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THE NEW AUSTRIAN TUNNELLING METHOD (NATM)

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SUMMARY

The New Austrian Tunnelling Method (or expressed in German 'Die Neue Österreichische Tunnelbauweise' (NÖT)) was developed in the late 50s and at the beginning of the 60s. It was originally developed for weak ground, i.e. where the materials surrounding the tunnel require rock supporting works because they are overstressed. The method is by many regarded as synonymous with shotcrete because this method of rock support plays an important role. In practice, NATM involves a combination of several tunnelling aspects from ground characterization via rock mechanics and tunnel design to construction principles, rock support design and monitoring during tunnel excavation. The main principle of the method is, however, utilization of the bearing capacity of the ground surrounding the tunnel. This is practically done by letting the weak ground around the tunnel deform in a controlled way by applying a flexible rock support. Later, when the rate of displacement is less than a specified limit, the permanent rock support is installed, designed to withstand future loads. Where water shielding is required this is often incorporated where the permanent support is performed as concrete lining.

SAMMENDRAG

Den nye østrerrikske tunnelmetoden - 'Die Neue Österreichische Tunnelbauweise' (NÖT) - ble utviklet i slutten av 50-årene og begynnelsen av 60-årene. Metoden er spesielt utviklet for tunneldrift i svak grunn, dvs. under forhold der bergtrykket overskrider berg- eller løsmassenes fasthet slik at massene deformeres - trykkes - inn mot tunnelen ('squeezing rock').

Metoden som bedre kan benevnes byggemåte ('Bauweise') er sammensatt av de viktigste elementene som inngår i tunnelbygging; fra bergmekanikk gjennom kontraktsutforming til driveprinsipper og instrumentering. Det som imidlertid ansees som viktigst i NATM, er erkjennelsen av at berg- eller løsmassene rundt en tunnel kan oppta en større eller mindre del av bergtrykket. Dette oppnås i svak grunn ved at en lar massene rundt tunnelen deformere seg kontrollert ved at det utføres en initiell (på stuff) sikring som hindrer rasutvikling, men tillater visse deformasjoner. Det er hovedsaklig bolter og sprøytebetong som inngår i denne sikringen. For å holde kontroll med deformasjonsutviklingen der disse er store, foretas instrumentering på visse steder innover i tunnelen; ofte måles også spenningene i sprøytebetongen. Permanent sikring som foretas senere når deformasjonsutviklingen viser foreskrevet avtagende tendens, dimensjoneres for fremtidige mobiliserte laster. Ofte inngår nødvendig vannskjerming som en del av denne (i trafikktunneler og lignende).

1. Introduction

"It is much more skilful to prevent rock load than to handle it."
Franz von Rziha (1872)

This important statement made more than a hundred years ago plays a most significant part of the New Austrian Tunnelling Method (NATM). NATM is a concept, or more precise, a mixture of design, contracting, excavation and active use of rock support experience. Müller (1978) concludes that "the NATM is rather a tunnelling concept than a method, with a set of principles, which the tunneller tries to follow". These features have been systemized into the NATM conception where the different parties involved have worked out a splendid cooperation.

The method was developed by L. von Rabcewicz, L. Müller. and F. Pacher from 1957 to 1964. It is essentially an empirical approach, evolved from practical experience. In practice, NATM involves the whole sequence of tunnelling aspects from investigation during design, engineering and contracting to construction and monitoring. An overview of NATM will therefore include most features involved in the execution of a tunnelling project and tends to be a comprehensive work on tunnelling. In this paper some of these elements have been described in more detail, while other, such as tunnel excavation - which may be important to many NATM applications -, is only briefly included. Neither is another important feature of NATM dealt with, namely, the accumulation and active use of construction experience.

In the author's opinion the principles are only a part of the method as they form the base for the design, but they are so closely linked to practical execution of the tunnel construction that NATM should rather be regarded as a tunnelling system.

2. Development of the New Austrian Tunnelling Method

Central Europe has long traditions in tunnel construction. A number of large tunnel projects had been already completed in the ninteenth century (St. Gotthard, Arlberg and Simplon railway tunnels). From these and other tunnelling projects several adverse tunnelling features, many of them unexpected, were encountered. They sometimes caused enormous construction challenges and problems. Among them the squeezing effect in weak ground had often been a significant problem in alpine tunnelling. Fig. 1 shows an example of the 'old Austrian tunnelling method' where heavy rock support is installed successively in several headings and drifts; a method that was very time-consuming and expensive.

The need for improved tunnel excavation and rock support techniques has, therefore, existed for decades. Several pioneers in Austria have made important field observations which have contributed to develop the science of tunnelling. Between the two World Wars the development in tunnel engineering and design in Europe was led by Professor Stini (1950); his textbook on tunnel geology includes a classification of rock masses, and very detailed and well documented treatise of adverse conditions in tunnelling.

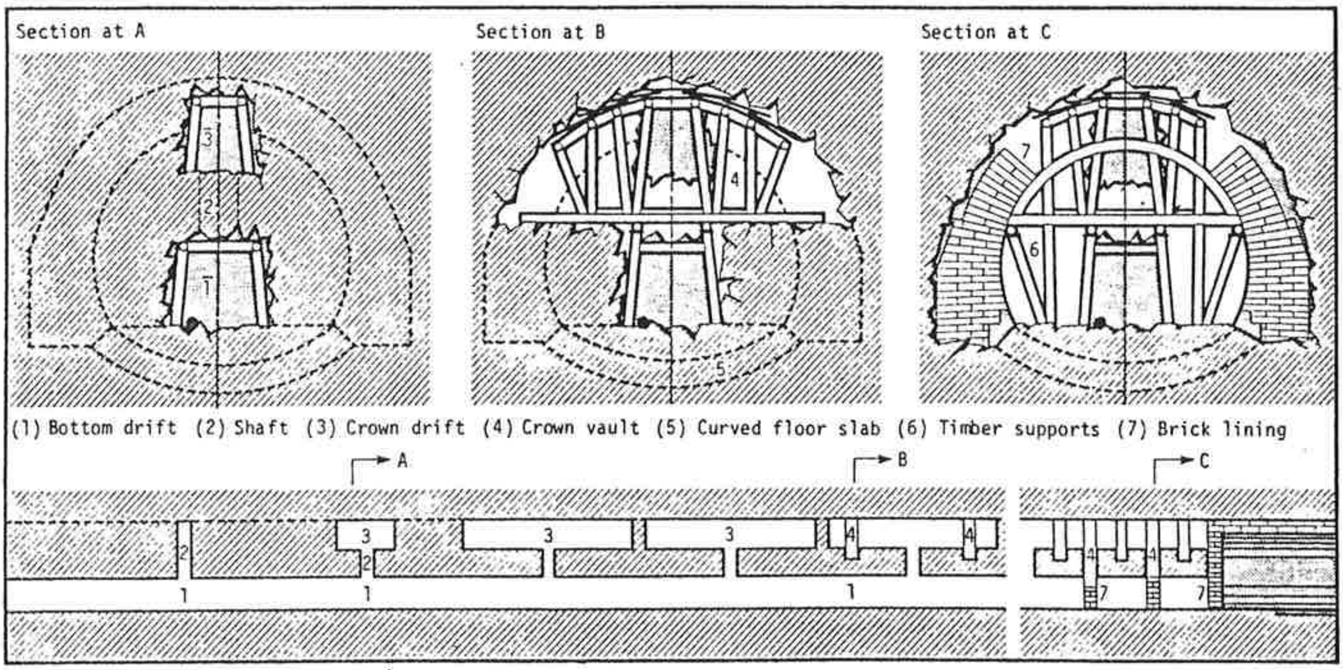


Fig. 1. Example of the old Austrian tunnelling method where a rigid tunnel support is installed step by step in several sections of the tunnel profile. (After Braun, 1980).

Later, the well known 'stand-up time' classification system was presented by Lauffer (1958). The very important contribution of this was the emphasis of the significance of the time an opening can stand unsupported in different qualities of rock masses related to the "active span". The active span is the width of the tunnel or the distance from support to the face in case this is less than the width of the tunnel, as shown in Fig. 2.

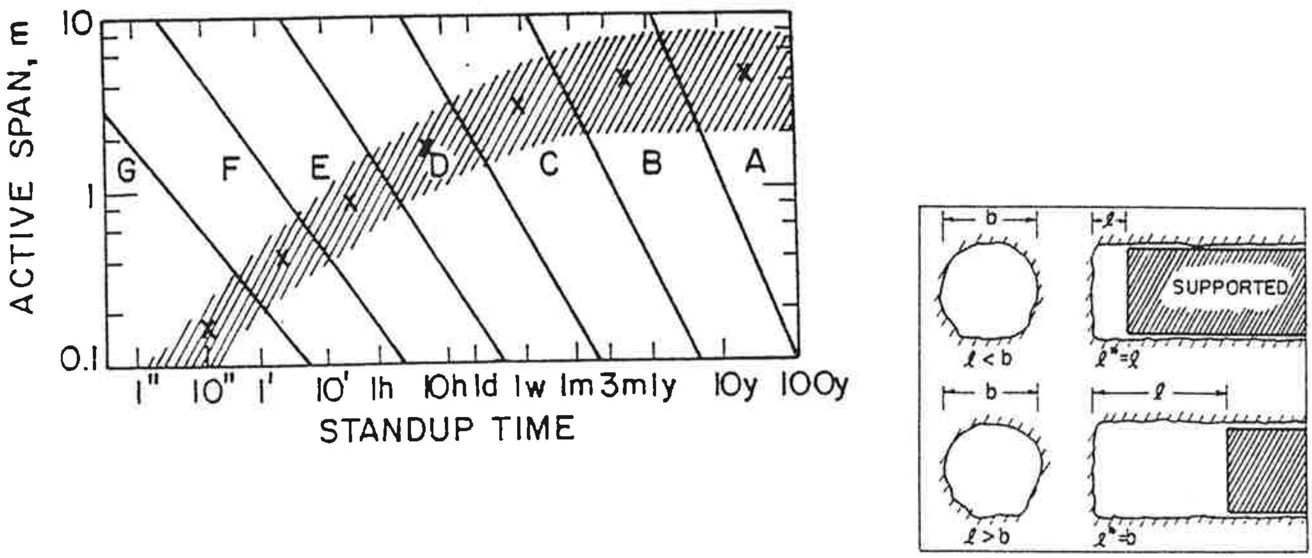


Fig. 2. Left: Active span versus stand-up time. "A" is best rock mass, "G" worst rock mass. Shaded area indicates practical relations. Right: Definition of active span (l^* = l for upper, l^* = b for lower example). (After Brekke and Howard, 1972, based on work by Lauffer, 1958.)

The stand-up time diagram by Lauffer is based on a classification of the behaviour of rock mass used in Austria at that time. The main point that should be made with regard to this chart is that an increase in tunnel size leads to a drastic reduction in stand-up time since the allowable size of the face obviously must be related to the allowable active span. Thus, while a pilot tunnel may successfully be driven full face through a fault zone it may prove impossible in terms of stand-up time to drive a large heading through the same zone, even with the help of spiling and breastboarding. Lauffer has not, however, given a further description in his paper the type of rock mass conditions the various classes represent.

At the same time Prof. von Rabcewicz in collaboration with Müller and Pacher made developments towards new principles for tunnel construction. To separate this from the earlier tunnelling practice it was called the 'New Austrian Tunnelling Method' (NATM) or in German 'Die Neue Österreichische Tunnelbauweise' (NÖT).

The development of NATM made use of earlier experience gained from decades of tunnelling but has taken advantage of the new support technology in rock bolting and sprayed concrete (shotcrete) that was made available in the late '50s and at the beginning of the '60s. Officially, the NATM was introduced by Rabcewicz at the 13th Geocolloquium in Salzburg 1962. This new trend in Austrian tunnelling gained national attention in 1964 when it was applied during the construction the Schwaikheim tunnel, under consulting guidance of Rabcewicz and Müller. International recognition was achieved in 1964 when Rabcewicz published a paper on NATM in the Water Power magazine.

3. The main principles of NATM

It is important to note that NATM has been developed for tunnelling in weak ground. Weak ground is here defined as material which, in tunnelling, requires the use of structural supports, either to re-establish equilibrium or to limit displacements around the tunnel. The rock material itself may be soft or hard. According to Rabcewicz (1975) the goal of NATM is:

> "To provide safe and economic support in tunnels excavated in materials incapable of supporting themselves - e.g. crushed rock, debris, even soil. Support is achieved by mobilizing whatever humble strength the rock or earth posesses.".

He further explains:

> "It uses surface stabilization by a thin auxiliary shotcrete lining, suitably reinforced by rockbolting and closed as soon as possible by an invert.".

As a part of NATM 'the dual-lining support' (initial and final support) for tunnels was introduced. This is the concept of letting the rock and the intial support deform before the final or permanent support is applied so that loads are reduced. Rabcewicz (1975) pointed out three main principles of NATM:

1. It relies on the strength of the surrounding rock as an integrated part of the tunnel support. This is done by inhibiting rock deteroriation, joint opening, and loosening due to excessive rock movements.
2. It uses protective measures like supporting tunnel walls with shotcrete and installing rock bolts in unstable rock. In many cases, a final support by inner lining is not needed - i.e. for water conduits, short road tunnels.
3. It involves intallation of sophisticated instrumentation at the time the initial shotcrete is sprayed, to provide information to design the final support.

Later, other important features have been introduced, such as contractual arrangements, excavation procedures and more advanced design methods.

NATM features a qualitative ground classification system in which the ground is described behaviourally, and the rock mass is allocated a ground class in the field, based on field observations. The classification has been included in the Austrian standard Önorm B 2203 as further described in Chapter 8. New projects are classified based on previous experience from tunnelling in the region and a detailed geotechnical investigation. An example of this is shown in Fig. 3.

In practice, the NATM combines ground conditions, excavation procedure, and tunnel support requirements. It is basically a 'build as you go' approach based on monitoring, backed by theoretical considerations.

4. Design principles applied in NATM

The main achievements and contributions in NATM are the introduction of systematic rock support and in-situ measurements based on rock mechanics theories. This makes use of a ground quality classification divided into 7 groups which contain principles of suitable rock support. Where the amount and system of rock supporting works should be better documented the ground response rock support interaction curves are often used. A basic principle in the design of rock support is, as mentioned earlier, to take advantage of the load-bearing capacity of yielding rock masses surrounding the tunnel. These principles are described in this chapter.

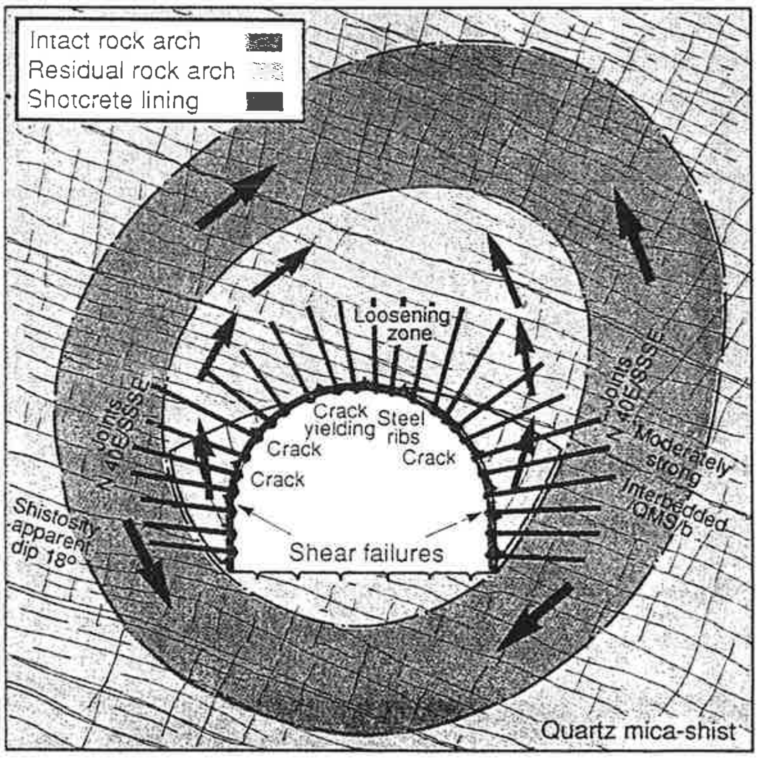
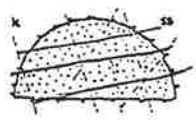
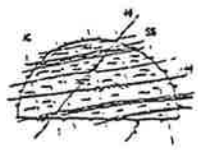
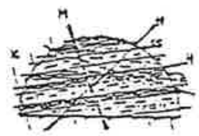
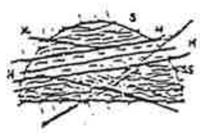
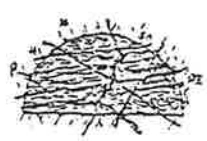


Fig. 4. The various zones around a tunnel. (after Hagenhofer, 1991)

Fig. 3. ***Example of ground classification developed for the Loktak project. (after Golser et al., 1981).***

Type	Rock Mass Reaction	Rock Mass Characterization Structure	Chem. Feature	Water	Rocktype	Structure N (top heading) S	Support	Exc	Geomechanical Behaviour
	1	2	3	4	5	6	7	8	9
II	stable to slightly friable	massive bedded wide joint spacings slightly jointed	chemical intact local dis-integration	local water-drops heavy rain no influ-ence	SANDSTONE		support in roof fullface	BLASTING	uniaxial compressive strength of rock mass (σgd) is higher than tangential border stress σt, permanent equilibrium is reached by: type I - local provisions, additional provisions in popping rock (high primary stress) type II - strength-ening of the rock arch in the roof.
III	friable rock	medium bedded medium joint spacings, locally clay-filled fissures locally shattered	chemical intact, local dis-integration occasional kaolinite films	moder-ately switting, water of imbibi-tation, low joint-water-pressure	SANDSTONE SILTSTONE + s u b o. s h a l e		support in roof and side walls fullface	BLASTING MECHANICAL EXCAVATION AM 50	
IV	moderate-ly squeezing rock	close bedded, shistose, close jointed, local mylonites and clayfilled fissures crushed zones	partly chem. alterated kaolinite films swelling clay minerals occasionally pyrite	switting seepage water of imbibi-tion, joint water-pressure	SILTSTONE + s u b o. s h a l e SHALES + s u b o. s i l t s t. + s d s t.		support in roof and side walls, sealing of face invert closure fore-poling ex. in st.		limited strength of rockmass at circum-ference is reached or exceeded by secondary stresses, caused by stress rearrangements. systematic strength-ening of the rockarch with or without in-vert arch is required.
V	plastic-squeezing and swelling rock	foliation closely jointed mylonites sec. joint fillings slicken-sided	heavy dis-integration and alter-ation kaolinite films swelling clay minerals occasionally pyrite	local drop-water high influ-ence on bond strength of rock mass,			support in roof and side walls, sealing of face invert closure fore-poling ex. in st.		the strength of the rock mass is consi-derably affected by stress rearrangements rockmass behaves plastic and pressure exerting. V - medium VI - highly causing high rock pressure from all directions particu-larly perpendicular to primary principle compressive stress (side pressure) and bottom uplifts, imme-diate protection of all exposed rock sur-faces and invert arch required.
VI	highly plastic squeezing and swelling rock	plastic shales mylonites crushed slicken-sided with lineation graphitic films internal folding	highly dis-integrated kaolinite films swelling clay minerals	local drop-water high influ-ence on bond strength high swelling pressure			support in roof and side walls sealing of face invert closure fore-poling ex. in st.		

Legend: S = foliation; SS = beddings plane; K = joints; Z = shattered and crushed zone; M = mylonitic zone; P = plastic shales; H = slickensides; ex. in st. = excavation in steps

4.1 Utilization of the load-bearing capacity of the ground

An important property of rock masses is their ability to dilate or bulk as they yield close to a tunnel in *weak ground*. Several authors (among others Rabcewicz, 1964 and 1975; Pacher, 1975; Brown, 1979; Ward 1978) have stressed the importance of allowing the rock mass to dilate to some extent by yield and crushing, so that its potential strength can be fully mobilized. The high ground stresses close to the tunnel dissipate and the displacements do not extend far because the rock bulks (i.e. increases in volume) in a limited zone, see Fig. 4. By this the surrounding rock mass is transformed from a loading body into a load-carrying element and a reduced support is needed to confine the unstable ground close to the tunnel.

An important observation by Rabcewicz was that strong lateral ground stresses developed typical shear failures (and not flexion), as shown in Fig. 5. From this he saw the possibility to design the support as a thin lining to stop the detrimental breaking-up or loosening of the surrounding rock. Thus the opening up of fissures to cause detachment of rock particles is prevented, which improves the self-bearing capacity of the ground. This theory was verified in 1965 by model tests.

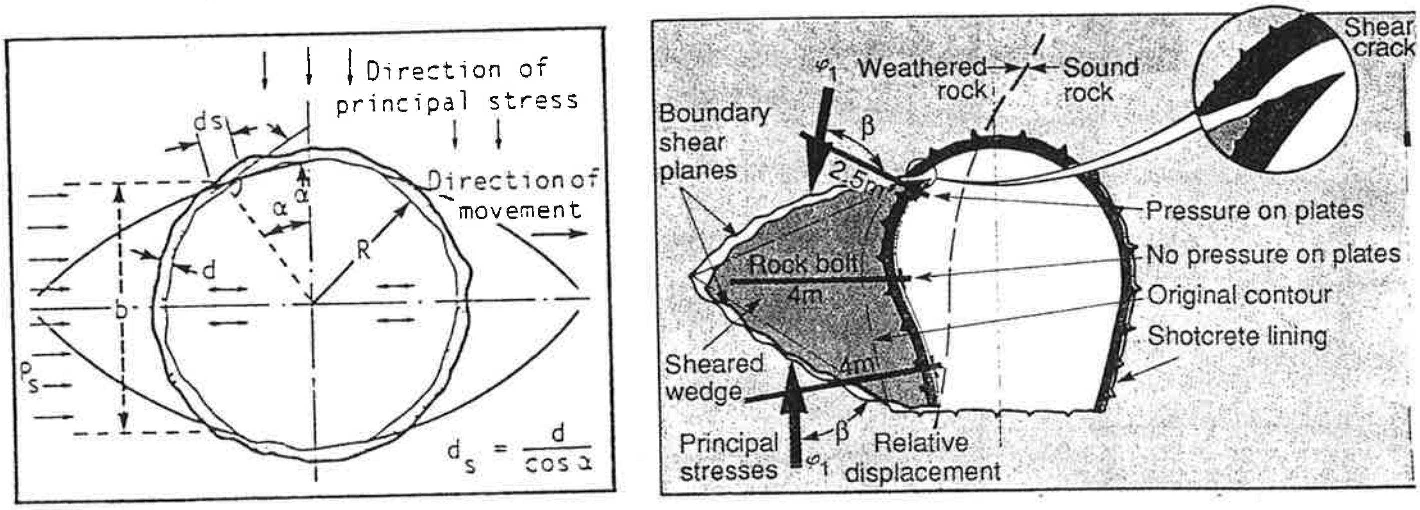


Fig. 5. Principles of typical shear failure phenomena caused by high lateral stresses shown as two examples. This feature is often used in NATM design (Rabcewicz and Golser, 1973). An important feature of shear failure is that it seldom endangers the workmen or interfers with the tunnelling operations. This gave Rabcewicz the idea to introduce an initial support to reduce or stop the loosening of the rock in the tunnel surface. (The left figure is after Sauer, 1988; the right after Hagenhofer, 1991.)

This concept of a systematic utilization of the inherent strength of the soil or rock mass surrounding the tunnel is by many regarded as the main feature in the NATM. It is practically achieved by allowing the rock masses around the underground opening deform in a controlled way. Initial and final support have therefore mainly a confining function; their main purpose is to establish a load-bearing ring to stabilize the rock masses that deform. It follows that the support must have suitable load-deformation characteristics and be installed at the right time. The use of this requires a knowledge of the inter-relationships between ground deformation and load, support deformation and load, and time.

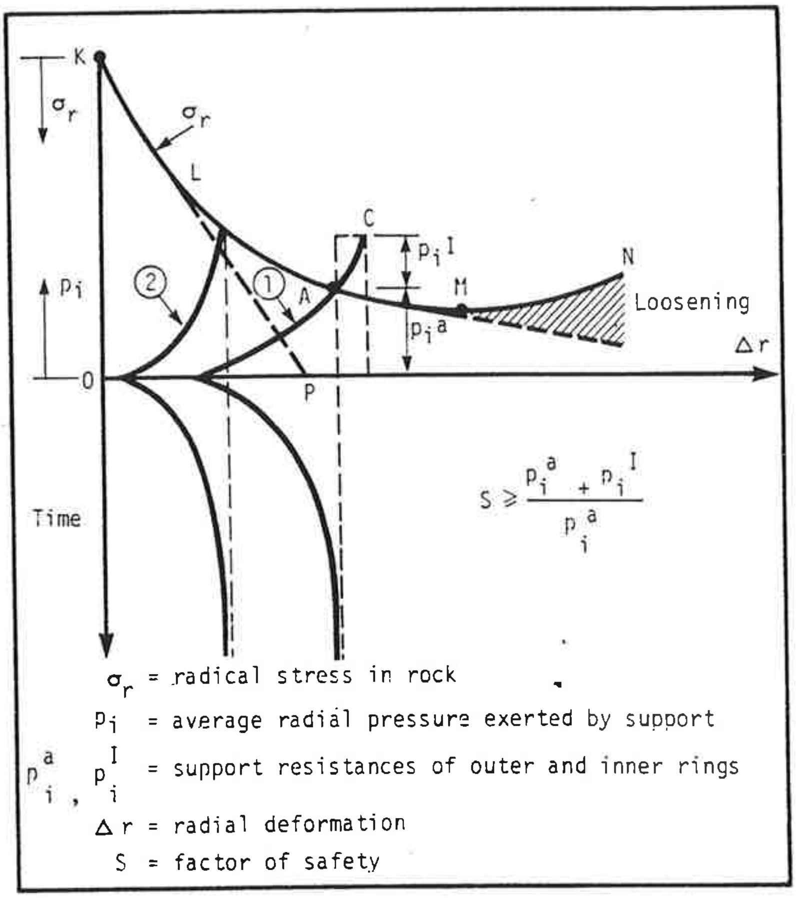


Fig. 6. An example of a ground support interaction diagram or a Fenner - Pacher curve, (after Brown, 1981). The numbers 1 and 2 show two different support systems installed at different times. The stiffness and timing of the support is further described in Sections 4.2 and 5.2.

The Fenner-Pacher curve applied by the Austrians for design is a ground response curve for rock - support interaction, Fig. 6. The curve provides a tool to optimize rock support: to help determine a favourable time for installation and an appropriate stiffness. Seeber et al. (1978) have worked out characteristic diagrams for different rock mass qualities exposed to various rock stresses.

The properties of the various rock supporting methods to be applied in the ground response curves are more complex than they appear at first sight as there is a great variety of systems and combinations of these in use. Hoek and Brown (1980) have given some information on the characteristics of various supporting types.

4.2 The timing of rock support installation.

The ground response curve in Fig. 6 indicates that the timing of rock support installation is an important factor for a favourable mobilization of the inherent strength of the rock mass. It is also important for an optimal dimension of the amount of rock support:

- if the rock support is installed too early, a heavier support is required to carry the resulting rock mobilized, while
- an installation made too late may cause deformations of the rock masses surrounding the tunnel that result in loosening and failures.

Since a tunnel statically is a thick-walled tube it is crucial in very weak ground to quickly close the invert using a lining. No section of the excavated tunnel surface should be left unsupported even temporarily in such ground. However, support should not be installed too early since the load-bearing capacity of the rock mass would not be mobilized; the rock mass must be permitted to deform sufficiently before the support takes full effect.

Braun (1980) stresses that, particularly in deep tunnels, the timing of the rock support (bolts and shotcrete) installations is extremely important. It is, however, difficult to predict the time factor and its variations during tunnelling even for experienced rock mechanics and mining engineers. In this connection NATM recommends the use of tunnel support measures to avoid undesirable deformations of the surrounding rocks to occur. The optimal NATM involves also additional verification calculations carried out during the execution.

5. Excavation and rock support principles

The practical execution of NATM involves a close cooperation between tunnel excavation and rock supporting works and that these two operations are planned and designed according to the ground conditions.

5.1 Excavation principles

Golser (1979) stresses that cost-effective tunnelling requires use of rapid, modern equipment for full face excavation in large cross sections. It might, however, be necessary for stability reasons to excavate smaller sections, for example an upper heading and benching, alternatively the arch, the core and the invert arch of the tunnel section.

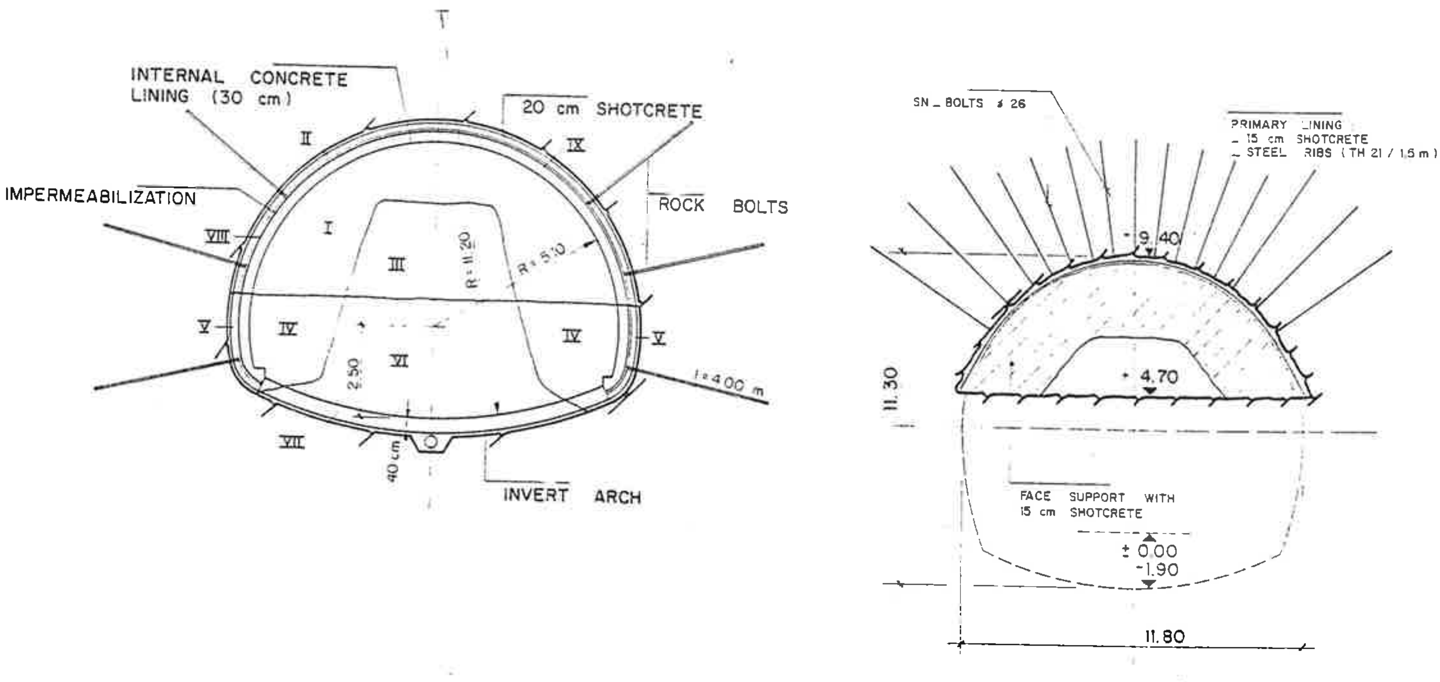


Fig. 7. Example of a sequential tunnel excavation, based on Amberg and Cristini (1986). The left figure shows a typical cross section and excavation sequences; the right shows excavation of the top heading of the tunnel.

In especially unstable ground, a further subdivision of these sections can be imperative, excavating alternatively the different zones illustrated for example in Fig. 7 from a highway tunnel close to Florence, excavated in clayey schists where the following excavation procedure was applied:

- In arch (I) leaving a central core in order to shore the walls and the arch. Immediately after excavation, execution of a first layer of reinforced shotcrete, collocation of light ribs and a further increase of the shotcrete thickness until final value (II)
- Excavation of central core (III) and side trenches by stage (IV), prestrengthening with a reinforced shotcrete lining and steel ribs (V).
- Excavation of the trench (VI), execution of the invert arch.

The stages above follow each other very closely in order to ensure the closing of the lining within a distance of 15 to 25 m from the face.

5.2 Rock support principles

The importance of a deformable rock support has been mentioned earlier. Rabcewicz (1975) stresses that it must be neither too stiff, nor too flexible:

- A stiff rock support will be carrying a larger load because the rock mass around the opening has not had the possibility to deform enough to bring the stress peak longer into the surrounding rocks (see Fig. 6).
- If the support is too flexible the deformation may become too large and unsafe conditions may arise.

This generally requires a support system consisting of systematic rock bolting and shotcrete. Whatever support system is used, it is essential that it is placed and remains in intimate contact with the surrounding ground and deforms with it. The two stages of rock support have been described by Rabcewicz and Golser (1973). They are carried out as:

- *The initial support* is often carried out as an outer lining designed to stabilize the rocks during excavation. It consists mainly of shotcrete, systematically bolted and reinforced by additional steel ribs if required. In addition a closing of the invert is carried out in very weak ground.
- *The final support* is often carried out as a concrete lining. It is generally not carried out before the deformations of the initial support have reached an acceptable, decreasing trend.

The initial support can partly or completely represent the total support required. It may consist of a thin layer of shotcrete combined with rock bolts. The second, final lining inside the initial one may be necessary for structural reasons:

1) when the initial lining is stressed beyond its elastic limit or
2) when squeezing or swelling from time-dependent loads will exceed its bearing capacity. A second lining may also be required for waterproofing.

The dimensioning of the final support is based on assessments based on results of systematic measurements of stress in the primary support element and/or deformations of the tunnel surface and the ground surrounding the tunnel. Where necessary, caused by the ground conditions, the support element is designed to absorb large movements, as shown in Fig. 8.

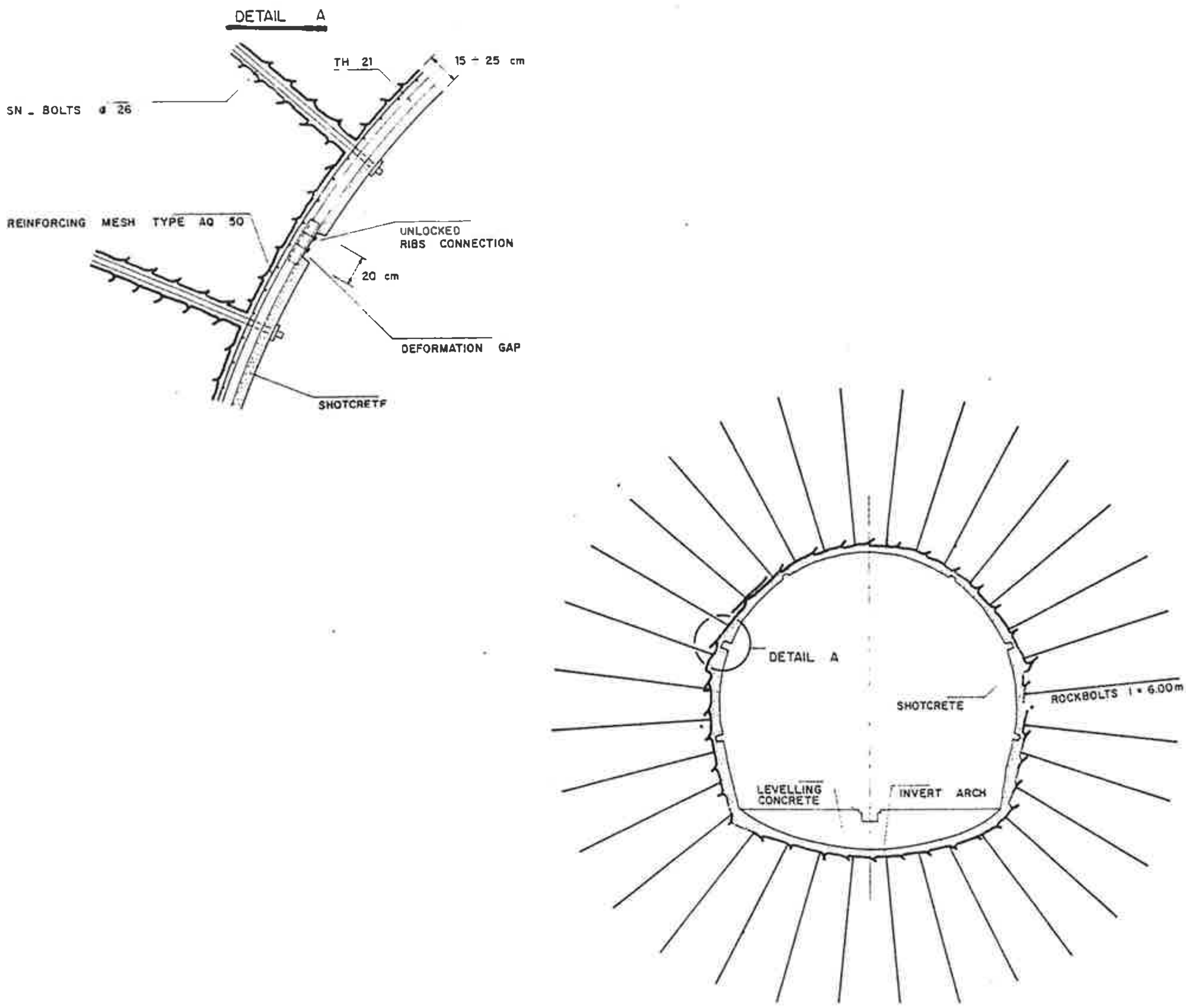


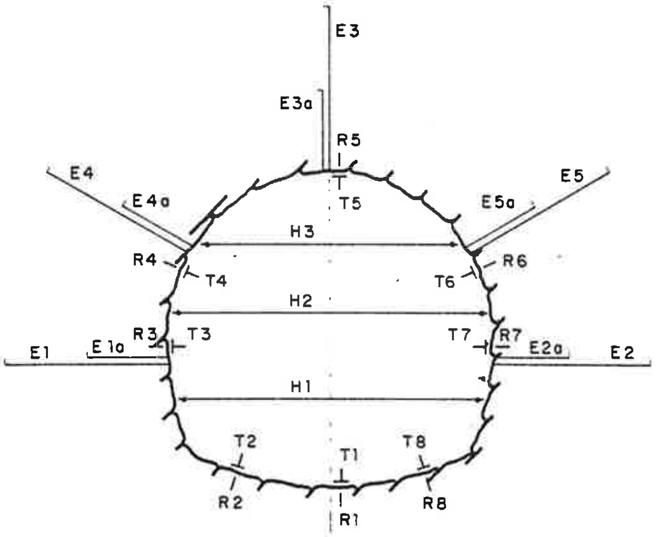
Fig. 8. *Where large deformations occur in the tunnel walls it is important to have a flexible lining. The figure is an example from the Taurn tunnel where the initial lining (by shotcrete and rock bolts) has been designed with a longitudinal joint. (after Amberg and Cristini, 1986)*

In order to investigate the real behaviour of the rock when the excavation is completed, the NATM is based on systematic in-situ measurements primarily of deformations and of stresses. Following the progress of the deformations, it is possible to recognize early enough an unacceptable trend and to act accordingly. This is further described in the next Chapter.

5. Instrumentation and monitoring during construction

Tunnelling in weak ground is, according to Ward (1978), much more of an art than soft ground tunnelling; that is why it is important to monitor the performance of the surrounding ground. Construction monitoring in these conditions is, therefore, not research, but a simple, essential and the only means of knowing whether construction is proceeding satisfactorily or whether it is likely to become a hazard to health, safety and further progress.

The measurement of deformations and stresses is the only way of really knowing, even if approximately, the mechanical characteristics of the rock mass and its behaviour during the excavation. Therefore, the measurement of deformation as a function of time has long been a principal means of studying rock mechanics phenomena. Controlled deformations of the underground opening is a provision required in NATM for the surrounding ground to develop its full strength safely, as excessive deformation will result in loss of strength. The significance of this measure, which was emphasized early by Rabcewicz, has steadily increased as the NATM has become more widely applied.



R1 -R8	Radial pressure pads
T1 -T8	Tangential pressure pads
H1, H2, H3	Convergency measuring lines
E1 - E5	Long extensometers (6 m)
E1a - E5a	Short extensometers (3 m)

Fig. 9. Examples of instrumentation in a tunnel (after Amberg and Cristini, 1986).

During the planning stage the location of measurement sections along the tunnel, is decided. The instruments are installed at the time when the initial support is placed. Since the readjustment of stresses generally takes a very long time, it is often essential from both the practical and theoretical point of view to also measure the deformations and stresses in the inner lining. The following measurements can be made, Figs. 9 and 10:

- Convergence measurements, usually with horizontal wires (giving only relative distances).
- Levelling of crown and invert (giving absolute distances).
- Measurement of displacement of rock mass with long extensometers, installed radially with different measurement positions.
- Placement of anchors to work in a manner similar to the extensometers.
- Monitoring of original stress in the rock mass with stress cells at the perimeter, placed radially and tangentially.
- Monitoring of the loading with stress cells placed radially in the concrete.

These observations recorded by sophisticated instruments yield information for convergence control and, if possible, the build up of a load in the support. The results are used mainly for the following purposes:

1 The stress-time or deformation-time graphs worked out from the measurements give a high degree of safety, as any undesired situation can be recognized in time before it develops into a possible dangerous, unwieldy problem.
2 In cases where the measurements show that the lining has been over-dimensioned, the lining can immediately be reduced accordingly when the same or similar ground conditions are encountered during further tunnel excavation.
3 The monitoring is used to decide the time for installation of a possible final lining.
4 The results from the measurements can be used to adjust ground response curves that have been worked out.

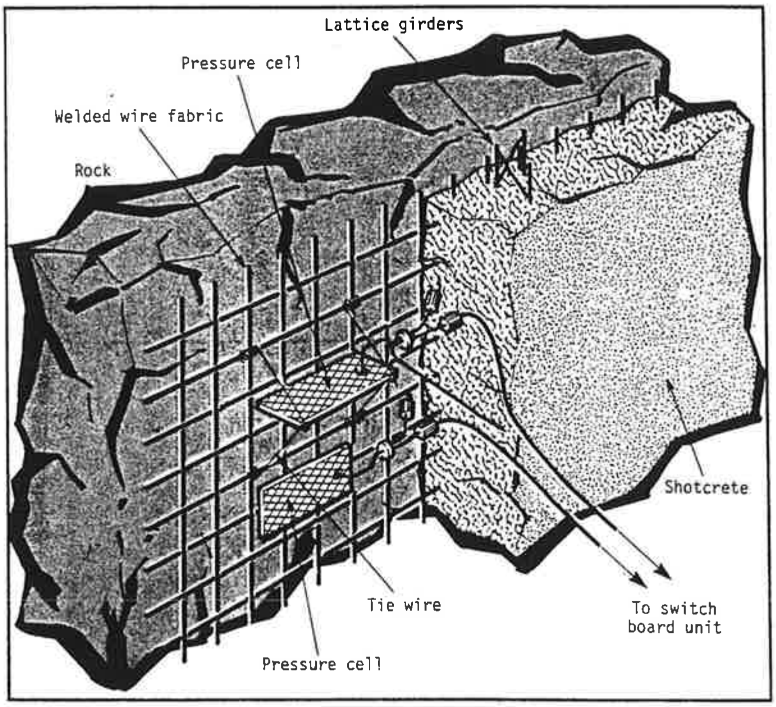


Fig. 10. Example of an instrumentation station (after Martin, 1984).

This information is then related to the characteristics of rock and of the cross section of the opening. When interpreted in an appropriate way, it is possible to adapt the type and dimensions as well as the timing of rock support to the actual ground conditions encountered during the excavation. In this context, it is obviously very important to have an effective calculation model which permits a quick interpretation of the data, and is the best possible representation of reality.

The use of modern in-situ measurements, and the application of advanced rock mechanics, have made it possible to observe the interaction of forces around the excavated opening and to ascertain that the state of equilibrium has been established.

7. The contractual arrangement applied in NATM

The NATM method of tunnelling is futher improved where appropriate contractual arrangements are made. The monitoring measurements on which NATM is based, presuppose that it is possible to make changes in support and construction methods adapted to the ground conditions encountered. This, however, is only possible if there are provisions in the contract that are open for changes during construction. Austrian contractual practice does contain a certain amount of valuable features in terms of sharing risk and decision making, encouraging flexibility in construction methods, and providing simple and equitable arrangements for settling disputes. This is another important feature for using NATM in tunnel construction.

As previously mentioned the contracts in Austria are based on Önorm B 2203 where the ground conditions are classified according to quality and behaviour. In this classification the principles for support are given, as shown in Table III. During excavation the actual rock mass conditions are recorded after each excavation round using the same system. The monitoring program is included in the specifications and bill of quantities.

The level of detail in the pre-construction description depends on the information available from site exploration and experience from earlier tunnels in the vicinity. Brosch (1986) reports that Austrian engineers believe that a qualitative ground classification and contract conditions are inseparable. Clearly, such principles could lead to disputes, but since the contractor is paid on the basis of 'as found' conditions, possible disagreements are minimized. If necessary, an expert 'Gutachter' (appraiser) is usually available at short notice to solve any disagreement.

Payment for support is based on the rock mass classification made during construction. In some countries this is not acceptable contractually, and this is why the method has received limited attention, for example in the United States.

In these various ways, the contract results in a greater sharing of responsibility and risk between owner, designer, and contractor than the contractual arrangements used in many other countries. The use of NATM requires that all parties involved in the design and execution of project - design and supervisory engineers and foremen - must understand and accept the implementation of NATM and adopt a co-operative attitude to decision making and the resolution of problems.

8. The ground classification used in connection with NATM

In Austrian tunnelling practice, the ground is described behaviourally and allocated a ground class in the field, based on field observations. The classification is qualitative without a numerical rating . This system for classification is highly adaptable and its application can be traced back to Lauffer (1958). A comparison with the terms used today and those of Lauffer is shown in Table I.

The qualitative ground description is associated, rather inconsistently, with excavation techniques together with principles and timing of standard support requirements.

Table I. Development of the Austrian characterization of rock masses

Lauffer (1958)	Önorm B 2203 (1983)	Önorm B 2203 (1993)	Suggested English term
A Standfest	F1 Standfeste	A1 Standfest	Stable
B Nachbrüchig	F2 Nachbrüchig	A2 Nachbrüchig	Slightly loosening
C Sehr nachbrüchig	F3 Leicht gebräch	B1 Gebräch	Ravelling
D Gebräch E Sehr gebräch	F4 Gebräch oder leicht druckhaft	B2 Stark gebräch	Strongly ravelling
F Druckhaft	F5 Stark gebräch oder druckhaft	C1 Druckhaft	Squeezing or swelling
G Sehr druckhaft	F6 Stark druckhaft	C2 Stark druckhaft	Strongly squeezing or swelling
	F7 Fließend		Running or flowing

During tunnel excavation the classification of rock masses is carried out at the tunnel working face, where a large proportion of the important parameters used in classification systems often are more or less impossible to establish. Brosch (1986) does not know of any Austrian experience with the common international classification systems (mainly the RMR and Q systems), although he admits that such experience would be a most desirable basis for the further development of engineering geology and would assist Austrian tunnel construction firms in handling foreign projects. A comparison between the NATM classification and the main international systems is given in Table II.

Table II. Approximate connection between NATM, Q system and RMR system (after Martin, 1988).

Austrian NATM system class	term	Q system rating	RMR (geomechanic) system class	term	average stand-up time	rating
F1	stable	> 50	I	very good rock	10 years for 15 m span	> 80
F2	slightly loosening	10-50	II	good rock	6 months for 8 m span	60-80
F3	ravelling	5-20	III	fair rock	1 week for 5 m span	40-60
F4	moderately squeezing	1-10	IV	poor rock	10 hours for 2.5 m span	0-50
F5	plastic squeezing and swelling	0.5-5	IV	poor rock	10 hours for 2.5 m span	30-50
F6	highly squeezing and swelling	0.05-1	V	very poor rock	30 min for 1 m span	10-20
F7	running	< 0.05	V	very poor rock	30 min for 1 m span	< 10

Although there are guidelines in the qualitative NATM clasification, the ground class is mainly determined from individual observations by the engineering geologist (Kleberger, 1992). The application of the NATM classification in ÖNORM B 2203 is shown in Tables III and IV. As seen this classification relates ground conditions, excavation procedure, and tunnel support requirements. The classification, which forms part of the contract, is adapted to a new project based on previous experience and a detailed geotechnical investigation.

Table III. The NATM classification applied in ÖNORM B2203 (1993)

CLASS and TERM	ROCK MASS CONDITIONS	REQUIREMENTS TO ROCK SUPPORT FUNCTION AND/OR EXCAVATION MEASURES
A1 Stable	Elastic behaviour. Small, quick declining deformations. No relief features after scaling. The rock masses are long-term stable.	No need for rock support after scaling. Not necessary to reduce length of rounds, except for technical reasons.
A2 Slightly loosening	Elastic behaviour, with small deformations which quickly decline. Some few small structural relief surfaces from gravity occur in the roof.	Occasional rock support in roof and upper part of walls necessary to fasten loose blocks. The length of rounds might only be limited for constructional reasons.
B1 Ravelling	Far-reaching elastic behaviour. Small deformations that quickly decrease. Jointing causes reduced rock mass strength, as well as limited stand-up time and active span*) . This results in relief and loosening along joints and weakness planes, mainly in the roof and upper part of walls.	Systematic rock support required in roof and walls, and also of the working face. The cross section of the heading depends on the size of the tunnel, i.e. the face can contribute to stability. The length of the rounds must be reduced accordingly and/or systematic use of support measures like spiling bolts ahead of the face.
B2 Strongly ravelling	Deep, non-elastic zone of rock mass. The deformations will be small and quicly reduced when the rock support is quickly installed. Low strength of rock mass results in possible loosening effects to considerable depth followed by gravity loads. Stand-up time and active span are small with increasing danger for quick and deep loosing from roof and working face.	Rock support of the whole tunnel surface is required, often also of the working face. The size of the heading should be chosen to effectively utilize stabilizing effect of the face. The effect of the rock support is mainly to limit the breaking up and maintain the 3-dimensional stress state. The length of the round must be adjusted according to the support measures ahead of the working face.
C1 Squeezing or swelling	"Plastic" zone of considerable size with detrimental structural defects such as joints, seams, shears. Plastic squeezing as well as rock spalling (rock burst) phenomenas. Moderate, but clear time-dependent squeezing with only slow reduction of deformations (except for rock burst). The total and rate of displacements of the opening surface is moderate. The rock support can sometimes be overloaded.	Comprehensive rock supporting works of all excavated rock surfaces is required. The size of the unsupported surface after excavation has to be limited according to the support measures performed ahead of the face. The large deformations require use of special support designs, for example deformation slots or other flexible support layouts. The support should be installed to maintain the 3-dimensional state of stress in the rock masses.
C2 Strongly squeezing or swelling	Development of a deep squeezing zone with severe inwards movement and slow decrease of the large deformations. Rock support can often be overloaded.	

*) Active span is the width of the tunnel or the distance from support to face in case this is less than the width of the tunnel), see Fig. 2.

Table IV. The principles and amount of rock support in the NATM classification (Bieniawski, 1989)

Class	Construction Procedure	Support Procedure: Principle	Crown	Springline	Invert	Face
I	Check crown for loose rock When popping rock is present placement of support after each round	Support against dropping rock blocks	Shotcrete: 0–5 cm Bolts: cap = 15 t Length = 2–4 m Locally as needed	Bolts: cap = 15 t Length = 2–4 m locally	No	No
II	Crown has to be supported after each round Bolted arch in crown	Shotcrete support in crown Bolts: cap = 15 t Length = 2–4 m One per 4–6 m	Shotcrete: 5–10 cm with wire fabric (3.12 kg/m^2) Bolts: Length = 2–4 m locally	Shotcrete: 0–5 cm	Bolts L = 3.5 m if necessary	
III	Shotcrete after each round; other support can be placed in stages	Combined shotcrete—bolted round in crown and at springline	Shotcrete: 5–15 cm with wire fabric (3.12 kg/m^2) Bolts: cap = 15–25 t Length = 3–5 m	Shotcrete: 5–15 cm Bolts: 15–25 t Length: 3–5 m One per 3–5 m^2	Adapt invert support to local conditions	Adapt face support to local conditions
IV	Shotcrete after each round Bolts in the heading have to be placed at least after each second round	Combined shotcrete—bolted arch in crown and springline, if necessary closed invert	Shotcrete: 10–15 cm with wire fabric (3.12 kg/cm^2) Bolts: fully grouted Cap = 25 t Length = 4–6 m One per 2–4 m^2	Same as crown	Slab: 20–30 cm	
V	All opened sections have to be supported immediately after opening. All support placed after each round	Support ring of shotcrete with bolted arch and steel sets	Locally linerplates Shotcrete: 15–20 cm with wire fabric (3.12 kg/m^2) Steel sets: TH21 spaced: 0.8–2.0 m Bolts: fully grouted Cap = 25 t Length = 5–7 m One per 1–3 m	Same as crown but no linerplates necessary	Invert arch ≥40 cm or bolts L = 5–7 m if necessary	Shotcrete 10 cm in heading (if necessary) 3–7 cm in bench
VI	As Class V	Support ring of shotcrete with steel sets, including invert arch and densely bolted arch	Linerplates where necessary, shotcrete: 20–25 cm with wire fabric Steel sets: TH21: 0.5–1.5 m Bolts: cap = 25 t L = 6–9 m One per 0.5–2.5 m^2	Same as crown	Invert: ≥50 cm Bolts: 6–9 m long if necessary	Shotcrete 10 cm and additional face breasting

[a] After John (1978); arrangement by Steiner and Einstein (1980).

9. Strengths and limitations of the NATM

From the first experiences made in the 60s until today, a large number of tunnels have been executed with success using the NATM, some of which were constructed in very difficult ground (marl, graphite-clayey schist, etc.). There are, however, also examples of downfalls and other unpleasent experiences where NATM has been used (as the cave-ins in some German high speed railway tunnels).

9.1 Some of the benefits using NATM

Where the NATM approach has been successfully used in a wide range of tunnelling conditions, the versatility and adaptability of the method have been demonstrated from its basic principles and from the flexibility of the rock support applied (shotcrete and rock bolts) as an initial and final support material. In most of the tunnelling projects large savings have been made as well as time savings compared to traditional tunnelling.

A significant advantage of using shotcrete according to NATM is the possibility of adjusting its thickness to the actual rock mass condition, i.e. by the application of further shotcrete layers or by combining it with rock bolts. As a further reinforcing measure in the NATM, light steel ribs of the channel-section type are used, connected by overlapping joints and fastened to the rock by bolts. The ribs serve primarily as a protection for the tunnelling crew against rockfall and as local reinforcement, to bridge across zones of geologic weakness. The static share of the ribs in the lining resistance is relatively low.

Where NATM is used in conjuction with drill and blast or mechanical excavation, the flexibility of these methods can also be fully utilized. Tunnelling by NATM can therefore be applied for various sizes of tunnel cross sections as well as for various excavation sequences, for example for pilot headings and benching.

9.2 Limitations in the NATM

NATM has its greatest benefit for tunnels constructed in weak ground, i.e. materials that have a lower strength than the rock stresses they are exposed to. This includes also hard rocks exposed to high rock stresses where rock burst or spalling takes place. Tunnels excavated in stable and slightly loosening rock (class A1 and A2 in Önorm B 2203 (1993)) will, however, benefit slightly from use of the NATM concept.

Haak (1987)) and Amberg and Christini (1986) conclude that there are also other ground conditions where NATM is not applicable. These may be rock masses without co-action between the rock blocks, such as in highly jointed, brittle rocks with loss of no cohesion. Here, the interaction between shotcrete and rocks will not take place, excluding an important prerequisite for the NATM: the interaction of a relatively yielding tunnel support with the vault-like bearing rock mass.

Other examples where the use of NATM can be inadequate, or at least very little advisable, are:

- In inhomogeneous rock masses where stress concentrations on the lining can cause sudden outfalls and collapse.
- In some tunnels with high anisotropic rock stresses or loads where the co-action and arching effect do not develop.
- In urban areas where the ground is not allowed to deform in order to avoid building settlements.

At last should be mentioned the rather unsystematic use of geo-data in the ground classification where use of numerical characterization has not been established.

10. What is new in NATM?

The method relies, according to Bieniawski (1989), on the inherent strength (load-carrying capacity) of the rock masses surrounding the underground excavation which contributes to the main component in the rock support. This is achieved by a controlled deformation of the surface of the excavation allowing the inherent rock mass strength to be mobilized. To avoid loosening of the rock mass at the surface, a flexible rock support is installed generally consisting of rock bolts and shotcrete.

The NATM has sometimes been assumed to be synonymous with the use of shotcrete during tunnel excavation, probably because the NATM people often have stressed the great advantages of applying this rock supporting method in weak ground tunnelling.

Of the several principles and features which are followed when NATM is applied during execution of a tunnel project, the main feature is the concept for conservation and mobilization of the inherent strength of soil or rock mass by the application of a self-supporting ring around the tunnel. This effect has been independently advocated and applied by others, also before the development of NATM. It is likely, however, that the NATM was among the first to practically utilize this effect based on theoretical considerations.

The other main achievement of NATM is the practical use of instrumentation and monitoring. Brown (1981) is of the opinion that the early practitioners of the NATM probably were the first to make systematic use of instrumentation as an integral part of their approach to underground construction. This has resulted in significant improvements to the quality of field instruments.

Brown (1981) has summarized the role of NATM as the following:

> "The original applications of the NATM were to tunnels constructed in the Alps in rocks subjected to high in-situ stresses. Many of its essential features are far from new but it must be acknowledged that the originators and developers of the NATM have made a vital contribution to the art and science of tunnelling by encompassing them all in one unified approach."

Also, Muir Wood (1979) is of the same opinion:

> "I do find in Austria a splendid matching and continuity between design and construction."

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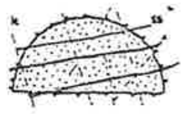
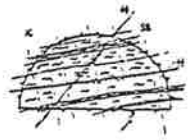
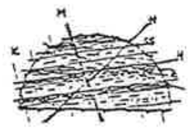
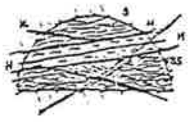
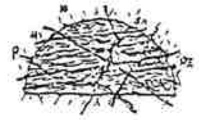
Fig. 3. Example of ground classification developed for the Loktak project. (after Golser et al., 1981).

Type	Rock Mass Reaction	Rock Mass Characterization: Structure	Chem.Feature	Water	Rocktype	N Structure (top heading) S	Support	Exc	Geomechanical Behaviour
	1	2	3	4	5	6	7	8	9
II	stable to slightly friable	massive bedded wide joint spacings slightly jointed	chemical intact local dis- integration	local water- drops heavy rain no influ- ence	SANDSTONE		support in roof fullface	BLASTING	uniaxial compressive strength of rock mass (σgd) is higher than tangential border stress σt, permanent equilibrium is reached by: type I - local provisions, additional provisions in popping rock (high primary stress) type II - strengthening of the rock arch in the roof.
III	friable rock	medium bedded medium joint spacings, locally clay- filled fissures locally shattered	chemical intact, local dis- integration occasional kaolinite films	moder- ately switting, water of imbibi- tation, low joint- water- pressure	SILTSTONE + subo. shale		support in roof and side walls fullface	MECHANICAL EXCAVATION AM 50	
IV	moderate- ly squeezing rock	close bedded, shistose, close jointed, local mylonites and clayfilled fissures crushed zones	partly chem. alterated kaolinite films swelling clay minerals occasionally pyrite	switting seepage water of imbibi- tion, joint water- pressure	SHALES + subo. siltst. + sdst.		support in roof and side walls, sealing of face invert closure fore- poling ex. in st.		limited strength of rockmass at circumference is reached or exceeded by secondary stresses, caused by stress rearrangements, systematic strengthening of the rockarch with or without invert arch is required.
V	plastic- squeezing and swelling rock	foliation closely jointed mylonites sec. joint fillings slicken- sided	heavy dis- integration and alter- ation kaolinite films swelling clay minerals occasionally pyrite	local drop- water high influ- ence on bond strength of rock mass,			support in roof and side walls, sealing of face invert closure fore- poling ex.in st.		the strength of the rock mass is considerably affected by stress rearrangements rockmass behaves plastic and pressure exerting. V - medium VI - highly
VI	highly plastic squeezing and swelling rock	plastic shales mylonites crushed slicken- sided with lineation graphitic films internal folding	highly dis- integrated kaolinite films swelling clay minerals	local drop- water high influ- ence on bond strength high swelling pressure			support in roof and side walls sealing of face invert closure fore- poling ex. in st.		causing high rock pressure from all directions particularly perpendicular to primary principle compressive stress (side pressure) and bottom uplifts, immediate protection of all exposed rock surfaces and invert arch required.

Legend: S = foliation; SS = beddings plane; K = joints; Z = shattered and crushed zone; M = mylonitic zone; P = plastic shales; H = slickensides; ex. in st. = excavation in steps