Mastering of Squeezing Rock in the Gotthard Base

The term "squeezing rock" originates from the pioneering days of tunnelling in the Alps. The various descriptions of rock pressure were already classified into three groups, namely loosening rock pressure, swelling pressure and squeezing pressure. Thus, the observed rock behaviour was often described using terms like spalling, swelling and squeezing. As long ago as the last century, it was understood that these three types of rock pressure were caused by fundamentally different physical mechanisms (Kovári 1975). They may also act in a superimposed way; and thus it is conceivable that, in a rock of low strength containing clay minerals, the failure processes are accompanied by the swelling phenomenon.

Squeezing rock is characterised by the tendency to reduce the cross-section of the opening (Figure 1). The reduction in size of the opening in course of time is called "convergence". The actual creep potential of the rock under the given stresses is a basic requirement for the occurrence of squeezing rock. Since the lining resists the convergence, the pressure acts as a reaction, so that rock pressure and rock deformation are directly related to one another. With respect to the lining, the rock pressure is regarded as a loading, and with respect to the rock, it acts as a lining resistance; thereby, two distinct aspects (action and reaction) of the same phenomenon are expressed. If the rock pressure exceeds the bearing capacity of the lining, it will be damaged or even destroyed, and the rock deformations continue until a new state of equilibrium is reached. By not fulfilling the planned clearance of the minimum excavation line with the temporary lining, re-profiling the rock is unavoidable (Figure 2). Such repair work is time-consuming and involves high costs.

Recently, within the framework of a research project (Staus & Kovári 1996) commissioned by the project management of AlpTransit of the Swiss Federal Railways and the BLS AlpTransit Co., the experiences gained in the last 25 years in traffic tunnels in squeezing rock zones were studied and presented according to unified points of view. The report is intended to heighten our awareness of the various forms of squeezing rock, and thus consolidate our understanding of the underlying relationships. Since the rock behaviour is inseparable from the construction and operation methods and type of temporary lining, it was essential, besides information of the actual geological conditions, to describe the construction process as precisely as possible. Of the seventeen case studies from Italy, Japan, Austria, Turkey and Switzerland, the conclusion is that the trend in modern traffic tunnel construction is to excavate large areas in the tunnel profile, even in squeezing rock conditions, in order to allow a high degree of mechanisation. Usually this necessitates a systematic support of the face which, thanks to the technological developments in recent decades, can be rationally executed.

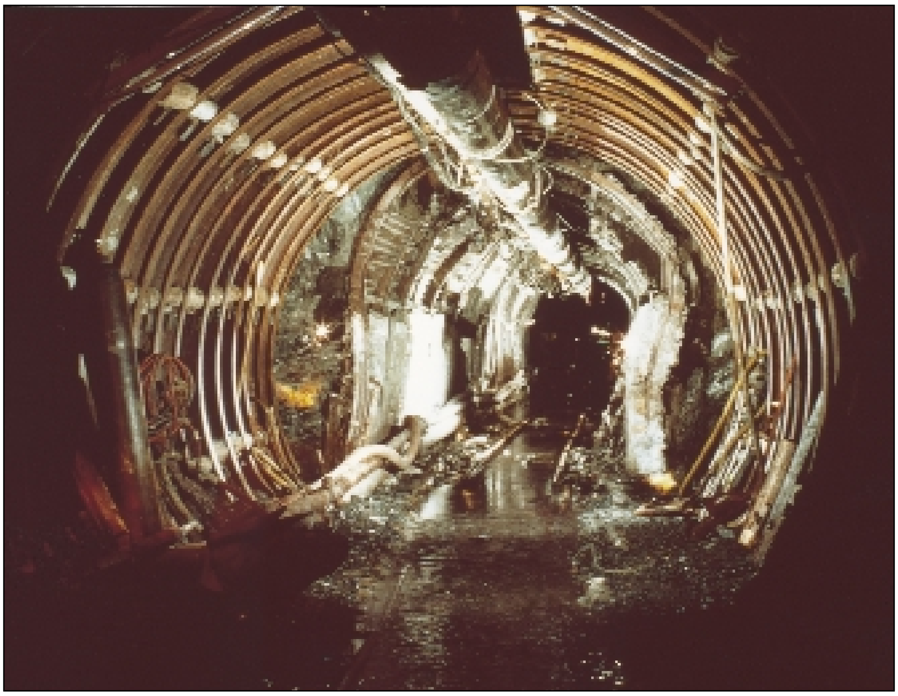


Fig. 1: Squeezing rock reduces the cross-section.

Constructional Experience

From worldwide experience in tunnelling in squeezing rock, the following empirical facts emerge (Kovári 1998):

Large long-term deformations or large long-term rock pressures only occur in rocks of low strength and high deformability. A pronounced creep capacity is an important prerequisite for the occurrence of this type of rock pressure. Phyllite, schist, serpentine, claystone, tuff, certain types of Flysch, and weathered clayey and micaceous metamorphic rocks are typical examples of such rock types. In excavating a 42 m stretch in the Simplon tunnel "a rock was encountered appearing as a dough, mainly consisting of soft micaceous limestone". Overcoming this short section took seven months (Pressel 1906).

The rock pressure decreases with increasing rock deformation. In earlier days, in extreme squeezing rock conditions, a big contraction of the cross-section was accepted and the subsequent re-profiling, as well as changing the type of lining, were the only possibilities of controlling the rock pressure.

The existence of ground water or high pore pressures aids the development of rock pressure and rock deformation. This observation is confirmed repeatedly by the favourable effect of rock drainage using an advanced pilot or parament tunnel.

As a rule, the rock deformation is not uniformly distributed over the excavation cross-section. Often, bottom heave is practically irrelevant, although in the side walls and the roof large deformations occur. In many cases, moreover, the deformation of the face, and its stability, respectively, do not present any practical problems. For full face excavation of large cross-sections the stabilisation of the face is necessary, involving time-consuming measures (Lunardi 1995).

The intensity of the rock deformations and of the rock pressure, respectively, in a stretch

Fig. 2: Reprofiling of the rock is unavoidable.

of squeezing rock usually varies considerably. For the same excavation support, the same depth of overburden and the same lithological type, often sudden changes of convergence of several magnitudes difference may be observed over a short distance. This is one of the main reasons for setbacks, which in some cases may occur despite wide experience and a well-founded knowledge of the engineers in charge.

The influence of the depth of overburden on the types of rock pressure could not, up till now, be empirically observed in an unambiguous way. The reason for this is that the changing deformation properties and strength properties have a much greater influence on convergence and pressure than the overburden effect. Thus our knowledge of the unfavourable influence of overburden is based on theory.

Finally, we would like at this point to draw attention to the following:

Firstly, it is not possible to give a precise quantitative definition of the term "squeezing rock". There is agreement, however, that the time effect is one of the most pronounced and unmistakable characteristics of squeezing rock. It is governed – as explained above – by the combined action of the rock properties, the pore water pressures and the depth of overburden.

Secondly, in overcoming a stretch of squeezing rock, it is particularly important to have a correctly formulated contract between the client and the contractor. This is understandable, since the costs per tunnel metre are high and the rate of advance is slow.

Squeezing Rock Phenomenon

The first theoretical works to explain the phenomenon of squeezing rock are closely related to the construction of the approximately 20 km long Simplon Tunnel, which has a maximum depth of overburden of 2,100 m. The Simplon Tunnel I was constructed in the period 1898 – 1906, and Simplon Tunnel II between 1912 and 1921. The long construction time for the second tunnel was due to the European war. The Alpine geologist Heim warned in an article (1878) that was much acclaimed by professional colleagues at the time, that, in his opinion, insuperable difficulties would be encountered when tunnelling at great depth. He maintained that "for each rock one needed to envisage a column so high that its weight exceeded the strength of the rock and therefore the foot of the column would be crushed. Depending on the strength of the rock this column will be higher or lower, but the envisaged conditions would always occur." Under "strength" Heim understood the uniaxial strength of the rock (Kastner 1962). He believed that reaching this strength, "hydrostatic conditions" would dominate and he coined the term "latent plasticity". Further, he assumed that "the internal friction would be so reduced under the all round pressure that a stress redistribution would occur without cleavage and the rock begins to flow, just like ice flows in a glacier (Heim 1878). The material would try to flow into the tunnel opening. From this he concluded that, beyond a certain critical depth, depending on the type of rock, the tunnel construction work would become impossible to control technically. It was Wiesmann (1912), one of the chief supervising engineers on the construction of the Simplon Tunnel, who discovered the error in the reasoning of Heim. Firstly, for the behaviour of the rock surrounding the tunnel it is not the uniaxial, but the triaxial, compressive strength that applies: "The bearing capacity of enclosed bodies, this is the governing rock strength". He could already consult the results of the von Kármán's (1911) triaxial tests on marble from the year 1905. Secondly, the behaviour of a rock in a plastic state cannot be compared to that of a fluid. In a viscous (Newtonian) fluid it is only a question of time until a hydrostatic stress state develops. Due to internal (Coulomb) friction, however, rocks behave quite differently. After the creep and relaxation processes fade away there remains, due to the cohesion and internal friction, a deviatoric component of stress state which allows a difference in principal stresses – for axisymmetrical conditions between the radial and tangential stresses – in the rock surrounding a tunnel. As one of the first, Wiesmann recognised the significance of the stress redistribution in the vicinity of an underground opening, as well as the influence of the failure state on the stress redistribution, in that he called the zone of rock affected by stress redistribution a "protective zone". Wiesmann argued in a qualitative way, basing his considerations on experience known to him of tunnelling in squeezing rock, on the findings from triaxial tests and on the stress conditions in an elastic plate containing a hole under in-plane loading. He recognised, and also gave clear reasons for, the relationship between rock pressure and deformation: "With each fraction of a millimetre with which the rock mass moves, the amount of pressure acting (on a lining) decreases".

The first computational model for describing the stress redistribution in a plate with a hole in it taking into account a failure criterion comes from the bridge engineer Maillart (1923), who in 1923 considered the idea of a "protective zone" to be outdated. In fact, this represents a considerable scientific advance, to speak of separate plastic and elastic regions, whereby the rock mass is stressed to the limit of its triaxial strength or where this is no longer the case. From Maillart we also get the pregnant formulation "As long as we require a tunnel lining, which can withstand an external rock pressure, the strength of the rock will be increased and thus enabled to develop a self-carrying capacity". The subsequent internationally well established theoretical developments led to the "characteristic line method", which permits quantitative assessment of the rock pressure. Under characteristic line, one understands the functional relationship between the radial displacement at the edge of a hole and the resisting force acting there. Thus, the characteristic line is limited purely theoretically to the axisymmetric conditions: this applies both to the cross-sectional shape (circle) and to the material properties (homogeneity, isotropy), the primary state of stress (hydrostatic condition) and the lining resistance. For a detailed analytical derivation of the characteristic line see references (Fritz 1981, Kovári 1986, Brown 1983, Panet 1995).

The first works on its practical application to the determination of the rock pressure come from Mohr (1964) and Lombardi (1971). In 1964, Pacher proposed a characteristic line of a special kind, which should enable the tunnelling engineer to optimise the lining resistance. Müller (1978) spoke of the "Pacher concept of the deliberate stress relaxation, to achieve a minimum of lining thickness for temporary lining". He maintains that "the theory of the overall concept of the New Austrian Tunnelling Method (NATM) is based on Pacher's characteristic line for linings". In 1994 Kovári raised objections to such a theory and clearly showed that the optimisation of the lining resistance following NATM is fundamentally flawed, since its requirement of having a trough-shaped rock characteristic line according to Pacher has no proper theoretical basis (Kovari 1994). Kolymbas (1998), in a recently published text book in

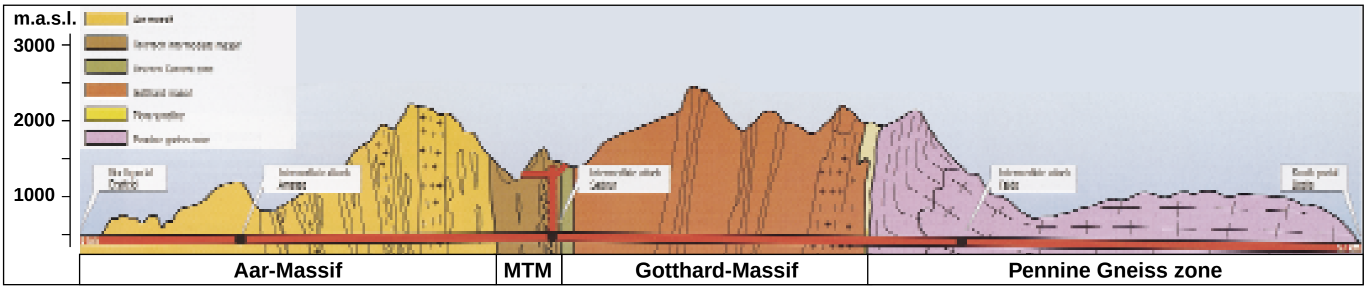


Fig. 3: Longitudinal geological profile of the Gotthard Base tunnel.

his chapter on NATM supports the Pacher curve, in that he writes, it is reasonable, although it could not up till now be verified by measurements or numerical simulations. Thereby, Kolymbas justifies those who have always doubted the reports of Rabcewicz, Müller and others regarding the alleged use of the Pacher curve for model projects of NATM. The realisation of the large recently planned projects in the Alps makes the retraction of the claims of NATM more urgent than ever.

Squeezing Rock in the Gotthard Base

The overall project of the Gotthard Base Tunnel has been described in detail elsewhere (Gehriger 1994, Kovári 1995, Zbinden 1997), which is why we restrict ourselves here to a summary of the basic elements.

The 57 km long Gotthard Base Tunnel forms together with the 34.5 km long Lötschberg Base Tunnel the heart of the AlpTransit project, which was accepted by a clear majority in two Swiss referendums. The project offers the possibility of moving the greater part of the cross-alpine freight traffic from road to rail and to ensure the connection of Switzerland to the European High Performance Rail Network for passenger traffic. The trains will pass below the Alps at a maximum gradient of 1.25 % at depths of up to 2,300 m. Due to the increased speed for passengers and freight in both base tunnels the system is like that of a flat railway system.

For the base tunnel, a system comprising two single-track tunnels with cross connections and the possibility of a change of track was chosen. Two multi-functional stations allow, amongst other things, an emergency stop and can be reached from outside by means of access tunnels and a vertical shaft; the latter serves during the construction phase as a point of intermediate attack. The vertical shaft in the vicinity of the village of Sedrun is situated in a zone of rock with favourable rock properties, has a depth of 800 m and an inner diameter of 7.5 m. The sinking of this blind shaft was begun in 1998 and completed in February, 2000.

Geology

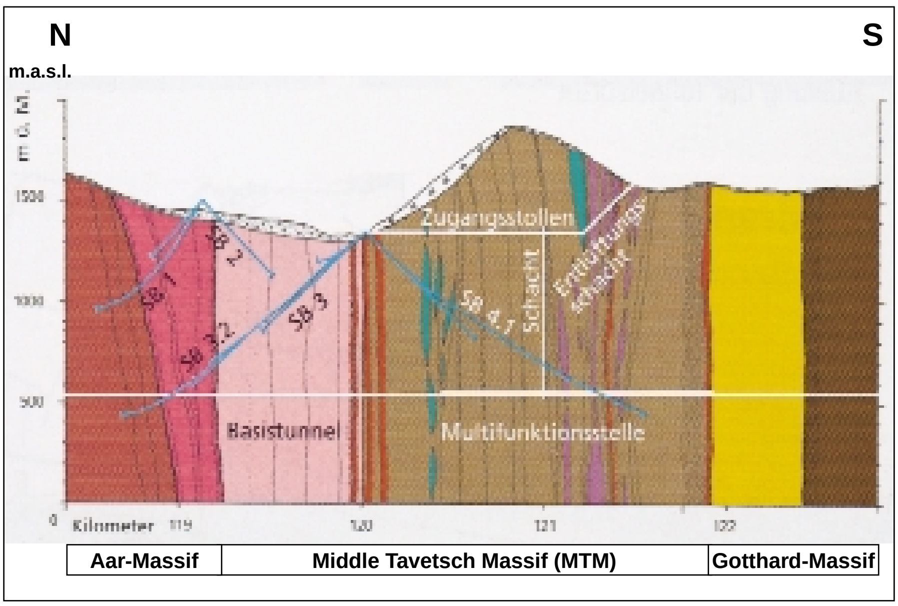
In Figure 3 the longitudinal geological profile is shown. Besides the Aar-Massif, which will also be penetrated by the Lötschberg Tunnel in its southern section, the Gotthard Base Tunnel will pass through further complex crystalline rocks: the Middle Tavetsch Massif, the Gotthard-Massif and the Pennine Gneiss Zone; they consist predominantly of granite, gneiss and schist or slate. Of special importance are the long stretches in the Middle Tavetsch Massif (MTM) and in the Clavaniev Zone (CZ) bordering in the northern part, where rocks of low strength and high deformability are expected (Figure 4).

The old crystalline Middle Tavetsch Massif consists of highly varying rock types: gneisses to a succession of steep zones of soft phyllites and schists. In the period 1995-97 two deep trial boreholes were sunk in the southern and northern parts of the Middle Tavetsch Massif to a depth of almost 2,000 m (Figure 4). They supplement three older shorter boreholes and both reached to below the level of Gotthard Base Tunnel. In the southern part of the Middle Tavetsch Massif, with the start of the construction of the two access tunnels to the top of the blind shaft in Sedrun over a length of 1.3 km, the first *in situ* information could be obtained; it was shown that the rock encountered so far is better than was predicted. In the northern part, on the other hand, a more unfavourable distribution of the rock properties must be reckoned with than originally predicted: about 70% of a zone of 1.1 km width consists of weak rocks, including kakiritic phyllite (a rock reduced to a loose condition during the formation of the mountains; formerly the term mylonite was employed). Our subsequent discussion concentrates on this stretch as well as the conditions in the above mentioned Clavaniev Zone, which consists of clayey kakirite, phyllitic schist and gneiss types of the Aar-Massif.

Sedrun Concept

The construction lot Sedrun stretches over a distance of about 6 km of the tunnel, the tunnel being driven from the foot of the shaft northwards and southwards. This lot also includes the construction of one of the technically difficult multifunctional stations. The construction concept for driving the tunnel tubes in squeezing rock is based on the carefully prepared geotechnical model of the rock, whose elements were determined using the results of trial boreholes. Important findings with regard to the mechanical properties of the rock were also given by the laboratory tests on the core samples. Of particular interest were the drained and undrained triaxial tests with accurate determination and control of the pore water pressures. Despite

Fig. 4: Trial boreholes in the region of the Middle Tavetsch Massif.



the complex structure of the kakiritic phyllite remarkably uniform and useful results were obtained from the total of 39 tests carried out (Vogelhuber 1998).

The geotechnical model corresponds overall to a several hundred metre-long homogeneous, isotropic rock mass with a depth of overburden of 900 m. The rock consists of a series of qualitatively different rock zones, described by an appropriate geotechnical model. For the rock type considered to be the least favourable the following values of elastic (i.e.Young's) modulus E, angle of internal friction (and cohesion c were assumed: E = 2 GPa, (= 23°, c = 250 kPa (drained conditions).

In the following, we leave out a discussion of the computational investigations and mention only that the characteristic line method provides useful information on the combined action of the most important influence factors like material parameters, primary state of stress, lining thickness and the rock deformations. The influence of the pore water pressure, creep and other time-dependent effects as well as the construction stages were not investigated in the computations, since they would require a knowledge of the specific material behaviour (constitutive model) with the corresponding material parameters. The latter, however, could not be determined with sufficient reliability and accuracy for practical purposes. The limitations of numerical methods in tunnelling are not given by the computational methods at our disposal, but by the difficulty of describing the actual stress-strain relations and the primary state of stress in the rock mass.

To recognise the limitations of geological predictions in relation to the average material properties and their variability along the tunnel as well as the inadequacy of the above mentioned statical computational methods have increased the importance of the planning and design work. Selecting the shape of the cross-section, method of construction and operation as well as support measures should be made such that even in the poorest rock zones a high degree of mechanisation for the excavation work is possible. By "poorest" zones is meant the rock regions in which, as a consequence of the excavation work, especially large deformations (inward movement of the face, reduction of the cross-section) or by their retardation or prevention the development of extremely high rock pressures can be expected. The project engineers (Ehrbar & Pfenninger 1999) were responsible for the task, based on the available information, of defining a length of the "poorest" zone, to represent it in a model and to design the corresponding normal cross-section including the construction concept. In better rock conditions the concept of the chosen procedure should retain its validity, in that the individual measures should only need to be adjusted in their intensity (reduction).

The design for the Middle Tavetsch Massif and the Clavaniev Zone has in the order of their importance the following elements (Figure 5): circular tunnel cross-section, full face excavation, uniform systematic anchoring of the face, deliberate over-excavation to accommodate the convergence, steel arch linings closed to a ring with sliding connections (Toussaint-Heintzmann), uniform radial anchoring around the cross-section as well as a closed ring of shotcrete lining in the region behind the face.

In their heaviest size the steel arch linings are chosen such that at uniform convergence two rings, one lying within the other, are given, whereby both a considerable lining resistance and a high level of safety against lateral buckling results. The shotcrete lining is to be applied after the closure of the steel arch linings and the full exploitation of the estimated convergence, respectively, to prevent a further reduction in cross-section of the tunnel opening. Lances in the roof region and the immediate support of the face guarantee safe working conditions. The length of the anchors in the face are at least 6 m, whereby this is achieved by overlapping the original 12-18 m long anchors. The final lining with a thickness of maximum 1.20 m of unreinforced cast-in-place concrete follows the tunnel excavation at a distance of about 300 m. Its dimensioning is based on the assumption that the temporary support in the course of the long operating life (roughly 100 years) may completely loose its statical function due to corrosion. Besides the high rock pressures the final lining has to withstand a water pressure corresponding to a height of about 100 m. Detailed investigations showed that a top heading excavation could not be carried out in the most unfavourable rock zones, which is why only full-face excavation was considered further. To back up this decision it was possible to draw upon useful Italian experience (Lunardi 1998 & 2000, Hentschel 1998).

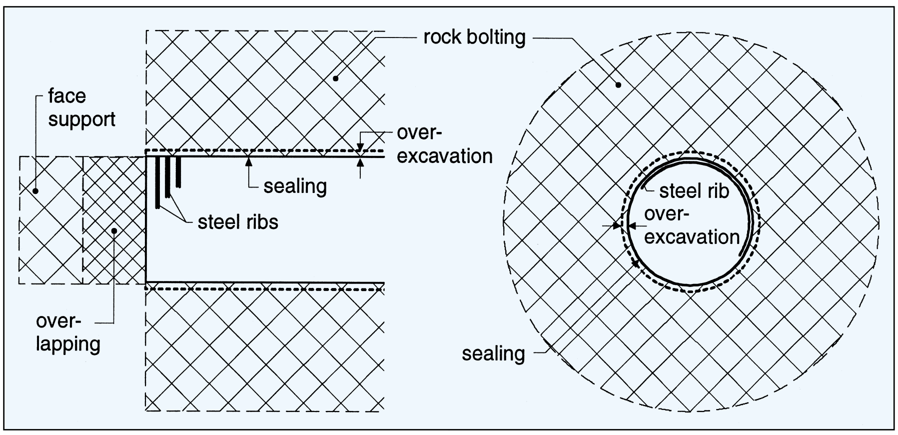
The critical hazard scenarios, which will apply to the tunnel excavation in this zone, are collapse (instability) of the face, local spalling of the same, exceeding the planned limiting values of the convergence as well as fall of rock from the roof region. The instability of the face and inadmissible convergence are "announced" by time-dependent and forerunning rock deformations up ahead. The measurement of the distribution of the axial displacement of the face and of the radial displacement in the rock and at the boundary of the excavation provide useful indications for the current (i.e. immediate) assessment of the rock behaviour. The spatial distribution, the amount and the time variation of the rock deformation, depending on the rock conditions encountered, help to define the stepwise use of the planned safety measures. In this respect the characteristic property of squeezing rock – the creep behaviour – turns out to be an advantage for the engineer.

Table 1 shows the dimensions of the

Table 1. Squeezing rock: data on excavation and support measures

Tunnel excavation radius	5.09 – 6.54 m
Over-excavation	0.30 – 0.70 m
Area of full section	81 – 134 m^2
Length of round	1.0 m
Steel fibre reinforced concrete (protective layer)	0.05 m
Steel arch	TH 44/70
spacing	1.00 – 0.33 m
weight per TM	2.5 – 9.4 to
Shotcrete behind face	0.35 – 0.50 m
Rock bolting	
length	8 – 12 m
ultimate load	320 kN
anchor per TM	96 – 288 m
Anchoring of face	
length	12 – 18 m
ultimate load	320 kN
anchor per TM	80 – 210 m
Final lining	0.30 – 1.20 m

Fig. 5: Sketch of the proposed support measures and over-excavation in heavily squeezing rock.



possible modifications of the various support measures to the different geological conditions. The most remarkable feature is the size of the full section (134 m^2), which might result in the most difficult geological conditions. The increase of the excavation radius up to 6.5 m is necessary because of the large over-excavation for achieving the convergence, with a thick shotcrete lining and a greater thickness of the inner lining. From Figure 6 the extra measures for material and time planned for the most unfavourable rock zones can be clearly seen. The corresponding tunnel cross-section is shown in Figure 7.

Final Remarks

In the construction lot Sedrun of the planned Gotthard Base Tunnel lengthier stretches of squeezing rock are expected. Due to over-excavation and the time-consuming and expensive support measures the rate of excavation will drop to 1 m or less per working day, which is why this lot could decisively influence the length of construction time for the whole tunnel. In the design and construction, therefore, great importance was attached to the possibilities of using high mechanisation. The best possibility here is given by full-face excavation, since more working space is available for the use of high performance equipment. Excavation in full section, however, is alone for statical reasons necessary, since in very difficult rock conditions partial excavation, as for example top heading advance, could lead to uncontrollable construction situations. Using the method described here the systematic support of the face for providing safety against collapse and safe working conditions forms a major component of the overall construction method.

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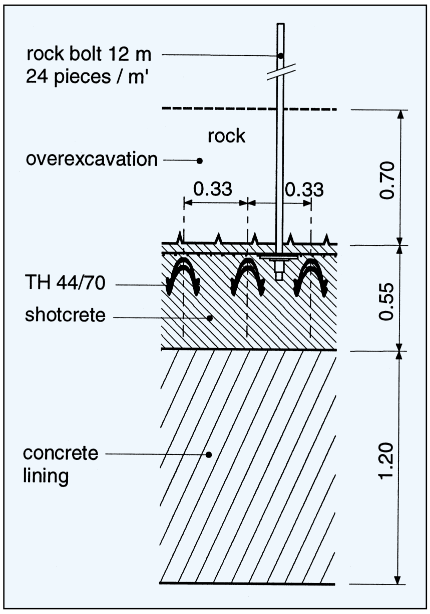

Fig. 6: Section through the temporary and permanent linings normal to the tunnel axis (least favourable rock zone) after complete exploitation of the estimated convergence.

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Fig. 7: Sketch of the tunnel cross-section for over-excavation and the lining thicknesses after completely exploiting the estimated convergence.