Tunnelling Experiences with NATM in Soft Rock

L'Expérience avec la Construction de Tunnel par NATM dans les Roches Tendres

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SYNOPSIS The driving and supporting measures described below apply with respect to the quarzite-mylonite-zones and to the phyllite-zones in the eastern section of the Ganzstein Tunnel, Styria, Austria. The softrock encountered there were relatively stable directly following excavation. However, partially before and also partially shortly after sealing the surface with shotcrete, significant settlement and deformation of the tunnel lining was observed. The quarzite-mylonite-zones were driven with the help of boredpile walls placed on both sides of the tunnel and in addition with a grouted roof above the tunnel crown. Following the principles of the "New Austrian Tunneling Method" (NATM) and a comparison of cost estimates in the phyllite-zones, a"method with systemic overroofing and controlled invert completion" was selected alternatively to reinforcement of the supporting means with bolts. Through controlled deformation, this method permits the primary stress condition to be changed to the secondary stress condition, and thus to the new state of equilibrium; in addition, it also permits the rock to perform a supporting function. The respective roof deformations were estimated. The overroofing dimension was selected in such a manner as to largely limit the risk for local subsequent profiling.

INTRODUCTION

In connection with construction of the new Semmering Highway S6, the Ganzstein Tunnel permits the city of Mürzzuschlag to be bypassed. The road bypasses Mürzzuschlag to the south, without touching heavily populated areas. Coming from Vienna the tunnel enters the mountain near Auers Brook, passes beneath Michelbauer Heights, proceeds slightly below Steingraben Brook and then passes beneath Ganzstein Mountain, coming out again in the Ganz Valley. Although there will be 2 tunnel tubes at ultimate, this project consisted only of the northern tube, having a length of 2,135m (cf. Fig. 1).

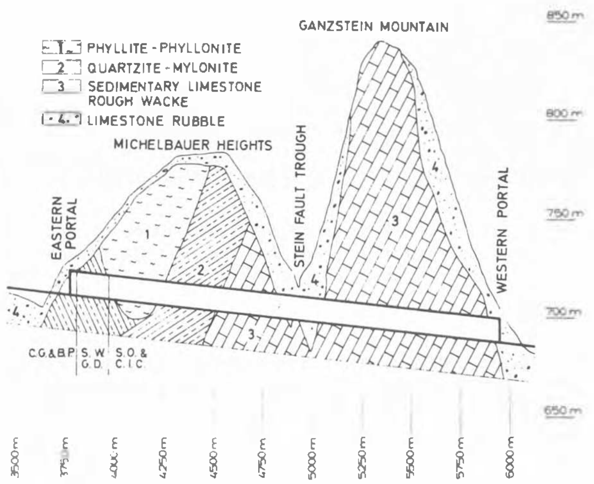

Fig.1 Longitudinal geological section (ratio of length to height: 1 to 10)

An almost circular, horizontal oval profile was selected for the tunnel. Because of the unfavourable rock mechanical conditions encountered, the invert was completed with an invert arch throughout for structural design reasons. The complete lining of the tunnel tube was of two-shell design. The thickness of the shotcrete outer-shell, constructed in accordance with the principles of the "New Austrian Tunneling Method" varied in accordance with the rock mechanical conditions, ranging between 15 and 25 cm. The inner shell was concreted with a thickness of 30 cm with the aid of a formwork transport wagon. Plastic film insulation was employed between outer, and inner shells throughout the entire length of the tunnel.

ADDITIONAL MEASURES FROM SURFACE

CEMENT GROUTING AND BORED PILES

The fact that the NATM can be economically employed in conjunction with ground improvement measures, even under difficult conditions with preceding extreme surface settlement in quarzite -mylonite zones was demonstrated by the Ganzstein Tunnel. (cf. Fig. 2).

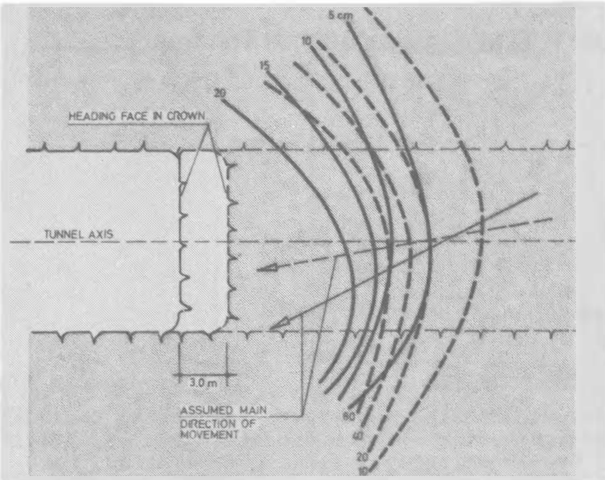

Fig.2 Preceding extreme surface settlement in quarzite-mylonite (top view).

As a result of test borings of the original tunnel route, it was assumed, that the tunnel route would be located entirely within the Semmering mesozoic with its quarzite sands and gravels. (cf. Fig. 3 and Fig. 4).

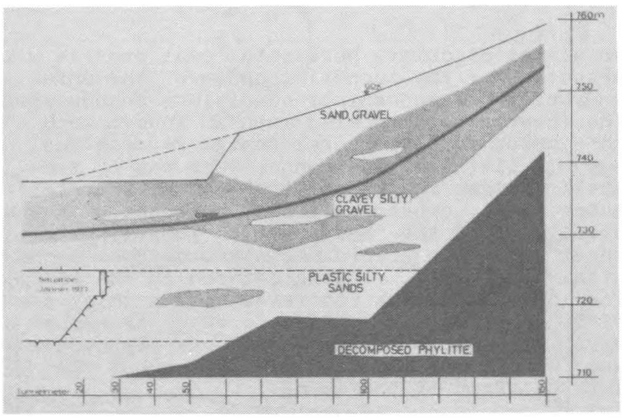

Fig.3 Detailed longitudinal geological section of eastern portal zone.

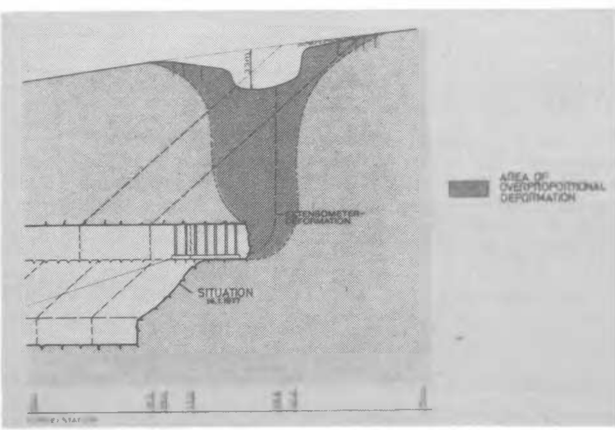

Fig.4 Plastifying behaviour of the rock.

During the course of the advance, it was found, that the Semmering mesozoic quickly dropped beneath the tunnel route, sloping to the south, thereby placing the cross section of the tunnel fully in the miocene with its phyllites and mylonites. The clayey silts, the major portion of which were fully saturated with water, produced extremely poor ground conditions which would have made further advance with the NATM impossible without additional ground improvement measures. (cf. Fig. 5).

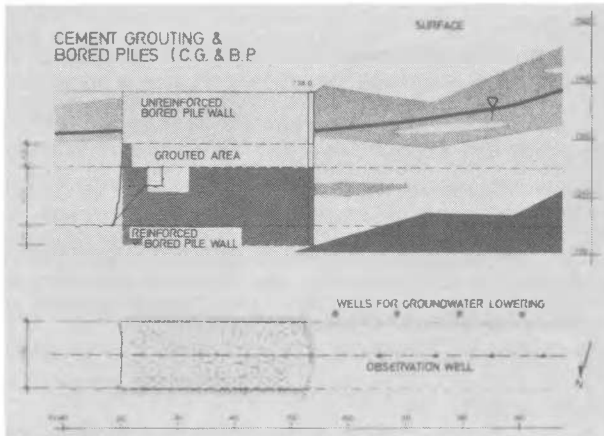

Fig.5 Additional ground improvement and side wall support measures from surface.

The time deformation behaviour in conjunction with a limit investigation for the stability of the heading face of the side wall gallery in ground class VI led to the conclusion that permanent ground improvement would be an absolute necessity in the critical area. The relatively high proportion of silt, amounting to 70 to 90 %, and the given permeability values of approximately 10^{-5} to 10^{-6} cm per second made an injection sealing method in which the plastic ground would be broken open under high pressure appear suitable, with the resulting fissures being grouted with a cement suspension. In addition to this bored pile walls have been placed on both sides of the tunnel to give the deformation behaviour of the calotte a stable foundation and thus to stop settlement and deformations.

In the unfaulted area, the sand-gravel lenses in the silty ground, were drained by means of wells, thereby helping to stabilize the ground.

ADDITIONAL MEASURES FROM FACE

1. SIDE WALL GALLERY DRIVING

In ground class VI the support measures were executed in accordance with the modified official concept and an alternative proposal by the contractor.(cf. Fig. 6 and Fig. 7).

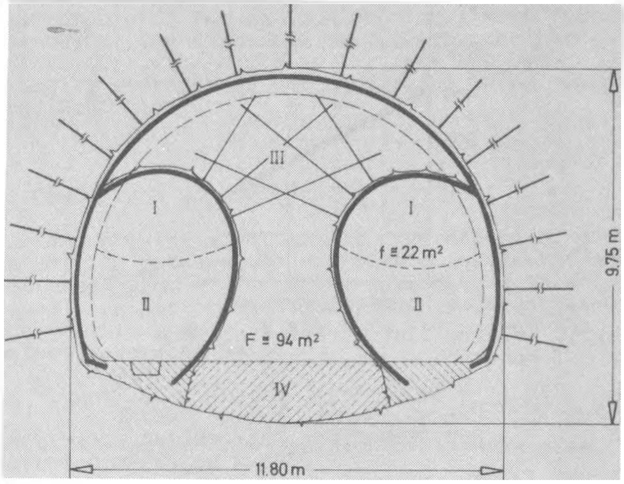

Fig.6 Excavation phases and support of side wall gallery driving (S.W.G.D.).

Side wall galleries with a slightly advanced crown were excavated with a staggered synchronized advance.The full excavation followed 20 to 30 m behind the side wall galleries.Experiences made with both side wall driving and full excavation were executed in two shifts around the clock and have been operationally satisfying.

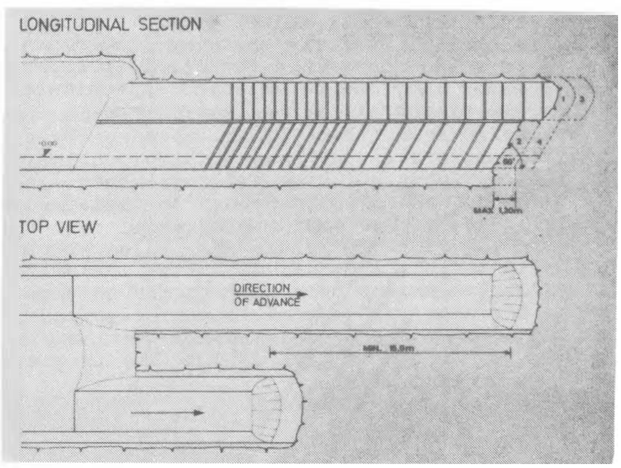

Fig.7 Tunnel driving with the help of side wall galleries.

The advance in the eastern portal of the Ganzstein Tunnel demonstrates solutions under homogenous soil mechanical conditions with highly reduced inherent supporting characteristics.

2. SYSTEMATIC OVERROOFING AND CONTROLLED INVERT COMPLETION

After succesfully holing through the extremely difficult eastern portal zone to a length of approx. 120 m with the aid of special measures as described above it was assumed that the tunnelling and marginal geomechanical conditions would improve. Consequently , side wall driving was terminated and converted to driving in partial sections, with crown, 2 benches and subsequent invert completion.

It was then attempted to drive and support by means of conventional supporting, combinations. However the shotcrete outer shell and bolts placed by means of this method were significantly deformed (cf. Fig. 13) at the roof and at the two side walls. While these deformations could be delayed, they could not be stopped with sufficient speed. With known supporting measures alone, the unusually large ground movements, beneath an overburden of approx. 30 to 50 m, preceding the heading and propagating all the way to the surface, where they formed depressions could not be controlled.

MARGINAL TUNNELING CONDITIONS

The tunnel is generally placed in highly changing, medium to very-difficult tunnelling ground. The rock to be penetrated is Semmering mesozoic. In the eastern half of the tunnel, the area beneath Michelbauer Heights, which is the subject of the part of the report, highly teary to compressive, in part variable, originally firm types of rock, such as phyllite, quarzite, quarzite-mylonite and phyllonite were encountered. (cf. Fig. 1).

The phyllite and phyllonite encountered was relatively stable directly following excavation. However shortly after placing the primary support by means of shotcrete sealing, significant roof settlement continuing to the surface was observed (cf. Fig. 8).

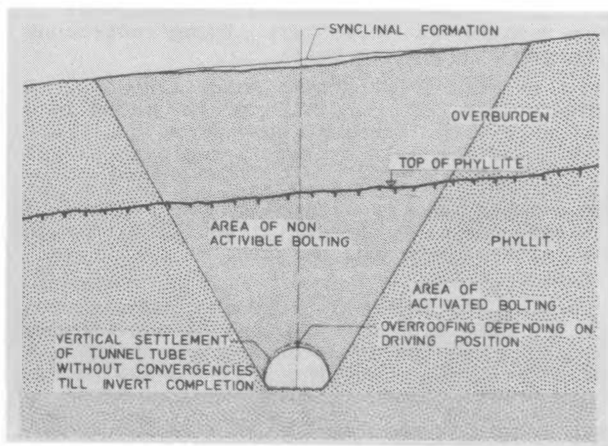

Fig.8 Areas of overstressed bolts and non activatable bolt forces.

Because of the rapid initial deformation, a relatively high reroofing dimension was therefore required, in spite of systematic bolting.

COMPARATIVE STRUCTURAL-DESIGN CALCULATIONS

The following is a portrayal of the assumptions and results in the calculation of a ficticious bolting system, which would have been required with the employment of conventional driving and supporting methods; however in this case, the calculation was performed for comparison purposes only and the bolt system was not installed.

1. Rod-static assessment

With the aid of the STRESS programme system, a rod-static investigation was performed on the computer system, at the Engineering Office of Beton- und Monierbau.

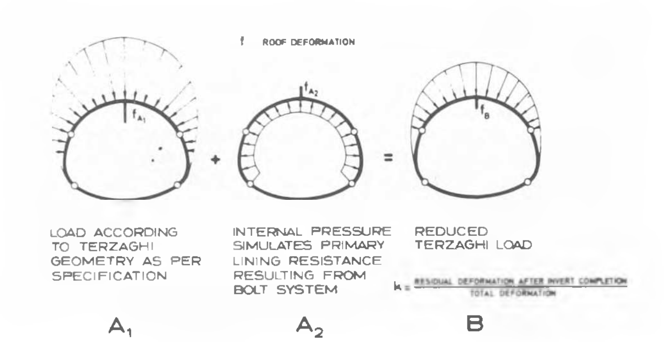

Fig.9 Schematic load assumptions

On the basis of two comparable load schematics (cf. Fig. 9) the following conclusions were drawn respect to the primary lining resistance to be provided by the bolts.

Load A results from partial loads A1 and A2. A1 simulates the load condition of the tubes in accordance with Terzaghi, however without bolts, while in A2 the bolt system is employed as internal pressure.

Load B takes into consideration the reduced load in accordance with Terzaghi, resulting from the quotient K of residual deformation over total deformation.

By varying the internal pressure load resulting from the bolt system in A2, the calculation was iterated until there was coincidence between the roof deformation resulting from A and B and the subsequently measured deformation.

Since the self-supporting effect of the ground is determined primarily by the cohesion, its magnitude was varied, with the interrelationship between cohesion and primary lining as a result of active bolting being investigated.(cf.Fig.10).

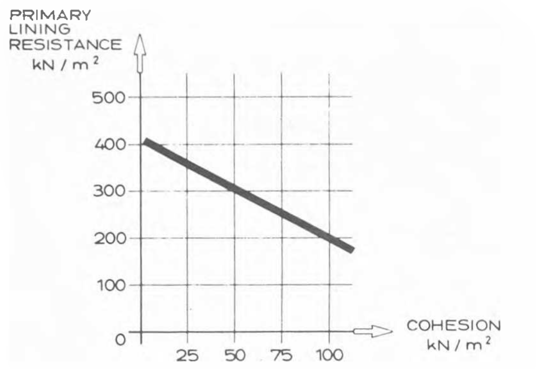

Fig.10 Primary lining resistance and cohesion

2. Assessment in accordance with Netzer (1976)

With this assessment method, the rock support ring, tensioned by means of bolts, is investigated with the theory of thick-walled tubes. The relationships of the perforated plate apply with respect to the area outside this ring, i.e. for the promary lining resistance of the shotcrete lining, whereby compatability conditions must be observed with respect to the interaction of these two systems.

Upon condition that the primary lining resistance is placed in time, the calculation can approximate theoretical elasticity principles. It was further assumed that there would be no radial deformation after placement of the full primary lining resistance (bolt system + uninterrupted shotcrete shell). Strictly speaking, the dimensioning formulae apply only for circular excavation, however they can be employed approximately in the case of the profile of this tunnel.

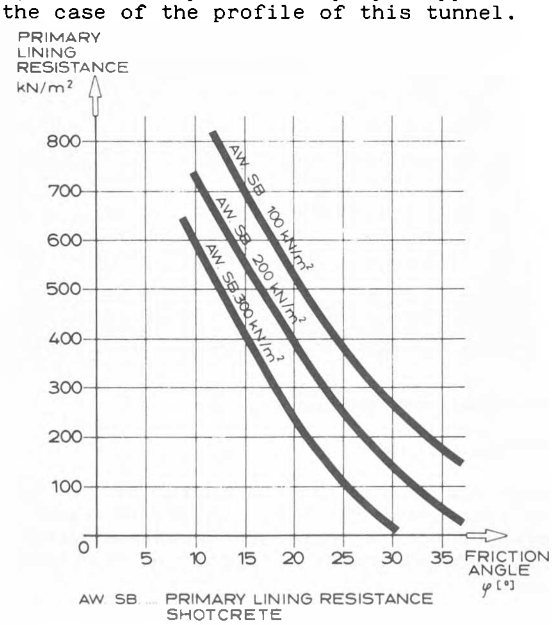

Fig.11 Primary lining resistance of the bolt system as a function of friction angle and primary lining resistance of the shotcrete shell.

A parameter assessment was again performed; in this assessment the percentage of the required primary lining resistance from the bolting system was calculated for various primary lining resistances of the shotcrete shell, with the parameters (friction angle of the rock and compressive strength of the shotcrete) being varied. (cf. Fig. 11).

3. Comparison of the results

A trend showing a relatively high degree of comparability between the rod-static assessment and the assessment of the thick-walled tube in accordance with Netzer was found. This substantiated the decision, made at the site, to employ the method described below.

SYSTEMATIC OVERROOFING AND CONTROLLED INVERT COMPLETION METHOD

Initially, an attempt was made to employ the rapid invert completion method, known from the construction of subway tunnels. In subway tunnel construction, generally involving small overburdens of approx. 10 to 15 m and relatively small cross sectional areas of approx.30 to 40m^2, this method is successful because any deformation occurring can be stopped following invert completion. In the project described here, on the other hand, rapid invert completion was followed by areas of destruction of the shotcrete outer shell, with only slightly reduced roof settlement and side wall convergencies.

1. Description of the method

The underlying concept of the construction method described below was to employ controlled deformation to achieve a new state of equilibrium, while minimizing the actually required bolting as a result of the high costs. On the basis of the experience gained from the tunnel driven up to this point, the expected deformations were estimated, with a corresponding overroofing dimension being stipulated at the site. The risk of having to reroof locally in the event of major deformation was shared by both owner and contractor. (cf. Fig. 12).

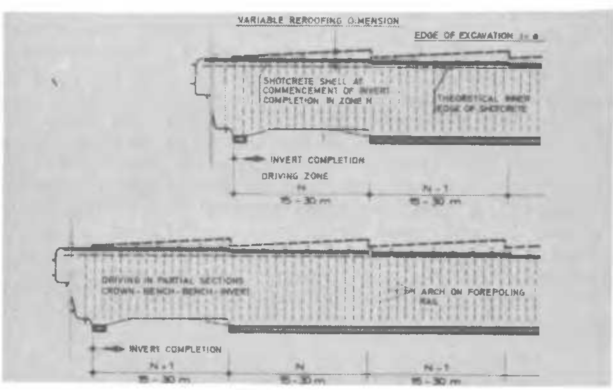

Fig.12 Driving schematic with systematic overroofing and controlled invert completion.(S.O. & C.I.C.)

Since deformation was influenced so greatly by the time of invert completion, tunnel construction aspects, in conjunction with operations considerations, led to the selection of the "tunneling with systemic overroofing and controlled invert completion" method. The overroofing, performed in increments over the respective zones, resulted in differing roof settlement; after stipulation of the corresponding invert completion time, as a function of the measured rate of deformation, this settlement was able to be mastered. (cf. Fig. 13).

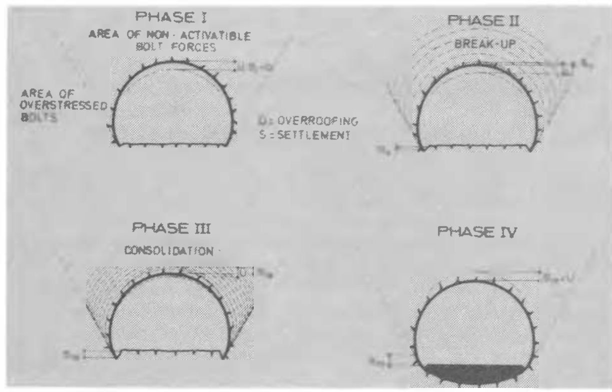

Fig.13 Deformation mechanism, cross sectional view.

a) Phase I - Stipulation of the overroofing, depending upon the location in the driving zone
 - Formation of crown and side walls, by zone

b) Phase II - Break-up begins above the roof
 - Vertical deformation of the tunnel frame commences
 - No horizontal convergence

c) Phase III - Consolidation begins above the roof after surface settlement
 - Settlement of tunnel frame gradually ends

d) Phase IV - After primary deformation has ended (approx. 10 - 12 days), completion of the invert, in zones, in accordance with the overroofing dimension.

At the beginning of the driving zone to be constructed (viewed in the driving direction), the outer shell was approx. 15 days old, at the end of approx. 5 days; this resulted in differing residual deformations, which were correspondingly taken into consideration in the overroofing. The invert was completed in accordance with the individual driving zones, (averaging approx. 15 m) in a half-sided manner, in partial zones of 2 m. Prior to commencement of a new driving zone, soil was placed in order to protect the fresh shotcrete invert. The overroofing was approx. 10 - 15 cm greater at the beginning of the zone than at the end. The remaining deformation stradled the theoretical inner edge of the shotcrete and only had to be corrected in certain areas. (cf. Fig. 14).

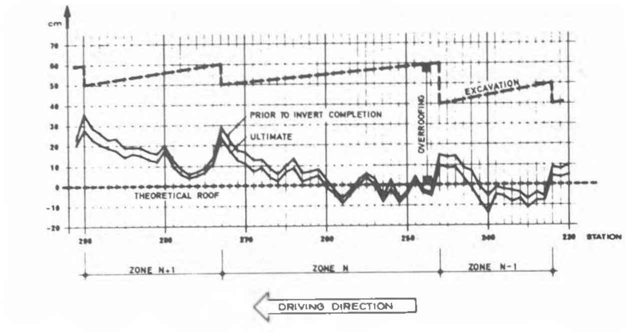

Fig.14 Choiced overroofing and measured roof settlement.

It was thus evidenced that with proper estimation of the expected deformation, through appropriate selection of the overroofing dimension, and taking into consideration the invert completion time, the desired profile could be produced with a minimum of support, without expensive supplementary bolting.

2. Interpretation of the measurement results

The deformation, analysed statistically for all driving zones, were measured at the centre of the roof and investigated separately at the beginning and end. They clearly showed the relationship between settlement and the time of invert completion. The dashed curve in Fig. 15 represents the averaged settlement line at the centre of the zone.

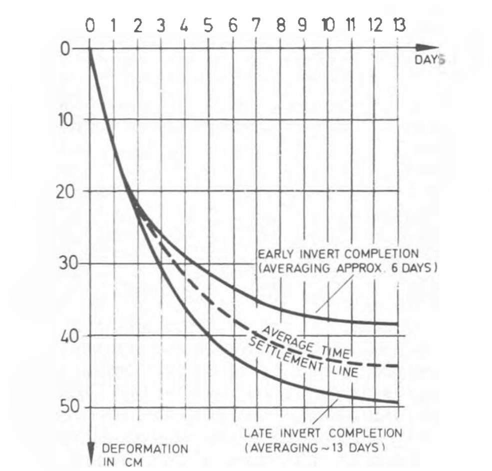

Fig.15 Averaged, idealized roof settlement, as a function of time.

It can be recognized that
- approx. 55 % of the 20-day settlement occurred during the first 3 days, and
- roof deformations gradually come to a stop asymptotically after completion of the invert.

The measured results confirmed the correctness of the empirically performed matching of overroofing, shell deformation and invert completion. However they apply only for the specific case of the phyllite zone in the Ganzstein Tunnel. Application of these considerations to projects of a different nature on an empirical basis would be possible.

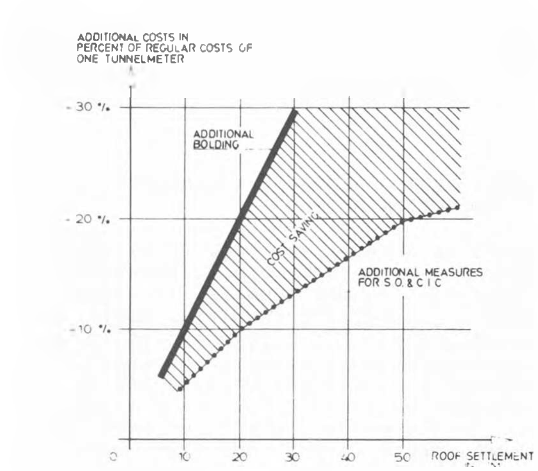

Fig.16 Estimated relativ costs for additional bolting compared with additional measures for driving with systematic overroofing and controlled invert completion. (S.O. & C.I.C.)

CONCLUSION

For driving through the relatively short quarzite -mylonite-zone, additional measures executed from the surface like bored pile walls on both sides of the tunnel and grouted zones within the roof area, showed technically well feasible. Compared with other possible ground improvement methods the proposed and executed measures habe even been more economical.

The estimated, but not executed, relative high bolting expenses required in the phyllite zones (cf. Fig. 16) and the operational difficulty of having to provide the primary lining resistance quickly by means of bolting led to the employment of the method of systematic overroofing in conjunction with controlled invert completion and controlled deformation for driving and support. The underlying concept behind this method, i.e. enabling the tension situation of the rock to shift through deformation while simultaneously maintaining the required safety for those working in the tunnel, was able to be realized through precise matching of the two tunnel construction parameters, roof settlement and invert completion, with the high degree of adaptability of the "New Austrian Tunneling Method".

In view of the specific marginal geomechanical conditions encountered in the phyllite zone, characterized by a high initial deformation rate, the method selected achieved the desired objective from both engineering and economic standpoints.

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