“Assessing the impact of squeezing ground on TBM excavation”

ANDREAS FERRER SERLEV

FRANCESCO AMBERG, RAFAEL ROJAS, EDUARDO ALONSO

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Summary

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By Andreas Ferrer Serlev and supervised by Francesco Amberg, Rafael Rojas and Eduardo Alonso.

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This study takes part in a major project that aims to provide a ready to use tool which will give information of squeezing phenomena to engineers and TBM designers in terms of

- Quantification of squeezing rates of the ground around the TBM
- Quantification of squeezing pressures that build up against the TBM shield
- Quantification of risks of jamming of the TBM
- Assessment of technological measures to cope with squeezing conditions

This project is conceived to carry out the following work:

1. Modeling of the process of shield TBM excavation through squeezing ground with FLAC 5.0 SP, for which the following parameters will be considered:

**Geometrical parameters**
- Diameter of the tunnel $D$: As it defines the geometry of the excavated tunnel
- Overburden of the tunnel $H$: As the in-situ hydrostatic stress field will be directly linked to the overburden

**Initial conditions**
- In situ stresses: It will be calculated directly from the overburden ($\gamma.H$), as a hydrostatic stress field will be supposed

**Ground parameters**
These parameters are relative to the chosen rheological model or general creep model used to characterize squeezing phenomena.

**TBM parameters**
- Length of the shield $L$: The shield will be modeled as a perfectly rigid support
- Over-coring of the cutter-head
- Skin friction, depending on the operational state of the TBM (standstill or advancing)

**Construction parameters**
- Advancement rate of the TBM (case Normal Operation)
- Standstill duration (case Exception)

2. Development of a visco-elastic model and implementation with FLAC. This will require formulating a mathematical expression for the hyperbolic creep law and the following
programming with FLAC. In terms of FLAC commands, a user-defined constitutive model will be created. The formulation will be based on Phienwej's et al study *Time-Dependant Response of Tunnels Considering Creep Effect*, 2007.

3. Step 1 and 2 will be merged and tested to guarantee the functionality of the designed excavation process when the surrounding ground is constituted by the hyperbolic creep law.

4. Parametric study of the variables that play a role in the process of shield TBM excavation through squeezing ground. If steps 1, 2 and 3 result successful, parametric study will be carried out to better understand the influence of the mentioned parameters, for both cases of Normal Operation and Exception, in the event of a shielded TBM excavating through squeezing ground. This will be made by means of the numerical analysis of a large number of cases where different values are assigned to the variables.

**Keywords**: TBM, squeezing ground, assessment, Hyperbolic creep law, constitutive model, advancement rate, overcoring.
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1. INTRODUCTION

Although the mechanism of squeezing failure of tunnels has not been clearly understood yet, it is generally envisaged as the time dependent reduction of the tunnel cross-section due to large deformations of the surrounding medium. Excavation through squeezing ground conditions is particularly delicate for the case of TBM excavation (tunnel boring machine), due to the difficulties that arise whenever tunnel convergences occur with considerable magnitude at a short distance from the face within short periods of time, leading to TBM jamming as a worst scenario.

In order to better understand the causes leading to squeezing ground phenomena and come up with solutions for reducing its consequences on TBM excavations, several Swiss based organizations have signed in for a CTI project called *TBM in difficult ground conditions*. Basically, the project will focus on the study of TBM in squeezing ground conditions, blocky rocks and mixed face conditions. The participants of the project are Lombardi Consulenti, Herrenknecht, Amberg, BG Ingénieurs and EPFL (Ecole Polytechnique Fédérale de Lausanne).

As a part of this major project, the following study was carried out between Lombardi Consulenti and the EPFL’s Laboratory of Rock Mechanics (LMR), with a clear goal: developing a the basis of a new user-friendly model with FLAC for creep calculations with the eventual objective to facilitate the following information to those responsible for a good functionality of the TBM:

- Quantification of squeezing rates of the ground around the TBM
- Quantification of squeezing pressures that build up against the TBM shield
- Quantification of risks of jamming of the TBM
- Assessment of technological measures to cope with squeezing conditions

For such purpose, we based the theory development of the creep model on the study carried out by N. Phienwej et al. *Time-Dependant Response of Tunnels Considering Creep Effect*, 2007. According to this study, the empirical visco-elastic model called Hyperbolic law can represent with accuracy and simplicity the effect of creep around a tunnel.

The modeling of geo-engineering processes involves special considerations and a design philosophy. this project exposes the methodology to follow when developing and implementing a user defined constitutive model on FLAC and solving a given problem. Furthermore, analyses and designs for structures and excavations in or on rocks and soils must be achieved with relatively little site-specific data, and an awareness that deformability and strength properties may vary considerably. It is impossible to obtain complete field data at a rock or soil site. In order to overcome such constraints different methodologies will be used and explained.
2. Interest of the proposed research

TBM excavation represents a potentially very fast method of excavating and supporting a rock tunnel. However, when unfavourable conditions are encountered without warning, TBM’s inflexibility leads to consequences often far greater than in a tunnel excavated by means of traditional techniques. Among those unfavourable conditions, squeezing ground is to be highlighted, as it will cause difficulties whenever tunnel convergences occur with considerable magnitude at a short distance from the face within short periods of time, leading to TBM jamming as a worst scenario.

This Research will attempt to provide a ready-to-use quantitative assessment tool to quantify the risks involved in shielded TBM excavation through squeezing-prone ground and serve as a guideline to adapt the TBM design and construction procedures for optimum results.

The reasons why the development of such a tool might be of great interest are several:

1. The use of TBM’s for the excavation of tunnels has not stopped increasing since the moment they were first employed, some 50 years ago. Nowadays, it is rare the tunnel of a relative length that is not, partially at least, excavated by means of TBM. In Switzerland alone, the use of TBM has increased dramatically during the last years.

![Figure1: Evolution in the use of TBM in Switzerland](image)

2. In most developed countries the transportation plans are favouring more and more the creation of international networks of high speed train lines. The technical requirements of such lines, where maintaining low grade rates and radius is essential, normally imposes the construction of deep tunnels as soon as mountain ranges are to be crossed. As a result, we witness nowadays a proliferation of the construction of base tunnels for transportation, normally large infrastructure projects with high strategic and economic value. The same applies to the construction of new lines of motorways and water conveyance, where deep tunnels become more and more often needed. However, as it has already seen in this
document, squeezing problems are especially sensitive to the overburden of the tunnel. Relatively weak rock masses that did not pose problem in the past when boring a tunnel under some hundreds of meters will very surely display large time-dependent convergences as soon as a new tunnel is bored with an overburden of some thousand meters. Thus, squeezing problems might logically become more commonplace than in the past.

3. As it will be exposed in the state of the art, under the generic name of “squeezing behaviour” there exist a multitude of totally different mechanisms that trigger such a phenomenon. It is a complex phenomenon, not really well understood nor studied by the scientific community. The same applies to the industrial side of tunnelling engineering, where trial and error is still the most common tool to fight against this event.

4. The effect of squeezing ground on TBM excavation very often leads to quite “impressive” negative effects: machines are jammed, large sections of the tunnel need to be re-bored, and on some extreme occasions (tunnel Yucambú-Quibor) the TBM’s even need to be “abandoned” in the bosom of the mountain. This is a problem that, frequently, has a strong social impact, normally also due to the controversial budget implications that the construction of tunnels have nowadays.

5. The result of squeezing conditions on TBM performance means extremely serious negative effects on the budget and construction planning.

6. Even if the excavation of tunnels by means of TBM has become commonplace, the technology associated to these machines has still a large need of improvement when situations that differ from the optimum one are encountered along a tunnel. It could be said that in the last years the demands imposed to these machines have been increased at a far higher pace than the technological measures needed to cope with those new demands have been found. Squeezing ground effect on TBM reminds us the existence of those limitations.

7. As this study is framed under a major project involving construction firms, it has been of certain relevance the fact of tackling a practical subject, aiming at finding results that might be of direct use, in order to give an answer to the needs had by tunnelling industry. Therefore, a guidance has been convened with Lombardi Engineering to assess and guide through possible solutions.

8. Few studies have been carried out where the connexion between squeezing phenomena and TBM excavation has been made. Some of the most recent studies are those carried out within the frame of the EurekaBuild project TISROCK (2007), carried out by several universities (Graz, Rome) and some industrial actors. Other studies where squeezing problem and its effect on TBM have been tackled have been carried out by Anagnostous and Ramoni (2007-2008).
3. STATE OF THE ART

3.1. Definitions of squeezing ground

From the engineering point of view, two main types of time dependent rock behaviour have to be distinguished, that are usually referred to as "squeezing" and "swelling". Special care must be taken in distinguishing these two completely different mechanisms, which display very similar consequences: large time dependent convergences around the tunnel during and after its excavation.

It is possible to find common arguments amongst the different definitions of squeezing rock behaviour that have been collected from different authors: Barla (1995), Terzaghi (1946), Deere (1981), Jethwa (1981), Panet (1996), Kovari (1988), Einstein (1990), Gioda (1982), Singh (1988), Aydan (1993), etc. In the light of the different definitions available in the literature, the main aspects related to the squeezing phenomena can be shortlisted as follows:

I. Squeezing response of the rock to excavation implies large ground deformation around a tunnel that can take place during and well after its excavation.

II. This deformation is first of all produced by the disturbance of the primitive stress field as a consequence of the excavation of the tunnel. The rock mass around the opening is strained under the influence of induced stresses and deforms accordingly. In case of competent rock masses, these displacements are elastic in nature and remain generally within 1% of the tunnel radius. A softer rock mass might fail and form a plastic or broken zone around a tunnel opening, undergoing plastic deformations. This mechanism is not genuine to rocks displaying squeezing behaviour, as it just represents the behaviour of an elasto-plastic material subjected to the redistribution of the stress field due to the excavation of the tunnel.

III. The real particularity of squeezing phenomena is the aspect of the time dependency of the rock mass behaviour. This time dependency is explained in the literature by two different factors:

a) Creep caused by exceeding a limiting shear stress

- Creep in the particles of the intact material (viscous behaviour or unstable crack propagation)
- Creep along the interfaces between particles, due to a complete shear failure around the tunnel
- Creep along larger scale discontinuities such as bedding and foliation surfaces, joints and faults

These creep mechanisms involve the three well known components (primary, secondary and tertiary) and typical combinations thereof. Usually, the creep mechanisms underlying squeezing is of visco-plastic nature but, particularly at low stresses, some of the strains may be recoverable i.e. visco-elastic behaviour occurs. Creep usually occurs at stress levels below the short term shear strength of a material. The results of short duration strength tests are thus not very useful for determining creep susceptibility and the type of creep mechanism.

b) Consolidation – dissipation of pore water pressure in low permeability rock masses

Consolidation might be understood, as well as creeping processes, as another mechanism of time-dependency that contributes to the generation of a squeezing behaviour. In fact, in the vicinity of the working face, creep and consolidation are in general superimposed on the spatial stress redistribution. The mechanism of consolidation is relevant for tunnelling through water-bearing, low permeability ground.
IV. It might be accompanied but not mistaken with swelling phenomena, which is due to volume increase caused by water being uptaken by certain clay minerals (Montmorillonite) and often occurs without yielding.

V. Can occur without volume change, however, it might be associated with volume increase in dilatant materials

VI. It is affected by factors such as: rock mass strength, in situ stresses, pore water pressure, permeability, mineralogy, joint orientation, construction procedures, penetration rates, support measures, etc

VII. It is observed in weak rocks such as phyllite, mudstones, siltstones, salt, potash and/or sheared metamorphic and igneous rocks.

3.2. Mechanisms that explain squeezing ground

Even if the squeezing behavior is not yet fully understood, a broad range of completely different mechanisms which might explain this phenomenon can be found in literature.

3.2.1. Complete shear failure

This mechanism can be observed in rock masses of low strength and high deformability, as long as the particular combination of induced stresses and material properties pushes some zones around the tunnel beyond the limiting shear stresses at which creep starts (Einstein 1990, Peck et al. 1972, Hopper et al. 1972, Ladanyi 1974, Jethwa et al. 1984, Gioda & Swoboda 1999, Barla 1995).

This is the case of continuous ductile rock masses, like sedimentary soft rocks such as mudstones, claystones, shales, sandstones, certain kinds of flysch) or in masses with widely spaced discontinuities (Aydan et al., 1993, Verman et al. 1998, Bai et al. 1991,Phien-wej 1991, Barla 1995, Bhasin & Grimstad 1996). When a tunnel is driven into this kind of rock masses, the ground advances slowly into the opening without visible fracturing or loss of continuity (Gioda and Cividini, 1996). It is also the case of rock masses composed of very weathered and/or sheared metamorphic and igneous rocks (Aydan et al 1993), like altered gneiss, schists, phyllites and tuffs (Kovari 1998-2001). In this case, the continuous inward deformation of the rock may take place together with the separation of rock fragments, or blocks, from the roof and walls of the excavation (Gioda and Cividini 1996).

The gradual increase of ground deformation or pressure during tunnel excavation is associated with spatial stress redistribution taking place in the vicinity of the advancing face (Lombardi 1973). This new stress state around the tunnel might involve the complete process of shearing of the medium. In this way, ground squeezing is considered by some authors as a simple consequence of high stresses and yielding of the ground around the tunnel (Peck et al. 1972, Hopper et al. 1972, Ladanyi 1974, Jethwa et al. 1984).

But the explanation for squeezing behaviour needs also to take into account the character of time dependency. That is the reason why creep has been cited as the prime mechanism causing ground squeezing.

![Figure 2: Representation of general shear failure of the rock mass around the tunnel (Aydan, 1993)](image-url)
It is assumed that the time dependency of this shear failure is due to anyone or a combination of submechanisms, namely:

- Creep (or otherwise expressed viscous behaviour) in the particles of the intact material such as the grains of rock. Creep of individual particles may be due to viscous behaviour of the crystal structure (salt and potash rocks, Cividini 1996, Gioda & Swoboda, 1999).

- Creep along the interfaces between particles of the materials (case of soft sedimentary rocks) or through shear movement and rotation of rock elements (case of very fractured and sheared metamorphic and igneous rocks). This creep is caused by a redistribution of the contact forces between the particles or rock fragments induced by the shear stresses. This produces reorientation, relative movements of the particles, and perhaps bending and breakage of them (Gioda and Cividini 1996).

These creep mechanisms involve three components (primary, secondary and tertiary) and typical combinations thereof. When gradual increase of pressure takes place associated with spatial stress redistribution occurring in the vicinity of the advancing tunnelling face, the rock mass might exhibit instantaneous strains.

If the stresses are sustained longer, the primary (transient) creep or attenuating creep occurs (Chi-Wen Yu, 1998). This primary creep is characterized by a strain rate decreasing with time and usually exhibits a reversible nature (Gioda & Cividini 1996).

At higher stresses, the secondary creep or steady state creep might become apparent. This secondary creep will appear only if the stress level overcomes a given limit. The creep strain rate is approximately constant (Gioda & Cividini 1996).

If the applied stresses approach or pass a certain threshold, the strain will increase rapidly and a tertiary creep or accelerating creep will appear and lead to eventual failure of the rock even if this threshold is inferior to the yield plastic limit of the rock mass (Chi-Wen Yu 1998, Gioda and Cividini 1996). This last effect is particularly important for the stability of tunnels driven into squeezing rocks (ISRM 1995). In fact, tertiary creep governs the value of the so called “stand up time”, or time span during which part of the opening close to the excavation face can remain unsupported without major risks for its stability (Gioda & Cividini 1996).

Figure 3: Representation of the different components of creep in rocks (Chi-Wen Yu 1998).
Usually, the creep mechanisms underlying squeezing are of visco-plastic nature but, particularly at low stresses, some of the strains may be recoverable i.e. visco-elastic behaviour occurs. Creep usually occurs at stress levels below the short-term shear strength of a material (Einstein 1990, Dusseault and Fordham 1993, Malan 2002).

Creep and thus squeezing can occur without volume change. In cases of dilatant behaviour (perhaps the case of very sheared and fractured metamorphic igneous rocks), squeezing will be associated with volume increase.

### 3.2.2. Buckling failure

This kind of failure is initiated from the flexural tensile buckling of the inter-bedded formation (see figure below). This type of failure is generally observed in metamorphic rocks (phyllite, mica-schists) or thinly bedded ductile sedimentary rocks (mudstone, shale, siltstone, sandstone, evaporitic rocks). The squeezing behaviour is generally characterized by a relatively slow deformation in comparison to the one that occurs in the event of rock-bursting. However, for tunnels with a shallow overburden and steep bedding planes, buckling failure may cause a sudden collapse of the tunnel without any prior excessive deformation. According to Hsu et al. (2004), the discontinuity spacing of those inter-bedded formation is a factor influencing the width of the buckling failure: “For the same thickness of the formation, a smaller spacing will have a wider buckling range (...).”

![Figure 4: Representation of the buckling failure](image)

### 3.2.3. Shearing and sliding failure

This kind of mechanism is observed in relatively thickly bedded sedimentary rocks and involves sliding along bedding planes and shearing of intact rock (Aydan et al. 1996). Also according to Malan (2002), the intersection of the tunnel by prominent bedding planes containing infilling material will cause significant shear displacement on these bedding planes, leading to large deformations that can built up in time.
One special case of this mechanism is the cave-in failure, more likely to occur for a tunnel under a shallow overburden, especially with a steeply interbedded formation located at the sidewalls, and for rocks which exhibit strain-softening behaviour and which are subjected to low horizontal in situ stresses. The progressive sliding failure along the bedding planes may extend to the ground surface in case the overburden is small enough. Therefore, an increase of horizontal stress will help to reduce the amount of squeezing deformation for this kind of tunnel (Hsu, 2004).

3.2.4. Squeezing due to stress relaxation in foliated rock

Metamorphic rocks containing phyllosilicates are prone to creep and relaxation (Kolymbas 2006), as in the long term only small or vanishing shear stress can be sustained in the foliation planes. The orientation of schistosity (or foliation) imposes a mechanical anisotropy to such rocks. It appears reasonable to assume that stress relaxation affects only shear stresses acting upon planes of schistosity. As a consequence, tunnels that cross the planes of schistosity (figure a) perpendicularly are not affected by squeezing, even at high depths. In contrast, tunnels whose axes have the same strike as the schistosity planes (figure b) can be considerably affected by squeezing. In this case of squeezing mechanism, the deformations occur asymmetrically, being larger in the direction normal to the foliation due to the smaller Young’s modulus in that direction (Kolymbas 2006).
3.2.5. **Squeezing due to consolidation processes**

Some authors, like Anagnostous and Ramoni (2007), claim that squeezing behaviour can also be explained by another mechanism: the consolidation of the ground surrounding the tunnel when tunnelling takes place through water bearing and low permeability ground, happening simultaneously with the stress redistribution caused by the advancing tunnel heading.

Thus, creep and consolidation generally occur in a superimposed way. The squeezing of saturated ground generally leads to a rise in water content ("plastic dilatancy"). This occurs more or less rapidly depending on the permeability of the ground. In case of a low-permeability ground, the water content cannot change immediately after excavation. Since the pore water hinders dilatancy, negative excess pore pressures develop, which will dissipate over the course of the time. According to Anagnostous and Ramoni, “tunnel excavation in a saturated and relatively impermeable ground will trigger a transient seepage flow process, where the state of the ground will pass from a short term behaviour characterized by a constant water content ("undrained conditions"), to a long term behaviour governed by the steady-state pore pressure field ("drained conditions"). Seepage flow and ground deformation will be coupled to each other. The pore water pressures and the effective stresses will change with time, the latter leading to additional deformations or, in the presence of a lining or a TBM shield, to increase loading, producing a squeezing behaviour. The undrained conditions are more favourable because short term suctions strengthen the ground, which is called "dilatancy hardening".

3.2.6. **Time dependent microcracking in brittle / quasi-brittle hard rocks**


Malan (2002) talks about the creation of a “fractured zone” around the tunnel, where multiple cracks grow and interact, deeply influenced by existing discontinuities or bedding planes.

According to Boukharov and Chanda (1995), the creep nature or such rocks is determined by their mineralogical and structural features. The reason is that, as opposed to relatively homogeneous rocks as limestone where the properties of the rock do not differ significantly from the properties of the rock forming mineral, the properties of heterogeneous rocks such as granites depend not only upon the minerals constituting it, but upon the structural features of the rock as well: size difference of individual
mineral crystals, boundaries between crystals, pores and microcracks occurring in and/or between the crystals.

Brittle creep in rocks occurs due to the growth of pre-existing microcracks (Boukharov and Chanda (1995), Shao et al. (2003, 2005), Malan (2002)). The crack propagation events cause the observed creep displacement. Under compressive stresses, sliding wing cracks seem to be the principal propagation mode of microcracks (Shao and Chau, 2005). Due to roughness of crack surfaces in hard rocks, crack sliding may induce an associated aperture which is the origin of volumetric dilatancy in these materials. On propagation, cracks intersect other cracks. The new intersecting crack array may be unstable and depending on the length and orientation of the cracks, further growth and propagation could take place. As the array grows in size, it will intersect other cracks leading again to array growth. Intersection can then result in increasingly large arrays of cracks which grow by an accelerating process until they extend through the specimen and it finally fails. In other words, intersection may result in accelerating creep that leads to failure. The probability of an intersection occurring depends on the density of cracks in the rock. A high density leads to a high probability of intersection. As Cruden (1974) pointed out, the onset of accelerating creep occurs when the “hot” crack arrays growing make up a critical density. Thus, brittle creep is for the author a process of accumulation of displacements caused by microcrack propagation.

According to some authors like Shao and Chau (2005), Potyondy (2006), microcracks within the rock will grow due both to instantaneous stress induced growth and sub-critical growth. Creep will mainly be due to sub-critical propagation of microcracks. Under a certain applied stress, if the equivalent tensile force in a given crack reaches a critical value represented by the material toughness, there is an instantaneous propagation of the microcrack and then a new mechanical equilibrium is obtained. During the time at these fixed stresses, the stress corrosion process takes place and leads to time dependent slow propagation of cracks. It is then needed to define a long term residual material toughness, smaller than the original, which determines the threshold for sub-critical crack propagation. When the equivalent tensile forces on the crack remain higher than this threshold, the sub-critical crack propagation continues and the creep strain increases. If the cumulated crack length in the most unfavourable orientation reaches a critical radius, there is an accelerated creep leading to material failure.

The phenomenon of sub-critical crack growth can, according to Atkinson and Meredith (1987), be due to stress corrosion. Stress corrosion involves a thermally activated surface reaction between the rock and an environmental agent such as water. The activation energy for the reaction is reduced by tensile stress such that the reaction will be most rapid at sites of large tensile stress. Such sites exist at the tips of strained defects and lead to preferential corrosion at the tips, thereby increasing the defect length and maintaining conditions for continued growth.

In tunnelling, this phenomenon leads to the creation of a “fractured zone” around the tunnel, where multiple cracks grow and interact, deeply influenced by existing discontinuities or bedding planes.

Figure 8: Evolution of the fracture zone through time.
3.2.7. Swelling

As explained before, swelling and squeezing phenomena often happen in a superimposed way, with similar consequences but completely different mechanisms. According to the definition proposed by the ISRM Commission on Swelling Rock (1995), the swelling mechanism “is a combination of physico-chemical reactions involving water and stress-relief”. Since swelling is related to an increase of the water content, the duration of this phenomenon and its rate are markedly influenced by the permeability.

In a clayey rock the absorption of water is associated with an increase of the distance between the solid particles that, in turn, produces a reduction of the interaction forces connecting them. The decrease of the particle bonds reduces the overall shear resistance of the rock. As a consequence, in the presence of a non negligible shear stress level, swelling may be associated with the development of time dependent deviatoric strains. Swelling presents mainly in over-consolidated clays moderately stiff, with a consistency from medium to high and with low natural water content, close the plastic limit. This phenomenon is also present in clayey rocks that contain clay minerals such as Montmorillonite which has a high swelling capacity, or in rocks with a tendency to a volume increase through absorption of water such as Anhydrite.

3.3. Specific problems due to squeezing ground behavior on TBM excavation

The hazards associated with squeezing ground concern both the machine and the back-up area. Due to the fixed geometry and the limited flexibility of the TBM the room to be allowed for ground deformations is restricted. Convergences which exceed 5% of the tunnel radius are to be considered problematical (Kovári 1986). The consequences of squeezing can range from large tunnel closures and high pressures exerted by the rock mass on the shield of the TBM to more extreme conditions, when the friction produced by the ground in contact with the machine cannot be counteracted by the available thrust and the TBM becomes jammed (Steiner 1996, Einstein and Bobet 1997). Therefore, even if any tendency to instability at the face is likely to be overcome as any squeezing is excavated as part of the cutting process, in severe squeezing conditions when face extrusion may become important, severe problems might also be experienced at the cutting face.

As the TBM types are different with respect to the thrusting system, the type of support and the existence or not of a shield, different hazard scenarios have to be considered, that will depend on the machine type:

a) Gripper TBMs

These TBMs are today generally equipped with a short shield (canopy, cutter head shield). Depending on the rheological behaviour of the ground, a high radial ground pressure acting upon the cutter head or the canopy as well as an extremely high extrusion rate of the core can also immobilise the machine. Normally, the excavation speed is high enough to avoid such problems. The short length of the shield has a positive influence. If the TBM is moving the risk of a shield jamming is lower (deformations occur mostly only after the passage of the machine). Maintaining a high advance rate may nevertheless, be difficult in poor ground, because support installation needs more time and squeezing may also reduce the performance of the back-up system (e.g. re-profiling works, differential heave or twisting of the tracks). In the extreme case of a standstill, the TBM can be freed if the installed thrust force and torque are high enough and the ground can provide a sufficient reaction to the gripper forces.

b) Single shielded TBMs

These TBMs are longer than gripper machines. The bigger length increases the risk of becoming trapped in squeezing ground. On the other hand, single shielded TBMs have the advantage of a higher advance rate in poor ground, although feedback effects are possible for these machines too. For example, high water inflows or unstable tunnel walls may make installation or backfilling of the lining difficult and therefore slowdown advance. A single shielded TBM is jacked against the segmental lining. The possible
thrust force and torque depend not only on the design of the machine (installed thrust force and torque) but also on the structural design of the segmental lining and the quality of annulus grouting. The lining has to be designed for the combined action of ground pressure and maximum jacking forces, in order to avoid overstressing or inadmissible ovalisation.

c) Double shielded TBMs

These TBMs install the lining simultaneously with boring, thus achieving higher performances than single shielded TBMs. These machines are longer, however, particularly in small diameter tunnels. In weak ground prone to squeezing, the bracing by the gripper may be impossible and, furthermore, additional problems may occur with the extension and compression of the telescopic joint. The machine is then operated in single shield mode with jacking against the segmental lining. In this case, the same remarks as for the single shielded TBM apply. But, in the latter case, there is a main difference respect to the single shielded TBM, the length of the shield, being much longer in the case of a double shielded TBM, leading to higher friction forces when the rock mass enters in contact with the shield and higher risks of TBM jamming.

The design of the TBM plays an important role. The improving TBM technology allows the installation of higher thrust force and torque and the reduction of the machine length also for double shielded TBMs (easier to realize for bigger diameters). The shield can also be slightly "conical" (2-3 cm in radius). The friction between shield skin and ground can be reduced (up to 50 %) by lubricants such as bentonite (Gehring 1996). A moderate amount of squeezing can be accommodated by using extendable gauge cutters when such ground is encountered. This solution allows an increasing of the boring diameter up to 30 cm (Wolff & Goliasch 2003) and can be easily handled by gripper TBMs; for shielded TBMs, lifting of the centerline of the shield is necessary (Voerckel 2001). However this technology is not yet well developed and is of very uncertain value in long reaches of squeezing ground (ITA 2003). The trouble-free application of it seems to be possible only in very soft rocks.

![Figure 9: Solution for radial overcut by increasing the excavation diameter (Voerckel, 2001).](image)

The problems associated with excessive deformations of the tunnel during excavation in squeezing conditions are of great concern for both designers and contractors. When squeezing rock conditions are expected, several decisions are open to debate: TBM choice, need of overcutting, installed thrust and torque, the length of the shield, etc.

As it is well known, the value of the TBM, in terms of direct project costs, is relatively insignificant. Failure to achieve the desired results and maintain the time schedule, however, significantly affects the project. This is why from the outset of the tunnel project it is important to adopt the approach of utilizing the best possible equipment and excavation procedures (Barla & Pelliza, 1996). This is especially true in the case of tunnelling through ground displaying squeezing ground behaviour.

The different hazard scenarios have to be considered, being of especial relevance the machine type that is going to be employed as well as the especial technology with which the TBM will be fitted out in order to cope with the expected squeezing situation. In the worst case scenario of squeezing behaviour, the
situation might make the use of a TBM unworkable (Steiner 1996, Einstein and Bobet 1997). So, it is evident the relevance of prior identification and quantification of the squeezing potential of a tunnel that might be excavated by means of a TBM.

The identification of the degree of potential squeezing that the ground might display would in this way enable:

In terms of construction planning:

- A better selection of the excavation method (TBM or drill and blast) and a more accurate assessment of the TBM performance.
- A better planning of the alignment of the tunnel
- A better planning of the construction techniques and procedures adapted to the evaluated risk of squeezing behaviour (i.e. to plan a pre-treatment of the ground, using conventional excavation of the critical zone, etc.)
- To foresee the need of pursuing a high rate of excavation
- To foresee the need of keeping standstills as short as possible. If an identified critical zone has to be crossed, exhaustive maintenance work should be accomplished in advance and the necessary logistical precautions taken to allow for continuous operation in the critical zone (Ramoni & Anagnostou 2007).

In terms of the choice of the most appropriated TBM.

The identification of squeezing potential of the ground would enable:

- The choice of the more adapted type of TBM: open, single shielded or double shielded
- In extreme cases, the choice of special TBMs, like an outershield TBM (Walking Blade Shield) with parallel blades that are supported on hydraulic rams and can move independently in both axial and radial directions.
- The need of providing the TBM with especial features, like overcutting, a more pronounced conical shape for the shield, a higher thrust force and torque, reduction of the shield length, lubrication of the shield to reduce the skin friction, use of compressible backfill in the segmental lining, use of especial yielding segments for the lining in shielded TBMs (Tisrock project, 2006), etc.

3.4. Approaches for assessing the squeezing ground behavior

From Terzaghi’s first attempt of roughly quantifying the squeezing behaviour (1946), a number of approaches have been proposed by various authors. These approaches can be classified, attending to the followed approaches, in:

1. Qualitative Empirical methods
2. Qualitative Semi-Empirical methods
3. Quantitative Analytical Continuum Elasto-plastic models: Closed form solutions
4. Quantitative Analytical Discontinuum Elasto-plastic models
5. Quantitative Continuum Rheological models
6. Quantitative Analytical Continuum models based on physical mechanisms (material degradation and damage mechanics)
7. Quantitative Discontinuum Rheological models
8. Quantitative Analytical Discontinuum models based on physical mechanisms (material degradation and damage mechanics)
9. Quantitative Empirical Rheological models

### 3.4.1. Qualitative Empirical methods

These empirical approaches are essentially based on classification schemes, supported by the study of different case histories of tunnelling through squeezing ground.

Singh et al. (1992), plotted a clear cut demarcation line to differentiate squeezing cases from non-squeezing cases, based on 39 case histories, by matching data on rock mass quality $Q$ (Barton et al. 1974) and overburden $H$.

He calculated the equation of the line as:

$$H = 350 Q^{1/3} \text{ [m]}$$

Obtaining the conclusion:

- **For squeezing conditions**
  $$H >> 350 Q^{1/3} \text{ [m]}$$
- **For non squeezing conditions**
  $$H << 350 Q^{1/3} \text{ [m]}$$

### 3.4.2. Qualitative Semi-Empirical Methods

The semi-empirical approaches give indicators for predicting squeezing, but also providing some tools for estimating the expected deformation around the tunnel and/or the support pressure required, by using closed form analytical solutions for a circular tunnel in a hydrostatic stress field. The common starting point of all these methods for quantifying the squeezing potential of rock is the use of the “competency factor”, which is defined as the ratio of uniaxial compressive strength $\sigma_c/\sigma_{cm}$ of rock/rock mass to overburden stress $\gamma H$. This competency factor was initially proposed by Muirwood (1972), and later used by Nakano (1979), Barla (1995), Aydan (1996) and Hoek (1999).
So, Jethwa et al. (1984) define the degree of squeezing on the basis of the competence factor, defined as:

\[ N_c = \frac{\sigma_{cm}}{p_0} = \frac{\sigma_{cm}}{\gamma H} \]

where:
- \( \sigma_{cm} \) = rock mass uniaxial compressive strength;
- \( p_0 \) = in situ stress;
- \( \gamma \) = rock mass unit weight;
- \( H \) = tunnel depth below surface.

According to the value of this competency factor, the squeezing behaviour is determined as follows:

<table>
<thead>
<tr>
<th>( N_c )</th>
<th>type of behaviour</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;0.4</td>
<td>highly squeezing</td>
</tr>
<tr>
<td>0.4-0.8</td>
<td>moderately squeezing</td>
</tr>
<tr>
<td>0.8-2.0</td>
<td>mildly squeezing</td>
</tr>
<tr>
<td>&gt;2.0</td>
<td>non squeezing</td>
</tr>
</tbody>
</table>

A plot of the \( p_u / p_0 \) ratio is given versus \( \Phi_p \), for different values of \( \sigma_{cm} / 2 p_0 \) and a set of residual friction angles \( \Phi_r \), always for a residual cohesion:

![Figure 10: Representation of Jethwa's qualitative semi-empirical method.](image)

Figure 10: Representation of Jethwa's qualitative semi-empirical method.
3.4.3. Quantitative Analytical Continuum Elasto-Plastic models: Closed form Solutions

These are methods for analysis of the onset of yielding within the rock mass, as determined by the shear strength parameters relative to the induced stress, as well as the extent of the total “yielded” or “plastified” zone around the tunnel, deformations around the tunnel and stress exerted on the tunnel support. It is important to point out that these models do not consider the time-dependency of the squeezing phenomena. These solutions can very useful in order to gain insights into tunnel behaviour when the excavation takes place in rock masses which exhibit squeezing conditions (Barla, 2001).

The tunnel is assumed to be circular and the rock mass subjected to a hydrostatic in situ state of stress. The rock is assumed to behave as an elasto-plastic-isotropic medium, being possible to use different models: elastic-perfectly plastic, elastic plastic with brittle behaviour, elastic-plastic with strain softening or strain hardening.

The equilibrium solution for the rock support interaction analysis is given by the intersection of the “rock characteristic line” and the “support characteristic line”. This is the essence of the so called “convergence-confinement” method.

This method is not able to tackle the time-dependency of the squeezing behaviour, unless using the method by also taking into account a degradation of deformation and strength characteristics of rocks as a function of time by utilising information obtained from creep tests.

Calculation of the rock mass characteristic curve:

A comprehensive set of solutions of the elasto-plastic type has been given by Panet (1996), in his book on the “Convergence-confinement” method. More recently, a mechanically rigorous elasto-plastic solution for the problem of unloading a cylindrical cavity in a rock mass that obeys the Hoek-Brown yield criterion has been given by Carranza Torres and Fairhurst (1999).

Calculation of the Support Characteristic Curve

It relates the confining pressure acting on the support to its deformation. The “support characteristic lines” can be computed by a set of equations (Hoek and Brown, 1980; Brady and Brown, 1985), which allow one to determine the stiffness $k_i$ and the maximum support pressure $p_{\text{max}}$ for typical support systems (concrete lining, shotcrete, steel sets embedded in shotcrete, rock bolts, etc).
3.4.4. Quantitative Analytical Discontinuum Elasto-Plastic models

When squeezing behaviour is due to sliding failure, the rock mass must be studied as a discontinuum medium. Hsu et al. (2004), have modelled the rock mass by representing the bedding planes in the stratified rock masses and considering Coulomb slip models with a capability of weakening upon failure for the discontinuities, and an elasto-plastic with strain-softening model for the intact rocks. Peak and residual shear strengths of the intact rocks and bedding planes are used in the model. The tensile strength and dilation angle of the material and discontinuities are assumed to be zero, since the strength of weak rock may have deteriorated and become completely lost with time due to weathering, water-softening or swelling after stress relief.

This is the example of a model in which the time-dependency is not taken into account, based on elasto-plastic behaviour but where the mechanism of squeezing is so linked to the existence of discontinuities that the rock mass must be studied as a discontinuum medium.

3.4.5. Quantitative Analytical Continuum Rheological models

The analytical methods based on elasto-plastic rock behaviour study ground squeezing as a simple consequence of high stresses and yielding of the ground around the tunnel (Peck et al. 1972, Hopper et al. 1972, Ladanyi 1974, Jethwa et al. 1984). But the explanation for squeezing behaviour needs also to take into account the character of time dependency. That is the reason why creep has been cited as the prime mechanism causing ground squeezing (Phienwej et al. 2007). In order to describe rock creep, many rheological models have been developed. These models can be classified into:

a) Visco – elastic models

b) Visco-elastic-plastic models

c) Elasto-visco-plastic models

d) Elasto-visco-plastic-damaged models

e) Other models

3.4.5.1. Visco-elastic models

Visco-elastic models are simple rheological models which generally comprise basic mechanical models, such as a spring and dash pot, to simulate a range of time-dependent behaviour. The different combination of these spring and dash pot model lead to different rheological models. These models include the elastic model, viscous model, Maxwell model, Kelvin model, Generalized Maxwell model, Generalized Kelvin model, Burgers model and others.
<table>
<thead>
<tr>
<th>Type</th>
<th>Property</th>
<th>Stress-Strain-Time Relation in One Dimension</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spring</td>
<td>Elastic</td>
<td>$\varepsilon = \frac{\sigma}{E}$</td>
</tr>
<tr>
<td>Newtonian</td>
<td>Viscous</td>
<td>$\varepsilon(t) = \frac{\sigma \cdot t}{3\eta}$</td>
</tr>
<tr>
<td>Maxwell</td>
<td>Visco-elastic</td>
<td>$\varepsilon(t) = \frac{\sigma}{E} + \frac{\sigma t}{3\eta}$</td>
</tr>
<tr>
<td>Kelvin</td>
<td>Visco-elastic</td>
<td>$\varepsilon(t) = \frac{\sigma}{E} [1 - \exp\left(-\frac{E_b \cdot t}{3\eta_b}\right)]$</td>
</tr>
<tr>
<td>Generalized Maxwell</td>
<td>Visco-elastic</td>
<td>$\varepsilon(t) = \frac{\sigma}{E} [1 - \exp\left(-\frac{E_b \cdot t}{3\eta_b}\right)] + \frac{\sigma \cdot t}{3\eta_m}$</td>
</tr>
<tr>
<td>Generalized Kelvin</td>
<td>Visco-elastic</td>
<td>$\varepsilon(t) = \frac{\sigma}{E} [1 - \exp\left(-\frac{E_b \cdot t}{3\eta_b}\right)] + \frac{\sigma}{E_m}$</td>
</tr>
<tr>
<td>Burgers</td>
<td>Visco-elastic</td>
<td>$\varepsilon(t) = \frac{\sigma}{E} [1 - \exp\left(-\frac{E_b \cdot t}{3\eta_b}\right)] + \frac{\sigma}{E_m}$</td>
</tr>
</tbody>
</table>

Note: $\sigma$ = stress, $\varepsilon$ = strain, $E$ = Young’s modulus, $E_b$ = bulk modulus, $E_m$ = shear modulus, $\eta$ = viscosity, $t$ = time, subscript $b$ denotes Kelvin model, subscript $m$ denotes Maxwell model.

Figure 12: Rheological visco-elastic models list and characterizing strain –stress relation.

The schematic representation by means of springs and dashpots as well as the evolution of deformations in time is shown in the following figure for the following visco-elastic models:

- a) Maxwell model
- b) Kelvin model
- c) Generalized Maxwell model
- d) Generalized Kelvin model
- e) Burger’s model (after Goodman, 1989)

Figure 13: Schematic representation of the rheological models and strain’s time dependence.
The Kelvin model and the generalized Kelvin model can only simulate the primary creep or rock. The Maxwell model and the generalized Maxwell model can only describe the secondary creep of rock. The Burger’s creep model, derived from combinations of the Kelvin model and Maxwell model, can describe the elastic strain, the primary creep and the secondary creep. Goodman (1980,1989) stated that for many practical purposes for rocks, the Burgers creep model is preferable and will suffice for the description of most rock creep behaviour if proper parameters are selected.

3.4.6. Quantitative Analytical Continuum models based on physical mechanisms (relaxation along foliation planes, material degradation and damage)

As opposed to purely rheological models, which could be considered as bare mathematical frameworks for the modelling of creep deformations, there are other models which attempt to explain the rheological behaviour of the rock by also taking into account physical mechanisms related to these deformations: damage mechanics, degradation of strength and elastic properties, relaxation along the foliation planes of certain rocks due to the “sliding” along mica crystals, etc. Hereafter, some of the main models that fall into this category are analysed.

3.4.6.1. Model for rheological behaviour or foliated metamorphic rocks (Kolymbas, 2006)

Kolymbas (2006) proposes a model to account for rheological behaviour of metamorphic rocks containing phyllosilicates. His theory is that these rocks are prone to creep and relaxation because in the long range only small or vanishing shear stress can be sustained in the foliation planes. So, the orientation of schistosity or foliation imposes a mechanical anisotropy to such rocks. The author assumes that stress relaxation affects only shear stresses acting upon planes of schistosity. As a consequence, tunnels that cross the planes of schistosity perpendicularly are not affected by squeezing, even at high depths. In contrast, tunnels whose axes have the same strike as the schistosity planes can be considerably affected by squeezing.

3.4.6.2. Models based on Damage Mechanics for brittle and quasi-brittle rocks: isotropic and anisotropic damage

As it has been shown, different approaches exist to model the rheological behaviour of general rocks. However, when trying to explain the rheological behaviour of brittle or quasi-brittle rocks, it has been stated of special interest the use of models based on damage mechanics. As opposed to purely rheological models, which could be considered as bare mathematical frameworks for the modelling of creep deformations, models based on damage mechanics attempt to explain the rheological behaviour of the rock by also taking into account physical mechanisms related to these deformations. Among the different damage models that have been developed for the description of induced damage there are two main families: micromechanical approaches and phenomenological models.

Micromechanical approaches: The main advantage of micromechanical approaches is the ability to account for physical mechanisms involved in the nucleation and growth of microcracks. For the construction of a micromechanical model, two steps are generally performed. The first step consists in the evaluation of the effective elastic properties of material weakened by microcracks. The second step is to propose a suitable damage evolution law for microcrack growth. The main features related to microcrack growth, opening and closure, friction, interaction between cracks, could be taken into account in such micromechanical models. The macroscopic behaviour of the rock is then obtained through a homogenization procedure. This renders these models difficult to be applied to practical applications.
Phenomenological models: These models use internal variables to represent the density and orientation of microcracks, for instance, scalar variable for isotropic damage, second and fourth rank tensor to describe anisotropic damage. The constitutive equations are generally formulated using the concept of effective stresses based on the principle of strain and energy equivalence and from the standard derivation of a thermodynamic potential. The damage evolution law is determined according to the principles of the irreversible thermodynamics. The main advantage of such models is that they provide macroscopic constitutive equations, which can be easily implemented and applied to engineering analyses. The main weakness is that some of the concepts and parameters involved in these models are not clearly related to physical mechanisms.

3.4.6.3. Models based on material degradation (Shao et al. 2003)

Shao et al. (2003) propose a new constitutive model for creep deformation in rock materials, starting from an elastoplastic model for the description of short term behaviour, and incorporating the time-dependent deformation in terms of evolution of microstructure. This evolution is accounted by assuming a progressive degradation of elastic modulus and failure strength of material. According to Shao et al. (2003), this specific model is used in order to describe the creep in sedimentary rocks (argilites), where there is a coupling between plastic flow due to sliding of clay sheets and damage growth due to propagation of microcrack around quartz and calcite grains. This could let us think that this model might be of a certain interest when trying to model metamorphic soft foliated rocks (where there might be plastic flow associated to sliding of mica sheets, and damage growth due to the quasi-brittle nature of these rocks).

3.4.7. Quantitative Analytical Discontinuum Rheological models

3.4.7.1. Study of Creeping associated to macro-discontinuities within the rock mass

It is generally accepted that the time-dependent behaviour of excavations in hard rock is governed mainly by the rheological properties of discontinuities surrounding the excavation. This is in agreement with other authors (Tan and Kang, 1980; Schwartz and Kolluru, 1984). Barla (2000) noted that in conditions where discontinuities dominate the squeezing behaviour, discontinuum modelling is the most appropriate model to simulate the behaviour of the rock.

![Figure 14: Schematic representation of creep process in hard rocks due to macro-discontinuities.](image)

Realistic modelling of the time-dependent behaviour of hard rock therefore needs to simulate the rheological behaviour of discontinuities and the interaction between these discontinuities. Samtani et al. (1996) developed a viscoplastic interface model for use in finite element programs. Napier and Peirce (1995) developed a boundary element method for solving multiple interacting crack problems in which several thousand elements can be treated. This formulation has also been used by Malan (2002) to simulate the time-dependent behaviour of excavations in hard rock when large discontinuities rule the
behaviour of the rock mass. For this, a viscoplastic displacement discontinuity interface model was developed. Malan (2002) postulates that the intact rock material behaves elastically and all inelastic behaviour, including viscoplastic effects, are controlled by the presence of multiple interacting discontinuities. Shear slip on these discontinuities happens in a time-dependent fashion. This allows for a progressive redistribution of stress near the edges of the tunnel (mine edges in his study). The detailed formulation is reflected by Napier and Malan (1997).

3.4.8. Quantitative Analytical Discontinuum models based on physical mechanisms (damage, stress corrosion)

3.4.8.1. Napier and Malan (1997)

Napier and Malan (1997) propose a discontinuum viscoplastic formulation to relate the rate of slip on a crack to the shear stress acting on the crack. A procedure is outlined by the authors for the solution of a collection of interacting cracks in a series of timesteps and for the computation of energy changes in the crack assembly during each timestep. The authors show that this model is able to simulate complex material behaviour if applied to a random assembly of cracks, giving results that represent very well the primary, secondary and tertiary creep phases of rocks containing an initial population of weak flaws. The model has also proofed to cast good predictions of movements at tunnel level (deep mines in South Africa).

The model works by supposing that the problem region of interest is covered by a specified mesh of potential crack surfaces $S_d$ and that each arc of the mesh is a straight line segment that is divided into one or more elements.

As it can be seen in the figure above, approximately one tenth of the flaws, depicted by heavy lines in the figure, are assumed to be “weak” and to have no cohesion or tensile strength. The remaining flaws are assumed to be “strong” and to have a uniform strength. Specific material properties of the weak and strong flaws must be given, and the flaw size statistics relative to the dimension of the sample. Summing up, the following parameters must be taken into the model:
One of the advantages of this model is that it might take into account initial anisotropy of the rock, by imposing a certain biased statistic distribution of open cracks within the rock mass. However, the large quantity of parameters that are needed make the use of the model quite complex.

3.4.9. Quantitative Empirical time-dependent models

3.4.9.1. Hyperbolic and Power creep empirical laws

In order to model and quantify the time-dependent behaviour of squeezing ground around tunnels, creep has been commonly described by rheological models, models based on damage mechanics or other physical processes (stress corrosion, strength degradation, etc) and last, empirical models. The rheological models, as it has been already exposed, are composed of a number of mechanical elements (springs, dashpots and sliders) to account for the stiffness, viscous properties and strength of the ground. The mathematical relationship of strain, stress, and time is then derived directly from the way the elements are connected, and the parameters of the models are obtained from curve-fitting of creep test data of the material. On the other hand, models based on damage mechanics, material degradation, stress corrosion, etc, are models whose parameters mainly represent physical entities. However, the main drawback of these two groups of models is that, in order to closely describe the real relationship of strain-stress-time in the ground, they would require a large number of elements (Kaiser 1979), that would, in turn, result in complicated mathematical equations, having a large number of parameters that are not so simple to determine.

This problem is even larger when it comes to the study of the behaviour of a rock mass around a tunnel excavation, due to the big scale effect normally present in rock mechanics problems. According to Sulem (1987, 1994), the use of complex rheological models to account for the behaviour or rock masses at tunnel level has a major drawback: the determination of their parameters, based on data from geological and geotechnical investigations, is very delicate. Shape and scale effects may appear in phenomena like the viscosity or consolidation of a rock mass and it is very difficult to extrapolate laboratory results to field situations.

Empirical creep models, however, are derived directly from the observed relationship of time, stress, and strain or strain rate of creep test results. These models could also be “calibrated” or derived from observed relationships obtained by performing “tests” at real tunnel scale, that is, by using directly the tunnel monitoring results, as these results are the only that represent the real behaviour of the rock mass at the scale of the tunnel.

The empirical models are usually expressed in simple mathematical forms with a small number of parameters. According to Phienwej et al. (2007), empirical models have been successfully used to describe observed creep behaviour of soil and rock. The commonly used empirical creep models are such as the power law (Obert 1965, Phienwej, 2007), the exponential law (Singh and Mitchell 1968; Semple 1973), and the hyperbolic law (Mesri et al. 1981, Phienwej, 2007). Creep behaviour of most rocks is found to be adequately described by the power law (Obert 1965; Singh 1975; Campos de Orellana 1996). For clayey soils, weak shale, mudstone, and faulted rocks, the exponential law and the hyperbolic law are more commonly used (Semple 1973; Febres-Cordero 1974; Mesri et al. 1981; Li and Wang 1998).

These empirical laws have been used in developing visco-elastic solutions for prediction of tunnel closure and ground pressure on supports by numerous authors: Aiyer 1969; Semple 1973; Hanafy and Emery 1979; Phienwej 1987; Sulem et al. 1987; Schubert et al. 2003, etc
The power law is a creep law that provides a visco-elastic analysis of the ground. This law is commonly used for rocks, specially rock salt, potash and evaporates. Based on the original expression suggested by Obert (1965), the relation between strain, stress and time is expressed as follows:

$$\varepsilon_{ai} = k \sigma^\alpha \left( \frac{t}{t_1} \right)^\lambda$$

Where $\varepsilon_{ai}$ = axial strain at time $t$

$\sigma$ = stress difference given by $\sigma_1 - \sigma_3$

$\alpha$ = index of the power function between stress and strain

$\lambda$ = index of power function between strain and time, which is called creep parameter

$k$ = constant at the reference time $t_1$ and related to the modulus of the material

The variable $t_1$ of one hour is mostly used for reference of stress-strain relationship parameters of geomaterials.

If one considers strength limit of the material and the validity of the normalization principle between stress and strain, the power law may be expressed as

$$\varepsilon_{ai} = KD_i^\alpha \left( \frac{t_i}{t_1} \right)^\lambda$$

Where $K$ = dimensionless constant; and $D_i$ = stress level expressed as the ratio of $(\sigma_1 - \sigma_3) / (\sigma_1 - \sigma_3)_f$

The hyperbolic law, mostly used for clayey soils and soft rocks, was firstly introduced by Mesri et al. (1981), and applied to visco-elastic analysis of tunnel closure by Phienwej (1987). The hyperbolic law has an advantage to other empirical creep models in that all of its parameters have physical meaning and are related to the mechanical properties of the materials (i.e. modulus and strength). This makes it more attractive than others (Phienwej 1987; Lin and Wang 1998). Besides, the hyperbolic creep law is found to be more practical for simulation of closure response of squeezing ground then the Power creep law because it is capable of modeling ground yielding around the tunnel when the magnitude of $R_f$ is close to 1.0 (Phienwej 2007). The hyperbolic creep equation, which defines the time-dependent nonlinear stress strain relationship in the hyperbolic form, is expressed as:

$$\varepsilon_{ai} = \frac{1}{E_u/q_f} \frac{D_i}{1 - R_f D_i} \left( \frac{t_i}{t_1} \right)^\lambda$$

Where $\varepsilon_{ai}$ = axial strain at time $t$

$\lambda$ = creep parameter

$E_u$ = initial tangent modulus

$D_i$ = $(\sigma_1 - \sigma_3) / (\sigma_1 - \sigma_3)_f$ or stress level
(σf − σ0) or qf = maximum stress difference that is governed by the strength parameters of the material

Rf = hardening parameter that is equal to Rf = 1 - 1 / \left( E_{qf} / q_f \right) \varepsilon_f, where \varepsilon_f is the strain at failure.

The power creep law and hyperbolic creep are valid only for a stress level (ratio of stress to strength) not greater than one (Phienwej 2007). Since tunnel squeezing is often related to creep as well as yielding of the ground in the vicinity of the opening, by using the viscoelastic analysis, the initial elastic stress in the area close to the opening wall will be allowed to exceed the strength limit of the ground in the initial step. In order for the hyperbolic creep law to be applicable in such an analysis, a modification of the creep law is necessary so that calculation for stress above the strength limit in the subsequent timestep to the initial elastic step can be accommodated. The stress-strain relation of the material for stress level greater than one is extrapolated linearly from the tangent modulus at the stress levels between 0.99 and 1. Accordingly, the extended portion of the hyperbolic law for stress level greater than one is

\[ \varepsilon_{ai} = \left( \frac{1}{E_{tf}/q_f} \right) \left( D_t - 1 \right) \varepsilon_f \left( \frac{t}{t_1} \right) \lambda \]

Where \( E_{tf}/q_f = 0.01/(\varepsilon_f - \varepsilon_{0.99}) \); and \( \varepsilon_f \) and \( \varepsilon_{0.99} \) = strain at stress levels 1 and 0.99, respectively.

Phienwej et al. (2007), has formulated the tunnel creep closure solution adopting these empirical laws. For this, a simple plane strain axis-symmetric viscoelastic solution is developed to predict time-dependent tunnel wall closure and support pressure. The analysis is developed in accordance with the approach originally suggested by Aiyer (1969). It is assumed in the formulation of the plane strain axis-symmetric analysis that a circular tunnel is created instantaneously in an isotropic medium under a uniform stress field. The immediate response of the medium is linear elastic and the subsequent response is governed by a creep law expressing the relationship between stress, strain and time. The hyperbolic or power creep law is superimposed on the equations of equilibrium and compatibility of the Lame’s elastic thick walled cylinder to obtain a governing differential equation for the problem. The equation is then solved for a given boundary condition using a time marching technique. This solution was already used in order to predict the squeezing behaviour of the Stillwater Tunnel in Utah (Phienwej 1987). This study has been developed further by the author, by allowing consideration of the location of support installation from tunnel face and the variation in tunnel support types according to the convergence confinement method developed by Hoek and Brown (1980).

In this model, total strain \( \varepsilon \) is assumed to consist of elastic strain component, \( \varepsilon^e \), and creep strain component \( \varepsilon^c \), where the former is linearly related to stress and the latter is nonlinearly related to stress by the creep model used in the analysis (power law or hyperbolic law, for instance). The following assumptions are taken: creep occurs at a constant volume and creep strain component in the direction of the tunnel axis is negligible.

Consideration of various support types in the analysis can be adopted, following the approach of confinement – convergence method (Hoek and Brown 1980). The location of the support installation is also taken into account, as well as the face effect, by using the expressions of Panet (2001), following the approximation of the relationship between initial stress relief and distance from tunnel face. The yet to relieve confinement to the ground or the fictitious support pressure in plane strain tunnel is given by the Panet expression. This value is then used to calculate the portion of displacement already developed before the installation of support. The formulation of the analytical solution can be found in detail in Phienwej (1987) and Thakur (2003).
By comparing the results given for a same case by the power creep law and the hyperbolic creep law, Phienwej et al. (2007), concludes that the analysis using the hyperbolic law can more adequately simulate the yielding of ground around the tunnel with increasing elapsed time than the power law.

Phienwej (2007) tests the visco-elastic solution with monitoring data from five tunnels excavated in poor rock masses. The strength and elastic modulus of the ground are estimated from the rock mass conditions evaluated with the GSI (Hoek and Brown, 1997). For the hyperbolic parameters, \( E_u/(\sigma_1-\sigma_3) \) is considered to vary from 200 to 250 and \( R_f \) is fixed at 0.9.

Based on these assumptions the creep parameter \( \lambda \) is back-calculated by fitting the time-dependent monitoring data with the prediction by the model. By performing this back-analysis, the parameter \( \lambda \) adopted the following values:

a) \( \lambda = 0.105 \), for phyllitic quartzite of poor quality, Nepal

b) \( \lambda = 0.04 \) for schistose rocks in the Frejus Tunnel, Alps

c) \( \lambda = 0.031-0.033 \) for marly soil

d) \( \lambda = 0.115 \) for shear zones of claystone and slate, India.

e) \( \lambda = 0.3 \) for siltstones, claystones and mudstones with water infiltration. According to the author, what seems to have disturbed the “normal” value of \( \lambda \) for this kind of rocks is the fact that the squeezing behaviour was due to creep AND consolidation of pore water pressure.

As a conclusion, the author summarizes that it can be seen that for general cases of continuing tunnel closure of squeezing ground, tunnel closure can be reasonably predicted by the visco-elastic analysis using the hyperbolic creep law with the range of hyperbolic parameter of \( E_u/(\sigma_1-\sigma_3) \) around 150-200, \( R_f \) around 0.9 and the creep parameter \( \lambda \) around 0.05-0.15. The range of the parameters for some kind of soft rocks is also given by the author in the following table:
Special care must be taken when the squeezing mechanisms is a mixture of creep and consolidation due to dissipation of pore water pressure. In this case, the values of these parameters can vary considerably (Phienwej 2007).

3.5. **Instruments for the analysis of squeezing ground behavior**

3.5.1. **Laboratory and in-situ testing**

As it has been exposed, in order to analyse the behaviour of squeezing rock around the excavated tunnel, numerous models exist. These models can be rheological, based on damage mechanics or empirical. But in all cases, these models will need the determination of a number of parameters: creep parameters and/or consolidation parameters that will rule the time-dependent behaviour of the rock, and strength.
parameters (in some occasions, peak and residual values) that will constitute the plastic sliders of the models.

In order to obtain these parameters, different possibilities exist. Among them, obtaining the parameters as estimations given in literature for similar rocks, or obtaining them by correlation to geomechanical know parameters of the rock mass are a possibility. However, in most cases, laboratory testing (or in-situ, when available) is the most accurate and often only way to proceed.

a) Obtaining the value of the parameters in the literature, for similar rocks.

This option is only suitable when very rough estimations are enough, and when qualitative or semi-quantitative methods are to be applied, like those from Aydan (1993), Hoek and Marinos (2000), Singh (1992), Goal (1995), Jethwa (1984), etc. Therefore, this option would lack generalisation and would impose that the "sample" case from which the parameters are inferred shares with the studied case a large number of almost identical variables. Therefore, this option is obviously quite impractical.

b) Obtaining the value of the parameters by means of correlations to geomechanical parameters of the rock mass.

This method is already commonly used to provide values of the strength and elastic modulus of the rock mass, even when we lack the values for the intact rock. For instance, Kitagawa (1987) compared Q values and determined the material strength constants of Japanese rocks from many measurements.

Nowadays, from geomechanical parameters it is only possible to draw rough estimations about the squeezing potential by using qualitative or semi-quantitative methods of assessing the squeezing behaviour, and where no creep parameters are needed: Aydan (1993), Hoek and Marinos (2000), Singh (1992), Goal (1995), Jethwa (1984), etc.

However, it is still not available a method of "linking" the geomechanical parameters to the rheological ones, if we want to proceed to a more accurate analysis of the squeezing behaviour in tunnelling using quantitative models. The feasibility of this task, would depend very much on which is the chosen model to explain the rheological behaviour of the rock: a too complex model which needs a too large number of parameters to be rule the rheological behaviour of the rock mass would be impractical. This will be the main objective of the proposed research.

c) Obtaining the value of the parameters by means of laboratory testing of samples of intact rock.

In the case of using quantitative methods of approaching the squeezing phenomena (rheological models, models based on empirical time-dependent models, etc), testing of intact rock samples at the laboratory is the most common way to get values of the parameters that will define the constitutive relations of the rock in the model. The way and number of tests to be performed will depend on the complexity of the model to be used and on the number of parameters to be obtained. For most of the models-approaches for squeezing behaviour already exposed in this review, the laboratory testing procedure is generally undertaken by the vast majority of their authors (Bonini and Barla (2007); Sterpi and Gioda (2007); Cristescu and Hunsche (1998); Gioda and Cividini (1996); Chi-Wen Yu (1998); Cantieni & Anagnostou (2007); Kovari, Vogelhuber and Anagnostou (2000); Shao et al. (2005); etc).

However, the main drawback is that the values obtained will be valid at laboratory scale. This will obviously pose a problem if we are more interested in the behaviour of the rock mass around a tunnel when excavated, as the representative parameters must be found at the scale of the tunnel. Hence, the parameters that are obtained through laboratory tests should be calibrated or "scaled-up" in order to model reliably the behaviour of the excavation.

Typical laboratory tests for obtaining the needed parameters for modelling squeezing ground are:

- Triaxial tests and/or Uniaxial compression strength tests at controlled strain to get the strength parameters.
- Triaxial creep tests, where stress is kept constant in order to visualize the creeping effects, and under different stress levels thresholds, this latter in order to "activate" primary, secondary and tertiary creeps.

- Shear creep tests

- Relaxation tests, where deformations are imposed, and the decay of stress levels is monitored (what is directly linked to the rheological behaviour of the rock)

- In the case of highly impermeable rocks where consolidation processes are expected to take place, tests must be performed in drained and undrained conditions, always measuring the pore water pressure during the execution of the test.

- Mineralogy tests, grain size distribution, permeability, oedometer tests, clay content, swelling tests, etc

d) Obtaining the value of the parameters by means of "in situ" testing, on samples that are normally bigger than those tested in the laboratory. These tests could, if applied to big volumes of rock, get closer to the identification of the real parameters of the "rock mass", and not only of the intact rock.

The commonly used methods include:

- The borehole dilatometer test or plate loading test (Goodman, 1980 & 1989): simple, economic testing method, but the test volume is still too small to be representative of the in-situ rock mass.

- The plate loading test: commonly used for major underground openings projects, even though it is much more expensive than the borehole test. It was suggested as a standard test method for determining in situ creep characteristics or rock by ASTM (D4553-90, 1995).

- In situ triaxial creep test (Pai et al., 1991), which may be a better method for obtaining the creep parameters of a rock mass because the testing conditions are well controlled. This method is seldom used, due to its high cost. Even though, Chi-Wen-Yu (1998) tried to get the creep parameters for the rock by carrying out triaxial laboratory tests of intact rock and in addition also from in-situ triaxial creep tests of a rock block 130 cm high with a square cross section of 65 cm by 65 cm, carried out directly in the tunnel. There was a big difference between the orders of magnitude of the creep parameters of the in situ bigger tested rock cube and the laboratory values. But even though, the author confirmed that even the parameters obtained with the in-situ triaxial test still needed "calibration" or "scaling up" by means of monitoring data in order to accurately represent the observed behaviour of the rock mass around the excavation. So, the conclusion was that even with the considerable size of the in situ tested block, the scale effect was still markedly present. However, the trends obtained in the two tests (laboratory and in situ) were very similar to those displayed in the monitoring of the tunnel excavation.

Figure 17: Setup of In-situ Triaxial Creep Test
e) Obtaining or calibrating the parameters by means of “back-analysis” of the results given by monitoring and observation during the excavation of real tunnels.

As both laboratory and field tests may have the shortcomings of the scale effect, monitoring structural response to construction activity, which might be regarded as a full scale test, has been used extensively to obtain the most representative material properties through back-analysis. However, due to the number of parameters involved and the stress-dependent nature of these parameters, direct back calculation of the parameters is rather difficult when the model is too complicated (Chi-Wen-Yu, 1998). This method would consist, in fact, in assuming that in order to draw useful assumptions for tunnelling engineering, we need to take into account just “experiments” that are performed at the concerned scale, that is to say, data obtained from real tunnelling excavation.

3.5.2. Numerical calculation

As it has been already exposed in this review, a broad array of different methods exists to approach and model the squeezing behaviour on rocks. Some of them, like the empirical methods, can be approached directly without the need of numerical calculations, as they are based on qualitative appreciations of the squeezing potential according to the values of ratios, like the competency factor. The same can be said about the semi-empirical and analytical methods were the theories of elasticity and plasticity can be applied to simple cases of circular tunnels in isotropic and homogeneous rock masses subjected to hydrostatic in situ stresses, making it possible the formulation of analytical closed form solutions.

However, in order to study the time-dependent component of squeezing due to creep and/or consolidation, and by taking into account more realistic situations as far as geological conditions, construction procedures and geometry is concerned, numerical calculations must be performed.

Significant advantages are envisaged by using numerical analyses at the design stage, when very complex support/excavation sequences, including pre-support/stabilisation measures are to be adopted, in order to stabilize the tunnel during construction (Barla, 2002). Very powerful computer codes have been developed and are now available for the stress and deformation analysis of tunnels. It is therefore possible to develop reliable predictions of tunnel behaviour with these tools, provided a proper understanding of the real phenomena as observed in practice is available. With respect to closed-form solutions, anisotropic in situ stress fields can now be considered, together with multiple excavation stages, the influence of the face advance, and the important three-dimensional conditions that occur in the vicinity of the face, the consequence of linear placement delay, etc. (Barla, 2002).

In general, tunnelling engineering is perhaps one of the areas of applied soil and rock mechanics in which the numerical models for stress analysis are more frequently adopted in practice (Clough, 1990). According to Gioda and Snowboda (1999), their frequent use depends on several reasons related to the complex characteristics of the tunnelling problems: strong influence of the excavation and construction procedures, their technological details and the stress/strain distribution in the rock surrounding the opening and in its support system. This represents the main drawback for the analytical solutions, or for the approximated “standard” methods of analysis, which in most cases cannot consider this process with sufficient approximation. On the contrary, the excavation/construction steps can be simulated in a numerical analysis with a degree of accuracy which in principle is limited only by the required computational effort (Gioda and Cividini 1999).

Another important aspect of tunnelling problems that can be accounted for in a numerical analysis is their complex geometrical nature. This is not only related to the shape of the opening, but also to the presence of discontinuities in the rock mass, of non-homogeneous or non-isotropic layers. Also, the extension to 3D problems is straightforward, the mail limit being again the required computer time. Finally, these methods are able to solve problems, frequently met in tunnelling engineering, characterized by a non-homogeneous initial stress distribution and by non-linear, time-dependent of multi-phase behaviour (Gioda and Cividini 1999). This is specially the case of the analysis of squeezing behaviour, were creep and consolidation play an essential role.
3.5.2.1. **Classification of NM attending to the approach of study and behaviour taken into account**

a) Numerical models based on the so-called convergence-confinement method, used in order to study the squeezing behaviour assuming the ground as an elasto-plastic continuum. In these cases, time-dependency is not taken into account.
b) Numerical models that take into account the time-dependency of squeezing behaviour, due to creep. These models can be fitted with complex rheological constitutive equations (visco-elastic, elastic-visco-plastic, elastic-visco-plastic-damaged, etc), constitutive equations based on damage mechanics or material degradation, or empirical creep equations (power law, hyperbolic law, etc).
c) Numerical models that take into account the time-dependency of squeezing behaviour, due to consolidation. These models can use the classical consolidation theory of Terzaghi (Terzaghi and Jelinek 1954).
d) Models that can take into account the rheology of rock discontinuities (visco-elastic, visco-plastic, etc), as exposed in point 7.6.

3.5.2.2. **Classification of NM attending to the 2D or 3D approach**

**2D models**

The advancing process of a tunnel, which has an essentially 3D nature, has been studied by different authors by introducing different hypothesis in order to reduce the general and more complex 3D case to a simpler (and cheaper) 2D scheme (Gioda and Swoboda, 1999). In these cases, the effect of the advancing face during excavation (which is a 3D effect) is taken into 2D models by simulating the excavation through a stepwise reduction of the tunnel boundary tractions from the in-situ original stresses to zero (Panet, 1993; Cantieni & Anagnostou 2007).

Two simplifications exist that allow simplifying the problem from 3D into 2D:

**Axisymmetric 2D analysis**

Axisymmetric analyses allow modelling a 3-dimensional excavation which is rotationally symmetric about an axis. The input is 2-dimensional, but because of the rotational symmetry, in fact a symmetric 3-dimensional problem is being analyzed (rocscience, 2008).

The mathematical formulation of an Axisymmetric finite element analyze is actually similar to Plane Strain (and plane stress) problems. By symmetry, the two components of displacement in any plane section of the excavation through its axis of symmetry define completely the state of strain, and therefore, the state of stress. Instead of analyzing a unit out-of-plane depth, the analysis is performed on a unit radian (rocscience, 2008).

In order to model the squeezing behaviour of real tunnels, Sterpi and Gioda (2007), have carried out analyses in axisymmetric conditions. The advancing process is explicitly modelled by gradually removing, at a suitable rate, the elements representing the excavated portion of the medium. Besides the limiting assumptions of axisymmetric regime, this analyses allows for the direct evaluation of the face effects with an accuracy that depends only on the refinement of the finite element grid along the tunnel axis (Sterpi and Gioda, 2007).
However, the axi-symmetric model does not allow for a detailed simulation of the installation of the tunnel stabilisation measures and support (Barla, 2003), and has its limitation in the restriction of its use to the case of hydrostatic and constant distribution of initial stresses (Sterpi and Gioda, 2007).

**Plane strain 2D analysis**

Plane Strain assumes that the excavation is of infinite length normal to the plane section of the analysis. In a Plane Strain analysis it is calculated (rocsience, 2008):

- the major and minor in-plane principal stresses (Sigma 1 and Sigma 3),
- the out-of-plane principal stress (Sigma Z)
- in-plane displacements and strains

By definition, the out-of-plane displacement (strain) is zero in a Plane Strain analysis.

In practice, as the out-of-plane excavation dimension becomes less than approximately five times the largest cross-sectional dimension, the stress changes calculated assuming Plane Strain conditions begin to show some exaggeration because the stress flow around the “ends” of the excavation is not taken into account. This exaggeration becomes more pronounced as the out-of-plane dimension approaches the same magnitude as the in-plane dimensions (rocsience, 2008).

This 2D simplification, based on the so-called convergence-confinement method (Panet et al. 2001), is frequently adopted in tunnel design particularly to account for the elastic-plastic behaviour of rock (Sterpi and Gioda, 2007). However, some limits have been observed when the interest is focused on the effects that the excavation advancing has in the vicinity of the tunnel face (Kielbassa and Duddeck, 1991; Eberhardt, 2001). In the plane strain scheme the excavation process is modelled through the gradual reduction of ground pressure on the excavated boundary. In elastic-plastic applications, this reduction is generally applied at a constant rate. This, however, represents a first drawback in the time-dependent case. In fact, in analyses carried out by Sterpi and Gioda (2007), the effects of the tunnel advance are non-linearly related to the distance of the face from a given tunnel section. This non linearity has its direct consequence in the non-linear development of the tunnel closure with time, even for a constant advancing rate. Another drawback, according to Sterpi and Gioda (2007), concerns the choice of the constant rate of reduction of the ground pressure on the tunnel boundary. A possible way to evaluate the plane strain advancing rate is to relate it to the actual rate of excavation and to the length, along the tunnel axis, of the zone influenced by the heading effects (around 3 times the tunnel diameter) (Sterpi and Gioda, 2007).

Some authors have used this 2D analysis in order to study the squeezing behaviour in tunnelling construction, like Malan (2002).

**Comparisons between axysimmetric and plane-strain approaches in squeezing modelling**

Sterpi and Gioda (2007) compared the results given by the plane strain and axisymmetric assumptions in the modelling of squeezing behaviour by means of a elastic-visco-plastic-damaged model, concluding that it could be observed that the plane strain scheme did not provide accurate results in the vicinity of the excavation face, due to its intrinsic approximation introduced in the reduction of the rock pressure on the opening contour. On the contrary, long term closure, far away from the face, practically coincided in the two cases (plane strain and axisymmetric). This conclusion is of special relevance when the study of short term squeezing phenomena affecting TBM machines is targeted. The reason is that deformations that are likely to pose potential problems for the TBM operation are those occurring directly behind the face, where the TBM shield stands during excavation. So, this suggests using an axisymmetric scheme, instead of the more popular plane strain one, for the evaluation of the short term effects in the presence of time dependent behaviour of the rock (Sterpi and Gioda, 2007).
3D models

Contrary to the simplification of a 2D analysis, the application of three-dimensional (3D) models can achieve a far more realistic prediction of the rock mass deformation near the excavation face as well as of the rock loads acting on the TBM shield and the support system (Graziani et al., 2007). As it has been exposed, in 2D models the deformation of the tunnel and the stress distribution around it (including the extent of the plastic zone, where appropriate) are assumed to occur independently of the tunnel face. However, if the attention is posed on the excavation and support methods currently adopted, it is clear that by doing so important features of tunnel behaviour are being neglected. It appears to be a simplification of the real problem, particularly when squeezing rock conditions are to be dealt with (Barla, 2002). So, it is clear that 3D models will be far more reliable than conventional 2D simplification models. However, the price to pay is a much bigger complexity and time-consuming running time for the calculations.

Models of this kind, particularly Finite Difference Models (FDM), like FLAC3D, have been used in order to model the squeezing behaviour and its effect on the operation of TBM’s. This has been the case of the studies within the frame of the TisRock EurekaProject (2007), intended to analyse the operation of TBM’s in squeezing ground condition (Felsbau magazine November 2007).

3.5.3. Observation and Monitoring during excavation

As already described, an increased ability to carry out design analyses of tunnels by using closed-form analytical solutions and numerical methods, to a high degree of complexity if required, is available. However, it is also apparent that in squeezing rock conditions these analyses can gain in value if associated to observation and monitoring, which should become an integral part of the construction scheme. With the additional information thus obtained, the design can be adjusted accordingly (Barla, 2002). This is of special importance in the case of the study of squeezing phenomena. As it has been pointed out by numerous authors (Gioda and Cividini, 2007; Chi-Wen Yu (1998); Barla M., and Bonini (2002); Sulem and Panet (1987); Bonini et al (2007); Z.Guan et al (2007); Phienwej et al (2007) etc), the considerable scale effect from laboratory level to tunnel level is very significant. This is the reason why the parameters adopted to rule the constitutive laws that will simulate the rock mass, might be, either:

- Obtained directly from back-analysis of monitoring data or at least (directly at “tunnel-scale”)

or

-Obtained on the basis of laboratory tests and scaled up-calibrated to the tunnel level by means of monitoring back-analysis.

Barla (2002) says that observation and monitoring in the case of squeezing rock phenomena are intended to fulfil the following main objectives:

- To evaluate the stability of the tunnel and of the face
- To extrapolate observed behaviour to sections yet to be excavated
- To provide factual documentation of tunnel performance as a function of rock conditions and construction methods adopted.
- To provide valuable data for interpretation and back analysis, in order to clarify design assumptions and improve models of behaviour for rock mass and rock-structure interaction

Primary observations comprise: rock mass characteristics including rock mass classification; characters of discontinuities, faults and shear zones; water inflow; amount of overbreak; type and quantity of support measures; etc. In squeezing rock conditions these observations may become extremely valuable for a first
sight rock identification, considering that in the most difficult cases it may be impossible to obtain samples for testing (Barla, 2002). These observations might suffice if the assessment of squeezing behaviour will be done by means of qualitative approaches.

Instrumented observations comprise: instrumented tunnel sections for measuring displacements, deformations, pore pressure, etc. around the tunnel and ahead of the advancing tunnel face, including the structural components used for support. In all cases this implies accurate installation, monitoring and maintenance so that the data obtained are made available in a timely fashion (Barla, 2002).

It is noted that observation and monitoring in the special case of TBM tunnelling are characterized by a different degree of constraint depending on the excavation/support options adopted and the rock conditions encountered during face advance. In general, with conventional methods a reasonable time is available or observation, although in cases the need for early support installation may be the cause of difficulty. In TBM excavated tunnels, there is the particular difficulty that the face is not accessible and the rock mass just behind the TBM head may be difficult to observe even with open TBM-s, when the rock conditions are difficult and need early installation of support (Barla, 2002).

A summary of instrumentation specially suited for squeezing ground conditions is given as follows (Barla 2002):

a) Convergence measuring points, between which accurate measurements are made with tape extensometers (accuracy of measurements: \( \pm 0.5 \) mm), and/or geodetic measurements from a remote theodolite station (accuracy of measurements: \( \pm 1.0 \) mm) without disruption to normal operations (relative movement is measured with the tape extensometers while absolute movement with geodetic measurements)

b) Extensometers measurements for determining relative displacements between points in a borehole in the direction of the borehole axis (accuracy of measurements: \( \pm 0.5 \) mm); these measurements are generally performed by multiple point borehole extensometers installed in boreholes oriented in a desired direction around the tunnel.

c) Sliding micrometer measurements for determining relative displacements between points in a borehole drilled in a direction parallel to the tunnel axis, starting from the face (accuracy of measurements: \( \pm 0.02 \) mm/m). These measurements are carried out by means of a portable measuring probe and are possible concurrently with face advance

d) Strain measurements in the support elements (shotcrete and concrete linings, steel ribs, dowels or anchors, etc.) by means of strain gauges attached to steel members of imbedded within shotcrete/concrete (accuracy of measurements: \( \pm 0.1 \) mm); also used are load cells, concrete stress cells, hydraulic flat jacks...

e) Pore water pressure measurements, by means of piezometer cells installed in boreholes; pneumatic, vibrating wire and electrical resistance piezometers can be used; whichever is adopted, it is important to know the conditions of installation and the ground in which the piezometers cell has been placed.
As the ISRM (1994) suggests, for squeezing ground conditions it is always advisable, whenever possible, to use observation and monitoring of a test tunnel (access tunnel, adit tunnel, pilot tunnel, etc). The test tunnel is to be excavated well in advance of the actual tunnels in order to obtain the following information (Barla, 2002):

- Identification and quantification of the squeezing behaviour, mainly the ratio of rock mass strength to in situ stress as an indication of the stability conditions of the rock mass surrounding the advancing tunnel

- In situ observation and monitoring of tunnel convergences and deformations around the tunnel, including the tunnel face and support/pre-support measures
Comparison of predicted and observed performance in order to improve the computational approach used and to obtain the rock mass parameters for final design.

- Analyse the tunnel response during face advance, by comparing different support measures and excavation sequences, in the attempt to experience passive, active and intermediate design concepts.

Special care must be devoted, during performance monitoring of the test tunnel, to the evaluation of the time dependent behaviour of the rock mass. This is a rather difficult task, especially if one is to determine the rock mass creep parameters to be used in the constitutive laws which will be applied for design purposes. Successful examples of application of this type have been reported by Sulem et al (1987). As described by Sulem (1994), one need to be careful to distinguish amongst the monitoring data the effect of the face advance and the time-dependent behaviour of the rock mass.

3.5.4. Relation between monitoring – numerical modelling – laboratory parameters

As it has been exposed in this review, different models and approaches exist for assessing the squeezing behaviour. Amongst the most complex ones are the quantitative time-dependent approaches. These models rely on the knowledge of strength, creep and in some occasions also on consolidation parameters (low permeability ground). Once these parameters are reliably selected for the rock mass, they can be integrated into the numerical model where a certain constitutive equation has been taken to represent the behaviour of the rock mass. So, it is obvious that the accuracy in the determination of these parameters is paramount to understand and correctly model the behaviour of the squeezing ground.

Different methods have been exposed in order to directly obtain these parameters. Among them, mechanical tests are of higher reliability. These tests will be carried out in situ or at laboratory level. In situ tests affecting large volumes of rock mass will always be more representative of the real mechanical parameters of the rock mass. However, large scale tests are extremely expensive and impractical in most of the cases. Thus, laboratory tests, and eventually some in-situ tests affecting sizeable but workable volumes of rock are most of the time carried out (Chi-Wen Yu, 1998).

The scale effect problem

As it has been already exposed in this document, the main drawback of laboratory tests is the fact that the parameters obtained correspond to the intact rock, and not to the rock mass. This difference has been found by several authors to be reflected in their order of magnitude, but not in the trend, that are essentially identical, at least in the case of relatively homogeneous rock masses (Chi-Wen Yu 1998).

In fact, some authors, like Chi-Wen Yu (1998) have found that results obtained by triaxial tests on rocks with a size of up to 65 x 65 x 130 cm are still subjected to the scale effect.

As another example, Barla M., and Bonini (2002), in the first attempt to simulate the behaviour of a tunnel through tectonised clay shales, verified that by integrating the parameters obtained in laboratory tests into a numerical model unrealistically high values were obtained. Laboratory data needed to be scaled-up significantly in order to obtain a better correlation with the monitored behaviour.

According to Sulem and Panet (1987) the determination of the parameters normally needed for conventional rheological models is very delicate, since shape and scale effects may appear in phenomena like viscosity or consolidation of a rock mass and it is very difficult to extrapolate laboratory results to field situations. According to these authors, ground characteristics and the time-dependent parameters can only be properly determined by back analyzing the long-term closure observations.
So, in order to get the real parameters for the rock mass level and solve this “scale” problem, two approaches can be adopted:

a) Obtaining or calibrating the parameters by means of back-analysis, based on the monitoring results obtained at the construction site

In general, back-analysis techniques have been introduced in geotechnical engineering for determining the “average” mechanical parameters of soil/rock masses on the basis of field measurements performed during excavation or construction works, or for the evaluation of the rock pressure acting on linings or support structures on the basis of deformation measurements (Kovari and Fritz 1997). These procedures can also be seen as a practical tool to be adopted in the context of the observational method (Bjerrum 1960) and of its application to tunnelling engineering according to Rabcewicz (1972).

In the particular case of squeezing ground behaviour, these techniques have an especial relevance. As it has been mentioned by different authors, back-analysis permit to obtain the real value, at the scale of the tunnel, of the different parameters that define the constitutive equations of the model adopted to represent the behaviour of the rock, or might at least be used to calibrate or scale up parameter values obtained at the laboratory level.

In this way, Chi-Wen Yu (1998) affirms that for engineering problems related to squeezing, it is not practical to assess the parameters based merely on testing. From his experience in simulating the creep behaviour of some tunnels through soft sedimentary rock and much sheared hard rock, which can be considered as relatively homogeneous, calibrating the creep parameters using monitored tunnel deformation and guided by the laboratory results may be the most practical approach in obtaining the parameters for simulation.

Phienwej et al (2007), use monitoring results of different tunnels in order to obtain, by means of backanalysis, the values of the parameter λ that rules the creep behaviour in its empirical-based hyperbolic law.

b) Using “scaling-rules” that allow the conversion of parameters at “laboratory level” to those at “rock-mass level”

This is a possible approach that has already been frequently adopted in tunnel engineering for assessing the stiffness and shear strength characteristics of the rock mass consists in relating them to those of intact samples tested in laboratory through a measure of the conditions of the rock mass in situ. The engineering practice has proved that these scaling rules are effective, although some uncertainty still affects the empirical relationships between laboratory and in situ parameters. This procedure could also be extended to the evaluation of the mechanical parameters of rheological models, according to some authors (Sterpi and Gioda 2007). However, very few experimental data have been reported in the literature, and there are still no plausible scaling relationships that might be applied in a generalized way.
4. Model Implementation

4.1. Objectives for the Model Analysis

As exposed in the “State of the Art”, the goal of this project is to give a ready to use tool that can allow a parametric study of the different parameters involved with squeezing ground during the construction of a tunnel driven by a shielded TBM, in order to quantify and qualify their influence for both cases of Normal Operation and Exception.

Basically, for the development of such tool the work has to be divided in two steps: the computer design of the physical process, relative to the excavation of the tunnel and the boundary conditions, and the implementation of the quantitative model used to calculate creep: the hyperbolic creep model. As it will be seen, both actions are closely linked, and represent a conditioning factor to each other.

For this study a finite difference method based program will be used: FLAC 5.00 S.P created by ITASCA. The numerical formulation for this calculation methodology will not be exposed on this project, however, it can be found at the FLAC manual BACKGROUND — THE EXPLICIT FINITE DIFFERENCE METHOD

For creating a conceptual picture of the physical system we will assume the following major hypothesis which will be the same for all the possible scenarios:

1. Among the range of different mechanisms that might explain squeezing phenomena (see state of the art section), the study will be focused on total shear time dependent failure, whose incidence is normally greater on TBM excavation (i.e. where TBM jamming is likely to happen), and whose characteristics make possible a generalized study.

2. As mentioned, consolidation might be also one of the mechanisms that explain rheological behavior in rocks of low permeability and with water content, being normally coupled to the creep phenomena of the rock. However, as opposed to rock creep (which is not well understood), we already dispose in soil mechanics of successful techniques to model the consolidation phenomenon. This study will not take into account the effect of consolidation. Furthermore, no water pore pressure will be considered.

3. A hydrostatic stress field will be supposed as in situ natural stresses before the tunnel is excavated. The reasons for this simplification are several:

   a) Squeezing problems are normally displayed in deep tunnels and in relatively soft rock masses, where normally the in situ stresses are closer to a hydrostatic stress field.

   b) Sadly, the measurement of the in situ stresses is not accomplished in most of the tunnels. That is why this parameter is normally estimated by assuming that the vertical stress is one of the principal stresses and it is due exclusively to the weight of the rock overburden. So, assuming a hydrostatic field of stresses is very often the only possibility of guessing the in-situ stresses for normal tunneling projects.

   c) This assumption simplifies very much the need of data, since it is possible to define the whole stress field by means of just one parameter: the tunnel overburden.

   d) Good results have been obtained by different authors by assuming this simplification, when trying to match monitoring results with data obtained from models built upon the assumption of a hydrostatic field of stresses.

4. An isotropic and homogeneous rock mass will be supposed. The reasons for this simplification are several:
a) The squeezing mechanism that will be taken into account is total shear time dependent failure that occurs in overstressed soft rock masses. The studied rock masses will be soft sedimentary rocks or very sheared and broken harder metamorphic rocks. In this case, the assumption of an isotropic and homogeneous rock mass is fair enough.

b) The only way of providing a general and more universal method of assessing the squeezing ground impact on TBM excavation is by dealing with mechanisms that are not structurally-controlled and too dependent on the particularities of the geology around the tunnel. This recommends the assumption of an isotropic and homogeneous rock mass, that is perfectly plausible for the studied case of soft rock masses where no time-dependent failure due to sliding along macro-fractures or buckling of rock layers are considered.

5. The research will be focused on the use of shielded TBMs (single shield or double shield). The reason is that squeezing behavior is normally displayed by poor quality rocks, where the use of shielded TBMs (and not open) will always be recommended by good engineering judgment. For this assumption, we will suppose a perfectly conical TBM.

The main innovation of this project will be the implementation with FLAC of the hyperbolic creep model. The reasons why it has been decided to implement this methodology are listed followingly:

a) As opposed to other laws (like the power law), more applicable to materials whose time dependent behaviour is due mainly to viscous behaviour of the crystal structure (potash, salt), hyperbolic law is mostly used for clayey soils and soft rock masses (sedimentary rocks or very sheared and broken metamorphic rocks). This is the most generic case of geology through which in the practice TBM experience problems due to squeezing behaviour, due to the main mechanism of squeezing of total shear time dependent failure.

b) The hyperbolic law has an advantage to other empirical creep models in that all of its parameters have physical meaning and are related to the mechanical properties of the materials (i.e. modulus and strength). This makes it more attractive than others (Phienwej 1987; Lin and Wang 1998). In fact, from the three parameters that define the rheological behaviour of the rock, two are directly linked to geomechanical parameters of the rock mass, leaving just one parameter that might be taken as a “pure” creep parameter. This will be of great relevance when trying to correlate the rheological behaviour of the rock mass to its geomechanical characteristics, since we will have just one main parameter to correlate.

c) For the eventual formulation of a parametrized equation for predicting squeezing behaviour, it is important to reduce the amount of variables characterizing a rockmass. In this sense, the hyperbolic model can be defined by three variables which

d) By comparing the results given for a same case by the power creep law and the hyperbolic creep law, Phienwej et al. (2007), concludes that the analysis using the hyperbolic law can more adequately simulate the yielding of ground around the tunnel with increasing elapsed time than the power law.

e) This time-dependent law is easy enough to be “fitted-out” into a numerical analysis software like Flac 2D, by means of a simple Fish function (programming language used by this software)
4.2. Creating a Conceptual Picture of the Physical System

4.2.1. Axisymmetric 2D Model

For the proper design of the tunneling driven by a shielded TBM, it is necessary to identify the parameters that may play an important role on the results outcome. A particularity of FLAC is that, whenever building up a grid, the user must associate a constitutive model to it. More than one constitutive model may be used. In FLAC these models are: Mechanical models, creep models, thermal models or User-defined FISH constitutive model. These parameters will dictate the gross characteristics of the numerical model, such as the design of the model geometry, the types of material models, the boundary conditions, and the initial equilibrium state for the analysis.

**Geometrical parameters**

1. Diameter of the tunnel \( D \): As it defines the geometry of the excavated tunnel
2. Overburden of the tunnel \( H \): As the in-situ hydrostatic stress field will be directly linked to the overburden.
3. Grid dimensions (X-Y). They will be proportional to the TBM’s length and radius.

**Initial conditions**

4. In situ stresses: It will be calculated directly from the overburden \((\gamma H)\), as a hydrostatic stress field will be supposed

**Ground parameters**

5. Geomechanical parameters of the rock mass. These parameters are subjected to the chosen constitutive model. As first attempt to reproduce a realistic TBM excavation with FLAC several calculations will be done with FLAC constitutive models in order to guarantee that the designed excavation model is valid. Each constitutive model implies the definition of variables that may be intrinsic to each model. For the testing of the model we will use a both a non time dependent model (Mohr Coulomb) and a time dependent model (Burger). Each model can be defined by the following variables:

**Mohr-Coulomb**

a) Unconfined Compressive Strength \((q_u)\)
b) Angle of friction \(\phi\) and cohesion \(C\). \(C\) is extrapolated from the unconfined compressive strength as: \(\text{coes} = q_u/(2.*\text{cos(\phi*pi/180.))/(1.-\text{sin(\phi*pi/180.))})\)

\[ C = \frac{q_u}{2 \cos \phi} (1 - \sin \phi) \]

c) Elastic modulus \((E)\) and Poisson’s coefficient \((\nu)\). From these two variables, the bulk modulus \((K)\) and the shear modulus \((G)\) can be obtained as:

\[ K = \frac{E}{3(1 - 2\nu)} \]
\[ G = \frac{E}{2(1 + \nu)} \]
**Burger creep model**

a) This rheological model is described by a Maxwell model connected in series with a Kelvin model. Both models are defined by an elastic and a viscous behavior. The parameters defining this model are:

- \( m_k \) Bulk modulus (elastic volumetric response — no creep)
- \( m_{k1} \) Kelvin shear modulus
- \( m_{vis1} \) Kelvin viscosity
- \( m_k2 \) Maxwell shear modulus
- \( m_{vis2} \) Maxwell viscosity

**TBM parameters**

6. Length of the shield \( L \) and diameter \( D \).
7. Overcoring of the cutter-head: It will be modeled by assuming a free gap between the initial excavation profile close to the head and the position of the rigid element that represents the TBM shield. In fact, its effect is the same as assuming that the rigid support (TBM shield) will be “installed” after the ground deformations have stretched out along the overcoring space.
8. The shield will be modeled as a perfectly rigid support in an initial phase in order to reduce the number of variables. On further calculations, it will be assumed an elastic behavior of the shield. Considering the shield as a rigid element will cause an increase of the stresses around it, which in terms of TBM tunneling security will lead to safer conditions.
9. In order to reduce complications on the model design, it is assumed that the TBM is perfectly cylindrical. Usually, TBMs are given a slight conical shape in order to reduce friction with the rockmass.
10. Skin friction: It will determine the friction shear stresses that will be acting on the shield once the ground gets in contact with the TBM shield. Like the previous parameter, this variable won’t be taken into consideration in a first phase of the calculation but, in future calculations it will be modeled under two situations:
   - Static friction: For the Case “Exception”, as after a standstill, the friction forces are bigger (the machine starts moving) than when it is continuously moving.
   - Dynamic friction: For the Case “Normal Operation”, which is valid for the situation where the TBM is continuously advancing. It is a lower value than the static one.

**Construction parameters**

11. Advancement rate of the TBM (case Normal Operation): The advancing process will be explicitly modeled by gradually removing, at a rate given by the advancement parameter, the elements representing the excavated portion of the medium.
12. In FLAC 2D axisymmetry models it is not possible to use FLAC predesigned structure elements like rockbolts, beams... for this reason, if it is required to model such a structural element, the user may consider to use a simplification of it by defining it with the grid function in FLAC. In this case, it will be necessary to define the parameters mentioned.

Having considered the assumptions and design parameters explained above, the proposed 2D axisymmetric model capable of reproducing the advancement of a TBM through a certain rock mass or soil taking into account the listed parameters is the following:
This is a plane section of the tunnel seen from the z-axis (perpendicular to the drawing plane). When FLAC calculates the solution in 3D it applies a rotational axisymmetry along the Y axis. The FLAC grid is configured for such an analysis by specifying the command CONFIG axisymmetry at the beginning of the data file. For this configuration, a cylindrical coordinate system is invoked: x = 0 is the axis of symmetry; the positive x-direction corresponds to the radial coordinate; and the y-direction to the axial coordinate. The out-of-plane coordinate (the z-direction) is the circumferential coordinate. Only the positive x-direction may be used to create an axisymmetric grid; a grid may not be created in the negative x-direction. Any grid points that have x = 0 are automatically fixed in the x-direction. Fixing the top and bottom boundaries on the y direction is necessary to simulate the confinement of the rock mass. Finally, the model above pretends to simulate the TBM and the lining as rigid structures: the TBM’s shield and lining representative grid points are fixed on the x direction while the cutter head is not allowed to have movements on the Y direction.

The main goal of this model is to be able to study how the TBM excavation process disturbs the stress field in a given area surrounding the excavation and how this reacts to it. In order to obtain valid results the model has to be numerically stable by means of proportional geometric design (grid) and, when time dependency appears, the time increments responsible for strain have to be calibrated on a proper way. In this sense, FLAC provides several tools and tips to allow the user choose the right option. This will be analyzed on future sections of this study. Concerning the geometrical calibration FLAC’s manuals restrict the model in terms of:

1. Size and form of the grid elements: grid elements should have forms close to parallelepipeds avoiding small angles. Exceptionally, when an element is connected to the grid’s boundary, this element may convert into a triangle or contain a curved side. The ratio between sides should be as close to the unity for assuring accuracy in the results, however under certain situations it is allowed to reach a 5:1 ratio.
2. The location of grid’s boundaries. Knowing the correct size of the grid may not be obvious. According to FLAC’s manuals, for the excavation of a tunnel the user is advised to use a ratio of 1:10 times the radius of the tunnel. However, depending on the type of research done, this ratio can be either increased or decreased. A trial-error methodology may be adequate for defining the problem’s boundaries.

3. Combining grid boundaries and grid elements. For more accuracy in the results, it is advisable to increase the density of grid elements around the area of study interest and in regions of high stress or strain gradients and make them coarser as getting closer to the boundaries.

When modeling the physical process of TBM advancement there is only one section that will be studied. The TBM advances from the bottom of the model (see figure) following the Y axis until it reaches the top. The section corresponding to the half of the TBM’s path will be the area which will provide best results for two reasons:

1. It will be possible to model the effect of the advancement of the TBM, both the approximation to the section and the move off, thanks to the boundary design.

2. The density of grid elements and their shape has been designed in order to give this area the best numerical accuracy as possible.

A closer look of figure on this section is shown:

![Figure 21: Detailed view of the TBM's cutterhead, shield, overcoring and rockmass](image)

The following picture represents the same area as the former but it allows to better understand the design. The groups `rock_mass` and `rock_mass2` represent a same geological body, but, due to programming drawbacks, they had to be named distinctly. Furthermore, it is possible to appreciate the TBM’s cutter head and shield and the overcoring proposed. In this case, the cutter head is in direct contact with the rock.
When idealizing a physical system for numerical analysis, it is more efficient to construct and run simple test models first. Simple models should be created at the earliest possible stage in a project to generate both data and understanding. The results can provide further insight into the conceptual picture of the system. In this sense there are several considerations to be taken into account in order to simplify the main design:

- Idealized models pursue the simplification of the problem both in complexity and on time consuming. In order to minimize the time spend on the calculation process, the number of elements of the grid will be reduced (lower mesh density).

- Considering the TBM’s design, initially, we will assume an open shield TBM and no lining. This will allow a better analysis of the stress-strain field obtained around the section. A second step on this direction will be to model as a single body the TBM’s shield and the lining.

- Independently from the rock’s physical behavior, the TBM will be modeled as an elastic body. This model reduces significantly the number of variables needed for defining a constitutive model in comparison to other.

- When modeling the shield and the lining special attention must be given to the modeling of the overcoring. The overcoring represents an empty gap between the rock mass and the TBM. When big displacements occur, both bodies enter in contact in a sudden way. This physical action can be difficult to simulate by FLAC since numerical problems may arise. A possible way to simulate the overcoring requires the use of the “Interface” FLAC command which, on the same time, requires extra parameters to be defined.
Before developing and implementing the hyperbolic creep model, the previous designs will be tested with a non-time dependent constitutive model (Mohr-Coulomb) and a time dependent one (Burger’s model).

### 4.2.3. Testing the Idealized models

1. **2D axisymmetric; Open TBM advancement.**

The following idealized model represents a TBM when passing by the section of study. As seen on the following picture, the number of grid elements has been considerably reduced allowing an increase of the testing speed (though reducing results accuracy). No TBM shield, cutter-head and no lining are modeled. This means that free movements are allowed around the walls and excavation face.

![Idealized 2D axisymmetric model](image)

Two qualitative analyses will be held in order to justify whether the model is well defined or not: a first one will center on the stress field and the second on the displacements of the wall. When an initial hydrostatic stress field dominates the stress behavior of a solid body around a tunnel, it is possible to set the elastic and plastic boundaries depending on the radius of the tunnel ($r$), the magnitude of the initial pressure ($P_0$), the supporting pressure ($P_s$) and the characteristics of the rock mass ($C, \varphi$).
Comparing the elasto-plasticity theory with both the shear and radial stress fields for a Mohr-Coulomb and a Burger creep model the results show that the model proposed is physically possible:

The Mohr-Coulomb stress fields can easily separate both the states of elasticity and plasticity, however, the Burger model is a visco-elastic model, reason why no plasticity is found. In terms of stress behavior, the results can be considered correct. The different magnitude in the results can be explained because the Burger model needs extra variables to be defined: the Kelvin and Maxwell viscosities and a time dependent parameter (TBM penetration rate: velocity). The chosen viscosities are probably not the best option for fitting the results obtained by the Mohr-Coulomb model.
Analyzing the effect of the TBM advance (considering penetration ratio when possible) regarding wall displacement, the results obtained also lead us to confirm the functionality of the model.

These results represent the total displacement suffered by the wall at the cross section 0,00 due to the position of the TBM’s excavation front. The total convergence of the wall should deduct the displacement recorded on the moment when the TBM excavates the 0,00 cross section, simulating the physical excavation.

The speed can play an important role when soft rocks are drilled. For higher penetration ratio less time the walls remain unsupported. In terms of time dependent strains, this means less time to develop important displacements. In our case, though no lining is taken into account, increasing the speed implies a reduction of the problems creep time. This means that the wall experiments displacements for less time. The results show a difference of 1 mm less displacement when being excavated at higher speeds, a result which, although being small, confirms the functionality of the idealized model.

2. 2D axisymmetric; shielded TBM advancement and lining

In this case several improvements have been done in comparison to the previous design. In first case, an elastic body has been modeled as the TBM shield and lining (no physical difference is considered). In order to guarantee numerical accuracy, the number of grid elements has been increased (both on the X and Y directions). Finally, for simplicity, overcoring is still not taken into consideration. A figure representing the two bodies (rock mass and TBM’s shield) is shown below. The constitutive model defining the bodies is also shown.
The following picture represents the effect of the TBM on the radial displacement of the rock mass in contact with the shield. The rock mass has an elasto-plastic behavior (Mohr-Coulomb):

The behavior is equivalent to the one shown in figure 26 and the supporting action of the TBM’s shield is clearly appreciated for two reasons: first, the total radial displacement is much lower (around 4 cm) and, secondly, the highest displacements occur moments before the TBM reaches the zone of measurements.
(section 0.00). Actually, it can be seen that the radius of influence of the TBM advancement is around 2 times the radius of the tunnel.

Concerning the stress fields, the following graphic show the stress field (radial and shear) generated by the interaction between a rock mass with an elasto-plastic behavior and a lining or TBM shield with an elastic behavior:

![Stress fields at rockmass and lining](image)

Figure 29: stress fields produced behind the tunnel wall and on the supporting body.

When applying a time dependent constitutive model, in this case, in order to keep the coherence with the previous tests, the burger model, the outcome results do not show any numerical instabilities and the actual behavior of the results seem correct. As an example the radial displacement due to TBM advancement is shown at figure 30. The viscous behavior of the rock mass can be noted since the displacements take place during a longer period.

![Figure 30: Radial displacement caused by TBM advancement restricted by a rigid shield and lining when a visco-elastic rheological model defines the rock body.](image)
4.3. Hyperbolic formulation

The particularity of the hyperbolic law proposed by Phienwej is that the strains are directly related to stress as a function of time. However, because the hyperbolic creep model is an empirical solution, the actual constitutive model able to describe this phenomenon is unknown. In order to define a constitutive model, it was decided to use Maxwell’s constitutive model. The reasons for using Maxwell’s model are basically two:

1. As a constitutive model, the Maxwell model is defined by an elastic component (spring) and a viscous component (Dashpot) which are connected in series. According to Phienwej 2007: “The immediate response of the medium is elastic and the subsequent response is governed by a creep law”. This statement leads to assume that the methodology used by Phienwej is also conceived as a combination in series of an elastic element and a viscous element.

2. For implementing the hyperbolic creep model with FLAC it is necessary to program it as a “User Defined Constitutive model” (UDM). The task of the constitutive models is to supply a new set of stress components, given strain increments and the old set of stress components. This requires programming in FISH language skills. The Maxwell model is a “built-in model” which will reduce complications for successfully compiling it.

Hereafter, Phienwej’s formulation of the hyperbolic creep law is used and reinterpreted, if necessary, in order to build a constitutive model that will relate strain and stress as follows:

\[
\dot{\mathbf{u}}_m = \frac{\mathbf{F}}{k} + \frac{\mathbf{F}}{\eta}
\]

According to Phienwej, 2007, two assumptions are done: creep occurs at a constant volume and the creep strain component on the direction of the tunnel axis is negligible. This second assumption is important when calculating the maximum stress difference \(q_f\): the deviatoric component of \(\sigma_2\) is zero \((\sigma_{22}^d = 0)\) and \(\bar{\sigma} = \frac{\sigma_1 + \sigma_3}{2} = \sigma_2\).

As already mentioned at the state of the art, Phienwej 2007 defines the stress level as

\[
D_i = \frac{\sigma_1 - \sigma_3}{(\sigma_1 - \sigma_3)_f}
\]

In a triaxial test the deviatoric stress level corresponds to the stress difference \(\sigma_1 - \sigma_3\)

We will assume that the deviatoric stress magnitude defined as

\[
\bar{\sigma} = \sqrt{\frac{3}{2}} \sqrt{\sum (\sigma_{ij}^d)^2}
\]

Can be used for quantify the deviatoric stress magnitude in a general case, and in particular to the case defined by Phienwej.
The deviatoric stresses are obtained as follows:

\[
\begin{align*}
\sigma_{11}^d &= \sigma_{11} - \bar{\sigma} \\
\sigma_{22}^d &= \sigma_{22} - \bar{\sigma} = 0 \\
\sigma_{33}^d &= \sigma_{33} - \bar{\sigma}
\end{align*}
\]

These results are then substituted at eq [2]. Special attention must be set if working on plane strain conditions since shear stresses are present. The resulting equation is

\[
\bar{\sigma} = \sqrt{\frac{3}{2}} \sqrt{\sigma_{11}^d + \sigma_{22}^d + \sigma_{33}^d + 2 \sigma_{12}^d}
\]

If the calculated deviatoric stresses are in principal directions, the maximum stress difference at failure can be calculated as

\[
q_f = \frac{\sqrt{3}}{2} (\sigma_1 - \sigma_3)_f
\]

From the Mohr-Coulomb circle theory the stress difference magnitude at failure (maximum stress difference) is obtained by

\[
(\sigma_1 - \sigma_3)_f = 2 \sin \varphi' (\sigma_{iso} + CA)
\]

Where

\[
CA = \frac{c'}{\tan \varphi'}
\]

Finally, substituting eq. [5] in [4] we obtain the expression for calculating \(q_f\):

\[
q_f = \sqrt{3} \sin \varphi' (\sigma_{iso} + CA)
\]

In order to define the stress level \(D_i\), a different approach has been considered than what Phienwej did. Instead of using the stress difference proposed by him, we will use equation [2] which gives the deviatoric stress magnitude. Like this, \(D_i\) is defined as

\[
D_i = \frac{\bar{\sigma}}{q_f}
\]

The creep strain general definition is:

\[
\varepsilon_{cr} = \frac{\sigma}{\eta}
\]

Or

\[
\frac{\Delta \varepsilon_{ij}^d}{\Delta t} = \frac{\sigma_{ij}^d}{\eta} \Rightarrow \varepsilon_{ij}^d = \frac{\sigma_{ij}^d}{\eta}
\]
The viscous part of the deviatoric strain-rate is coaxial with the deviatoric stress tensor (normalized by its magnitude, $\bar{\sigma}$) and is given by
\[
\dot{\varepsilon}_{ij}^d = \frac{3}{2} \dot{\varepsilon}_{cr} \frac{\sigma_{ij}^d}{\bar{\sigma}}
\]

As explained at the state of the art, according to the stress level $D_i$, the hyperbolic creep law is defined two equations:

For $D_i < 1$
\[
\varepsilon_{ai} = \frac{1}{E_u/q_f} \frac{D_i}{1 - R_f D_i} \left( \frac{t_i}{t_1} \right)^\lambda
\]

For $D_i \geq 1$
\[
\varepsilon_{ai} = \left( \frac{1}{E_i f} \frac{(D_i - 1) + \varepsilon_f}{q_f} \left( \frac{t_i}{t_1} \right) \right)^\lambda
\]

Differentiating both equations respectively:
\[
\dot{\varepsilon}_{cr} = \frac{\varepsilon_u}{E_u/q_f - R_f \bar{\sigma}} \left( \frac{\lambda}{t_1} \right) \left( \frac{t_i}{t_1} \right)^{\lambda-1}
\]

Where $\varepsilon_u = q_f / E_u$

\[
\dot{\varepsilon}_{cr} = \varepsilon_u \frac{D_i}{1 - R_f} \left( \frac{\lambda}{t_1} \right) \left( \frac{t_i}{t_1} \right)^{\lambda-1}
\]

For the latter equation ($D_i \geq 1$), it has been assumed that $E_i f / q_f = 0.01 / (\varepsilon_f - \varepsilon_{0.99})$ can be re-written as $E_i f / q_f = 0.01 / \varepsilon_f - 0.99 \varepsilon_f = 1 / \varepsilon_f$; and then combined with
\[
R_f = 1 - \frac{1}{\left( \frac{E_u}{q_f} \right) \varepsilon_f} \Rightarrow R_f = 1 - \frac{\varepsilon_u}{\varepsilon_f} \Rightarrow \varepsilon_f = \frac{\varepsilon_u}{1 - R_f}
\]

Equaling [8] and [9] and substituting by [12]
\[
\frac{1}{\eta} = \frac{3}{2} \frac{\varepsilon_{cr}}{\bar{\sigma}}
\]
\[ \eta = \frac{2}{3} \varepsilon_u \left( q_f - R_f \bar{\sigma} \right) \frac{t_1}{A} \left( \frac{t_1}{t_i} \right)^{\lambda - 1} \]

If \( D \geq 1 \)

\[ \eta = \frac{2}{3} q_f \frac{t_1}{\varepsilon_u} \left( 1 - R_f \right) \left( \frac{t_1}{t_i} \right)^{\lambda - 1} \]

15

16

17

The viscous parameter \( \eta \) is not a material property but a time dependent parameter.

Finally, in order to know the exact relationships between the different components defining formula 1, the new constitutive model will assume the same mathematical approach as if dealing with a Maxwell substance. Denoting the new value of force by \( F' \), and the old value by \( \bar{F} \), over a timestep of \( \Delta t \), we can rewrite Eq. [1] as

\[ \frac{\Delta u}{\Delta t} = \frac{F' - F^o}{k \Delta t} + \frac{F' + F^o}{2\eta} \]

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This is a central difference equation, since the velocity is calculated at the midpoint between the instance when \( F' \) and \( \bar{F} \), are defined. Solving for \( F' \)

\[ F' = (F^o C_1 + k \Delta u) C_2 \]

\[ C_1 = 1 - \frac{k \Delta t}{2\eta} \]

\[ C_2 = \frac{1}{1 + \frac{k \Delta t}{2\eta}} \]

19

An equation identical to Eq. [18] can be written for the relation between deviatoric stresses and strain increments:

\[ \sigma^d_{ij} = \left( \sigma^d_{ij} C_1 + 2G \Delta \varepsilon^d_{ij} \right) C_2 \]

\[ \Delta \varepsilon^d_{ij} = \Delta \varepsilon_{ij} - \frac{1}{3} \Delta \varepsilon_{ij} \delta_{ij} \]

\[ \sigma^d_{ij} = \sigma_{ij}^o - \frac{1}{3} \sigma_{ij}^o \delta_{ij} \]

\[ C_1 = 1 - \frac{k \Delta t}{2\eta} \]

\[ C_2 = \frac{1}{1 + \frac{k \Delta t}{2\eta}} \]

20

21
Here, $\Delta \epsilon_{ij}$ are the components of the “input” strain-increment tensor, $\sigma^\circ_{ij}$ are the components of the previous stress tensor, and $G$ is the shear modulus. For the volumetric component of stress and strain, we assume that there are no viscous effects — elastic relations apply, as follows:

$$\sigma^{iso} = \frac{1}{3} \sigma^\circ_{kk} + K \Delta \epsilon_{kk}$$  \hspace{1cm} (22)

Where $K$ is the bulk modulus. The final stress tensor is given by the sum of the deviatoric and isotropic parts

$$\sigma_{ij} = \sigma^d_{ij} + \sigma^{iso} \delta_{ij}$$  \hspace{1cm} (23)

The material properties required for this model are shear and bulk moduli (for the elastic behavior) and the viscosity, which is calculated by Eqs [15] or [16]. Under an applied shear stress, the material flows continuously, but it behaves elastically under an applied isotropic stress.

### 4.3.1. Hyperbolic creep law testing

Focusing on the 3D axisymmetric model we first implemented, we realize that, if we want to simulate the advancing of the TBM through a given soil, a speed is needed and, consequently, a time must be given. When working with the creep option, FLAC sets a global solving time $t_i$ to the whole grid. In a 2D model this represents no problem, but in a simulation of the tunnel advance some particularities have to be taken into account:

- In the initial stages of the solving process, when the tunnel drive into the first part of the model, this is, when $t_i$ is very low, the value of $\eta$ becomes minimum. The effect of low values of viscosity produces high strains and displacements of the material.

- In the final stages, when the tunnel drive in the second part of the model $t_i$ and $\eta$ and higher producing low displacements.

The hyperbolic model considers the viscosity $\eta$ as a function depending on the total creep time $t_i$, this means that the creep behavior is not constant along the longitudinal direction of the tunnel. In this sense, when the TBM would reach the final part of the grid, this is, when creep time would have high values, the viscosity at that moment would have high values and, consequently, the soil at that area would experiment much lower strains than those in the beginning.

A possibility to overcome said limitation of the constitutive model is to substitute the value of $t_i$ in equations [15] and [16] by $t_i - t_0$, being $t_0$ a time parameter variable along the longitudinal direction. The problem is how to define $t_0$.

$t_0$ could be defined in a certain cross section equals to $t_i$ when the tunnel face reaches the considered cross section. This choice implies that the creep time becomes negative ahead of the tunnel face. To avoid absurd creep parameters a lower boundary for $t_i - t_0$ must thus be added, for example 1 h, allowing excluding any numerical problem.

The remaining physical problem is, that the ground ahead the face that is already influenced by the approaching tunnel excavation, i.e. for a distance of 1, 2 or even 3 times the diameter, depending on the ground strength, remains for a certain period with a fixed creep time, equals to the lower boundary, producing high deformations.
In these cases the extrusion (longitudinal displacement) and the radial displacement at the face are both overestimated, while the convergence of excavation boundary underestimated. The latter is underestimated because corresponds to the final displacement minus the displacement at the face.

Another possibility is to define \( t_0 \) in a certain section equals to \( t_i \) when the tunnel face approaches a certain distance from the considered section. In this case the tunnel convergence may be underestimated because the creep time at the tunnel face is greater than 1 increasing the creep parameter \( \eta \) significantly.

Figure 31: consequences of choosing a global creep solving time start at the advancing front.
The choice of $t_0$ remains arbitrary but may influence the results significantly.

We realized that the application of the empirical creep model of Phienwej, developed for a plain-strain 2D-Model can produce quite different results when applied to a 3D-Model, including the longitudinal direction and simulating the tunnel advance.

The hyperbolic creep is an empirical model that was developed for a 2D analysis. Since in this model the distance from the face is not explicitly considered, it is not possible to simulate the stopping of a TBM. The use of the hyperbolic creep in an axisymmetric model where the longitudinal direction is also modeled can change significantly the behavior of the creep law.

Reaching this point, it was decided to implement this theory into a 2D plane strain model in order to suppress the numerical incongruities appeared in the 2D axisymmetric study. The main drawback when switching into a 2D plane strain model is that several parameters that where clearly defined in the previous model will now be suppressed: basically, these parameters are those referred to the tunnel advance like penetration ratio, and the dynamic friction. Another drawback will be the loss of a realistic behavior of the advancement simulation: it won’t be possible to calculate the face advancement effect on the radial and face displacements. In order to overcome this limitation, Filippos Manolas proposed to use Pane’s equation for calculating the stress-strain field behind the tunnel face.
4.4. 2D plane strain model

A new idealized design of the physical problem is now proposed:

The advantage of using a 2D plane strain model is that structural elements can be used. In this case the “Beam structure” command has been found suitable to represent the TBM’s shield. A beam element represents a structural member in which bending stiffness is important (e.g., footing, foundation, retaining wall, tunnel lining). Structural elements reduce the complexity of defining the boundary conditions and allow the user to model more simple grids. In terms of operational cost, this reduces significantly the calculation time.

4.4.1. Idealized 2D plane strain model

As explained at the 2D axisymmetric section, tests must be done to idealized models to assure the good functionality of the designs. In this case, a non supported tunnel will be designed in order to be able to compare our results with those Phienwej obtained and published in 2007. Excluding structural elements will guarantee that the comparison strictly involves the two hyperbolic creep laws formulated.

The model proposed is a symmetric 2D model. Because of its symmetric design, no movements are allowed in the Y axis for x=0. The bottom and top of the model are fixed simulating tunnel’s confinement. A higher density of grid elements can be considered thanks to the axisymmetry of the model, which will provide more accurate results in terms of magnitude and numerical behavior.
4.4.2 Results and qualitative analysis

One of the major and most common problems when implementing user-defined constitutive models is the numerical instabilities that may appear when performing the calculations. On one hand, FLAC gives several ways to control the results while performing the calculations (by keeping low unbalanced forces and equilibrium ratios) and, on the other, it allows the user to define calculation steps in order to minimize any instability. Also, a proper grid defining will help reduce these problems. Following, an example of numerical instability and the corrected version are shown:

Figure 34: idealized case of study. Boundaries are fixed simulating confinement and a force proportional to the in-situ stress is applied.

Figure 35: representation of numerical stable result and an unstable one.
The blue graphic corresponds to a calculation where FLAC has performed automatically the calculations using the corrective and controlling instruments it possesses for lowering numerical instability. As seen, the results can’t be considered as valid results.

For correcting this problem, in this case it has been adopted several correction measures. First of all, it has been separated the problem solving into 5 steps, according to the total creep time calculations. The more creep time increases the higher the timesteps $\Delta t$ used for the calculations can be. For instance, for a creep time of one day it has been set a timestep of 100 seconds (864 steps), whereas for a creep time of 1000 days, the timestep is 10800 seconds (setting the number of steps at 7200). FLAC proposes several ways for calibrating these timesteps depending on the rheological model. For a general creep model FLAC estimates a maximum timestep model as:

$$\Delta t_{max}^{cr} = \frac{n}{G}$$

Quoting FLAC’s manual: “In some cases, it may be preferable to avoid a continuous adjustment of the timestep which may create “noise.” For this purpose, after a timestep change has occurred, there is a user-defined “latency period” (e.g., 100 steps) during which no further adjustments are made, allowing the system to settle. Normally, the timestep will start at a small value, to accommodate transients such as excavation, and then increase as the simulation proceeds. If a new transient is introduced, it may be desirable to reduce the timestep manually and then let it increase again automatically”. In this sense, the problem has been initialized with the option “creep mode calculation off” ($\Delta t=0$) and stepped until both the equilibrium ratio and the unbalanced forces become lower than a specific threshold (FLAC recommends an equilibrium ratio of around $1e-3$ and the unbalanced force, in our case, less than 1KN is acceptable). Because the following calculation initializes creep calculations the displacement speeds are set to zero in order to minimize any numerical shock and the calculations are performed by very little timesteps (10 seconds during 2 hours).

A result that confirms the qualitative good functionality of the model is the calculation of the viscosity over time. As seen at equations [15] & [16], viscosity is inversely proportional to the creep time. Our results confirm this:

![Viscosity's time dependent behavior](image_url)

**Figure 36**: inverse of the viscosity's time dependent behavior.
4.4.3 Quantitative analysis

Following FLAC’s problem solving methodology, once the qualitative analysis has been performed and concluded that the coupling between the numerical formulation and the modeling of the engineering problem result in numerically stable and physically admissible values, a quantitative analysis must be done for determining if the magnitude of the results are correct. This can be done in several ways, however, since we only dispose the data published by Phienwej in 2007, our quantification will be limited by the results comparison.

The general case of study the input parameters are: radius of the tunnel \( r = 3.5 \text{ m} \); in-situ stress \( (P_0 = \gamma h) = 10 \text{ MPa} \); Young modulus \( E = 700 \text{ MPa} \); Poison’s ratio \( (\nu) = 0.3 \); friction angle \( (\phi) = 25^\circ \); unconfined compressive strength \( (q_u) = 3.5 \text{ MPa} \); and the hyperbolic parameters \( E_u/(\sigma_1 - \sigma_2)/E_u = 250, \ R_f = 0.9 \) and the creep parameter \( \lambda = 0.07 \).

As a first result Figure 37 and 38 are the results obtained with our approach and Phienwej’s respectively. As seen our results reproduce with a good level of accuracy Phienwej’s for the input values and a creep period of 40 days.

Figure 37: Tunnel’s wall radial displacement results depending on time obtained by running a reference case of study.
Quoting Phienwej, “the $\lambda$ parameter controls the time rate of decrease in the modulus of the material”. These values are back-calculated from different sets of data and are used to classify different rock types (see figure 16). Analyzing the effect of hyperbolic creep parameter $\lambda$ we obtain certain discrepancies. Comparing figure 39 (according to our tests) and 40 (Phienwej’s results), for lambdas reaching 0.07 a good result correlation is obtained. However, when lambdas exceed this value, the displacements obtained in our tests are found to be much higher.

Figure 39: Effect of lambda on diametral closure according to our tests.
Another aspect interesting to study when comparing both hyperbolic formulations is the influence that the strength parameters of the material have on the outcome results. The following two figures show in terms of diametral displacement the influence when varying the strength parameters $q_u$ (Uniaxial compressive strength) and the friction angle. In fact, the $q_u$ is a function of the cohesion ($c'$) and the friction angle as it follows:

$$q_u = f(c', \phi)$$

Analyzing the results a good correlation with Phienwej’s results are found for the reference case and $q_u=1750$ KPa. However, a huge difference for the case $\phi=12.5$ can be observed; a difference of more or less 50% of what Phienwej publishes.

Figure 40: Phienwej’s results on calculating diametral closure depending on lambda

Figure 41: Influence of the strength parameters on wall displacements found with our approach.
Other tests can be done for comparing results. A good comparison can be done by the stress fields generated behind the tunnel’s wall after the excavation. Since Phienwej provides his results on both the shear stress magnitude and the radial stress (figure 44) a test has been done on this way. The results are shown here after:

![Stress fields behind opening wall measured at different days](image)

Figure 43: Stress fields generated as consequence of the tunnel excavation.
First of all, it is interesting to see that a model which is technically visco-elastic can reproduce a stress behavior which is relative to elasto-plastic models. This can be explained by the Mohr-Coulomb failure criteria on which the hyperbolic creep law’s formulation has been based.

Analyzing the values, the limit plasticity-elasticity is found in both cases around 5m from the tunnel wall, though, in our results it’s slightly less. Concerning the stress magnitudes, Phienwej encounters slightly higher values (both peak values and values at tunnel’s wall). This is in part normal if we consider that usually our displacements are higher.

Focusing on the time dependency, it can be seen that with time, a certain relaxation appears (decrease of the values) since the strains are much lower.

A way for contrasting the stress values around the tunnel opening is to see the stress level measured along time. Figure 45 provides the variation of time of the stress level $D_i$ in a zone close the tunnel’s wall and another one several meters away. As seen, the area close to the wall experiences very high stress levels and it is not reduced as much as expected.
4.5 Reformulation of the hyperbolic creep law

Having concluded that the reason for obtaining different results from those presented by Phienwej is not due to numerical instabilities or coding mistakes, another hypothetic solution can be given. Phienwej’s theory development is conceived to perform plane strain calculations. When FLAC performs calculations in plane strain conditions it needs to define the complete strain-stress tensor (for plane strain conditions). This may lead to a conceptual error when defining the stress level \( D \) and the stress tensor itself. As seen in the previous section, \( D \) can reach values over 1.5 in the proximities of the tunnel opening and be maintained long after the excavation. Hereafter a different approach is proposed for defining the stress level.

As seen, the general expression for the stress level is

\[
D_i = \frac{\bar{\sigma}}{q_f}
\]

where

\[
\bar{\sigma} = \sqrt{\frac{3}{2}} \sqrt{\sum (\sigma_{ij}^{d})^2} \Rightarrow \bar{\sigma} = \sqrt{\frac{3}{2}} \sqrt{\sigma_{xx}^{d^2} + \sigma_{yy}^{d^2} + \sigma_{zz}^{d^2} + 2\sigma_{xy}^{d^2}}
\]

For defining \( q_f \) it is assumed that \( q_f \) is the deviatoric stress magnitude of the principal stresses at failure:

\[
q_f = \sqrt{\frac{3}{2}} \sqrt{\sum (\sigma_i^{d})^2} \Rightarrow \bar{\sigma} = \sqrt{\frac{3}{2}} \sqrt{\sigma_1^{d^2} + \sigma_2^{d^2} + \sigma_3^{d^2}}
\]
For transforming the plane strain stresses into principal stresses the following transformations are required in FLAC:

\[
\sigma_1 = \frac{1}{2} \left( \sigma_{xx} + \sigma_{yy} - \sqrt{(\sigma_{xx} - \sigma_{yy})^2 + 4\sigma_{xy}^2} \right)
\]

\[
\sigma_2 = \sigma_{zz}
\]

\[
\sigma_3 = \frac{1}{2} \left( \sigma_{xx} + \sigma_{yy} + \sqrt{(\sigma_{xx} - \sigma_{yy})^2 + 4\sigma_{xy}^2} \right)
\]

When operating with FLAC it is important to make sure that the relation \(\sigma_1 < \sigma_2 < \sigma_3\) (compressions stresses are negative) is guaranteed. A controlling code can be programmed for assuring this relation.

Using the Mohr-Coulomb failure criteria, the maximum shear stress at failure \((\Delta \sigma)\) is defined as

\[
\Delta \sigma = \frac{2CA - \sigma_1 - \sigma_3}{2} \sin \varphi'
\]

As a result, the principal stresses at rupture will be

\[
\sigma_{2R} = \sigma_2
\]

\[
\sigma_{1R} = \frac{\sigma_1 + \sigma_3}{2} - \Delta \sigma
\]

\[
\sigma_{3R} = \frac{\sigma_1 + \sigma_3}{2} + \Delta \sigma
\]

The deviatoric principal strains at failure can be calculated by the general form \(\varepsilon_{iR} = \varepsilon_{iR} - \varepsilon_{iSO}\) and finally substituted at eq [25] for obtaining the deviatoric stress magnitude at failure \(q_r\).

In an attempt to reproduce on a better way Phienwej’s model, another modification has been done: As mentioned, \(E_{tf}/q_f = 0.01/(E_f - \varepsilon_{0.99})\). On a first approximation it was supposed that \(\varepsilon_{0.99} = 0.99 \varepsilon_f\). Using Phienwej’s formulation,

\[
\varepsilon_{0.99} = \frac{0.99}{q_f (1 - 0.99R_f)} E_u
\]

\[
\varepsilon_f = \frac{1}{(1 - R_f) \frac{E_u}{q_f}}
\]
4.5.1. RESULTS

After introducing these corrections into the hyperbolic theory development and implementing it by FLAC, tests shall be done in order to search for programming mistakes and numerical instabilities. The results will be now compared again with those presented by Phienwej and our first implementation and discussed.

The first test to be done is to determine the stress level after inputting the proposed changes. Figure 46 compares the values of Di over time of the new and old calculations in a zone close to the tunnel opening and another one several meters away from it.

![Figure 46: Figure comparing results from figure 45 with the new stress level values.](image)

The new results fit much better both the hyperbolic creep theory and the expected results. Quoting Phienwej 2007, “the hyperbolic creep law is only valid for stress level not greater than one. Since tunnel squeezing is often related to creep as well as yielding of the ground in the vicinity of the opening, by using the viscoelastic analysis, the initial elastic stress the initial elastic stress will be allowed to exceed the strength limit of the ground in the initial step. In the subsequent time steps, it should be reduced to the limiting value.” Since creep takes place during the period of study it is assumed as correct that the stress level remains slightly above 1. As a reminder, the strains due to stress levels over one are linearly extrapolated from the strain values at failure and 0.99 percent of it. Sticking to this, the greater the stress level is the greater the strain error by approximation can be.

Reducing the value of Di has a direct impact on the strain calculation since they are directly proportional. This means that the new strains (and consequently the displacements) should be smaller, which is needed regarding the results obtained previously.

Before testing the displacements, the stress fields are presented (Figure 47) for different creep time and locations behind the tunnel wall. Again, the stress values are lower than those presented by Phienwej (Figure 44) which means that the strains are still probably too big. Doing a quick qualitative analysis, the current results simulate more clearly the relaxation and the increment of the plastic radius.
Finally, some results on displacements depending on hyperbolic's creep law and on grounds strength parameters are presented in figures 44 and 46.

It is remarkable the fact that though reducing the stress level at zones close to the opening wall the displacements have become even higher than in the previous case. This may be explained basically for two reasons: on one hand, the introduction of eq. [28] increases the value of the strains at the threshold of failure in comparison to the formulation used on the first theory development (eq [14]). On the other, zones enough separated from the tunnel opening which don’t experiment yielding nor plastification may have an increase on the stress level which, in terms of strain, implies also an increase. Though talking of small increases, since the affected area can be quite extensive, the total displacement can result high. As an example, figure 45 show the radial displacement field magnitude all around the considered model.
The last test done to the model pretends to compare the resulting displacements depending on strength parameters with those calculated with the previous model and the ones found by Phienwej (figures 41 and 42). The results presented in figure 46 confirm that the new displacements are greater than both those published by Phienwej and the ones found with our first analysis. The trends show a considerable increase on the results outcome for the cases where the strength parameters have been reduced. Though the effect of these parameters is important when defining a solid mass, the difference in the results we present is mostly dominated by the effect of the linear approximation of the strains when the stress level is above one. Actually, some tests have been carried out in order to confirm this statement and it has been clearly observed that both models, if eq. [28] is not considered, are good correlated. However, the first formulation must be corrected so a stress level decrease is produced. Generalizing, the second formulation is more general than the first one.

Figure 45: Picture of FLAC’s emerging window plotting radial displacement.

Figure 46: Influence of the strength parameters on wall displacements found with the new hyperbolic approach.
5. CONCLUSIONS

The stress-strain fields created behind the tunnel opening as a result of the disturbance of the old stress field is a very complex physical process which cannot be fully predicted because of the multitude of heterogeneous characteristics defining a rock mass or ground, both on the macroscopic and microscopic levels. Each of the parameters which define such a solid mass is responsible for the particular reaction of the ground when an underground excavation is performed. Furthermore, other variables may influence the generation of the new stress-strain field such as the penetration rate. However, as an attempt to predict the effects of the disturbance of an old stress field, simple and conceptual models can be accepted to represent with enough accuracy the physical implications related underground works.

This study had set as principal objective to model the excavation of a tunnel by a shield TBM and study the ground reactions with the hyperbolic creep law. The first intent to conclude a 3D TBM advancing analysis with FLAC (modeled in 2D axisymmetric conditions) had to be dismissed because of the impossibility of FLAC to define time dependent ground properties (in our hyperbolic creep law formulation, viscosity is a time dependent parameter) together with the advancement of the TBM. Furthermore, FLAC couldn’t simulate with accuracy the effect of a TBM in standstill situation because the hyperbolic creep law is an empirical model that was developed for a 2D analysis. Since in this model the distance from the face is not explicitly considered, it is not possible to simulate the stopping of a TBM. The use of the hyperbolic creep law in an axisymmetric model where the longitudinal direction is also modeled needs further assumptions that, eventually, can change significantly the behavior of the creep law.

As a result of these constrains two options were initially considered: first of all, the possibility of reformulate our hyperbolic law was considered, however, because of the complexity of the overall effort and uncertainty of the results, this option was dismissed. The second and chosen option was to design a 2D plane strain model representing the tunnel (and TBM) and surrounding rock mass maintaining the hyperbolic formulation. Comparing our results with those Phienwej published (Time-Dependent Response of Tunnels Considering Creep Effect, 2007) it is clearly seen that the stress-strain fields obtained by both formulations are different. The explanation for this relays basically on the approaches to define the stress level done by both formulations: In the case of Phienwej, he defines a stress level which is calculated in terms of stress difference (like in a triaxial test) obtained with the hyperbolic formulation. In FLAC, the strain tensor is obtained directly from the stress tensor. This was a constraint for using the same approach as Phienwej. In order to overcome this problem, we decided to calculate the stress level by using the general definition for stress magnitude found in Eq [2] (the stress difference in the triaxial test is a particular case). In other words, we formulated a general case for the hyperbolic law.

In order to obtain more accurate results some correcting measures could be taken into account. For instance, Filippos Manolas used Pane’s formulation in order to take into consideration the effect of the excavation face. Phienwej, in his formulation, uses the thick walled cylinder solutions as boundary conditioning. Furthermore, the fact of using a conditioned formulation should be taken into consideration when back calculating the lambda parameter. As a topic of investigation lambda’s could be back calculated with both formulations an see the correlation between them.

Finally, it is important to highlight the procedure when solving problems with FLAC. As a reminder, when using FLAC the next steps should be followed: 1, Define the objectives for the model analysis; 2, Create a conceptual picture of the physical system; 3, Construct and run simple idealized models; 4, Assemble problem-specific data; 5, Prepare a series of detailed model runs; 6, Perform the model calculations; 7, Present results for interpretation.
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Appendix

FLAC CODES

2D axisymmetric modeling of TBM advancement

```
;;;;;; *.dat;;;;;;;; executes the *.fis files
new
  config axi
  call modello_MOHR_Lin.fis
  call scavo_MOHR_lin.fis
;
  set R_ex=3.5
  set L_Head=10
  set th_conc=.10
  set e_conc=20.7e6
  set nu_conc=0.15.
  set phi=25
  set qu=3.5e3
  set e_mod=700e3
  set nu=0.3
  set gamma=25
  set prof=400
  set nome_file= 'cas051b.sav' ;condicions a la meitat del túnel
  set sratio=0.005

modello_MOHR_Lin
save cas051a.sav ;condicions inicials
scavo_MOHR_lin
save cas051c.sav ;condicions una vegada excavat el túnel

set plot jpg color
set output cas051_plast.jpg
plot pen gr plast

set output cas051_rd.jpg
plot pen table 1

set log off

res cas051b.sav
```
set plot jpg color

; title
cas051b.sav
;
set output cas051_model.jpg
plot pen gr green fix apply red blue
;
set output cas051_disp.jpg
plot pen boun disp red
;
set output cas051_sxx.jpg
plot pen boun sxx fill
;
set output cas051_szz.jpg
plot pen boun szz fill
;
set output cas051_t2.jpg
plot pen table 2
;
set output cas051_t3.jpg
plot pen table 3

;;;;;;;;;;;;;; Defines basically Grid parameters, boundary conditions and ground conditions;;;;;;;;;;;;;;

DEF modello_MOHR_Lin

float R_ex L_Head L_press F_press R_press R_int e_mod
float e_conc nu nu_conc th_conc coes phi gamma prof qu

R_max=10.*R_ex
R_int=R_ex-th_conc
L_max=8.*L_Head
Delta_X=R_ex/4.
Delta_Y=L_Head/5.
NE_Y=int(L_max/Delta_Y)+30
NE_X0=4
NE_X1=2
NE_X2=int((R_max-R_ex)/Delta_X/3.))+5
NE_X=NE_X0+NE_X1+NE_X2
; els NE són els números d'elements (paral·lepipeds de la malla). Per poder ; representar els nodes hem de definir les línies sobre els quals estan ; posats:

\[ i_0 = NE_X 0 + 1 \]
\[ i_1 = NE_X 0 + NE_X 1 + 1 \]
\[ i_{\text{max}} = NE_X + 1 \]
\[ j_{\text{max}} = NE_Y + 1 \]
\[ y_{\text{min}} = L_{\text{max}} / 2. \]
\[ y_{\text{max}} = L_{\text{max}} / 2. \]

; 
\[ k_{\text{mod}} = e_{\text{mod}} / 3. / (1. - 2. * \nu) \]
\[ g_{\text{mod}} = e_{\text{mod}} / 2. / (1. + \nu) \]
\[ s_{\text{nat}} = 4 \gamma \text{prof} \]
\[ \text{coes} = qu / (2. * \cos(\phi \pi / 180.) / (1. - \sin(\phi \pi / 180.))) \]

command
grid NE_X NE_Y
gen 0,Y_min 0,y_max R_int,y_max R_int,y_min i=1,i0 j=1,j_{\text{max}}
gen R_int,y_min R_int,y_max R_{ex},y_max R_{ex},y_min rat=1,1 &
i=i0,i1 j=1,j_{\text{max}}
gen R_{ex},y_min R_{ex},y_max R_{max},y_max R_{max},y_min rat=1,2,1 &
i=i1,i_{\text{max}} j=1,j_{\text{max}}
model mohr
prop dens=gamma bulk=k_{\text{mod}} shear=g_{\text{mod}} fric=phi cohesion=coes
ini sxx=s_{\text{nat}} syy=s_{\text{nat}} szz=s_{\text{nat}}
apply sxx=s_{\text{nat}} i=i_{\text{max}}
fix y j=1
fix y j=j_{\text{max}}
end_command
end

;;;;;;;;;;;;;;;;;;; TBM's Advancement effect is programmed in this file ;;;;;;;;;;;;;;;;;;;;;;;;
DEF scavo_MOHR_lin
    string nome_file
    j_zm=jzones/2
    j_gm=j_zm+1
    k_conc=e_conc/3./(1.-2.*nu_conc)
    g_conc=e_conc/2./(1.+nu_conc)
    NE=NE_X0+NE_X1
    ii1=NE_X0+1
loop j(1,jzones)
    jj=j+1
    gg=int(L_press/delta_y)
    Jl=j4gg
command
    model null i=1,NE_x0 j=j
    model elas i=ii1,NE j=j
    Prop bulk=k_conc shear=g_conc i=ii1,NE j=j
    solve
end_command
    yf=y(i1,jj)
    y_half=y(i1,j_zm)
    Y_front=yf-y_half
    rdisp=xdisp(i1,j_gm)
    rstress=sxx(i1,j_zm)
command
    table 1 yf rdisp
    table 2 yf rstress
end_command
if y_front=0. then
    rdispf=rdisp
end_if
if y_front>=0. then
rconv=rdisp-rdispf
command
table 3 yf rconv
end_command
end_if
if y_front=0. then
command
save @nome_file
end_command
end_if
if y(i1,jj)>=39.6 then
loop k(i1,izones)
xdist=x(k,j_gm)-R_ex
radial_str= sxx(k,j_zm)
tan_str= szz(k,j_zm)
command
table 5 xdist radial_str
table 6 xdist tan_str
end_command
end_loop
end_if
end_loop
end

2D PLANE STRAIN MODEL
grid 30,30
model m

`gen circle 15.0 15.0 10.0`

`prop density=2.6 bulk=1.19E6 shear=1.087E6 cohesion=500.0 friction=25.0 &
   dilation=3.0`

`;`  
`gen adjust`  
`;`  
`fix y j=1`
`fix x i=1`
`fix x i=31`
`;`  
`set large`  
`set grav=10`
`ini syy -5980.0 var 0 780`
`ini sxx -5980.0 var 0 780`
`ini szz -5980.0 var 0 780`
`;`  
`apply nstress -680.0 j=31`
`;`  
`mod null reg 15 15`
`;`  
`def lining`
`r=9.85`
`x1=15.0`
`y1=15.0+r`
`dangle=2.0*pi/24`
`ang=pi/2.0`
`loop n (1,24)`
`ang=ang+dangle`
\[ x_2 = r \cos(\text{ang}) + 15.0 \]
\[ y_2 = r \sin(\text{ang}) + 15.0 \]

command

struct beam beg x1,y1 end x2,y2 prop 1001
end_command

\[ x_1 = x_2 \]
\[ y_1 = y_2 \]
end_loop
end
lining

struct prop 1001 e=1e12 height 0.50 width 1.00

int 1 aside from node 1,24 to node 1 bside long from 16,26 to 16,26
int 1 ks 2e9 kn 2e9 fric 30

save mesh0.sav
step 25000
save mesh.sav

2D Plane Strain model symmetric

;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;;
;Project Record Tree export
;Title:101.1 TBM Squeezing

;... STATE: EUREKA05_A ....
call deep_tunnel.fis
call Hyperbolic_creep.fis
;
;...... grid parameters
set fact1=0.00 ; ov_cor=fact1*R_int ; for overcoring
set fact2=10 ; L_X=fact2*R_ex ; X grid
set fact3=10 ; L_Y=fact3*R_ex ; Y grid
set fact4=3 ; L_Y2=fact4*L_Y/fact3
set fact5=5 ; L_X2=fact5*L_X/fact2
;

;;;;;;;;;;;;;;;;;; design
set R_int=3.5
set e_mod=700e3
set nu=0.3
set qu=3500.
set phi=25.
set gamma=25.
set prof=400.
set lamda=0.07
set eps_u=0.004 ;;;;; its the invers of the so called EU/qf ;;;
set Rf=0.90
;
depth_tunnel

;;;;;;;;;;;;;;;;;;; GRID, BOUNDARIES; GROUND PARAMETERS and other settings;;;;;;;;;;;;;;;;;;;;;;;;;;;;
def deep_tunnel
    float fact1 fact2 fact3 fact4 fact5 R_int
    float e_mod nu qu phi gamma prof
    float lamda eps_u Rf

    ;;;;; tunnel parameters ;;;;;
    ov_cor=fact1*R_int
    R_ex=r_int+ov_cor
    ;;;;; Mesh parameters ;;;

\[ L_X = \text{fact2} \times R_{\text{ex}} \]
\[ L_{X2} = L_X \times (\text{fact5} \times L_X / \text{fact2}) \]
\[ L_Y = \text{fact3} \times R_{\text{ex}} \]
\[ M_{L_Y} = 4L_Y \]
\[ L_{Y2} = \text{fact4} \times L_Y / \text{fact3} \]
\[ M_{L_Y2} = 4L_{Y2} \]
\[ \text{NEX1} = \text{int}(8 \times R_{\text{ex}}) \]
\[ \text{NEY1} = \text{int}(8 \times R_{\text{ex}}) \]
\[ \text{Delta}_X = L_{X2} / \text{NEX1} \]
\[ \text{Delta}_Y = 2 \times L_{Y2} / \text{NEY1} \]
\[ \text{ratio}_X = 1.2 \quad ;;;;;; \text{remember to change @ grid} \]
\[ \text{ratio}_Y = 1.2 \]
\[ \text{NEX2} = \text{int}((\log((L_{X2} / \text{ratio}_X - 1) / (\Delta_X \times \text{ratio}_X)) + 1) / \log(\text{ratio}_X) + 1) \]
\[ \text{PARAM1} = \log(((L_{Y2} - L_Y) / \text{ratio}_Y - 1) / (\Delta_Y \times \text{ratio}_Y) + 1) \]
\[ \text{NEY1U} = \text{int}((\text{PARAM1} / \log(\text{ratio}_Y) + 1) \]
\[ \text{NEY1D} = \text{NEY1U} \]
\[ \text{NEY} = \text{NEY1} + \text{NEY1D} + \text{NEY1U} \]
\[ i1 = \text{NEX1} + 1 \]
\[ i_{\text{max}} = i1 + \text{NEX2} \]
\[ j1 = \text{NEY1D} + 1 \]
\[ j2 = \text{NEY1D} + \text{NEY1} + 1 \]
\[ j_{\text{max}} = j2 + \text{NEY1U} \]

;;;; wall, roof, base;;;;
\[ \text{aaa} = \text{int}(R_{\text{ex}} / \Delta_X) \]
\[ \text{bbb} = \text{int}(R_{\text{ex}} / \Delta_Y) \]
x_center=aaa+1
x_centernode=x_center+1
Y_center=int(j_max/2.)
y1=y_center-bbb
m_y1=y1+1
y3=y_center+bbb+1
m_y3=y3+1
base=-R_ex
roof=R_ex

;;;;; distance from tunnel ;;;;;;
x_center_bis=x_center-1.

;;;;; elastic and strength parameters ;;;;;;
k_mod=e_mod/3./(1.-2.*nu)
g_mod=e_mod/2./(1.+nu)
s_nat=gamma*prof
coes=qu/(2.*cos(phi*pi/180.)/(1.-sin(phi*pi/180.)))

;;;;; building the mesh ;;;;;;;;,
command
grid NEX NEY
model e
gen 0,ML_Y 0,ML_Y2 L_X2,ML_Y 2 L_X2,ML_Y rat 1,0.8333  i=1,i j=1,j1
gen same 0,L_Y2 L_X2,L_Y2 same i=1,i1 j=j1,j2
gen same 0,L_Y2 L_X2,L_Y2 rat 1,1.2 i=1,i1 j=j1,j_max
gen same same L_X,ML_Y2 L_X,ML_Y rat 1.2,0.8333 i=i1,i_max j=1,j1
gen same same L_X,L_Y2 same rat 1.2,1 i=i1,i_max j=1,j2
gen same same L_X,L_Y same rat 1.2,1.2 i=i1,i_max j=j2,j_max
gen circle 0,0 R_ex
;
model m_hyperbolic
    prop dens 2.5 m_phi=phi m_coh=coes
    prop m_k=K_mod m_g=g_mod
    set m_lamda=lamda m_eps_u=eps_u m_Rf=Rf
;

model null region 1,y_center
ini sxx=s_nat syy=s_nat szz=s_nat
appl sxx=s_nat i=i_max
set large
fix x i=1
fix y j=1
fix y j=j_max

his 1 nstep=2 unbal
his 2 crtime
his 3 xdis  i=x_center j=y_center
his 4 sxx  i=15 j=y_center
his 5 syy  i=15 j=y_center
his 6 szz  i=15 j=y_center
his 7 sig1  i=15 j=y_center
his 8 sig2  i=15 j=y_center
his 9 sig1_f i=15 j=y_center
his 10 sig2_f i=15 j=y_center
his 11 m_D  i=15 j=y_center
his 12 m_vis i=15 j=y_center
his 13 m_D  i=x_centernode j=y_center
his 14 m_vis i=x_centernode j=y_center
save Eureka05_a.sav

;... STATE: EUREKA05A_B ....
set cr 3600 ; initialization at one hour
set crdt 0  ; time step of one hour
solve
save Eureka05A_b.sav

;... STATE: EUREKA05A_C ....
ini xv=0 yv=0
set crdt 10
step 720 ; + 2 h  (time=3h)
set crdt 100
step 756 ; + 21 h  (time=1day)
save Eureka05A_c.sav

;... STATE: EUREKA05A_D ....
set crdt 600
step 1296
save Eureka05A_d.sav

;... STATE: EUREKA05A_E ....
set crdt 3600
step 2160
save Eureka05A_e.sav

;... STATE: EUREKA05A_F ....
set crdt 10800
step 7200
save Eureka05A_f.sav

;*** plot commands ***
:plot name: his unbal
plot hold history 1 line
;plot name: his crtime
plot hold history 2 line
;plot name: his disp
plot hold history 3 line vs 2
;plot name: his stress
plot hold history 4 line 5 line 6 line
;plot name: his s1-s3
plot hold history 7 line 8 line 9 10
;plot name: his m_D
plot hold history 11 line 13 line
;plot name: his m_vis
plot hold history 12 line 14 line
;plot name: m_D
plot hold m_d fill bound
;plot name: sig1
plot hold sig1 fill bound
;plot name: disp
plot hold displacement lred bound

end_command
end
; Phjenwej et al. in paper “Time-Dependent Response of Tunnels Considering Creep Effect”
; International Journal of Geomechanics, Juli/August 2007
;
; NB: Maxwell is composed by a viscous dashpot and an elastic spring in series
;
;---------------------------------------------------------------------------------------
;
; set echo on
def m_hyperbolic
  constitutive_model
  f_prop m_k m_g m_phi m_coh ; material properties
  float m_Rf m_lamda m_eps_u ; creep parameters
  f_prop sig1_f sig2_f m_D m_vis ; output grid variables
  ;
  float $c1d3 $c4d3 $x_con $y_con $temp ; parameters
  float $dev $dev3 $de11d $de22d $de33d ; partition strain
  float $s0 $s11d $s22d $s33d $si ; partition stresses
  ;
  float $s1 $s2 $s3 ; principal stresses
  float $s1_f $s2_f $s3_f ; principal stresses at failure
  float $s1d_f $s2d_f $s3d_f $si_f ; partition stresses at failure
  float $t1 $par1 $par2 $par3 $tau_max ; parameters
  ;
case_of mode
  ;
  ;
; Initialisation section
  ;
  ;

case 1
    if m_g <= 0.0 then
        m_g = 1e-20
    end_if

; Running section

; -------------------

case 2
    $c1d3 = 0.333333333
    $t1   = 3600.

;--- partition strains ---

    $dev  = zde11 + zde22 + zde33
    $dev3 = $c1d3 * $dev
    $de11d = zde11 - $dev3
    $de22d = zde22 - $dev3
    $de33d = zde33 - $dev3

;--- partition stresses ---

    $s0   = $c1d3 * (zs11 + zs22 + zs33)
    $s11d = zs11 - $s0
    $s22d = zs22 - $s0
    $s33d = zs33 - $s0
    $si   = sqrt(3./2.) * sqrt($s11d^2 + $s22d^2 + $s33d^2 + 2.*zs12^2)

;--- principal stresses ---

    $s1 = 0.5 * ( zs11+zs22 - sqrt((zs11-zs22)^2 + 4.0*zs12^2) )
    $s2 = zs33
    $s3 = 0.5 * ( zs11+zs22 + sqrt((zs11-zs22)^2 + 4.0*zs12^2) )

    if $s2 < $s1 then
        $s2 = $s1
    end_if
$s1 = zs33
end_if
if $s2 > $s3 then
    $s2 = $s3
    $s3 = zs33
end_if

;--- principal stresses at failure ---
$$par3 = 2.* m\_coh/tan(m\_phi*pi/180.) - $s1 - $s3$$
$$\tau\_max = $par3 * \sin(m\_phi*pi/180.) / 2.$$  
$$s2\_f = $s2$$
$$s1\_f = ($s1+$s3)/2. - $\tau\_max$$
$$s3\_f = ($s1+$s3)/2. + $\tau\_max$$
$$\sigma1\_f = s1\_f$$
$$\sigma2\_f = s3\_f$$

;--- partition stresses at failure ---
$$s1d\_f = s1\_f - s0$$
$$s2d\_f = s2\_f - s0$$
$$s3d\_f = s3\_f - s0$$
$$\sigmai\_f = \sqrt{3/2.} * \sqrt{(s1d\_f^2 + s2d\_f^2 + s3d\_f^2)}$$

;--- calculation of Maxwell viscosity according to hyperbolic creep law ---
$$m\_D = \sigmai/\sigmai\_f$$
if $m\_D < 1.e-20$ then
    $m\_D = 1.e-20$
end_if
if $m\_D >= 1.$ then
    $$\text{par1} = (100.*m\_D-99.) \div (1.-m\_Rf) - (99.*m\_D-99.) \div (1.-0.99*m\_Rf)$$
else
    $$\text{par1} = m\_D \div (1.-m\_D*m\_Rf)$$
end_if
end_if

$\text{par2} = \text{t1}/m_{\text{lamda}}(\text{crttime}/\text{t1})^{(1.-m_{\text{lamda}})}$

$m_{\text{vis}} = 2./3. * \text{si} / m_{\text{eps}_u} / \text{par1} * \text{par2}$

if $m_{\text{vis}}<1.e-20$ then

$m_{\text{vis}}=1.e-20$

end_if

[--- Parameters 4444444444444]

$\text{temp} = \text{crtdel} / m_{\text{vis}} / 4.0$

$\text{x_con} = 1.0 / (2.0 * m_g) + \text{temp}$

$\text{y_con} = 1.0 / (2.0 * m_g) - \text{temp}$

[--- new deviator stresses ---]

$\text{s11d} = (\text{s11d} * \text{y_con} + \text{de11d}) / \text{x_con}$

$\text{s22d} = (\text{s22d} * \text{y_con} + \text{de22d}) / \text{x_con}$

$\text{s33d} = (\text{s33d} * \text{y_con} + \text{de33d}) / \text{x_con}$

$\text{z12} = (\text{z12} * \text{y_con} + \text{zde12}) / \text{x_con}$

[--- isotropic stress is elastic ---]

$\text{s0} = \text{s0} + m_k * \text{dev}$

[--- convert back to x-y components ---]

$\text{z11} = \text{s11d} + \text{s0}$

$\text{z22} = \text{s22d} + \text{s0}$

$\text{z33} = \text{s33d} + \text{s0}$

[; Return maximum modulus ;]

case 3

$\text{c4d3} = 1.3333333$

$\text{cm_max} = m_k + \text{c4d3}\text{m_g}$

[;-----------------------]
; Add thermal stresses
; ---------------------

case 4
    ztca = ztea*m_k
    ztcb = zteb*m_k
    ztcc = ztec*m_k
    ztcd = zted*m_k
end_case

end

;opt m_hyperbolic

set echo on