In the case of elastic ground, the conventional Convergence-Confinement method generally leads to overestimate the load on the lining (stemming from the underestimation of the ground stress release factor), while in the case of low-strength elasto-plastic ground, the load on the lining is generally underestimated. In these latter cases, the main discrepancy between \( \lambda_{\text{Panet}} \) and back-calculated \( \lambda \) is found for the ground conditions characterized by lower values of the elastic modulus, when the support of the tunnel plays a determining role in assuring stability.

### 5.6 Some Critical Remarks

The numerical analyses that have been carried out are characterized by many simplified assumptions concerning the behavior of both the ground and the shotcrete and the simulation of the tunnel excavation. Two of these simplified assumptions are analyzed here in order to evaluate their effect on the obtained results.

The first simplified assumption regards the constitutive model assumed for the shotcrete. The behavior of this material is described by means of an elastic law characterized by time-dependent stiffness (see Equation 5.5). As pointed out in Paragraph 3.2, several empirical expressions can be found in the literature to describe the increase in time of the strength. They read as (Chang, 1994):
\[ f_c(t) = a_1 \cdot f_{c,\infty} \cdot e^{\frac{a_2}{t^{a_3}}} \]  

(5.8)

where \( f_c \) is the uniaxial compressive strength in MPa, \( t \) the time in days, and \( f_{c,\infty} \) the uniaxial compressive strength at complete hydration. An interpolation of a wide range of experimental data results in the following values of the constants: \( a_1=1.105 \), \( a_2=-0.743 \), and \( a_3=0.7 \). The value of \( f_{c,\infty} \) strongly depends on the shotcrete technology employed. It can vary between 20 MPa and 60 MPa.

The numerical analysis characterized by the values of non-dimensional parameters \( E/S=60 \) (\( K_s/E=10.7 \)), \( c/S>0.7 \) (elastic ground), and \( \tau_{1R}/\tau_{hyd}=1.17 \) (\( v=2 \) m/d) was repeated adopting for the shotcrete an elasto-plastic model with both elastic and strength properties increasing in time. The evolution of the uniaxial strength in time is afforded by increasing values of cohesion, while the friction angle is assumed equal to \( \phi=30^\circ \). Three different values of \( f_{c,\infty} \) are selected in the new numerical analyses: 20, 30 and 40 MPa.

Figure 5.12 shows the radial displacement of the tunnel wall \( u_r \) as a function of the face distance \( z \) obtained for different values of the final strength cohesion of shotcrete \( f_{c,\infty} \). The numerical analysis where the elastic constitutive law is employed for the shotcrete gives the lowest value of the radial displacement. Adopting an elasto-plastic model for the shotcrete, with \( f_{c,\infty}=20 \) MPa, increases the final radial displacement of the tunnel by about 50%. For higher values of \( f_{c,\infty} \) intermediate results are obtained.

The consequence of introducing an elasto-plastic behavior for the shotcrete is evident also through the evaluation of the circumferential force in the lining. In Figure 5.13, the mean circumferential force \( n_\varphi \) in the shotcrete lining is expressed as a function of time, i.e. indirectly as a function of the tunnel face distance. For completeness, the evolution of the shotcrete strength in time is also reported. The assumption of an elasto-plastic material model for shotcrete with \( f_{c,\infty}=20 \) MPa reduces the circumferential force in the shotcrete lining up to 25%. Plastic deformation occurs especially at the beginning of the loading, when the strength of the material is low. The lining is globally less loaded and the stress is more uniformly distributed inside each segment.
These results highlight the important role played by the constitutive model of shotcrete in determining the state of stress and deformation in the tunnel. Other features of shotcrete behavior, such as chemical shrinkage and creep, were not taken into account and their influence on tunnel behavior is unknown. On the basis of these considerations it was decided to carry out a deeper investigation into the influence of shotcrete behavior on the static conditions of the tunnel. An advanced shotcrete constitutive model, featuring aging elasticity, plasticity, chemical shrinkage and creep, was applied to the analysis of the tunnel, as described in Chapter 6.

The second simplified assumption that will be discussed here regards the influence of the excavation scheme on the computed results. As described in Paragraph 3.5.4, the excavation scheme follows a step-wise procedure. The excavation of 1 m of tunnel is instantaneously performed in the numerical analysis as well as the installation of a segment of 1 m of shotcrete.

A new analysis was performed using 2-m excavation steps with the shotcrete lining being installed every 2 m.

Figure 5.13 compares the circumferential forces $n_\phi$ in the shotcrete lining obtained with excavation steps of 1 m (up) and 2 m (down). The average values of the circumferential force $\bar{n}_\phi$ within each shotcrete segment applied in the same excavation step are also reported. Both numerical analyses are characterized by the following values of the non-dimensional parameters: $E/S=60$ ($K_s/E=10.7$), $c/S>0.7$ (elastic ground), and $\tau_{1R}/\tau_{hyd}=1.17$ ($v=2$ m/d).

Increases in the excavation steps cause a larger variation between the maximum and minimum stresses inside each segment of the shotcrete lining. Nevertheless, the average value of the circumferential stress $\bar{n}_\phi$ within the shotcrete segment is only slightly lower for the 2-meter excavation step.
<table>
<thead>
<tr>
<th>PARAMETERS</th>
<th>NON-DIMENSIONAL PARAMETERS</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E=90, 900, 9000 \text{ MPa}; S=1.5 \text{ MPa}$</td>
<td>$E/S = 60, 600, 6000$</td>
</tr>
<tr>
<td>$c= 1.050, 0.525, 0.262, 0.131, 0.066 \text{ MPa}; S=1.5 \text{ MPa}$</td>
<td>$c/S = 0.700, 0.350, 0.175, 0.087, 0.044$</td>
</tr>
<tr>
<td>$K_s= 960 \text{ MPa}; E= 90, 900, 9000 \text{ MPa}$</td>
<td>$K_s/E = 10.7, 1.07, 0.107$</td>
</tr>
<tr>
<td>$z_0= 0.5 \text{ m}; R= 5 \text{ m}$</td>
<td>$z_0/R = 0.1$</td>
</tr>
<tr>
<td>$\nu= 0.25$</td>
<td>$\nu= 0.25$</td>
</tr>
<tr>
<td>$\phi= 20^\circ$</td>
<td>$\phi= 20^\circ$</td>
</tr>
<tr>
<td>$\psi= 0^\circ$</td>
<td>$\psi= 0^\circ$</td>
</tr>
<tr>
<td>$\tau_{1R}= \infty, 2.50, 1.25, 0.63, 0.00 \text{ d} (v= 0, 2, 4, 8, \infty \text{ m/d})$; $\tau_{\text{hyd}}= 0.42 \text{ d}$</td>
<td>$\tau_{1R}/\tau_{\text{hyd}} = \infty, 1.17, 0.34, 0.67, 0.00$</td>
</tr>
<tr>
<td>$\tau=0 \text{ h}; \tau_{\text{hyd}}= 0.42 \text{ d}$</td>
<td>$\tau/\tau_{\text{hyd}} = 0$</td>
</tr>
<tr>
<td>$\tau_{\text{sh}}= \infty, 1.25, 0.63, 0.31, 0.00 \text{ d} (v= 0, 2, 4, 8, \infty \text{ m/d})$; $\tau_{1R}= \infty, 2.50, 1.25, 0.63, 0.00 \text{ d} (v= 0, 2, 4, 8, \infty \text{ m/d})$</td>
<td>$\tau_{\text{sh}}/\tau_{1R} = 0.5$</td>
</tr>
</tbody>
</table>

Table 5.1: Parameters and corresponding non-dimensional parameters employed in the numerical analyses.
5.7 Figures of Chapter 5

(a) 

(b) 

Figure 5.1: Dimension of the structural model employed in the numerical analyses.
Figure 5.2: Particular of the mesh employed in the numerical analyses.

Figure 5.3: Excavation scheme.
Figure 5.4: Normalized radial displacement of the tunnel wall $u/\bar{u}_{r,\text{max}}$ as a function of the normalized tunnel face distance $z/R$ for different values of the non-dimensional parameters $E/S$ and $\tau_{1R}/\tau_{\text{hyd}}$ (elastic ground $c/S>0.7$).
Figure 5.5: Normalized radial displacement of the tunnel wall \( u_r/u_{r,\text{max}} \) as a function of the normalized tunnel face distance \( z/R \) for different values of the non-dimensional parameters \( E/S \) and \( \tau_{1R}/\tau_{\text{hyd}} \) (elasto-plastic ground \( c/S=0.87 \)).
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E/S = 60 (Ks/E=10.7)
c/S = 0.087
\( \tau_{IR}/\tau_{hyd} = 1.17 \) (v=2 m/d)

Figure 5.6: Distribution of radial stress in the ground in proximity of the face.

Figure 5.7: Axial forces, \( n_\phi \) and \( n_z \), and bending moments, \( M_\phi \) and \( M_z \), acting in each shotcrete segment in circumferential and longitudinal directions, respectively.
Figure 5.8: Equilibrium load in the shotcrete lining $q_{eq}$ for different values of the non-dimensional parameters $c/S$ and $E/S$. 
Figure 5.9: Convergence and Confinement curves for elastic ($c/S>0.7$) and elasto-plastic ($c/S=0.175$) ground given $E/S=60$ ($K_s/E=10.7$).

Figure 5.10: Back-calculation of the ground stress release factor $\lambda$. 
Figure 5.11: Back-calculated values of the ground stress release factor $\lambda$ for different values of $E/S$, $c/S$, and $\tau_{1R}/\tau_{hyd}$. 
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5.25

Figure 5.12: Radial displacement of the tunnel wall $u_r$ as a function of the face distance obtained for different values of final strength of shotcrete $f_{c,\infty}$.

Figure 5.13: Mean circumferential force $\bar{n}_\phi$ and strength $f_c$ of shotcrete lining as a function of time.
Figure 5.14: Circumferential force $n_\varphi$ and mean circumferential force $\bar{n}_\varphi$ in the shotcrete lining obtained for excavation steps of 1 (up) and 2 m (down).
6. The Role of an Advanced Shotcrete Constitutive Model for Reliable Prediction in Tunneling

6.1 Introduction

Chapter 5 provided a description of the numerical analyses for tunnels supported by shotcrete. The investigation was characterized by the assumption of the shotcrete having the behavior of an elastic material, with increasing stiffness controlled by means of an empirical material function.

The shotcrete behavior was found to strongly influence the static conditions of both ground and lining.

Consequently, further research was deemed necessary to better understand the influence of shotcrete behavior in tunneling. For the simulation of the mechanical behavior of shotcrete, a realistic material model, recently developed at the Vienna University of Technology, is employed in this chapter (see Paragraph 3.5.1). The effect of the hydration of shotcrete on strength, stiffness, and chemical shrinkage is considered. Moreover, microcracking as well as creep are accounted for.

For the description of the mechanical behavior of the ground, a viscoplasticity material model is employed.

The investigation of the ground-shotcrete interaction in tunneling is based on an extensive parametric analyses accounting for each phenomenon in the material model for shotcrete and different mechanical properties of the rock-mass (i.e., stiffness, strength, time-dependent
behavior). Moreover, the influence of the construction schedule of the tunnel is also investigated by adopting different excavation rates in the numerical analyses.

6.2 Constitutive Model for Ground and Shotcrete

The mechanical behavior of the ground is described by means of a multi-surface viscoplasticity model. It consists of the Drucker-Prager and the tension-cut-off criterion, reading:

\[ f_{DP}(\sigma, \zeta_{DP}) = \sqrt{J_2} + \alpha \cdot I_1 - \zeta_{DP} / \beta \quad \text{and} \quad f_{TC}(\sigma, \zeta_{TC}) = I_1 - \zeta_{TC} \]

(6.1)

where \( \zeta_{DP} \) and \( \zeta_{TC} \) represent the hardening forces of the Drucker-Prager and the tension-cut-off criterion, respectively. The parameters \( \alpha \) and \( \beta \) are computed from cohesion \( c \) and the angle of internal friction \( \varphi \), such that the Drucker-Prager meridian coincides with the compression meridian of the respective Mohr-Coloumb criterion. This yields:

\[ \alpha(\varphi) = \frac{2 \sin \varphi}{\sqrt{3(3 - \sin \varphi)}} \]
\[ \beta(c, \varphi, f_c) = \frac{f_c}{\sqrt{3c}} \left( \frac{3 - \sin \varphi}{2 \cos \varphi} \right) \]

with \( f_c(c, \varphi) = \frac{2c \cos \varphi}{1 - \sin \varphi} \)

(6.2)

which denotes the uniaxial compressive strength of the material. The evolution law for the viscoplastic strain tensor is chosen according to the law by Duvaut and Lions (1972),

\[ \dot{\varepsilon}^{vp} = -C^{-1}(\sigma - \sigma^\infty) \]

(6.3)

where \( C \) denotes the elastic material tensor. In Equation (6.3), \( \tau \) is the relaxation time and \( \sigma^\infty \) corresponds to the solution for rate-independent elastoplasticity, i.e., to the solution for infinitely slow loading.
The constitutive model described in Paragraph 3.5.1 is adopted for shotcrete.

6.3 Numerical Study

The described material models for ground and for shotcrete are employed for numerical analyses of a tunnel driven by full-face excavation and supported by shotcrete. The analyses were performed by means of the finite element method (FEM).

6.3.1 Geometric Dimensions and Initial Stress State

The geometric properties of the considered tunnel cross-section are given in Figure 6.1(a). A circular tunnel with a radius of $R=7$ m is chosen. After excavation, a closed shotcrete shell with a thickness $e$ of $0.3$ m is assumed.

The excavation of the tunnel is simulated by means of axisymmetric analyses. The dimensions of the structural model are shown in Figure 6.1(b). A local coordinate system is introduced, as described in Chapter 5.

Consideration of axisymmetric conditions implies that the initial state of stress is isotropic. Commonly, the stress level is set equal to the average of the horizontal and vertical component of the in situ stress state at the center of the tunnel, denoted as $S$. In the numerical model, $S$ is introduced by the application of a constant pressure $p_0$ at the top boundary of the model (see Figure 6.1(b)).

Figure 6.2 shows the FE discretization consisting of 15266 axisymmetric finite elements.
6.3.2 Excavation Scheme

During the simulation, the tunnel face moves from the right boundary of the structural model leftwards, until it reaches the center of the structural model at \(z=0\). The excavation of the tunnel is simulated by replacing the respective ground elements by cavity elements. The latter are characterized by a negligible small stiffness. Application of shotcrete is modeled by replacing the respective cavity elements by shotcrete elements. The shotcrete shell is discretized over the thickness by means of five rows of finite elements.

The length of one excavation step is set equal to 1 m. Hence, the length of the unsupported part \(z_0\) of the tunnel equals 1 m. This is a suitable value for tunneling in weak rock-masses characterized by applying shotcrete immediately after the excavation. The time assigned to complete 1 m of tunnel is divided into two parts: 2/3 of the time is devoted to the excavation, the remaining 1/3 to the application of shotcrete.

6.3.3 Material Parameters for Ground and Shotcrete

Because of the frequently changing geological conditions during the tunnel excavation, the influence of ground properties on the ground-shotcrete interaction is dealt with in this chapter. Accordingly, Young's modulus \(E\), the cohesion \(c\), and the relaxation time \(\tau\) of the ground will be varied. The original, starting parameters describing the ground behavior are listed in Table 6.1.

For the simulation of shotcrete the material properties listed in Table 3.2 will be employed. In order to consider the hydration kinetics of shotcrete in the following investigation, a characteristic time of the hydration process, \(\tau_{\text{hydr}}\), is introduced. It is related to the maximum slope in the evolution of the hydration degree \(\xi\). The evolution law of the degree of hydration is recalled here from Chapter 3

\[
\dot{\xi} = \tilde{A}(\xi) \exp\left(-\frac{E_a}{RT}\right),
\]  

(6.4)
where $\tilde{A}(\xi)$ represents the normalized chemical affinity (see also Paragraph 3.5.1).

In case of isothermal conditions with $T=293\, \text{K}$ and $E_a/R=4000\, \text{K}$, this slope is obtained from

\[
\max\left[ \frac{\partial \xi}{\partial t} \right] = \max\left\{ \tilde{A}(\xi) \right\} \cdot \exp\left( -\frac{E_a}{RT} \right), \quad (6.5)
\]

finally giving $\tau_{\text{hyd}}$ in the form

\[
\tau_{\text{hyd}} = \frac{1}{\max\left[ \frac{\partial \xi}{\partial t} \right]} = \frac{1}{\max\left\{ \tilde{A}(\xi) \right\}} \cdot \exp\left( \frac{E_a}{RT} \right) = \frac{1}{5.68 \cdot 3600} \cdot \exp\left( \frac{4000}{293} \right) = 41.5\, \text{h}
\]

(6.6)

6.3.4 Analyses Performed

The convergence of the tunnel, as well as the loading of the shotcrete lining, depend on the mechanical behavior of the shotcrete, the mechanical behavior of the ground, and the construction characteristics of the tunnel. In order to cover a wide range of tunneling conditions, five different numerical studies were performed:

- The first study focuses on the significance of the chemomechanical model for shotcrete. Four analyses characterized by different stages of modeling are performed.

- In the second study, the effect of different deformability of the rock-mass is investigated by assuming different values of Young's modulus $E$;

- In the third study, the ground cohesion values $c$ are changed in order to evaluate the tunnel conditions for different strengths of the rock-mass;

- The influence of the time-dependent ground properties on the deformation of the tunnel and the loading of the tunnel lining is investigated in the fourth study. Hereby, the characteristic time for the viscoplastic behavior of the ground, $\tau$, is changed;
• In the last study, the influence of the tunnel excavation rate on the numerical results is investigated.

As pointed out in Chapter 5, the problems can be analyzed in terms of non-dimensional parameters. Accordingly, the influence of the following non-dimensional parameters is investigated in this chapter: E/S, c/S, τ/τ_{hyd}, τ_{1R}/τ_{hyd}.

All the symbols are consistent with the definitions given in Chapter 5.

The non-dimensional parameters assumed in the reference analysis are listed in Table 6.2. With the exception of the first study, the complete mechanical behavior of shotcrete is considered (see analysis M4 in the next Paragraph).

### 6.4 Significance of the Chemomechanical Model for Shotcrete

In order to assess the effects of the different phenomena of shotcrete, such as chemical shrinkage, microcracking, and creep, four analyses have been performed:

• M1: In the first analysis, an elastic material response for shotcrete with constant material properties is employed. The incremental stress-strain law reported in Equation 3.14 for this type of analysis can be reduced to

\[
\Delta \sigma = C_\infty : \Delta \varepsilon \Rightarrow \Delta \sigma = C_\infty : \varepsilon, \quad (6.7)
\]

where $C_\infty$ refers to the elastic material tensor at the end of the hydration process.

• M2: In the second analysis, aging of shotcrete is considered. Moreover, chemical shrinkage is accounted for. The respective stress-strain law reads

\[
\Delta \sigma = C(\xi) : \left( \Delta \varepsilon - \Delta \varepsilon^s \right), \quad (6.8)
\]
• M3: Consideration of microcracking of shotcrete, which is controlled by the previously described multi-surface chemoplastic material model, is the essential feature of the third analysis. The stress-strain law takes the form of

\[
\Delta \sigma = C(\xi) \left( \Delta \varepsilon - \Delta \varepsilon^s - \Delta \varepsilon^p \right).
\]  

(6.9)

• M4: In addition to microcracking, the influence of creep of shotcrete is considered in the final analysis. Viscous and flow strains are added to the stress-strain law, giving

\[
\Delta \sigma = C(\xi) \left( \Delta \varepsilon - \Delta \varepsilon^s - \Delta \varepsilon^p - \Delta \varepsilon^v - \Delta \varepsilon^f \right).
\]  

(6.10)

The ground and the constructive details of the tunnel are modeled according to the reference analysis described in the previous Paragraph.

Figure 6.3 shows the radial deformation \( u_r/R \) of the tunnel wall at \( r=R \) as a function of the distance from the tunnel face \( z/R \), obtained at the time instant \( t=840 \) h. At this time instant, the tunnel face has reached its final position at \( z=0 \) m for an excavation rate of 2 m/day: \( t=70/2=35 \) days = 840 hours (see Figure 6.1(b)).

As regards the radial deformation of the ground ahead of the tunnel face, i.e., for \( z/R<0 \), little influence of the material model for shotcrete is observed. For all analyses, a pre-deformation in the radial direction of approximately 9 mm is obtained at the tunnel face. The pre-deformation decreases rapidly with increasing distance from the tunnel face. At \( z=-21 \) m, i.e., at a distance of 3R, the size of \( u_r \) is lower than 1 mm.

The radial displacement \( u_r \) behind the tunnel face, i.e., for \( z/R>0 \), is characterized by a saw-toothed shape. Each saw-tooth refers to one excavation step of 1 m. The change of \( u_r \) within one saw-tooth indicates the variation of deformation and loading within the respective shotcrete segment. For the sake of clarity, the distribution of the average value for each excavation step of 1 m has been added in Figure 6.3.

For all analyses, equilibrium between the ongoing excavation process (face effects) and the aging shotcrete lining is reached at a distance from the tunnel face of approximately \( z/R=2 \).
The continuous increase of the radial deformation observed for the analyses M2 to M4 even for \(z/R>2\) stems from chemical shrinkage of shotcrete.

The decrease of \(u_r/R\) at \(z/R>8\) stems from the influence of the boundary condition of the structural model at \(z/R=10\) (see Figure 6.1(b)).

Consideration of aging, microcracking, and, finally, of creep results in an increase in the compliance of the shotcrete lining and, hence, in an increase of the radial deformations of the tunnel (analyses M2 to M4 in Figure 6.3). Accordingly, use of the elastic material model in the context of the analysis M1 led to severe underestimation of the radial deformation and to a significant increase of the hoop force in the lining (see Figure 6.5).

Creep in the shotcrete is found to contribute mostly to the early deformation of the shotcrete located between the tunnel face at \(z/R=0\) and \(z/R=3\). For \(z/R>3\), the radial displacements obtained from the analyses M2 to M4 are almost parallel.

Similar to the situation for the radial displacement ahead of the tunnel face (see Figure 6.3), the influence of the employed material model on the maximum extrusion, i.e., on the horizontal displacement \(u_h\) of the ground ahead of the tunnel face at \(r=0\), is low (Figure 6.4). For all analyses, an extrusion of approximately 5 cm was obtained at the tunnel face. The elastic analysis (analysis M1) gives the smallest value whereas the analysis considering aging, microcracking, and creep (analysis M4) results in the largest value.

The distribution of the hoop force in the lining is depicted in Figure 6.5. The hoop force in the lining, as well as the longitudinal force introduce after, are normalized with respect to the final compressive force of the shotcrete \(n_{c,n}=f_{c,n,e}=39.6\cdot0.3=11.88\text{ MN/m}\). For the analyses M1 and M2, which are characterized by elastic and chemoelastic material behavior, respectively, of shotcrete, an abrupt increase in compressive loading in the lining right after the application of shotcrete is observed. In the analysis M1, \(n_{\psi}/n_{c,n}>1\) for \(z/R>0.5\), i.e. the maximum compressive strength of the shotcrete material is exceeded. Consideration of microcracking (analysis M3) results in a significant reduction in compressive loading in the region near the tunnel face. The respective distribution of the plastic stretch in the circumferential direction, \(\delta_{\psi}\) with
where $\varepsilon_\phi^p$ denotes the plastic strain in the circumferential direction and $e$ is the thickness of the shotcrete lining, is given in Figure 6.6. Near the tunnel face, the reduction in the compressive hoop force $n_\phi$ (Figure 6.5) is accompanied by an abrupt increase of the compressive plastic stretch $\delta_\phi^p$. Except for this region, however, the distribution of $\delta_\phi^p$ remains constant. Hence, no additional microcracking occurs. Accordingly, the distribution of $n_\phi/n_{c,\infty}$ in Figure 6.5 obtained from the analyses M2 and M3 are almost parallel.

Consideration of creep (analysis M4) leads to a further reduction in compressive loading. The hoop force at equilibrium is smaller than the respective force obtained from the analyses M1 to M3. The analysis based on elastic material response of the shotcrete (analysis M1) results in an overestimation of the mean value of the hoop force at equilibrium by 150%.

Figure 6.7 shows the distribution of the normalized longitudinal force $n_z/n_{c,\infty}$ in the tunnel lining. For analysis M1, an increase in compressive loading of the lining up to a distance from the tunnel face of approximately $z/R=1$ is observed. This increase has two sources: first, the deformations of the lining caused by excavation induce a bending moment in the lining, as illustrated by the distribution of the longitudinal stress component depicted in Figure 6.8. Tension is observed at the outer surface and compression at the inner surface of the lining. Because of the ground-shotcrete interaction, the in situ compressive state of stress in the ground is reduced ($\sigma_z<5.175$ MPa). This results, on the other hand, in the observed increase in compressive loading in the lining. The second source is related to the hoop force. Because of Poisson's effect, the increase in compressive loading in the circumferential direction (Figure 6.5) results in an increase in compressive loading in the longitudinal direction.

In order to assess the influence of the mentioned sources on the compressive loading in the longitudinal direction of the lining, analysis M1 was repeated, setting $\nu=0$ for the shotcrete. Hence, the respective distribution of $n_z/n_{c,\infty}$ shown in Figure 6.7, stems only from deformations in the lining caused by the excavation. Near the tunnel face, the excavation-induced bending of the ground-shotcrete compound structure dominates the development of
compressive forces in the longitudinal direction. For $z/R>0.7$, however, a reduction in compressive loading is observed which, as will be explained in the following, stems from longitudinal deformations in the lining.

Figure 6.9 shows the change in the radial and longitudinal displacement, $\Delta u_r$ and $\Delta u_z$, respectively, resulting from the final excavation step (excavation of the tunnel from $z=1$ to 0 m). The shotcrete lining in this part of the tunnel has not yet been installed. Figure 6.9 indicates that the magnitude of both $\Delta u_r$ and $\Delta u_z$ rapidly decrease in the part of the tunnel where the shotcrete lining has already been installed ($z \geq 1$ m). Accordingly, $\partial \Delta u_z / \partial z > 0$, which results in tensile loading and, hence, in the observed decrease of compressive loading of the lining in the longitudinal direction.

For analysis M1, characterized by $\nu = 0.2$ for shotcrete, the hoop force compensates the reduction in the compressive force by the longitudinal displacements. For $z/R>1$, the distribution of $n_z/n_c,\infty$ is almost constant (Figure 6.7).

For analysis M2, characterized by the consideration of the aging of the shotcrete, the larger compliance of the lining results in an increase of $\Delta u_z$ (Figure 6.9). Accordingly, the excavation-induced bending of the ground-shotcrete compound structure increases, resulting in larger compressive forces in the longitudinal direction close to the tunnel face (Figure 6.7).

For analysis M3, the higher compliance of the lining, resulting from the consideration of microcracking, leads to a further increase in compressive loading near the tunnel face.

Consideration of chemical shrinkage (analyses M2 to M4) results in a decrease in compressive forces at increasing distances from the tunnel face, i.e., with time (see Figure 6.7). For the analysis based on a chemoelastic material model for shotcrete (analysis M2), this results even in tensile loading of the lining. The maximum tensile strength of the material is reached at $z/R=7$ (n.b. tensile strength is set equal to 0.1 the compressive strength in the shotcrete model). Consideration of creep in the final analysis M4 reduces the loading arising from the deformations due to the excavation as well as the loading caused by chemical shrinkage. No tensile forces in the longitudinal direction are observed for this analysis.
It is noteworthy, that in contrast to the hoop force, no equilibrium was obtained for the longitudinal force $n_z$ in the lining. For analysis M1 characterized by $\nu=0$ for the shotcrete, a continuous decrease of $n_z/n_{c,\infty}$ is observed (Figure 6.7). Tensile loading, induced by longitudinal deformations at each excavation step (Figure 6.9), is transferred via the lining to the boundary of the structural model at $z/R=10$, resulting in the observed decrease in tensile loading throughout the entire lining (Figure 6.7). For the hoop force, on the other hand, it is well known from shell theory that the axial forces resulting from loads perpendicular to the axis of a cylindrical shell decline exponentially (see Timoshenko (1940)). Hence, in case of tunneling, where the cylindrical shell is the tunnel lining and the loads perpendicular to the axis of the lining arise from the excavation of the tunnel, equilibrium can be reached at a certain distance from the tunnel face.

### 6.5 Influence of Ground Deformability

The second study deals with the influence of ground elastic behavior on the observed ground-shotcrete interaction in tunneling. Three different values of Young's modulus of the ground are adopted: $E=450$ MPa (analysis E1), $E=2250$ MPa (reference analysis denoted here as E2), and $E=11250$ (analysis E3). It follows that $E/S=87, 435, \text{ and } 2174$, respectively.

Figure 6.10 shows the radial deformation $u_r/R$ of the tunnel wall as a function of the tunnel face distance $z/R$ at $t=840$ h. At the tunnel face, i.e. at $z/R=0$, radial displacement obtained in analysis E2 is about 4.5 times the value obtained in analysis E1, as well as the radial displacement obtained in analysis E3 is about 4.5 times the value obtained in analysis E2.

For $z/R>0$, $u_r/R$ greatly increases in the three analyses reaching almost equilibrium conditions for $z/R=2$ (as explained in Paragraph 6.4, the slight increase in convergence for $z/R>2$ is related to the chemical shrinkage of the shotcrete). Decreasing values of Young's modulus of the ground result in increasing tunnel radial deformations. Anyhow, as denoted in Chapter 5, the lower the ratio $E/S$ (i.e., the higher the ratio $K_s/E$ is), the greater the effectiveness of the lining in reducing tunnel convergence. Actually, the final value of the
radial displacement $u_r$ in analysis E3 is about 5 times larger than the corresponding radial displacement at $z/R=0$. The ratio between the final radial displacement and the radial displacement at the tunnel face is only 3.5 in analysis E2 and 2.3 in analysis E1.

As well as tunnel convergence, lower values of Young's modulus are associated with higher values of the longitudinal displacements of the tunnel for $r=0$ (Figure 6.11). At the tunnel face, i.e., at $z/R=0$, the maximum extrusion achieved in the analysis E2 is almost 4.5 times the maximum extrusion of analysis E1. The same ratio is obtained between the values resulting from analysis E3 and E2, respectively. The influence zone of tunnel excavation on ground displacements ahead of the tunnel face, i.e., for $z/R<0$, increases for lower values of the non-dimensional parameter $E/S$, both in the radial (Figure 6.10) and in the longitudinal (Figure 6.11) directions.

The distribution of hoop forces in the shotcrete lining is shown in Figure 6.12. The higher deformability of the ground results in larger loading of the shotcrete lining. For analysis E1, the final hoop force in the lining is very high, reaching $n_{qf}/n_{cs}=0.8$ at equilibrium.

The corresponding longitudinal forces are reported in Figure 6.13. As clarified in the previous Paragraph, the compressive longitudinal forces in the lining are related to the compressive hoop forces via Poisson's effect and to the excavation-induced bending of the ground-shotcrete compound structure while the tensile longitudinal forces are associated with the chemical shrinkage of the shotcrete and with the deformations in the longitudinal direction caused by the excavation. In analysis E1, the compressive longitudinal forces prevail because of the high value assumed by the hoop forces during the lining loading. On the contrary, the tensile longitudinal forces are predominant in analysis E3 where finally tensile stresses are induced in the lining for $z/R>0.35$. 
6.6 Influence of Ground Strength

The influence of ground strength is investigated in this third study. Different strengths of the rock-mass are assumed by adopting three cohesion values. On the contrary, the friction angle is maintained constant. The first analysis, denoted as C1, is characterized by a ground cohesion $c=0.2$ MPa ($c/S=0.04$). The second analysis is the reference analysis ($c=0.6$ MPa, $c/S=0.12$) and will be denoted in this study as C2. Finally, in the third analysis, called C3, the ground cohesion is set equals to $c=1$ MPa ($c/S=0.19$). The other non-dimensional parameters are set equal to $E/S= 2250/5.175= 435$, $\tau/\tau_{hyd}= 0/41.5= 0$, and $\tau_{1R}/\tau_{hyd}= 84/41.5= 2.02$.

The influence of decreasing or increasing ground cohesion on tunnel conditions is similar to that observed for decreasing or increasing Young's modulus of the ground in the previous Paragraph.

Figure 6.14 shows the radial deformation $u_r/R$ of the tunnel obtained in the three analyses. The larger value of radial displacement is achieved in analysis C1, both at the tunnel face (i.e., at $z/R=0$) and at a great distance from the face. Anyhow, as observed similarly in the second study of Yong's modulus, the lower the ground cohesion the lower the increment of radial deformation $u_r/R$ during the tunnel excavation if compared to the value measured at the tunnel face. As a result the ratio between the "equilibrium" radial displacement and the radial displacement at the tunnel face is equal to 1.95 for analysis C1, 3.56 for analysis C2, and 4.10 for analysis C3.

The longitudinal displacements of the ground ahead of the tunnel face for $r=0$ are reported in Figure 6.15. Decreasing values of ground cohesion result in increasing values of ground displacements ahead of the tunnel face. In analysis C1, ground extrusion for $z/R=0$ is approximately 6 times the value obtained in analysis C2 and 16 times the value obtained in analysis C3.

The larger deformation of the ground is associated here with larger loading of the shotcrete lining. The distribution of the normalized hoop force $n_q/n_{c,r}$ in the shotcrete lining is shown in Figure 6.16. The maximum hoop force in the shotcrete lining is obtained in analysis C1 where $n_q/n_{c,r}=0.6$. Lower values are achieved in analyses C2 and C3.
Higher values of compressive hoop forces in the shotcrete lining correspond to higher compressive longitudinal forces via Poisson's effect (Figure 6.17). In analyses C1 and C2 an increase in compressive longitudinal force is manifest until $z/R=2$. For $z/R>2$ the longitudinal force decreases in both cases because of the chemical shrinkage of the shotcrete. In analysis C3, the reduction in longitudinal force prevail, because of chemical shrinkage, even near the tunnel face leading to tensile loading for $z/R>0.8$. For $z/R>2$, the curves obtained in the three analyses are almost parallel.

### 6.7 Influence of Ground Viscosity

The fourth study reported in this chapter focuses on the influence of creep in the ground on the deformation and on the loading of the lining. In addition to the reference analysis characterized by disregard of viscous effects in the ground (in the following referred to as analysis V1), one analysis each with $\tau=1.5\; \text{h}$ (analysis V2) and $\tau=15\; \text{h}$ (analysis V3), respectively, were performed. The corresponding values of the non-dimensional parameter $\tau/\tau_{\text{hyd}}$ are: 0 (analysis V1), 0.036 (analysis V2), and 0.36 (analysis V3).

Consideration of creep in the ground results in a delay of the release of the in situ state of stress after excavation. The radial deformation $u_r/R$ at the tunnel face, i.e., at $z/R=0$ m, increases with decreasing values for $\tau/\tau_{\text{hyd}}$ (Figure 6.18). The deviations between the distributions of $u_r/R$ obtained from the analyses increase until a distance from the tunnel face of approximately $4R$. Large values of the characteristic time $\tau$ lead to delayed loading of the already mature shotcrete. Small values of $\tau$, on the other hand, result in loading of the young shotcrete and, because of a lack of support of the shotcrete lining, in an increase of deformations. As reported in the first study, chemical shrinkage leads to a continuous increase of $u_r/R$ even at larger distances from the tunnel face. For $z/R>8$, the results are influenced by the boundary conditions of the structural model.

The normalized extrusion of the tunnel face $u_z/R$ at $r=0$ is shown in Figure 6.19. For $z/R<-1$, $u_z/R$ is smaller than 0.001 and the influence of the different values for the characteristic time
of creep of the ground is negligible. Close to the tunnel face, however, rather large values are obtained for the longitudinal displacement. The largest $u_z/R$ value is approximately 0.008. It is obtained for analysis V1. Similar to the radial displacements (see Figure 6.18), creep in the ground reduces the ground deformation caused by the excavation: the larger the ratio $\tau/\tau_{hyd}$, the smaller the longitudinal displacement ahead of the tunnel face.

The distribution of the hoop forces is shown in Figure 6.20. Consideration of creep in the ground results in a delay of the release of the in situ ground stress. With increasing values for the characteristic time $\tau$, the increase in the compressive hoop force is delayed. The average hoop force at equilibrium, however, is the same for both analyses characterized by consideration of creep, i.e., for the analyses V2 and V3. Disregard of creep in the ground (analysis V1), on the other hand, results in larger deformations before installation of the shotcrete lining, see the distribution of $u_R/R$ close to the tunnel face in Figure 6.18. Hence, the deformations occurring in the round after application of the shotcrete lining result in a smaller loading of the lining, as depicted in Figure 6.20.

The distribution of the longitudinal force in the lining is shown in Figure 6.21. As pointed out in the first study, the distribution of the longitudinal force obtained from analysis V1 is a function of excavation-induced bending, the hoop forces via Poisson's effect, the deformations in the longitudinal direction, and chemical shrinkage. As regards analysis V2, a tensile longitudinal force prevails for $z/R>5$. The analysis V3 indicates tensile loading in the longitudinal direction already close to the tunnel face. Figure 6.22 shows the change in the radial and the longitudinal displacement components, $\Delta u_r$ and $\Delta u_z$, respectively, resulting from the final excavation as obtained from analyses V1 and V3. Besides quantitative deviations between the plotted displacement changes (excavation-induced bending of the ground-shotcrete compound structure is significantly smaller for analysis V3), qualitative differences are observed close to the tunnel face. In contrast to the results from analysis V1, according to analysis V3 the ground moves towards the tunnel face. This results in an increase in tensile loading in the lining as depicted in Figure 6.21, especially in the area close to the tunnel face, i.e., for $0<z/R<2$. For $z/R>2$, chemical shrinkage dominates the loading of the lining. For analysis V3, however, the delay of loading because of creep results in a continuous increase in the compressive hoop force (Figure 6.20). Hence, the increase in
tensile loading in the longitudinal direction because of chemical shrinkage is delayed by the increasing hoop force via Poisson's effect.

6.8 Effect of Tunnel Excavation Rate

Based on a closed control-cycle of the tunnel-driving process, consisting of "shotcreting -- rock bolting -- installing measurement devices -- observing/monitoring -- adjusting design parameters", the original design is continuously updated on site. Such updates involve the excavation rate of the tunnel. In order to investigate the influence of the excavation rate on the final load of the lining and on tunnel convergence, three different excavation rates were considered in this last study: 2, 4, and 8 m/day, corresponding to $\tau_{1R}/\tau_{hyd}=2.02, 1.01, 0.51$, respectively. The respective analyses are denoted as S1, S2, and S3. Because of the different excavation rates, the final position of the tunnel face at $z/R=0$ is reached at different time instants, namely $t=840, 420, \text{ and } 210 \text{ h}$, respectively.

Figure 6.23 shows the influence of the excavation rate on the radial displacement of the tunnel surface. The radial deformation $u_r/R$ is plotted as a function of the normalized distance from the tunnel face, $z/R$. It refers to the time instants $t=840, 420, \text{ and } 210 \text{ h}$ for analyses S1, S2, and S3, respectively. Expectedly, an increase in the excavation rate results in an increase in the deformation of the tunnel. At larger excavation rates, the shotcrete has less time for developing strength and stiffness. Consequently, the main deviations between the different excavation rates are observed close to the tunnel face ($z/R<2$) where the young shotcrete has not yet reached its final strength and stiffness for supporting the inward-moving ground mass. For a distance $z/R>2$ from the tunnel face, on the other hand, the obtained distributions are almost parallel. Again, boundary effects are observed for $z/R>8$.

In general, the larger compliance of the young shotcrete in case of a larger excavation rate results in a decrease in the loading of the shotcrete lining. The largest values of the hoop forces in the shotcrete lining are obtained for analysis S1, which is characterized by the smallest excavation rate (see Figure 6.24). The variation of the hoop force within a shotcrete
segment is greater than the variation obtained from analyses S2 and S3. The variation of the forces within one shotcrete segment is a function of the stiffness of the shotcrete near the tunnel face, which is greater for smaller excavation rates.

The distribution of the longitudinal force in the shotcrete lining is shown in Figure 6.25. Similar to the distribution obtained from the first study, the increase in compressive longitudinal forces close to the tunnel face is related to excavation-induced bending of the ground-shotcrete compound structure and to the hoop forces via Poisson's effect. Deformations in the longitudinal direction caused by the excavation and by chemical shrinkage result in a reduction in compressive loading for $z/R > 1$. In the analysis S3, the age of the shotcrete (210 h at $z/R = 10$) is significantly smaller than the shotcrete age in analysis S1 (840 h at $z/R = 10$). Hence, the rate of dilation resulting from shrinkage is smaller. This explains the different slopes in the distribution of the longitudinal force given in Figure 6.25. The distribution corresponding to the smallest excavation rate (analysis S1), i.e., to the oldest shotcrete, is characterized by the largest slope, whereas the distribution corresponding to the largest excavation rate (analysis S3), i.e., to the youngest shotcrete, has the smallest slope.
Table 6.1: Mechanical properties of the ground

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young’s modulus</td>
<td>E [MPa]</td>
</tr>
<tr>
<td></td>
<td>2250</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>ν [-]</td>
</tr>
<tr>
<td></td>
<td>0.33</td>
</tr>
<tr>
<td>Cohesion</td>
<td>c [MPa]</td>
</tr>
<tr>
<td></td>
<td>0.6</td>
</tr>
<tr>
<td>Angle of friction</td>
<td>φ [-]</td>
</tr>
<tr>
<td></td>
<td>18°</td>
</tr>
<tr>
<td>Dilation angle</td>
<td>Ψ [-]</td>
</tr>
<tr>
<td></td>
<td>18°</td>
</tr>
<tr>
<td>Relaxation time</td>
<td>τ [h]</td>
</tr>
<tr>
<td></td>
<td>0</td>
</tr>
</tbody>
</table>

Table 6.2: Parameters and corresponding non-dimensional parameters employed for the reference analysis in the numerical studies.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Non-Dimensional Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>E=2250 MPa; S=5.175 MPa</td>
<td>E = 435</td>
</tr>
<tr>
<td></td>
<td>E/S = 0.12</td>
</tr>
<tr>
<td>c=0.6 MPa; S=5.175 MPa</td>
<td>c/S = 0.07</td>
</tr>
<tr>
<td></td>
<td>ν = 0.33</td>
</tr>
<tr>
<td>z₀=0.5 m; R=7 m</td>
<td>z₀/R = 0.07</td>
</tr>
<tr>
<td></td>
<td>φ = 18°</td>
</tr>
<tr>
<td></td>
<td>ψ = 18°</td>
</tr>
<tr>
<td>τ₁R=84 h (v=2 m/d); τ₃ḥyd=41.5 h</td>
<td>τ₁R/τ₃ḥyd = 2.02</td>
</tr>
<tr>
<td>τ=0 h; τ₃ḥyd=41.5 h</td>
<td>τ/τ₃ḥyd = 0</td>
</tr>
<tr>
<td>τ₃ḥ=28 h; τ₁R=84 h (v=2 m/d)</td>
<td>τ₃ḥ/τ₁R = 0.33</td>
</tr>
</tbody>
</table>
6.9 Figures of Chapter 6

Figure 6.1: Numerical study: (a) cross-section of the tunnel and (b) geometric dimension of structural model.

Figure 6.2: Numerical study: finite element mesh consisting of 15266 axisymmetric four-node elements.
Figure 6.3: Numerical study based on different material models for shotcrete: radial deformation $u_r/R$ at $r=7 \text{ m}$ and $t=840 \text{ h}$.

Figure 6.4: Numerical study based on different material models for shotcrete: normalized extrusion $u_z/R$ at $r=0 \text{ m}$ and $t=840 \text{ h}$.
Figure 6.5: Numerical study based on different material models for shotcrete: distribution of the normalized hoop force $n_{\varphi}/n_{c,\infty}$ in the shotcrete lining at $t=840$ h.

Figure 6.6: Numerical study based on different material models for shotcrete: distribution of total and plastic stretch in the circumferential direction, $\delta_{\varphi}$ and $\delta_{p \varphi}$, in the shotcrete lining obtained from the analysis M3 at $t=840$ h.
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Figure 6.11: Numerical study based on different values for the Young's modulus of the ground: normalized extrusion $u_z/R$ at $r=0$ m and $t=840$ h.
Figure 6.12: Numerical study based on different values for the Young's modulus of the ground: distribution of the normalized hoop force in the shotcrete lining, $n_\phi/n_{c,\infty}$, at $t=840$ h.

Figure 6.13: Numerical study based on different values for the Young's modulus of the ground: distribution of the normalized longitudinal force in the shotcrete lining, $n_z/n_{c,\infty}$, at $t=840$ h.
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Figure 6.14: Numerical study based on different values for the cohesion of the ground: normalized radial displacements $u_r/R$ at $r=7$ m ($t=840$ h).

Figure 6.15: Numerical study based on different values for the cohesion of the ground: normalized extrusion $u_z/R$ at $r=0$ m and $t=840$ h.
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Figure 6.17: Numerical study based on different values for the cohesion of the ground: distribution of the normalized longitudinal force in the shotcrete lining, $n_z/n_{C,\infty}$, at $t=840$ h.
Figure 6.18: Numerical study based on different values for the characteristic time of creep of the ground: normalized radial displacements $u_r/R$ at $r=7$ m ($t=840$ h).

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the ground: distribution of the normalized hoop force in the shotcrete lining, $n_\varphi/n_{c,\infty}$, at $t=840$
h.

Figure 6.21: Numerical study based on different values for the characteristic time of creep of
the ground: distribution of the normalized longitudinal force in the shotcrete lining, $n_z/n_{c,\infty}$, at
$t=840$ h.
Figure 6.22: Change in radial and longitudinal displacement, $\Delta u_r$ and $\Delta u_z$, respectively, at $r=7$ m, for the final excavation step (analyses V1 and V3).
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Figure 6.24: Numerical study based on different tunnel excavation rates: distribution of the normalized hoop force $n_\varphi/n_{C,\infty}$ in the Shotcrete lining at the time instant at which the tunnel face reaches its final position at $z=0$ m.
Figure 6.25: Numerical study based on different tunnel excavation rates: distribution of the normalized longitudinal force $n_z/n_{c,\infty}$ in the shotcrete lining at the time instant at which the tunnel face reaches its final position at $z=0$ m.
7. Analysis of Monitoring Data from a Deep Tunnel in a Tectonized Clay Shale (Raticosa Tunnel, Italy)

7.1 Introduction

The Raticosa Tunnel is one of the several tunnels currently under construction for the new Bologna-Florence high-speed railway line (Italy). The tunnel, having an overburden of up to 500 m, is located in the Apennine range and half of its 10 km length crosses a tectonized clay-shale formation known as "Chaotic Complex". The construction method is based on full-face excavation combined with face reinforcement by means of fiber-glass dowels. Primary lining consists of a shotcrete layer and closed-ring steel sets. Due to the heavy squeezing ground conditions, as predicted in the preliminary investigation phase, the excavation of the tunnel was performed under the strict control of an extensive/comprehensive monitoring system.

In this chapter, face extrusion and tunnel wall displacements have been analyzed and correlated with the single construction stages and specific ground conditions (Boldini et al., 2002) (Boldini et al., 2003). The displacements measured near the tunnel face generally confirm the validity of the design criteria of face reinforcement and early invert closure, which assures tunnel stability before casting the final lining. Large, time-dependent, deformations of the ground have generally been recorded, causing local failure phenomena at some locations along the tunnel route. A tentative explanation of the observed behavior has been proposed on the basis of empirical models.
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7.2 Raticosa Tunnel, Italy

The Raticosa tunnel (10 km) is one of the tunnels under construction for the new Bologna-Florence high-speed railway line (Italy) that crosses the Apennine Chain. Starting from the Bologna side, half of the tunnel is excavated in a tectonized clay shale formation known as "Chaotic Complex" which, right from the design stage has represented one of the most critical points in the development of the whole project due to the poor geotechnical conditions and the high overburden of up to 500 m (Figure 7.1). The same formation is also crossed by the Osteria access tunnel, which runs almost parallel to the main tunnel before turning to intersect it.

Due to the expected heavy squeezing conditions, full face-excavation required face reinforcement by means of fiber-glass dowels. Primary lining consisting of a shotcrete layer and steel sets were adopted.

Tunnel deformation and loading conditions of the lining were carefully monitored during tunneling so as to ensure compliance with the construction quality requirements, verify the design assumptions and calibrate the reinforcement pattern at the face and the strength required for primary lining. Moreover the large amount of data gathered during construction represents a valuable tool for the back-analysis of the mechanical behavior of the tectonized clay-shale for which the geotechnical characterization in the preliminary investigation stage was difficult and affected by many uncertainties.

7.3 Geological Framework

Tectonized clay shale formations are rather widespread throughout southern and central Italy (A.G.I., 1985) (Picarelli et al., 2000) (Picarelli et al., 2002). The Chaotic Complex encountered during the excavation of the Raticosa tunnel belongs to the Liguridi units and reached the current position after intensive tectonic events which took place from the Miocene-Pliocene to the Plio-Pleistocene.
The chaotic structure of the formation is the result of such tectonic actions. Lithologically the Raticosa shale is mainly composed of a pelitic matrix with dispersed lithic components.

The pelitic matrix is constituted by a tight assemblage of hard clay scales of the size of millimeters to centimeters whose surfaces are curved, smooth and at times striated. A semi-quantitative mineralogical composition (Table 7.1) has been obtained by X-ray diffraction (XRD) analysis on several clay-shale samples taken at different depths from boreholes S12S and F4-2 and from the DISG block directly cut from the tunnel face. Mineral composition has been found to vary greatly in the three analyzed samples even if only the total fraction of clay minerals is considered. The deeper sample is characterized by a remarkably lower phyllo-silicate content (25-30% of the total dry weight), which increases to 45% for the more shallow samples. The more expandable clay mineral, i.e. smectite, is present in small proportions in the form of illite-smectite mixed layers.

The lithic components include calcareous, marly or arenaceous blocks for a total volume fraction varying between 0 and some tens %. Because of the low percentage of the lithic components, mainly present in the form of disarranged strata and fragments, the mechanical behavior of the Chaotic Complex is governed by the clay shale matrix.

The northern stretch of the Raticosa tunnel (Bologna side) crosses a large paleo-landslide area for approximately 500 m (Figure 7.1), where deformation phenomena are still active. Movements of more than 4 mm/year were measured in the 1993-95 period (i.e., before tunnel excavation) at the surface of the landslide body along borehole S12S equipped with an inclinometer tube (Figure 7.2). More recent measurements exhibit a more random trend, difficult to be soundly assessed. Moreover, the last measurements have been limited to the first 60 m since the inclinometer tube could not be moved further down.
7.4 Geotechnical Characterization

The geotechnical characterization of the clay-shale formation has met many difficulties and uncertainties, mainly related to the high sensitivity to sampling disturbance and specimen preparation of the pervasively fissured hard material. Figure 7.1 shows the location of the boreholes drilled during the investigation phase and the blocks taken at the tunnel face during the excavation.

A summary of the basic geotechnical parameters evaluated for the tectonized clay shale is given in Table 7.2.

The in situ measurement of hydraulic conductivity by borehole pumping tests has failed because of negligible water absorption, nevertheless indicating a very low bulk permeability, less than $10^{-11}$ m/s. Only inside the more permeable and loosened material of the paleo-landslide mass could meaningful pumping tests be executed, obtaining permeability values in the range of $3 \cdot 10^{-7} \div 2 \cdot 10^{-9}$ m/s.

At present no reliable information is available on pore pressure distribution inside the Chaotic Complex; still, it is reasonable to assume that the water table is at the ground surface and that the whole formation is almost fully saturated.

Piezometers installed in the ground during tunnel excavation (14 electric instruments up to a distance of 15 m from the tunnel wall) have not measured any pore water pressure so far; two Casagrande piezometers installed in borehole S12S inside the paleo-landslide area have recorded a piezometric head which is variable along the vertical and, in any case, lower than the depth of the measuring points from the ground surface (Figure 7.3). The deeper piezometer (A) shows a continuously time-decreasing trend of pore pressure with a steeper gradient as the tunnel face approaches the borehole position; piezometer (B), located in the transition zone at the boundary of the paleo-landslide exhibits a more irregular trend, probably influenced by deformation phenomena that are still active.

The lack of information about pore pressure distribution leads to much uncertainty about the effective stress at the tunnel depth.
The index properties obtained on borehole cores (according to ASTM procedures) do not vary significantly neither with the position along the tunnel route nor with depth (Figure 7.4). The clay-shale blocks taken at the tunnel face give more scattered results, especially the deeper samples. The natural water content $w$ is always lower than the plastic limit $w_p$ (by approximately 8%).

Only three values of the index properties are available for the paleo-landslide area; anyhow, the unit weight of total volume $\gamma$ seems to be lower than that obtained for the deeper material and the natural water content $w$ tends to reach the plastic limit $w_p$.

The index properties (particularly $w_L$ and PI) and the grain-size distribution of hardened clay scales strongly depend on the disaggregating technique adopted in preparing the samples. Only prolonged working with the spatula will destroy most of the diagenetic bonds within the shear lenses, thereby increasing the clay fraction (Picarelli et al., 2000). Actually, this could be less true for the clay-shales belonging to the Liguridi Units (A.G.I., 1985), characterized by high carbonate content as is the case of the Raticosa clay-shales (Table 7.1). An attempt to verify the influence of the disaggregating technique has been made on material taken from the DISG block; the clay fraction has been found to increase from 13% to 23% both after prolonged working with spatula and after repeated drying and wetting cycles, thus indicating a considerable de-bonding effect similar but still less than that associated with the transition from the deep clay-shale to the completely softened and destructured clay of the landslide mass.

The saturation degree is generally lower than unity also for very deep samples (Figure 7.4): this is quite a common finding in scaly clay formations, partly explained by the opening of the fissures due to the effect of looseness and stress release during sampling, particularly remarkable for deep samples.

A rough estimate of the sampling disturbance can be obtained by analyzing the result of the Huder-Amberg one-dimensional swelling test (Wittke, 2000). Figure 7.5 shows the typical result of a test carried out on a sample taken from a deep block ($h=430$ m). The reduction in the void ratio ($\Delta e=0.1$) between point A (start of the first loading phase) and point C (end of unloading) can be essentially attributed to the closure of fissures previously opened after sampling disturbance, which reduced the saturation degree to about 70%. Due to the
negligible volume increase recorded after adding water (point D, before unloading), a swelling pressure $\sigma_0 \leq 7$ MPa can be estimated.

During unloading, with water allowed to penetrate into the sample, swelling deformation is considerable and comparable with that observed in ordinary oedometer compression tests (Figure 7.6). In fact, as normally occurs with scaly clays, the Raticosa shale exhibits a very high swelling index $C_S$ and therefore also a high ratio between the swelling index and the compression index.

P-wave and S-wave velocities ($V_P$, $V_S$) measured on shale blocks taken at the face under an overburden of 480 m showed a dynamic shear modulus of around 2900 MPa; $V_P$ ranging from 2000 to 2500 m/s were measured in boreholes at a depth of around 150 m as well. The dynamic stiffness can be considered an upper limit of the actual in situ stiffness of the rock-mass subjected to excavation-induced loads.

The loading and unloading moduli given by dilatometer tests appear to be strongly scattered: the loading modulus is 140 MPa for a 0.5 to 2.5 MPa load increment and 3900 MPa for a 2.4 to 5.4 MPa load increment. Similarly, the unloading modulus is 250 MPa for a 2.5 to 0.5 MPa load reduction and becomes 3950 MPa for a 3.9 to 2.4 MPa load reduction.

The marked reduction in stiffness as unloading proceeds can be mainly related to the mobilization of shear displacements along discontinuities. It is therefore difficult to identify a true elastic modulus. The best estimated should be given by the upper range of unloading secant moduli.

The presence of the network of fissures strongly affects also the shear strength of the tectonized shale. To evaluate the strength parameters of the shales of the Raticosa tunnel, triaxial as well as direct shear tests were performed. The samples were obtained from investigation boreholes as well as from blocks cut at the tunnel face during excavation (Fig. 7.7).

Triaxial tests (CID and CIU tests) are deemed to afford the more reliable results. In fact, data interpolation by a linear Mohr-Coulomb criterion is characterized by a correlation coefficient as high as 0.97. More precisely, peak shear strength data seem to indicate two different
strength envelopes according to the depth of the sample, with a much higher cohesion for the samples taken at depths where \( h > 200 \text{ m} \). Beyond peak strength only a weak strain-softening is generally observed; a more brittle response has sometimes been observed for specimens with very low natural water content \((w = 3\div 4\%)\).

The 80% and 95% joint confidence regions for the strength parameters \( c'\cos\phi' \) and \( \sin\phi' \), obtained by means of the regression analysis of the triaxial strength data are shown in Figure 7.8. It can be observed that:

- there is a negative cross-correlation between the estimated values of the independent variables;
- the confidence interval is always very tight for the friction angle: i.e., \( \phi' = 15^\circ \) can be virtually considered as a characteristic value for all the samples;
- the confidence interval of \( c' \) seems to indicated a lower characteristic value, definitely equal to zero, for shallow samples and in the order of 200 kPa for deep samples, but owing to the small number of deep samples and moreover to the discontinuous shape of the apparent strength envelope, a variation of the cohesion in the range of 0\( \div 200 \text{ kPa} \) could be considered as a result of the natural dishomogeneity of the formation more than as a property strictly dependent on depth.

A set of 28 UU triaxial tests aimed at evaluating undrained strength were also carried out, but the results are likely to be strongly affected by the low saturation degree of the specimens. The maximum total deviatoric stress \((\sigma_1-\sigma_3)_{\text{max}}\) in fact continuously increases with the applied confining stress \(\sigma_3\), from an average value of 1.8 MPa for \(\sigma_3 = 1 \text{ MPa} \) up to 4.6 MPa for \(\sigma_3 = 10 \text{ MPa} \).
### Table 7.1. Results of the X-ray diffraction analyses.

<table>
<thead>
<tr>
<th>Borehole/Block</th>
<th>S12-S</th>
<th>F4-2</th>
<th>DISG</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Depth h (m)</strong></td>
<td>82</td>
<td>150</td>
<td>496</td>
</tr>
<tr>
<td><strong>Quartz + Feldspars (%)</strong></td>
<td>17 ÷ 22</td>
<td>41</td>
<td>30 ÷ 38</td>
</tr>
<tr>
<td><strong>Carbonates (%)</strong></td>
<td>35 ÷ 40</td>
<td>0</td>
<td>30 ÷ 35</td>
</tr>
<tr>
<td><strong>Phyllosilicates (%)</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Kaolinite (%)</td>
<td>&lt; 2</td>
<td>2 ÷ 12</td>
<td>&lt; 5</td>
</tr>
<tr>
<td>Chlorite (%)</td>
<td>40 ÷ 45</td>
<td>6 ÷ 7</td>
<td>2 ÷ 12</td>
</tr>
<tr>
<td>Illite (%)</td>
<td>21 ÷ 32</td>
<td>12 ÷ 25</td>
<td>15 ÷ 20</td>
</tr>
<tr>
<td>Illite-Smectite (%)</td>
<td>2 ÷ 11</td>
<td>12 ÷ 25</td>
<td></td>
</tr>
<tr>
<td>Others (%)</td>
<td>0</td>
<td>10</td>
<td>4 ÷ 8</td>
</tr>
</tbody>
</table>
Table 7.2. Index and mechanical properties of the Raticosa clay shale.

<table>
<thead>
<tr>
<th>Properties</th>
<th>Range</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit weight of total volume $\gamma$ (kN/m$^3$)</td>
<td>20.8 ÷ 24.1</td>
<td>22.8</td>
</tr>
<tr>
<td>Unit weight of solid $\gamma_s$ (kN/m$^3$)</td>
<td>26.7 ÷ 27.7</td>
<td>27.5</td>
</tr>
<tr>
<td>Natural water content $w$ (%)</td>
<td>2 ÷ 19</td>
<td>9</td>
</tr>
<tr>
<td>Liquid limit $w_L$ (%)</td>
<td>30 ÷ 48</td>
<td>38</td>
</tr>
<tr>
<td>Plastic limit $w_p$ (%)</td>
<td>16 ÷ 22</td>
<td>18</td>
</tr>
<tr>
<td>Saturation degree $S_r$ (%)</td>
<td>48 ÷ 94</td>
<td>77</td>
</tr>
<tr>
<td>Clay fraction CF (%)</td>
<td>3 ÷ 44</td>
<td>20</td>
</tr>
<tr>
<td>Compression index $C_c$</td>
<td>0.10 ÷ 0.17</td>
<td>0.12</td>
</tr>
<tr>
<td>Swelling index $C_s$</td>
<td>0.03 ÷ 0.06</td>
<td>0.05</td>
</tr>
<tr>
<td>P-wave velocity $V_p$ (m/s)</td>
<td>2000 ÷ 2500</td>
<td></td>
</tr>
<tr>
<td>S-wave velocity $V_s$ (m/s)</td>
<td>1000 ÷ 1100</td>
<td></td>
</tr>
<tr>
<td>Triaxial peak cohesion $c'_p$ (kPa) *</td>
<td>16 ÷ 540</td>
<td></td>
</tr>
<tr>
<td>Triaxial peak friction angle $\phi'_p$</td>
<td>15°</td>
<td></td>
</tr>
<tr>
<td>Direct shear peak (residual) cohesion (kPa)</td>
<td>170 (80)</td>
<td></td>
</tr>
<tr>
<td>Direct shear peak (residual) friction angle</td>
<td>14° (10°)</td>
<td></td>
</tr>
</tbody>
</table>

* the lower and upper limits refer to values obtained from samples taken at depths h below or above 200 m.
7.5 Design and Construction

The design of the tunnel was based on the ADECO-RS approach (Lunardi & Focaracci, 1999), whose distinctive feature is to highlight the key role of tunnel face stability for the overall static conditions of the tunnel; it also introduces the principles of the observational method into tunnel design.

Full-face excavation was preferred for the operational advantages it secures, such as enabling a high level of industrialized construction and safe working conditions; it also provides static benefits, mainly the possibilities of closing the support ring with an invert arch immediately behind the face and the extensive application of face reinforcement techniques by high-power mechanized equipment.

Figure 7.9a shows the typical cross-section of the tunnel (area 120±140 m²), with the primary lining consisting of a shotcrete layer (thickness 25±30 cm) and steel sets (including an arched strut in the invert); the final lining is in reinforced concrete (invert and sidewalls) and plain concrete (vault).

Construction stages are better represented by the longitudinal section of the tunnel in the vicinity of the face (Figure 7.9b) and by a typical recording of construction advance (Figure 7.10).

A thin layer of shotcrete is applied to the face (Figure 7.11) when the excavation is stopped to allow the installation of fiber-glass dowels in safety conditions. The face is reinforced by a set of 40÷50 fiber-glass dowels (ø32 mm, length 20÷24 m) grouted by a rapid-set cement mortar, then the next round of face advance (10÷12 m) is executed (Figure 7.12), applying a new stretch of primary lining every 1 m of excavation advance.

The concrete invert is cast within a distance of one tunnel diameter from the face, generally the length of each cast segment is 11 m; finally the concrete lining is closed by the vault at about 30 m behind the face.
If geotechnical conditions are worse than predicted, additional reinforcement can be applied also beyond the excavation profile, by installing the outer dowels with a small outward inclination.

Such higher rate of reinforcement was actually required only for the initial stretch of the tunnel that crosses at low depth the softened and remolded shale of the paleo-landslide area. The primary lining was adapted to the variable geotechnical conditions encountered along the tunnel route: after the large ground deformations experienced in the paleo-landslide area, heavier steel sets (HEA300/m instead of 2IPN220/m) embedded in shotcrete and closed by a steel strut at the invert were applied.

### 7.6 Analysis of the Monitoring Data

The whole construction process has been performed under the constant control of a monitoring system, mainly based on the geodetic survey of tunnel wall displacements (a measuring section every 30 m with 5 optical targets installed close to the face) and on axial deformation measurements carried out by a sliding micrometer along special tubes installed from the face in the direction of the tunnel axis. The initial length of the measuring tube is around 34 m but it shortens progressively because the plastic tube is cut as the excavation advances. The total number of installed tubes is enough to provide a representative picture of displacements throughout the tunnel length.

#### 7.6.1 Extrusion Measurements

Figure 7.13 shows two typical extrusion measurements recorded at two different chainages in the Raticosa tunnel.

The first (Fig. 7.13a) comes from the paleo-landslide zone (overburden 30 m). The instrument length decreased from the initial 33 m (zero reading) to the final 20 m,
corresponding to the last D and E readings, taken after 13 m of face advance. Every measurement exhibits an “extrusion” (i.e., axial displacement of the measuring pipe) which reaches a maximum at the tunnel face; extrusion values are progressively higher as the face advances, thus indicating that excavation-induced deformations affect the ground core for a distance significantly greater than 13 m.

A different trend is shown by the “extrusion” measured in the Chaotic Complex underneath a high overburden (Fig. 7.13b). The maximum extrusion is reached after a face advance of about 10 m (reading D) with no further increase in the next steps (reading E). It is therefore conceivable that inside the stiffer shale formation the influence zone of the face is limited to about 10 m.

From Figure 7.13, also the separate effect of time-dependent deformation can be appreciated: at chainage 30+171, no excavation occurred between readings D and E, meanwhile an approximately 21 mm/day face extrusion increase was measured; a much smaller deformation of only 1.2 mm/day was measured at chainage 33+676, during a face stop between readings A and B.

A picture of maximum extrusions $u_{e,max}$ recorded by the various instruments installed along the tunnel route is represented in Figure 7.14. In order to obtain more homogenous and comparable data, only extrusion measurements relative to the excavated length of the instrument, $\Delta l > 12$ m, have been considered in Figure 7.14. In fact, the face could have experienced an even larger extrusion than that recorded by the measurement system. The effective $\Delta l$ values and the time, $\Delta t$, elapsed between the zero reading and the final reading of each instrument are therefore represented as well.

At low tunnel depth, i.e. in the paleo-landslide area, maximum measured extrusions attain values even larger than 200 mm, which can be interpreted as the sign of actually unstable face conditions, as confirmed by the high deformation rate during face stops. On the contrary, inside the Chaotic Complex the maximum extrusion values are always of the order of 50 mm, almost independently of tunnel depth, thus denoting substantially stable face conditions.


7.6.2 Convergence Measurements

Figure 7.15 summarizes maximum convergence, crown settlement and longitudinal displacements measured by each monitoring section along the tunnel route. Apart from maximum measured convergence values exceeding 120 mm in the paleo-landslide area, the average convergence is around 40 mm in the tectonized shale.

Data scatter is higher for convergence than for extrusion: this cannot be explained only by the different levels of accuracy of the geodetic survey and the sliding micrometer measurements. A major role must be attributed to interaction with the primary lining, which is quite sensitive to installation unevenness.

Despite the data scatter, the monitoring data (Fig. 7.15) show a small but significant influence of the overburden on convergence; a linear regression of the data indicates an increase of about 5 mm every 100 m of depth. A similar trend is shown by the longitudinal displacements.

By observing that the average horizontal displacement of point 1 and 5 on the sidewalls is almost equal to the vertical displacement of point 3 (crown), it is argued that the bending distortion of the lining is limited; therefore there is an almost uniform hoop stress distribution between the inner and the outer sides of the lining.

Longitudinal displacements of the different points within the cross-section of the tunnel are almost equal. The whole section tends to move in the same direction of the face, but the displacement magnitude is much smaller.

A rough interpretation of the convergence data has been based on the curve-fitting technique proposed by Sulem et al. (1987).

Tunnel closure is represented by an empirical law $C(z, t)$, which takes into account the effect of both face distance $z$ and the time-dependent behavior of the ground:

$$C(z, t) = C_{\infty, z} \left[ 1 - \left[ 1 + \frac{Z}{Z} \right]^{-2} \right] \left[ 1 + m \left[ 1 - \left( 1 + \frac{t}{T} \right)^{-n} \right] \right].$$

(7.1)
where \( t \) is the time elapsed since the face passed through the monitoring section.

Relationship (7.1) depends on five parameters: \( C_{\infty z} \) ("instantaneous closure") and \( Z \) (a distance related to the tunnel radius \( R \)), which control the face effect; \( m \), \( T \) and \( n \) for the time-dependent part.

The free parameters can be determined by a non-linear regression of experimental data, for each monitoring section where a set of at least \( K=15\div20 \) readings were available, starting from a "zero" reading made close to the tunnel face (distance \( z_o \)) at time \( t_o \). The set of \( k \) equations to be solved, in the sense of minimum squares, assume therefore the form:

\[
\Delta C_{i, \text{measured}} = C(z_i, t_i) - C(z_o, t_o) \quad i = 1, k.
\]  

To reduce problems of ill-conditioning of the system of equations (7.2), exponent \( n \) was given the value of 0.3, as suggested by Sulem et al. (1987).

Figure 7.16 shows two examples of convergence curve fitting by means of Equation (7.1), respectively for a monitoring section located in the completely softened shale (chainage 30+343, overburden 50 m) and another one deep inside the Chaotic Complex (chainage 32+998, overburden 365 m). In the lower part of the figures the advance of the tunnel face as a function of time is also represented to make the interpretation of the observed behavior easier.

The estimated values of regression parameters, also reported in Figure 7.16, indicate a remarkable amount of time-dependent convergence, whose asymptotic values, controlled by the parameter \( m \), is equal to 1.8\div2.4 \) times the convergence due to the face advance.

The separate effect of time-dependent deformations can be clearly evidenced during a period of excavation pause, as the 20-day period in Figure 7.16b. In this case, when the face stopped at a distance of about 26 m from the monitoring section, the corresponding increase of convergence was about 15 mm; this measured value is well reproduced by the fitting procedure. This kind of behavior, observed also in other monitoring sections, stresses how important it is to closely respect the construction time-schedule in order to prevent
overloading phenomena of the primary lining before the final concrete lining becomes effective.
7.7 Figures of Chapter 7

Figure 7.1. Geological profile along the tunnel route and location of boreholes and blocks taken during the tunnel excavation.

Figure 7.2. Total displacements measured in inclinometer borehole S12S.
Figure 7.3. Hydraulic head measured by two Casagrande piezometers in borehole S12S.

Figure 7.4. Index and mechanical properties of the Raticosa clay shale.
Figure 7.5. Results of the Huder-Amberg swelling test (block GN04D).

Figure 7.6. Results of oedometer compression tests (samples from boreholes F4-2, F5-3, and S12-S).
max shear stress \( t = (\sigma_1 - \sigma_3)/2 \) (MPa)

mean effective stress \( s' = (\sigma_1' + \sigma_3')/2 \) (MPa)

\[
t = s' \sin \varphi' + c' \cos \varphi'
\]

Figure 7.7. Synopsis of peak shear strength data from triaxial tests and linear regression curves.
Figure 7.8. 80% and 95% joint confidence regions for peak strength data.

Figure 7.9. Transverse (a) and longitudinal cross-section (b) of the tunnel.
Figure 7.10. Recording of the construction advance of the Raticosa tunnel during July and August 2000.

Figure 7.11. View of the tunnel face supported by temporary shotcrete before installation of fiber-glass dowels (thanks to Nicola Grosso).
Figure 7.12. Detail of the tunnel face: fiber-glass dowels are easily destroyed by the hydraulic hammer.
Figure 7.13: Extrusions measured during face advance at two different tunnel chainages.
Figure 7.14: Maximum extrusion $\delta_{\text{max}}$ measured along the tunnel route (a); corresponding excavated length $\Delta l$ of the extensometer tube (b), and elapsed time $\Delta t$ after zero measurement (c).
Figure 7.15: Maximum horizontal convergence (a); crown settlement (b), and longitudinal displacement (c); distances, $\Delta_i$, $\Delta_f$, between the face and the monitoring section at the time of the initial and final readings (d), time elapsed $\Delta t$ between the initial and final measurements (e).
Figure 7.16: Curve fitting of convergence measurements at two monitoring sections.
8. **3D Numerical Analysis of the Raticosa Tunnel**

8.1 **Introduction**

This chapter deals with the numerical simulation of the Raticosa tunnel excavation and with the comparison of the results with the in situ measurements. The numerical simulation refers to the excavation of the tunnel between chainage 32+271 and 32+197 in July and August 2000. Here, the overburden was approximately equal to 250 m.

The analysis is based on a viscoplastic material model for the soil and on the thermochemomechanical model for the shotcrete developed at the Vienna University of Technology.

The numerical analysis of the Raticosa tunnel is performed in two separate steps. First, the thermochemical problem accounting for the thermally-activated nature of the hydration process of the shotcrete is solved. This analysis provides the temperature fields in the ground and in the shotcrete and the hydration extent in the shotcrete lining, serving as an input for the subsequent chemomechanical analysis. Then, the chemomechanical analysis gives insight into the ground-shotcrete interaction, the effect of face reinforcement by means of fiber-glass dowels, and the stress state in the shotcrete lining. In the analysis the ground is modeled as a dry material.

The numerical analysis aimed to analyze the tunnel conditions during both the steady-state advancement of excavation and the stops for the summer holidays. The results obtained are compared with the values of extrusion and convergence measured in the investigated sections of the Raticosa tunnel.
8.2 Description of the Investigated Tunnel Sections

The numerical investigation is devoted to the excavation of the Raticosa tunnel between chainages 32+271 and 32+197 in July and August 2000. Here, the tunnel is characterized by an overburden of about 250 m, which is half the maximum overburden of the whole tunnel. This part of the Raticosa tunnel was mainly excavated during the month of July because the construction works stopped for three weeks during August for the summer holidays. This fact offered the possibility of monitoring the tunnel deformations during both the normal excavation stage and the stops of face advancement.

In the part of the Raticosa tunnel considered in the numerical analysis, the tunnel construction was characterized by:

- the reinforcement of the tunnel face with 45 cemented fiber-glass dowels every 12.5 meters of excavation (the dowels were 24 m long giving an overlap of 11.5 m) (Figure 8.1);

- a closed primary lining consisting of a 30 cm-thick shotcrete shell and steel sets (HEA 300 every 1.04 m); and

- the support of the tunnel face by a shotcrete shell with a thickness of 10 cm.

The properties of the employed materials are listed in Table 8.1.

The tunnel was driven by full-face excavation in steps of 1.04 m.

The cross section of the tunnel is shown in Figure 8.1. It is defined by the radii R1, R2 and R3. The geometric characteristics of the cross-section are summarized in Table 8.2.

Figure 8.2 shows the location of the tunnel face, the invert arch, and the final lining for the considered part of the tunnel, driven in July and August 2000. The reinforcement of the tunnel face by fiber-glass dowels was performed every 12.5 m. The setting of the dowels took 3 days. During this time period, the invert arch was installed. The excavation of the tunnel from one face reinforcement to the consecutive face reinforcement, i.e., the excavation of 12.5 m, took approximately 5 days, giving a tunnel advance rate of 2.5 m/day.
8.3 Constitutive Model for the Ground

A viscoplastic material model is employed in order to describe the behavior of the tectonized clay-shales encountered during the excavation of the Raticosa tunnel.

The plastic material response of the ground is described by means of the Drucker-Prager criterion (see Figure 8.3a), reading:

\[ f_{DP}(\sigma, \zeta_{DP}) = \sqrt{J_2} + \alpha I_1 - \zeta_{DP} / \beta \]  \hspace{1cm} (8.1)

where \( \zeta_{DP} \) represents the hardening force of the Drucker-Prager criterion. The parameters \( \alpha \) and \( \beta \) are computed from the cohesion \( c \) and the friction angle \( \phi \) such that the Drucker-Prager meridian coincides with the compression meridian of the respective Mohr-Coloumb criterion. This yields:

\[ \alpha(\phi) = \frac{2 \sin \phi}{\sqrt{3}(3 - \sin \phi)}, \quad \beta(c, \phi, f_c) = \frac{f_c}{\sqrt{3}c} \frac{3 - \sin \phi}{2 \cos \phi}, \quad \text{with} \quad f_c(c, \phi) = \frac{2c \cos \phi}{1 - \sin \phi} \]  \hspace{1cm} (8.2)

denoting the uniaxial compressive strength of the material. An increase in the hardening force \( \zeta_{DP} \) from 0.4\( f_c \) to \( f_c \) in the context of strain hardening is considered, see Figure 8.3b.

In the tensile loading regime, the tension-cut-off is employed, reading:

\[ f_{TC}(\sigma, \zeta_{TC}) = I_1 - \zeta_{TC} \]  \hspace{1cm} (8.3)

with \( \zeta_{TC} \) representing the hardening force. For the tension-cut-off, an ideally-plastic behavior is assumed. Hence, \( \zeta_{TC} = f_t = \text{constant} \), where \( f_t \) is the uniaxial tensile strength.

The Drucker-Prager criterion and the tension-cut-off are combined in the context of multi-surface plasticity. The evolution equation for the plastic strain tensor is given by (Koiter, 1960):

\[ \dot{\varepsilon}^P = \dot{\gamma}_{DP} \frac{\partial f_{DP}}{\partial \sigma} + \dot{\gamma}_{TC} \frac{\partial f_{TC}}{\partial \sigma} \]  \hspace{1cm} (8.4)
where $\gamma_{DP}$ and $\gamma_{TC}$ represent the plastic multipliers of the Drucker-Prager criterion and tension-cut-off, respectively.

The extension of the described multi-surface plasticity model towards viscoplasticity follows the law proposed by (Duvaut and Lions, 1972), reading:

\[
\dot{\varepsilon}_{vp} = \frac{1}{\tau} C^{-1}\left(\sigma - \sigma^\infty\right) \quad \text{and} \quad \dot{\zeta}_{DP} = -\frac{1}{\tau}\left(\zeta_{DP} - \zeta_{DP}^\infty\right)
\]  

(8.5)

where $C$ denotes the elastic material tensor. In Equation 8.5, $\tau$ is the relaxation time, and $\sigma^\infty$ and $\zeta_{DP}^\infty$ correspond to the solution for rate-independent elastoplasticity, i.e., to the solution for infinitely slow loading.

The mechanical properties of the ground employed in the numerical analysis are selected in accordance with the values provided in RockSoil & FiatEngineering (1999). They are listed in Table 8.3.

### 8.4 Thermochemomechanical Material Model for Shotcrete

For the simulation of the shotcrete lining, the thermochemomechanical material model developed at the Vienna University of Technology is employed (see Paragraph 3.5.1 and references therein).

As pointed out in Chapter 3, the shotcrete is characterized by different thermal, chemical and mechanical processes during its aging, which are interdependent with each other. On the basis of experimental evidence, the mechanical processes are found not to have a manifest influence on thermal and chemical processes, which allows to split the thermochemomechanical problem into two parts. First, the thermal and chemical processes are accounted for (thermochemical analysis), resulting in the computation of the temperature field and the field of the degree of hydration of shotcrete. Then, the chemomechanical
problem can be solved (chemomechanical analysis), using the results of the former analysis as an input.

This approach will be applied to the numerical analysis of the Raticosa tunnel. The same mechanical properties of shotcrete as defined in Chapter 3 are considered here (see Table 3.2).

### 8.5 Thermochemical Analysis

During the hydration of the shotcrete, the latent heat \( l_\xi \) (kJ/(m\(^3\) shotcrete)) is released resulting in an increase in the temperature in the shotcrete lining and, hence, in heat conduction in the surrounding ground and heat radiation towards the tunnel cavity. Heat conduction in the shotcrete lining and the surrounding ground is described by the field equation (Ulm and Coussy, 1995)

\[
\rho c \dot{T} - l_\xi \dot{\xi} = \text{div} \mathbf{q} ,
\]

with \( \rho \) as the density and \( c \) as the heat capacity. \( \mathbf{q} \) is the heat flow vector. It is related to the temperature via the linear law of Fourier:

\[
\mathbf{q} = -k \cdot \text{grad} T
\]

where \( k \) is the thermal conductivity.
8.5.1 **Structural Model, Boundary Conditions, and FE Discretization**

For the determination of the temperature profiles resulting from (after) the hydration process, a 1D axisymmetric FE model is employed. The respective FE formulation for the solution of the axisymmetric thermochemical problem can be found in (Lackner and Mang, 2001).

The FE model refers to a section perpendicular to the axis of the tunnel. It comprises both the shotcrete lining and the surrounding ground. The thickness of the shotcrete lining at the Raticosa tunnel is 30 cm. The lining is discretized with 5 axisymmetric finite elements in the radial direction (Figure 8.4).

The surrounding rock is discretized with 36 finite elements (Figure 8.4).

At the inner surface of the shotcrete lining, heat radiation from the lining to the tunnel opening is considered. The underlying heat radiation law reads:

\[ q = \alpha_R (T - T_{\infty}) \]  \hspace{1cm} (8.8)

with \( T \) standing for the temperature at the inner surface of the lining and \( T_{\infty} \) for the temperature of the air in the tunnel cavity. \( \alpha_R \) is the radiation coefficient.

8.5.2 **Material Properties and Initial Temperature**

The thermal properties of the shotcrete and the ground employed in the numerical analysis are listed in Table 8.4.

The initial temperatures of the shotcrete lining and the ground are set equal to 20 and 10 °C, respectively. According to temperature measurements performed near the tunnel face of the Raticosa tunnel, the temperature in the tunnel opening, \( T_{\infty} \), is set equal to 25 °C.
8.5.3 Presentation of Results

Figure 8.5 shows the evolution of the mean temperature $\bar{T}$ in the shotcrete lining as a function of time. The maximum value of $\bar{T}$, 32.5 °C, is reached about 18 hours after installation of the lining.

The distribution of the temperature in the shotcrete lining and in the adjacent ground at different time instants is given in Figure 8.6. Temperature profiles like this serve as input for the following chemomechanical analysis.

8.6 Chemomechanical Analysis

8.6.1 Structural Model and FE Discretization

The structural model employed for the chemomechanical analysis allows to simulate both continuous excavation of the tunnel at a excavation rate of 2.5 m/day and the construction break during the summer holidays. The geometric characteristics of the structural model and the applied boundary conditions are identical to those employed in the numerical analyses of Chapter 6 (see Figure 6.5).

As described in Chapter 6, the excavation starts from the right boundary and is stopped when the tunnel face reaches the center of the model, i.e., after the excavation of 10R of tunnel length. Continuation of the analysis for three weeks without any excavation allows to simulate the situation during the summer holidays. A local coordinate system is introduced. The coordinate $r$ refers to the radial direction, whereas the $z$-coordinate is orientated along the longitudinal direction of the tunnel. $z=0$ refers to the final location of the tunnel face (Figure 6.5(b)).
A circular tunnel with a radius of $R=7$ m is chosen in order to approximate the real cross-section of the tunnel (see Figure 8.1). After excavation, the application of a closed 30 cm-thick shotcrete shell is considered.

The analysis is performed by means of the axisymmetric FE model described in Chapter 6, consisting of 15266 axisymmetric finite elements (Figure 6.6).

### 8.6.2 Excavation Scheme

During the numerical analysis, the tunnel face moves from the right boundary of the structural model to the left until it reaches the center of the structural model at $z=0$. The excavation of the tunnel is simulated by replacing the respective ground elements by cavity elements. The latter are characterized by marginal stiffness. Application of shotcrete is modeled by replacing the respective cavity elements by shotcrete elements. Similar to the FE discretization employed for the thermochemical analysis, the shotcrete shell is discretized over the thickness by means of five finite elements.

The length of one excavation step and, hence, the length of the unsupported part of the tunnel is set equal to 1 m. This value is in accordance to the excavation step adopted in the Raticosa tunnel during July and August 2000.

Analogously, the excavation rate is set equal to 2.5 m/day. The time assigned to complete 1 m of the tunnel is divided into two parts: 2/3 of the time is dedicated to the excavation, the remaining 1/3 to the application of shotcrete. For an excavation rate of 2.5 m/day, 1 m of the tunnel is completed in 9.6 h. Consequently, $2/3 \cdot 9.6 = 6.4$ h are assigned to the excavation process, and $1/3 \cdot 9.6 = 3.2$ h to the application of the shotcrete lining.
8.6.3 **Initial State of Stress**

The consideration of axisymmetric conditions implies that the initial state of stress is isotropic. As a standard, the stress level is set equal to the average of the horizontal and vertical components of the in situ state of stress at the center of the tunnel, given by:

\[
\overline{S} = \gamma \cdot h \cdot \left( 1 + K_0 \right) / 2
\]

where \( \gamma \) represents the weight per unit volume and \( h \) is the distance between the surface and the center of the tunnel. \( K_0 \) is the ratio between the horizontal and vertical in situ stress component, referred to as lateral pressure coefficient.

For the considered part of the Raticosa tunnel, \( h = 250 \) m, \( K_0 = 0.8 \), and \( \gamma = 23 \) kN/m\(^3\). Based on these values, the isotropic stress is obtained as \( \overline{S} = 5.175 \) MPa. In the structural model, the initial stress state was introduced by applying a constant pressure \( p_0 \) at the top boundary of the model, with \( p_0 = 5.175 \) MPa (see Figure 6.5b).

8.6.4 **Consideration of Dowels within the FE Model**

The main objective of the installation of fiber-glass dowels at the tunnel face is to increase the confinement in the ground ahead of the tunnel and, hence, the stability of the tunnel face. Moreover, the installation of fiber-glass dowels can prevent localized block detachments, improving the safety of working conditions. According to the construction scheme described in Figure 7.3, the orientation of the dowels is almost parallel to the tunnel axis.

In the FE model, the fiber-glass dowels are discretized by means of a chain consisting of two-node truss elements. The nodes of each truss element are connected to the respective nodes of the ground elements. Accordingly, no slip between the dowels and the ground is considered which, for the case of cemented fiber-glass dowels under moderate loading, is a reasonable assumption.
In the analysis, the excavation steps were set equal to 1 m. Accordingly, the distance between two subsequent setting of dowels increased slightly from 12.5 m (original design) to 13 m (numerical analysis). For an excavation length of 70 m as depicted in Figure 6.5b, the setting of dowels was simulated at the following positions of the tunnel face: z=65, 52, 39, 26, 13, and 0 m.

According to the performance of the Raticosa tunnel during July and August 2000, the time assigned to the installation of the dowels at the tunnel face was set equal to 3 days. In order to simulate the excavation break of August 2000 in the numerical analysis, the excavation was stopped after setting the dowels at z=0 m.

The layout of the face reinforcement considered in the numerical analysis is illustrated in Figure 8.7 for installation of the dowels at z= 0. The layout of two consecutive face reinforcements differs by the radial location of the dowels. The radial distance between two corresponding sets of dowels is 35 cm.

In axisymmetric analyses the dowels, which are set at different locations of the tunnel face, must be shifted towards the axisymmetric plane (see Figure 8.8). Consequently, the dowels described in Figure 8.7 represent a certain number of dowels. For the layout of the dowels used at the Raticosa tunnel, the different dowels of the axisymmetric model represent 4, 7, 6, 11, 10, and 7 dowels (see Figure 8.7).

For the simulation of the mechanical behavior of the dowels, a linear elastic-ideally plastic material model is employed. The mechanical properties of the fiber-glass dowels used in the numerical analysis are given in Table 8.5.

### 8.6.5 Presentation of Results

In order to assess the effect of the shotcrete lining and of the fiber-glass dowels on the deformations and loading of the primary lining, three different analyses were performed:

- **A1**: In the first analysis, neither the shotcrete lining nor the fiber-glass dowels were considered. In this analysis, the behavior of the unlined tunnel is investigated;
A2: Installation of the shotcrete lining is considered in the second analysis.

A3: Finally, the third analysis aims at simulating the real conditions at the construction site. Both the shotcrete lining and the reinforcement of the tunnel face by means of fiber-glass dowels are considered.

In all the analyses the ground is considered as a dry medium.

Figure 8.9 shows the radial displacements $u_r$ of the interface between the lining and the ground ($r=7$ m) as a function of the distance from the tunnel face $z$, obtained at the time instant $t=1032$ h. A saw-toothed shape characterizes the plotted distributions. Each saw-tooth refers to one excavation step of 1 m. The change of $u_r$ within one saw-tooth indicates the variation of deformation and loading within 1 m of the tunnel. At this time instant, the tunnel face has reached its final position at $z=0$ for a driving rate of 2.5 m/days and five excavation stops of three days for the installation of the face reinforcement ($t=70/2.5 + 5\cdot3 = 43$ days = 1032 hours).

In order to compare the different analyses, the same excavation stops were considered in analyses A1 and A2, even though no fiber-glass dowels are considered. Almost identical distributions for the radial displacement $u_r$ are obtained for analyses A2 and A3, indicating a small influence of the face reinforcement on the convergence of the tunnel. The installation of the shotcrete lining (analysis A2) led to a reduction of the radial displacement to 3/10 of the obtained displacement of the unlined tunnel (analysis A1). For analyses A2 and A3, a continuous increase of $u_r$ is observed. As pointed out in Chapter 6, this increase is a consequence of the chemical shrinkage of the shotcrete.

At the locations of the tunnel corresponding to the installation of the face reinforcement, a localized increase in the radial deformation $u_r$ was obtained from analysis A1. This increase is explained by ground creep during the three-days’ break. In analyses A2 and A3, on the other hand, the hardening shotcrete lining provides a support for the creeping ground. Hence, no localized increase in $u_r$ is observed for these analyses.

The extrusion at the tunnel axis, i.e., the longitudinal displacement $u_z$ of the ground ahead of the tunnel face at $r=0$, is shown in Figure 8.10. Again, no significant differences are detected.
between analyses A2 and A3: the face reinforcement results in a reduction of the maximum extrusion at $z=0$ from 7.6 cm (analysis A2) to 7.4 cm (analysis A3). Analysis A1 gives the highest value for the extrusion of 8.5 cm.

The distribution of hoop forces $n_\phi$ in the shotcrete lining is given in Figure 8.11. Near the tunnel face, the compressive loads in the lining ranged from 5 to 8 MN/m. The three-day break of the excavation for the installation of the face reinforcement results in a slight reduction of the compressive loading resulting from the creep in the shotcrete. The increase in stiffness during the three-day break leads to a sharp increase in loading after continuation of the excavation. Creep in the shotcrete, however, results in a relaxation of the hoop force. Finally, the hoop force ranges between 6 and 10 MN/m.

Figure 8.12 shows the distribution of the longitudinal force $n_z$ in the shotcrete lining. According to the results discussed in Chapter 6, the loading of the lining in the longitudinal direction has four reasons: close to the tunnel face, (a) excavation-induced bending of the ground-shotcrete compound structure results in compressive loading which is further increased by (b) the compressive hoop force via Poisson's effect; the compressive loading is reduced by (c) chemical shrinkage and (d) deformations in the longitudinal direction caused by the excavation (see decrease in compressive loading for $z>2R$ in Figure 8.12).

As regards the influence of the face reinforcement, similar results for the hoop force and the longitudinal force were obtained from analyses A2 and A3 (see Figure 8.11 and 8.12). Hence, face reinforcement has a very small effect on the loading of the lining.

The loading of the reinforcement is given by the average stress in the fiber-glass dowels:

$$
\overline{\sigma_d} = \frac{1}{n_d} \sum_{i=1}^{n_d} \sigma_{d,i} \tag{8.10}
$$

where $\sigma_{d,i}$ is the stress in the $i$-th dowel. $n_d$ represents the number of dowels at the considered cross section of the tunnel. In the area of overlapping dowels (see Figure 8.7), $n_d=90$. Otherwise, $n_d=45$. 


Figure 8.13 shows the distribution of $\sigma_d$ during the tunnel excavation from $z=13$ m to $z=0$. A new set of fiber-glass dowels is installed at $z=13$ m, while the previous one (installed at $z=26$ m) reaches until $z=2$ m. Hence, between $z=13$ m and $z=2$ m, $n_d=90$.

In the course of the excavation, the length of the dowels installed at $z=26$ m and $z=13$ m decreases continuously, resulting in an increase in the average stress. At $z=2$ m, one set of dowels ends, explaining the jumps in the distribution of $\sigma_d$. For all the excavation steps plotted in Figure 8.13, the dowels are activated only for the first 7 m ahead of the tunnel face by the inward movement of the ground.

Between time instants $t=1032$ h and $t=1608$ h, the position of the tunnel face remained unchanged in the numerical analysis, allowing for the simulation of the three-week construction break during the summer holidays. Figure 8.14 shows the radial displacements $u_r$ obtained from the analysis A3 at time instants $t=1032$ h and $t=1608$ h. Close to the tunnel face, the relatively high compliance of the young shotcrete resulted in an increase of $u_r$. The maximum increase of radial displacement $u_r$ is observed approximately between $z=1$ and 3 m, where it reaches 8 mm.

Figure 8.15 shows the change of extrusion $u_z$ at the tunnel axis for time instants $t=1032$ h and $t=1608$ h. Even though the face was reinforced with a new set of fiber-glass dowels before the start of the summer holidays, resulting in a total number of 90 dowels installed ahead of the tunnel face, an increase in the extrusion at the tunnel face by 1.7 cm is obtained from the analysis.

Whereas the hoop force $n_\phi$ in the shotcrete lining (Figure 8.16) does not significantly change during the summer holidays, the stress release in the ground at the tunnel face results in a sharp increase in the compressive longitudinal force in the final shotcrete segment installed at $0\leq z \geq 1$ m (Figure 8.17). For $z<4R$, chemical shrinkage of the shotcrete results in a reduction of the compressive loading in the longitudinal direction. For $z>6R$, a rather low effect of the excavation stop on the distribution of $n_z$ is observed.
As regards the loading of the dowels, the continuation of stress releases in the ground results in an increase of \( \sigma_d \) (Figure 8.18). The maximum increase of \( \sigma_d \) is observed at the tunnel face. \( \sigma_d \) increased by 50 MPa reaching the value of 250 MPa.

### 8.7 Comparison with In Situ Measurements

During the excavation of the Raticosa tunnel, the extrusion of the ground ahead of the tunnel face and the tunnel convergence were continuously monitored. Extrusion was measured by means of sliding micrometers along plastic pipes cemented in the ground from the center of the face in the direction of the tunnel axis.

Figure 8.19 shows the extrusion measured in three different plastic pipes located at different tunnel chainages, referred to as P19, P20, and P21:

- P19 was installed at km 32+270.67 on 2000/07/01 and its initial length was 24 m. The first measurement was performed four days later, on 2000/07/05, after approximately 8 m of excavation. The extrusion at the tunnel face at this time instant was 60 mm;

- P20 was installed at km 32+245.67 on 2000/07/18 and its initial length was 30 m. The first measurement was performed three days later, on 2000/07/21, after approximately 8 m of excavation. A second measurement on 2000/07/24 gave an extrusion at the tunnel face of 63 mm. At this time instant, 13 m have been excavated since the installation of P20;

- P21 was installed at km 32+208.17 right before the summer holidays, on 2000/08/08, and its initial length was 24 m. The first measurement was performed right after the summer holidays, on 2000/08/28, after that no excavation occurred. Accordingly, the measured extrusion of 14 mm at the tunnel face corresponds to the stress release in the ground during the three-week excavation break. A second measurement was performed
on 2000/09/01 after 13 m of excavation. Extrusion at the tunnel face was 78 mm, giving an extrusion caused mainly by the excavation of 78-14= 64 mm.

Extrusion at the tunnel face obtained from analysis A3 was 74 mm (see Figure 8.15). The difference between this value and the in situ measurements (60, 63, and 64 mm) ranges between 10 and 14 mm (see Figure 8.19). The lower values for the extrusion measured at the construction site can be explained by the deformation of the ground ahead of the tunnel face before the "zero measurement" of the sliding micrometer was performed. Disregarding these deformations leads to an underestimation of the extrusion by the measurements. Similar to the obtained numerical results, the extrusion measured at all the considered sliding micrometers decreases rapidly with increasing distance from the tunnel face. At a distance of 7 m from the tunnel face, the extrusion measured at the construction site and the extrusion obtained from the analysis A3 is below 1 cm. As regards the increase in the extrusion during the summer holidays, the measurement at P21 of 14 mm is lower than the respective numerical result of 17 mm (see Figure 8.15). Nevertheless, considering the uncertainties arising from the delayed "zero measurements" at the construction site, from the simplifications of the employed axisymmetric model, and from the deviation between the real driving rate and the constant excavation rate of 2.5 m/day considered in the numerical analysis, the numerical results for the extrusion are in good agreement with the available in situ measurements.

The convergence of the tunnel wall was monitored by means of a geodetic survey detecting the 3D displacements of 5 optical targets installed in the shotcrete lining as close as possible to the face. The distance between two adjacent monitoring cross-sections (MCSs) was approximately 30 m. During the time span considered in the numerical analysis (July and August 2000), four MCSs were installed, namely on 2000/07/03, 2000/07/12, 2000/07/28, and 2000/08/04 at tunnel chainages 32+268, 32+257, 32+231, and 32+218, respectively. The history of the horizontal convergence of the tunnel (i.e. the displacements between the optical targets 1 and 5 in the tunnel cross-section) at these MCSs was computed from the displacement measurements provided by the construction site (see Figure 8.20). Because the measurements started at a certain distance from the tunnel face, the convergence provided by the construction site underestimates the real convergence.
To compare the in situ measurements with the numerical results, the convergence values are plotted as a function of the tunnel face distance in Figures 8.21 to 8.24. For each value, a companion plot shows the time interval between the "zero" measurement and the respective measurement. The monitoring section located at tunnel chainage 32+268 shows an increment of tunnel convergence of about 4 cm. The "zero" measurement was performed at a distance from the tunnel face of 2 m. For the monitoring sections located at tunnel chainages 32+257, 32+231, and 32+218 the maximum observed convergence values are equal to about 4 cm, 2.5 cm, and 2.8 cm respectively (the "zero" measurements were made at a distance from the tunnel face of 4.5 m, 8 m, and 3.5 m in the three cases). During the excavation break in August 2000, the monitoring section at chainage 32+231, located at a distance from the tunnel face larger than 3R, detected an convergence increment of about 0.6 cm, while the monitoring section at chainage 32+218, much closer to the tunnel face, monitored a negligible increment in convergence.

The in situ measurements of horizontal convergence evaluated in this study are characterized by quite scattered values. Data scatter can be attributed to interaction with primary lining, which is quite sensitive to installation unevenness. Moreover it can be related to the chaotic nature of the ground, which can produce dissimilarities in mechanical behavior even for close tunnel sections.

The horizontal convergence observed at the MCSs located at chainages 32+268 and 32+257 can be correctly compared to the numerical results because the tunnel excavation rate was almost the same as that assumed in the numerical simulation. In general, the radial displacements \( u_r \) obtained in the numerical analysis, i.e., half the convergence of the tunnel, are smaller than the values measured in the Raticosa tunnel. For example, the increment in radial displacements \( u_r \) between \( z = 2 \) m and \( z = 28 \) m (i.e., between the same face distances experienced by the MCS at chainage 32+268) is equal to about 1.5 cm in analysis A3 resulting in an increment in convergence of only 3 cm (Figure 8.14).

The increment in radial displacement \( u_r \) in analysis A3 from \( z = 4 \) m on (i.e., the same face distance where the zero reading of MCS located at chainage 32+257 was made) is less than 1 cm, giving an increase in convergence less of 2 cm.

Many reasons can be found to explain this discordance.
The first is related to the axisymmetry of the numerical model, where the shotcrete lining is a perfectly closed ring. In situ, the invert arch is closed only by steel set placed every excavation step, allowing the ground to deform more in this area. Another reason is related to the mechanical properties used in the numerical analysis for the shotcrete material, which are probably too high for describing the mechanical behavior of the shotcrete employed in the Raticosa tunnel.

In general, according to the considerations done in Chapter 7, the value of ground cohesion adopted for the Raticosa clay-shales seems to high. This fact can explain the low influence of face reinforcement on tunnel static conditions as well as the underestimated calculated convergence.

The horizontal convergence measured at the MCSs located at chainages 32+231 and 32+218 can not be easily compared to the numerical results because the tunnel excavation was interrupted there for many days. The comparison can be limited to the displacement observed in the tunnel when no excavation occurred. After the simulation of the summer break the increment of radial displacement $u_r$ develops only for $z< 2R$ in the numerical analysis. This appears in be in contradiction with what was observed in the MCS located at chainage 32+231. The numerical result better agrees with what was monitored in the MCS located at chainage 32+218. Anyway, no clear indication on tunnel behavior comes from the observation of the horizontal convergence in the MCSs.

<table>
<thead>
<tr>
<th>steel arch HEA 300</th>
<th>Fe430</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shotcrete</td>
<td></td>
</tr>
<tr>
<td>compressive strength</td>
<td>at 48 hours $\geq 13$ MPa</td>
</tr>
<tr>
<td>(core samples with h/∅ = 1)</td>
<td>at 28 days $\geq 20$ MPa</td>
</tr>
<tr>
<td>fiber-glass dowel</td>
<td></td>
</tr>
<tr>
<td>reacting area</td>
<td>8.4 cm$^2$</td>
</tr>
<tr>
<td>tensile strength</td>
<td>900 MPa</td>
</tr>
<tr>
<td>Young's modulus</td>
<td>15000 MPa</td>
</tr>
</tbody>
</table>

Table 8.1: Raticosa tunnel, Italy: properties of lining materials and fiber-glass dowels.
Table 8.2: Raticosa tunnel, Italy: geometric properties of the cross section.

<table>
<thead>
<tr>
<th></th>
<th>x (m)</th>
<th>y (m)</th>
<th>R (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>M1</td>
<td>0.00</td>
<td>0.00</td>
<td>7.00</td>
</tr>
<tr>
<td>M2</td>
<td>1.81</td>
<td>-0.75</td>
<td>8.96</td>
</tr>
<tr>
<td>M3</td>
<td>0.00</td>
<td>1.23</td>
<td>7.50</td>
</tr>
</tbody>
</table>

Table 8.3: Mechanical properties of the ground employed in the numerical analysis (the peak strain $\varepsilon_p$ is required for determination of $\chi_{DP}$ (see Figure 6b: $\chi_{DP} = \varepsilon_p - f_y/E$).

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cohesion</td>
<td>0.6</td>
</tr>
<tr>
<td>Angle of internal friction</td>
<td>18</td>
</tr>
<tr>
<td>Young's modulus</td>
<td>2250</td>
</tr>
<tr>
<td>Poisson's ratio</td>
<td>0.33</td>
</tr>
<tr>
<td>Peak strain</td>
<td>0.03</td>
</tr>
<tr>
<td>Relaxation time</td>
<td>0.3</td>
</tr>
</tbody>
</table>

Table 8.4: Thermochemical analysis: material parameters for shotcrete and ground.

<table>
<thead>
<tr>
<th>Material</th>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shotcrete</td>
<td>Heat capacity</td>
<td>2428</td>
</tr>
<tr>
<td></td>
<td>Thermal conductivity</td>
<td>12.6</td>
</tr>
<tr>
<td></td>
<td>Latent heat</td>
<td>190000</td>
</tr>
<tr>
<td></td>
<td>Radiation coefficient shotcrete-air</td>
<td>40</td>
</tr>
<tr>
<td>Ground</td>
<td>Heat capacity</td>
<td>2300</td>
</tr>
<tr>
<td></td>
<td>Thermal conductivity</td>
<td>7.2</td>
</tr>
</tbody>
</table>

Table 8.5: Chemomechanical analysis: mechanical properties of the fiber-glass dowels.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Young's modulus</td>
<td>15000</td>
</tr>
<tr>
<td>Poisson's ratio</td>
<td>0.3</td>
</tr>
<tr>
<td>Yield strength</td>
<td>900</td>
</tr>
</tbody>
</table>
8.8 Figures of Chapter 8

Figure 8.1: Raticosa tunnel, Italy: cross section.

Figure 8.2: Raticosa tunnel, Italy: construction history for the months July and August, 2000.
Figure 8.3: Multi-surface plasticity model for soil and shotcrete: (a) yield surfaces in the $\left(\frac{I_1}{\sqrt{3}}\right) - \left(\sqrt{2J_2}\right)$-space and (b) strain-hardening of Drucker-Prager criterion ($f_c$: uniaxial compressive strength).

Figure 8.4: Thermochemical analysis: 1D axisymmetric FE mesh.
Figure 8.5: Thermochemical analysis: mean temperature $\bar{T}$ of the shotcrete lining as a function of time.

Figure 8.6: Thermochemical analysis: distribution of the temperature $T$ in the shotcrete lining and in the ground at different time instants.
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Figure 8.7: Chemomechanical analysis: location of truss elements representing fiber-glass dowels used for face reinforcement.

Figure 8.8: Chemomechanical analysis: shifting of e.g. three dowels towards the axisymmetric plane.
Figure 8.9: Chemomechanical analysis: radial displacement $u_r$ at $r = 7$ and $t = 1032$ h.

Figure 8.10: Chemomechanical analysis: extrusion $u_z$ at $r = 0$ and $t = 1032$ h.
Figure 8.11: Chemomechanical analysis: distribution of hoop force $n_\varphi$ in the shotcrete lining at $t = 1032$ h.

Figure 8.12: Chemomechanical analysis: distribution of longitudinal force $n_z$ in the shotcrete lining at $t = 1032$ h.
Figure 8.13: Chemomechanical analysis: average stress $\bar{\sigma}_d$ in the fiber-glass dowels during the excavation from $z= 13$ m to $z=0$ for the analysis A3 for different position of the tunnel face $z= 12, 11, ..., 0$ m ($n_d$= number of dowels).

Figure 8.14: Chemomechanical analysis: radial displacement $u_r$ at $r= 7$ at the time instants $t= 1032$ and $t= 1608$ h obtained for the analysis A3:
Figure 8.15: Chemomechanical analysis: extrusion $u_z$ at $r=0$ at the time instants $t=1032$ h and $t=1608$ h obtained from the analysis A3.

Figure 8.16: Chemomechanical analysis: distribution of hoop force $n_\phi$ in the shotcrete lining at the time instants $t=1032$ h and $t=1608$ h obtained from the analysis A3.
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Figure 8.17: Chemomechanical analysis: distribution of longitudinal force $n_z$ in the shotcrete lining at the time instants $t= 1032$ h and $t= 1608$ h obtained from the analysis A3.

Figure 8.18: Chemomechanical analysis: average stress $\sigma_d$ in the fiber-glass dowels at the time instants $t= 1032$ h and $t= 1608$ h obtained from the analysis A3.
Figure 8.19: In situ measurements: distribution of extrusion at different instants of time (the date in the parenthesis refers to the days at which the measurements were performed).

Figure 8.20: In situ measurements: horizontal convergence at different MCSs of the tunnel.
Figure 8.21: In situ measurements: history of convergence at km 32+268 ($\Delta =$ distance between km 32+268 and the tunnel face).

Figure 8.22: In situ measurements: history of convergence at km 32+257 ($\Delta =$ distance between km 32+257 and the tunnel face).
Figure 8.23: In situ measurements: history of convergence at km 32+231 ($\Delta = $ distance between km 32+231 and the tunnel face).

Figure 8.24: In situ measurements: history of convergence at km 32+218 ($\Delta = $ distance between km 32+218 and the tunnel face).
9. Summary and Conclusions

This thesis deals with the ground-shotcrete lining interaction in the context of tunneling using the NATM. The study was developed in two directions: numerical investigation and the analysis and interpretation of a real case (the Raticosa tunnel).

The ground-shotcrete interaction in tunneling was analyzed at first by adopting an elastic constitutive model for the shotcrete with increasing stiffness in time (Chapter 5). This interaction is investigated by means of axisymmetric analyses accounting for the three-dimensional nature of the excavation process. Different numerical analyses covering a wide range of NATM tunneling conditions have been performed. For this purpose, numerical analyses characterized by different ground mechanical properties and tunnel excavation rates have been simulated. The findings of the numerical analyses can be summarized in the following points:

- The reduction in final load due to the progressive hardening of the shotcrete, is proportionately less important as the strength of the rock mass decreases, but it is not negligible.

- The ground stress release factor to be assumed in the plane strain model increases with increasing ground stiffness ratio and with increasing tunnel excavation rate.

- The conventional Convergence-Confinement method generally leads to a largely underestimated load on the lining in the case of low-strength and low-stiffness ground.

Because of the strong influence of shotcrete behavior on tunnel static conditions, the numerical investigation has been extended in order to better understand the interaction phenomena between the ground and the shotcrete lining. For the simulation of the mechanical behavior of shotcrete, a realistic material model, recently developed at the Vienna University of Technology, has been employed. It allows to simulate the hydration process which leads to aging elasticity, chemical hardening, and shrinkage of the shotcrete.
Moreover, microcracking and creep are accounted for. Axisymmetric analyses of the excavation process have been performed in order to study the influence of shotcrete modeling on convergence and loading of the lining also near the tunnel face. For the purpose of investigating a wide range of conditions for tunneling based on the NATM, additional analyses characterized by different excavation rates and mechanical behavior of the ground have been performed. The mechanical behavior of the ground is described by means of a viscoplastic multi-surface material model. Based on the obtained results, the following conclusions can be drawn:

- As to the study focusing on the influence of shotcrete modeling, the fact of considering aging, microcracking, and creep, results in an increase in the compliance of the lining. Accordingly, an increase in the radial displacement of the tunnel surface and a decrease in the compressive hoop force in the lining are observed. Loading of the lining in the longitudinal direction is found to have four sources: (a) excavation-induced bending of the ground-shotcrete compound structure resulting in compressive loading further increased by (b) the compressive hoop force via Poisson's effect; the compressive loading is reduced by (c) chemical shrinkage and (d) deformations in the longitudinal direction caused by the excavation. The influence of the material model for shotcrete and, thus, of the stiffness of the lining on the extrusion of the ground ahead of the tunnel face is found to be negligible.

- Decreasing values of Young's modulus of the ground result in increasing tunnel convergence, face extrusion and loading of the shotcrete lining in the circumferential direction. Compressive longitudinal forces are induced in the shotcrete lining for low-stiffness ground whereas tensile forces are induced for high-stiffness ground. In the former case, high compressive forces in the longitudinal direction are associated with high compressive forces in the circumferential direction via Poisson's effect. In the latter case, low compressive forces in the longitudinal direction owing to low compressive forces in the circumferential direction are overcome by tensile loading due to chemical shrinkage of the shotcrete.

- The effect of decreasing or increasing ground cohesion was similar to the effect of decreasing or increasing Young's modulus of the ground described in the previous point.
The lower the ground cohesion the greater the tunnel convergence, face extrusion and hoop forces in the shotcrete lining. Compressive longitudinal forces in the shotcrete lining are obtained for low-strength ground. On the contrary, tensile stresses are induced in the shotcrete lining for high-stress ground;

- the fact of considering ground creep results in a reduction of the displacements of the tunnel surface. Loading of the lining is delayed for increasing values of the characteristic time of creep. The final hoop forces in the lining, i.e., the hoop force at complete stress release in the ground, however, are almost the same for all considered values of the characteristic time. As regards the extrusion of the ground ahead of the tunnel face, a significant influence of ground creep is detected.

- Finally, the excavation rate of the tunnel is found to have the greatest influence on the displacements and on the loading of the lining close to the tunnel face. In this area, the tunnel is supported by rather young shotcrete. In the case of high excavation rate, the shotcrete is too young to provide significant support for the inward moving rock masses, resulting in large deformations of the tunnel surface. A small excavation rate, on the other hand, provides sufficient time for the shotcrete to increase strength and stiffness in the course of the hydration process. Consequently, the lining is able to support the inward moving rock masses leading to smaller deformations and to larger loading of the lining. The distribution of the longitudinal force in the shotcrete lining is almost similar for all analyses: the main differences resulted from chemical shrinkage which is a function of the amount of hydration of the shotcrete and, hence, of the excavation rate. Accordingly, the largest reduction in compressive force in the longitudinal direction as a consequence of chemical shrinkage is obtained from the analysis characterized by the smallest excavation rate, i.e., by the oldest shotcrete.

In general, good mechanical properties of the ground result in small radial displacements and extrusions. According to the performed analyses, creep in the ground has the same effect. Whereas good mechanical properties result also in small loading of the tunnel lining, creep only delays the loading of the lining. It does not change the final loading.

The second part of the study focuses on an analysis of the monitoring data from a deep tunnel in tectonized clay-shale, called the Raticosa tunnel (Chapter 7). The full-face
excavation of the Raticosa tunnel has been associated with face reinforcement by means of fiber-glass dowels owing to the predicted high squeezing potential of the rock-mass. The primary lining consists of a closed ring of shotcrete and steel sets, installed right after the excavation of a cross-section of the tunnel. The assessment of the mechanical properties of the tectonized clay shales has been characterized by many difficulties mainly because of the high sensitivity of the pervasively fissured material to sampling disturbance. Moreover, no clear indications about the pore-water pressure regime are available. Large plastic time-dependent deformations of the ground have generally occurred, which have determined local failure phenomena at some locations along the tunnel route. This kind of behavior stressed how important it is to closely respect the construction time-schedule in order to prevent overloading phenomena of the primary lining before the final concrete lining becomes effective. A tentative explanation of the observed behavior is proposed on the basis of simplified analytical models.

In Chapter 8, the results of a numerical analysis of the excavation of the Raticosa tunnel are presented. Both the shotcrete lining and the fiber-glass dowels are considered in the numerical analysis. The mechanical behavior of the shotcrete is described by means of the thermochemomechanical material model developed at the Vienna University of Technology. For the simulation of the ground behavior, a viscoplastic material model is employed. The analysis is carried out considering the rock-mass as a dry material. Three different analyses are performed, investigating (i) the unlined and (ii) lined tunnel without face reinforcement and, finally, (iii) the lined tunnel with face reinforcement. From the numerical results obtained, the following conclusions can be drawn:

- The face reinforcement has a small influence on the deformations of the ground. The extrusion of the tunnel face is reduced by 2 mm. This result is obtained for rather high values of ground cohesion ($c=600$ kPa) estimated from the linear interpolation of strength data at high confining pressure. The real ground cohesion for low confining pressure could be much lower; in this case, the face reinforcement could be necessary to assure tunnel stability.
The consideration of shotcrete in the numerical analysis has a significant influence on tunnel deformations. The maximum radial displacement obtained from the analysis of the unlined tunnel is reduced by 70%.

The hoop force in the shotcrete lining ranges between 6 and 10 MN/m. The respective stress is lower than 85% of the final uniaxial compressive strength considered in the analysis.

The numerical results have been compared with in situ measurements provided by the construction site. The extrusion of three sliding micrometers as well as the horizontal convergence at four measurement cross-sections have been investigated. The following conclusions can be drawn from this comparison:

- the face extrusion obtained from the numerical analysis is in good agreement with the extrusion measured on site. The excavation break at the Raticosa tunnel during the summer of 2000 is accounted for in the numerical analysis, thus allowing for the estimation of the characteristic time of the viscoplastic response of the rock-mass.

- for tunnel convergence there is little agreement between the in situ measurements and the numerical results. The in situ measurement gives higher values of the increment of convergence at equal distances from the tunnel face. For a distance of about 3R/4R, the tunnel approximates equilibrium conditions in the numerical analyses, whereas this situation does not occur in situ. These discordances may be related to the overestimation of the mechanical properties of both the ground and the shotcrete in the numerical simulation. The final compressive strength of the shotcrete employed in the Raticosa tunnel is about only 50% of the value that was adopted in the numerical model.
10. Appendix I. References


Apeendix I. References


11. Appendix II. Notation

\[ a = \text{aggregate content}; \]
\[ a^0 = \text{shape function of the convergence equation for unlined tunnels}; \]
\[ a^s = \text{shape function of the convergence equation for lined tunnels}; \]
\[ \bar{\alpha} = \text{normalized chemical affinity}; \]
\[ c = \text{cohesion; constant of the flow-creep process of shotcrete; cement content; heat capacity;} \]
\[ c_p = \text{peak cohesion}; \]
\[ C = \text{tunnel convergence in the empirical law } C(z,t); \]
\[ C = \text{elastic material tensor}; \]
\[ C_C = \text{compression index} \]
\[ C_S = \text{swelling index}; \]
\[ C_E = \text{space of admissible stress states}; \]
\[ CF = \text{clay fraction}; \]
\[ C_\infty = \text{material tensor at complete hydration of shotcrete}; \]
\[ C_{\infty,z} = \text{instantaneous convergence in the empirical law } C(z,t); \]
\[ D = \text{tunnel diameter}; \]
\[ e = \text{void ratio; lining thickness}; \]
\[ E = \text{Young's modulus}; \]
\[ E^* = \text{reduced Young's modulus of ground in the stiffness reduction method}; \]
\[ E_\infty = \text{Young's modulus of shotcrete at complete hydration}; \]
\[ E_a = \text{activation energy for the hydration process}; \]
Appendix II. Notation

\[ E_t = \] Young’s modulus of the Kelvin body;
\[ f_b = \] biaxial compressive strength;
\[ f_c = \] uniaxial compressive strength;
\[ f_{c,\infty} = \] uniaxial compressive strength of shotcrete at complete hydration;
\[ f_{DP} = \] Drucker-Prager loading function;
\[ f_t = \] uniaxial tensile strength;
\[ f_{TC} = \] tension-cut-off loading function;
\[ f_y = \] elastic limit under uniaxial compressive loading; yield strength of fiber-glass dowels;
\[ f_\alpha = \] loading function;
\[ G = \] (normalized) fourth-order compliance tensor;
\[ H = \] softening modulus of the flow-creep process;
\[ h = \] tunnel depth;
\[ I_1 = \] first invariant of stress tensor;
\[ J_2 = \] second invariant of deviatoric stress tensor;
\[ J_{\infty}^V = \] asymptotic value of the viscous compliance;
\[ k = \] thermal conductivity;
\[ K_0 = \] earth pressure coefficient at rest;
\[ K_s = \] stiffness of the lining;
\[ l_\xi = \] latent heat;
\[ m = \] mass of formed hydrates; time-dependent parameter in the empirical law \( C(z,t) \);
\[ m_\infty = \] mass of formed hydrates at complete hydration;
\[ M_\phi = \] circumferential bending moment in the lining;
\[ M_z = \] longitudinal bending moment in the lining;
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$n$</td>
<td>time-dependent parameter in the empirical law $C(z,t)$;</td>
</tr>
<tr>
<td>$n_{c,e}$</td>
<td>final hoop force in the lining;</td>
</tr>
<tr>
<td>$n_d$</td>
<td>number of fiber-glass dowels;</td>
</tr>
<tr>
<td>$n_z$</td>
<td>longitudinal force in the lining;</td>
</tr>
<tr>
<td>$n_\phi$</td>
<td>hoop force in the lining;</td>
</tr>
<tr>
<td>$p_0$</td>
<td>constant pressure at the top boundary of the structural model;</td>
</tr>
<tr>
<td>$P_i$</td>
<td>internal pressure applied at the tunnel wall;</td>
</tr>
<tr>
<td>$q$</td>
<td>pressure at the extrados of the lining;</td>
</tr>
<tr>
<td>$\mathbf{q}$</td>
<td>heat flow vector;</td>
</tr>
<tr>
<td>$q_{eq}$</td>
<td>equilibrium pressure at the extrados of the lining;</td>
</tr>
<tr>
<td>$r$</td>
<td>radial coordinate of structural model;</td>
</tr>
<tr>
<td>$R$</td>
<td>tunnel radius; universal gas constant;</td>
</tr>
<tr>
<td>$R_p$</td>
<td>plastic radius of the tunnel;</td>
</tr>
<tr>
<td>$s$</td>
<td>mean stress</td>
</tr>
<tr>
<td>$S$</td>
<td>in situ stress; microprestress force of flow-creep process of shotcrete;</td>
</tr>
<tr>
<td>$\bar{S}$</td>
<td>mean in situ stress;</td>
</tr>
<tr>
<td>$S_r$</td>
<td>saturation degree;</td>
</tr>
<tr>
<td>$t$</td>
<td>time;</td>
</tr>
<tr>
<td>$t_{\text{max}}$</td>
<td>maximum shear stress;</td>
</tr>
<tr>
<td>$T$</td>
<td>temperature; time-dependent parameter in the empirical law $C(z,t)$;</td>
</tr>
<tr>
<td>$T_0$</td>
<td>reference temperature;</td>
</tr>
<tr>
<td>$\bar{T}$</td>
<td>mean temperature;</td>
</tr>
<tr>
<td>$T_\infty$</td>
<td>temperature of the air in the tunnel cavity;</td>
</tr>
<tr>
<td>$u$</td>
<td>tunnel convergence;</td>
</tr>
</tbody>
</table>
11.4

\( u_∞ \) = final convergence of unlined tunnels;
\( u_{eq} \) = equilibrium convergence of lined tunnels;
\( u_f \) = tunnel convergence at the face;
\( u_r \) = tunnel radial displacement;
\( u_{r,max} \) = maximum tunnel radial displacement;
\( u_z \) = extrusion;
\( u_{z,max} \) = maximum extrusion;
\( U \) = activation energy for the flow-creep process;
\( v \) = tunnel excavation rate;
\( v_P \) = P-wave velocity;
\( v_S \) = S-wave velocity;
\( w \) = ratio between \( f_y \) and \( f_c \), water content;
\( W \) = natural water content;
\( W_L \) = liquid limit;
\( W_P \) = plastic limit;
\( z \) = longitudinal distance from the tunnel face; longitudinal coordinate of the structural model;
\( Z \) = face-dependent parameter in the empirical law \( C(z,t) \);
\( z_0 \) = longitudinal distance from the tunnel face at which the lining is installed;
\( \alpha \) = material parameter of Drucker-Prager criterion; lining parameter of elastic media in the New Implicit Method;
\( \alpha^* \) = lining parameter of elasto-plastic media in the New Implicit Method;
\( \alpha_R \) = radiation coefficient shotcrete-air;
\( \beta \) = material parameter of Drucker-Prager criterion; chemical dilation coefficient; relaxation factor in the Stiffness Reduction Method;
Appendix II. Notation

\( \gamma \quad = \quad \text{unit weight of total volume; viscous slip of the flow-creep process of shotcrete;} \)

\( \gamma_s \quad = \quad \text{unit weight of solid volume;} \)

\( \delta_{\phi} \quad = \quad \text{total stretch in circumferential direction;} \)

\( \delta_{\phi}^p \quad = \quad \text{plastic stretch in circumferential direction;} \)

\( \Delta \quad = \quad \text{distance between the reference chainage and the tunnel face;} \)

\( \varepsilon \quad = \quad \text{total strain tensor;} \)

\( \varepsilon^f \quad = \quad \text{flow strain tensor;} \)

\( \varepsilon^p \quad = \quad \text{strain at the end of strain hardening;} \)

\( \varepsilon^p \quad = \quad \text{plastic strain tensor;} \)

\( \varepsilon^s \quad = \quad \text{chemical-shrinkage strain tensor;} \)

\( \varepsilon^v \quad = \quad \text{viscous strain tensor;} \)

\( \varepsilon^{vp} \quad = \quad \text{viscoplastic strain tensor;} \)

\( \varepsilon_{\phi} \quad = \quad \text{total strain in circumferential direction;} \)

\( \varepsilon_{\phi}^p \quad = \quad \text{plastic strain in circumferential direction;} \)

\( \varepsilon^u \quad = \quad \text{strain at the peak stress under uniaxial loading;} \)

\( \zeta_{\text{DP}} \quad = \quad \text{hardening force of Drucker-Prager criterion;} \)

\( \zeta_{\text{DP}}^\infty \quad = \quad \text{hardening force of Drucker-Prager criterion for rate-independent plasticity;} \)

\( \zeta_{\text{TC}} \quad = \quad \text{hardening force of tension-cut-off criterion;} \)

\( \lambda \quad = \quad \text{ground stress release factor;} \)

\( \lambda_{\text{Panet}} \quad = \quad \text{ground stress release factor obtained on the basis of the Guenot and Panet procedure} \)

\( \eta \quad = \quad \text{viscosity;} \)
Appendix II. Notation

\[ \eta_f = \text{viscosity of the flow-creep process of shotcrete}; \]
\[ \nu = \text{Poisson's ratio}; \]
\[ \xi = \text{degree of hydration of shotcrete}; \]
\[ \xi_0 = \text{percolation threshold of shotcrete}; \]
\[ \rho = \text{density}; \]
\[ \boldsymbol{\sigma} = \text{stress tensor}; \]
\[ \sigma_d = \text{stress in the fiber-glass dowels}; \]
\[ \overline{\sigma}_d = \text{average stress in the fiber-glass dowels}; \]
\[ \sigma_z = \text{longitudinal stress}; \]
\[ \sigma^\infty = \text{stress tensor for rate-independent plasticity}; \]
\[ \tau = \text{relaxation time of viscoplasticity}; \]
\[ \tau_{1R} = \text{characteristic time of the excavation process}; \]
\[ \tau_{\text{hyd}} = \text{characteristic time for the hydration process of shotcrete}; \]
\[ \tau_{\text{sh}} = \text{characteristic time for shotcreting}; \]
\[ \tau_w = \text{characteristic time of viscous-creep process of shotcrete}; \]
\[ \tau_{w,0} = \text{characteristic time of viscous-creep of shotcrete at complete hydration}; \]
\[ \phi = \text{friction angle}; \]
\[ \phi_p = \text{peak friction angle}; \]
\[ \chi = \text{homotetic ratio for the convergence expression in elasto-plastic media}; \]
\[ \chi_{\text{DP}} = \text{hardening variable of Drucker-Prager criterion}; \]
\[ \chi_{\text{DP}}^- = \text{hardening variable of Drucker-Prager criterion related to } \dot{\varepsilon}^u; \]
\[ \chi_{\text{TC}} = \text{hardening variable of tension-cut-off criterion}; \]
\[ \psi = \text{dilatancy angle}. \]
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