PRESSURE GROUTING OF AN EXTREMELY STIFF TUNNEL LINING

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ABSTRACT
The Prvonek Dam bottom outlet tunnel, designed with a horse-shoe cross-section and an unreinforced cast-in-place concrete lining, has been driven through very soft, hydro-thermally altered rocks of the Palaeozoic age. An unusually stiff structure, a concrete tube with the internal diameter 3.1 m and wall thickness 0.7-0.8 m, has been obtained due to bad excavation. The lining watertightness has been achieved by high pressure grouting, through the radial boreholes into the rock mass. With no previous experience with such problems, the designed technology was first checked at an instrumented test section. The performed measurements enabled the modification of the designed grouting procedure, reduction of total grouting works, and considerable savings. The test section results stressed the need for an active control of the grouting process and a strict instrumental monitoring of the lining deformation. Stress relaxation was deemed the main uncertainty in the analysis, requiring some conservatism in the design.

RÉSUMÉ
Le tunnel de vidange de fond du barrage Prvonek, élaboré avec une section en fer a cheval et un revêtement callé sur place non renforcé passe a travers de la roche tendre d’age Paléozoïque, altérée de façon hydro thermique. Une structure exceptionnellement rigide, un tube en béton avec un diamètre interne de 3,1m et des murs de 0,7 a 0,8m d’épaisseur, a été obtenue grâce a une mauvaise excavation. L’etanchiete du revêtement a été atteinte par la cimentation a haute pression a travers des trous de forage radiaux dans la masse rocheuse. Sans expérience antérieure avec de tels problèmes, la technologie du design a été testée dans un premier temps sur une section d’essai instrument. Les mesures prises ont permis la modification du processus de cimentation, la réduction des travaux de cimentation et des économies considérables. Les résultats de la section d’essai ont renforce l’importance du besoin d’un contrôle actif du procède de cimentation et un suivi stricte des instruments mesurant les déformations du revêtement. Une relaxation des contraintes a été jugée être l’incertitude principale dans l’analyse, ce qui requiert un design conservateur.

1. INTRODUCTION

It was common European practice in the 1970’s and 1980’s to achieve the watertightness of the lining of hydraulic tunnels under internal water pressure by prestressing using high pressure grouting. The grouting procedure, typically used in the former Yugoslavia, consisted of grout injection through the radial boreholes into the surrounding rock mass and annular gap between the rock and the lining. In this way, the lining prestressing was accompanied by consolidation of the surrounding rock, which resulted in the improvement of its deformation properties, and decrease of the rock heterogeneity, anisotropy and permeability.

The experience gained in Yugoslav practice referred entirely to large diameter circular tunnels with thin linings, in hard carbonate rocks, with marked fissure porosity and groutable joints of wide gaps. The grouting technology employed the concept of long boreholes (6-8 m) with large diameters (70 mm to 100 mm), thin grouting masses, strict regime of continuous grouting, and similar; see the Bajina Basta Pumped Storage Plant as a typical example (Nikolic and Markovic, 1992); then the Zavoj Dam (Kujundzic et al, 1992).

The prestressing success criteria were based on structural analyses of the interactive system tunnel lining – rock mass, and were expressed through the minimum necessary lining deformations, which were dependent on geometric and mechanical parameters, and loading. Having in mind a high degree of idealization in these analyses, testing sections were extensively used for carrying out the program of measurements in order to experimentally check the defined criteria and determine the optimal grouting technology (borehole arrangement, depths and diameters, grouting mass composition, injection regime) that was later applied without measurements. Nevertheless, the exact criteria were elusive and application of this grouting method was still a matter of skill and feeling, limited to the fields where some positive experience had already been gained.

This paper presents a case history that had no precedence in the prior experience described above. An extremely stiff tunnel in a very soft rock was prestressed by high pressure grouting using routine drilling and grouting equipment. The prestressing was accomplished only by an active management of the grouting regime and with instrumented monitoring of grouting effects. Although stress relaxation with time was higher than anticipated, successfully accomplished prestressing under these extreme conditions of the structure and the ground could offer a wider prospective for the application of grouting to achieve the watertightness of pressure tunnels.

2. THE SITE AND THE STRUCTURE

The Prvonek dam, a 90 m high rockfill dam, under construction since the end of 1980’s, is designed to supply water to the city of Vranje in southeast Serbia (Masala et al, 1995).

The dam site rock mass consists of Palaeozoic metamorphic rocks, hydro-thermally altered and weathered gneisses and micashists. The rock is highly heterogeneous and anisotropic foliated, markedly fractured, but the discontinuities are mostly fissures and tight joints while the unfilled joints of wide gaps are short. The intact rock is very soft, with the deformability modulus of about 2,000 – 3,000 MPa.

The part of the bottom outlet tunnel from the water intake tower to the outlet gate (Figure 1) was designed with a horseshoe cross section, a 3.10 m internal diameter and an unreinforced concrete lining 35 cm thick in the arch. This part of the tunnel is designed for two purposes: the upper part serves as the bottom outlet, while the lower part contains an embedded fresh water pipeline. This disposition practically precludes further interventions (remedial measures) in the tunnel during its use.

With the main gate at the downstream end, the tunnel will be under permanent internal water pressure of about 0.67 bars. The 240 m long downstream part of the tunnel, from the grout curtain to the outlet (Figure 1), will therefore operate as a tunnel under pressure. This tunnel section had originally been designed with steel lining, but the political turmoil in the region in the early 1990’s and associated economic problems rendered the designed solution as unfeasible. The client then proposed that the tunnel lining impermeability be provided by high pressure grouting, which was a common method in the current Yugoslav practice at the time.

The proposal and the problem were difficult and challenging. The experience gained in Yugoslav practice to that date referred to large diameter tunnels (6-8 m) with thin linings (typically 30-35 cm), in hard carbonate rocks with marked fissure porosity and groutable joints of wide gaps. There was no experience concerning the groutability of soft rocks with tight joints, as those present at the site, nor regarding their capability to permanently preserve the induced compressive stresses. Drilling problems were expected due to the considerable presence of quartz both as a mineral component and as intercalations. In addition, the tunnel itself was of an atypical cross-section and dimensions for the pressure grouting works.

3. TUNNEL CONSTRUCTION

The tunnel had been excavated according to the NATM method, with the primary reinforcement of anchoring bolts in the cement mortar. The quality of work was poor, considering the significant overbreakage and the asymmetrical cross-section. The concrete lining thickness was too large (70-80 cm instead of the designed 35 cm). A concrete pipe several times stiffer than the designed one was created, considerably endangering the designed prestressing concept by pressure grouting.

The concrete lining had been placed in two installments: first the prefabricated segmented tunnel invert, and then the cast-in-place upper arch part. The longitudinal concrete joints so obtained had not been sealed in any manner. The concrete sections were about 8 m long and rubber waterstops had been systematically placed at the transverse joints. The quality of work was low here as well, with a lot of segregation and concrete damage. During the water permeability tests and grouting, every joint was leaking as if there were no waterstops at all.

4. GROUTING

The original grouting concept was based on the previous experience with large diameter and thin lining tunnels like Bajina Basta and Zavoj and proposed three phases of work (Table 1):

- backfill grouting, aimed at the systematic filling of the gap between the lining and the rock in order to prevent the uncontrolled movement of grout along their contact;

- consolidation grouting, for the improvement and homogenization of the rock mass in the zone disturbed by blasting and for the tunnel sealing by closing the groundwater communication ways; and

- pressure grouting.

It was soon realized that both the grouting concept and the parameters, based solely upon experience, were uncritically "copied" to an unusual situation; therefore, it was decided to modify the designed procedure. Together with unfavourable geological conditions and bad construction, additional difficulties appeared:

- the contractor had never before dealt with pressure grouting so it did not have the appropriate equipment;

- there were practically no possibilities of the material and spare parts supply (drilling bits, pipes, valves, special joint sealing mixtures - mostly imported goods);

- the deadline for the entire works was very tight - the design modifications, the test section investigations, all grouting as well as the repetitions on the failed sections, were completed in September and October 1991.

4.1 Modified grouting method

An evaluation of the real capabilities of the contractor and the capacities of the site resulted in the following additional corrections to the design (Table 1):

- instead of ϕ 76 mm grout holes, the smaller ones ϕ 42 mm were accepted, since the contractor did not have the appropriate drilling bits nor the grouting packers ϕ 76 mm;

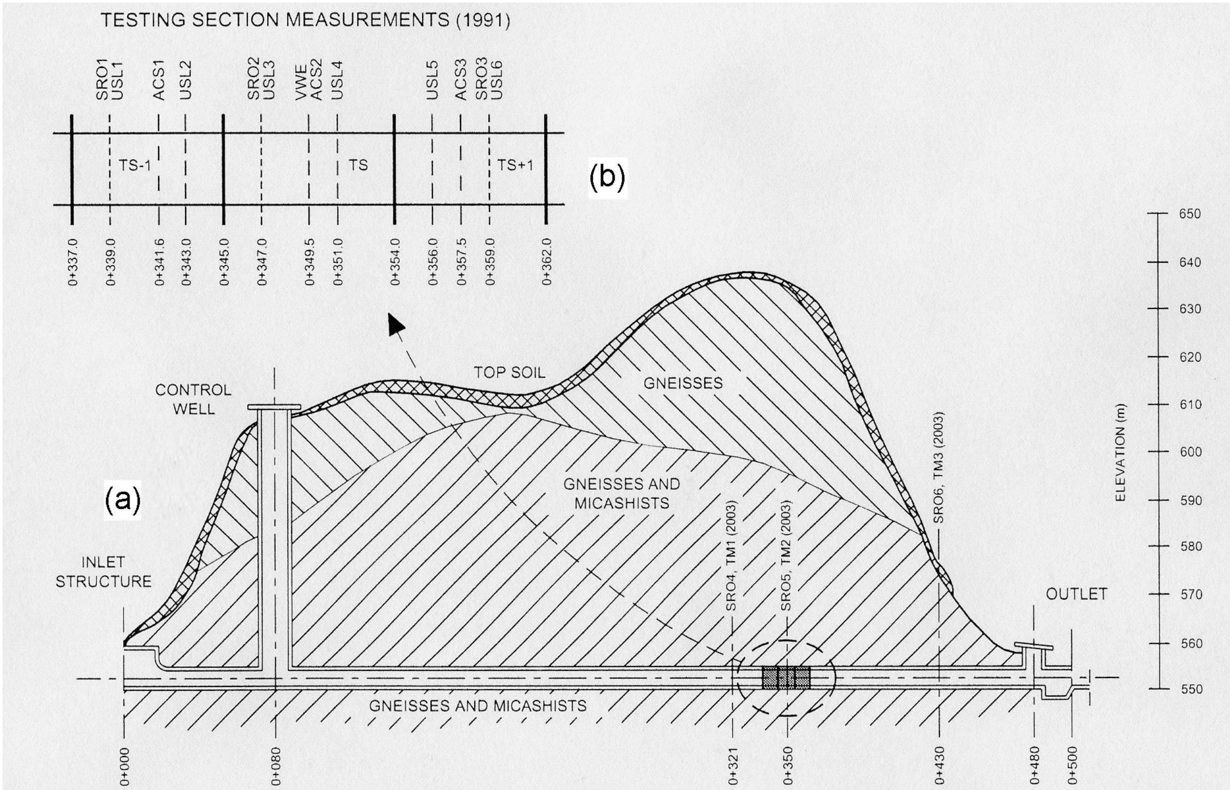

Figure 1. Prvonek Dam bottom outlet: (a) vertical section with geology; (b) grouting test section

Table 1. Grouting parameters

		Prvonek		
Parameter	Bajina Basta	Original	Test section	Modified
Internal diameter (m)	6.3		3.15	
Lining thickness (m)	0.35		designed 0.35 , actual 0.60 - 0.80	
Rock mass	carstified limestone		gneiss and micaschist	
Excavation method	TBM machine		drill and blast	
Lining type	cast-in-place unreinforced concrete			
Maximal internal water pressure (bar)	13.5		6.7	
Total grouted length (m)	5700		240	
Grouting section length (m)	10.0		8.0	
Injection profile distance (m)	2.5		4.0	
Number of boreholes in profile	10		6	
Borehole diameter (mm)	56	76	42	33
Borehole length (m)	6.0 - 3.0	2.0	2.0	1.0
Dry mixture cement to bentonite ratio C : B	95% C : 5% B	97% C : 3%	95% C : 5%	95% C : 5%
Final water to cement ratio W : C	2:1		2:1	
Maximal injection pressure (bar)	25	18	30	30
Total injection time (h)	-	-	5	5
Completion criterion - max. injection pressure maintenance without grout consumption (hours)	0.5	2	2	2
Number of repeated injections (%)	4.75%			14%
Dry mixture consumption (kg/m' of the tunnel)	200-5000	300	350	100-400
(kg/m^2 of the tunnel)	10-250	30	40	10-40

- instead of mechanical rotary drilling, the percussion drilling using manual pneumatic hammers was accepted (the only available equipment on the site).

It naturally became obvious that after such changes of the design terms the proposed modified grouting procedure had to be checked and confirmed on a test section. The goals for the test section were threefold:

- to investigate the possibilities of control of the prestressing effects by the change of the grouting regime, the grouting pressure and the grout density;

- to experimentally determine the amount of stress relaxation with time; and

- to investigate the non-uniformity of achieved deformations in the tunnel cross section and over the lining thickness.

5. TEST SECTION

The test section consisted of three 8 m long instrumented grouting sections (Figure 1). The grouting profiles were 4 m apart and contained 6 radial grouting holes (Table 1).

The boreholes were connected within a grouting network with the “outer mass circulation" (the main supply line with branches for each borehole + the return line). Each borehole was supplied with its own pressure gauge and stopcock. A stopcock was also placed at the return line to control the injection pressure and grout circulation.

The maximum injection pressure was about 30 bars. The total injection time was limited to five hours, after which cement started to sediment from the slurry.

The lateral sections TS-1 and TS+1 (Figure 1) were grouted 2 times; the first grouting for ground consolidation, the second for the lining prestressing. The main problem was uncontrolled movement of the grout along the tunnel (even 20 m from the grouted section), through the loosened rock mass ring around the lining, created by blasting during the excavation.

The middle section TS was grouted only once. The idea was to check whether it was possible to save both time and money by eliminating the consolidation grouting of the sections that were already confined by the grouting of two adjacent sections. This grouting had a real prestressing character due to the successfully prevented loss of the grout slurry by consolidation of the lateral sections.

5.1 Instrumentation and measurements

The key measurements on the test section referred to the concrete lining deformation (Figure 3).

The distribution of the tangential strains in the lining was made using the vibrating wire extensometers (VWE). The extensometers were especially useful for the monitoring of the decrease of induced prestressing deformations through time (stress relaxation).

The average circumferential strain on the lining intrados (ACS) has been measured using the Glötzl device (Lauffer, 1966). This simple, precise and reliable instrument was particularly convenient for a direct control of the injection process since it could integrate the changes of circumferential strain along the whole cross section of the tunnel into one numerical data.

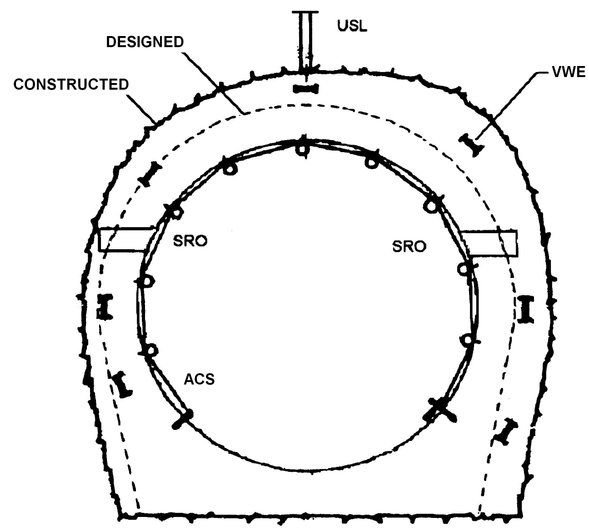

Figure 3. Test section measurements

The effects of the lining prestressing have been directly determined after the grouting by the strain relief method at the lining intrados using the overcoring method (SRO).

The ultrasonic longitudinal wave velocity logging (USL) of the rock mass was performed in the same boreholes before and after grouting. The measurements served for the determination of rock mass consolidation effects, as a supplement and alternative to the core drilling which was mostly of fairly low quality.

Engineering geological mapping with the classification according to the Q system had been carried out during the tunnel excavation. It served for the preparation of the rock mass deformability model along the tunnel.

Lugeon permeability tests had been planned before and after grouting, but the execution was completely unsuccessful due to the equipment, which was usually out of order, and abundant leakage through the tunnel lining. The estimated value of the ungrouted rock permeability at the test section was about 2-4 lit/min/m.

A desired value of the prestressing deformation had been previously calculated. The average circumferential strain at the lining intrados (ACS) was accepted as the design parameter since it enabled quick, direct comparison of the achieved prestressing deformation, measured by the ACS device during grouting, with the target value.

The calculation has been made according to the "analytical-experimental procedure" by Nikolic and Markovic (1992). The procedure was based upon a simple model of a thick wall pipe embedded in a continuum, with assumptions of the plane strain state, the elastic behaviour of concrete and rock and the axial symmetry of the problem. The priority was given to a simple model with clear input data to perform quick parametric analyses, rather than to a complex model with many influential parameters where the reliable input data determination would have presented the main problem.

The required permanent circumferential strain of the prestressing at the lining intrados was assessed as 70-80 x 10^{-6}.

5.2 Measurement results and interpretation

The measurement results on the central section TS are presented in Figure 4. They have shown the following. ACS measurement is a very reliable indicator of the grouting process regularity; it reacts quickly to the grouting pressure variations and clogging of grouting ways in the rock, accurately records changes of grout thickness, etc. Some additional prestressing effects are visible in the ACS measurements on the lateral sections TS-1 and TS+1.

The deformation distribution in a cross section can be very heterogeneous - the ratio min/max VWE was always within the range 0.20 to 0.25 (Figure 4c).

The difference in deformation measurements using VWE and ACS was the consequence of an irregular shape of the excavation and varying thickness of the lining. The ACS measurement was used as the reference further on. A 50% increase in the ultrasound longitudinal wave velocity after the grouting (Figure 4b) indicated successful consolidation grouting effects, with probable marked improvement of the rock mass stiffness. It should be noted that this rock improvement was not accounted for in the analyses of prestressing effects and the definition of the grouting success criterion.

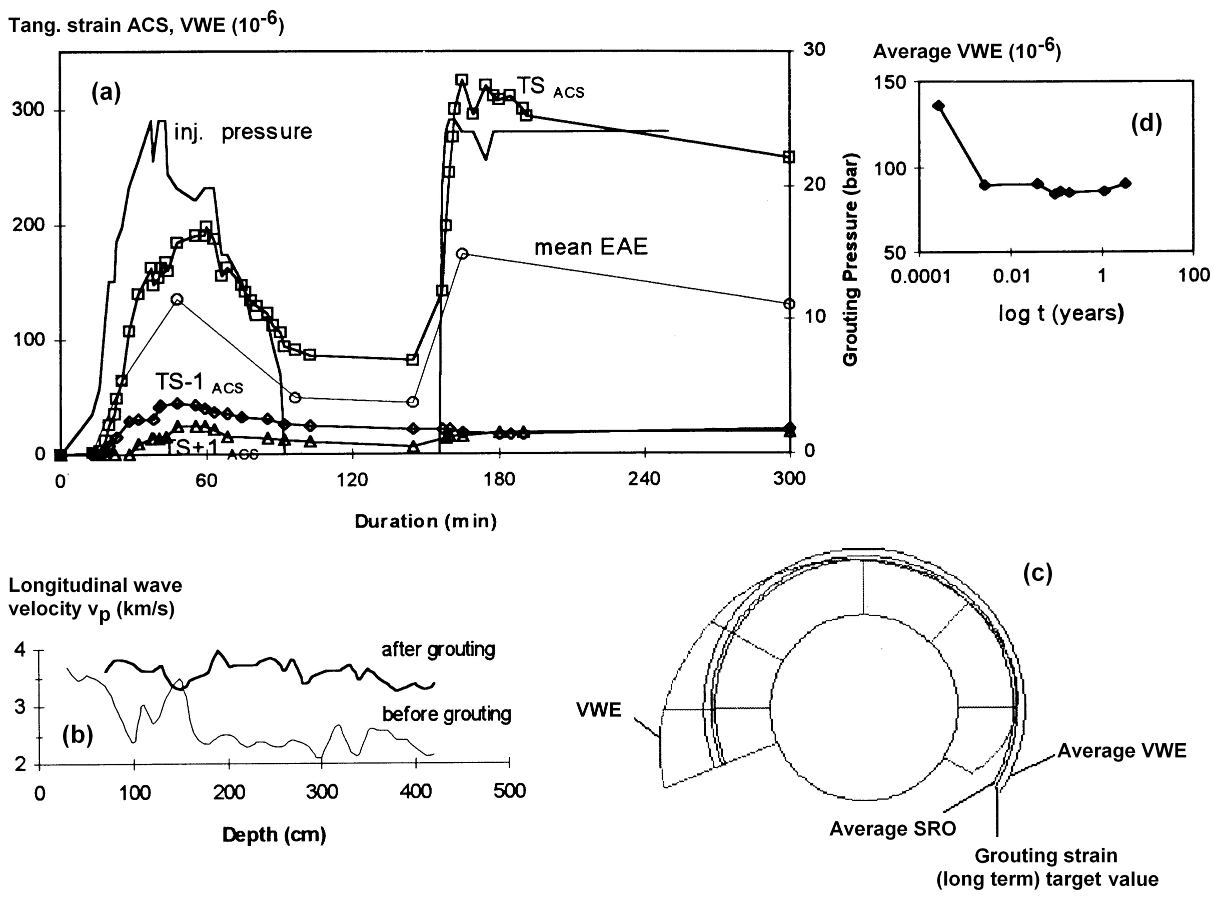

Figure 4. Test section measurement results: (a) Time graphs of tangential strain measurements in the lining during injection; (b) Ultrasound logging of surrounding rock mass before and after injection; (c) Distribution of strains in the lining a month after injection; (d) Strain relaxation during first 2 years after injection

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The SRO results confirmed the non-homogenous state of stress in the lining and the rapid loss of the prestressing effects immediately after injection, measured by the extensometers (Figure 4d). The average measured SRO strain of about 75×10^{-6} was at the very limit of acceptability (Figure 4c).

The test section measurements have shown that the injection process is very sensitive to variations in the grouting pressure and the slurry thickness. There are no guarantees that a prescribed grouting procedure will result in a regular and uniform grouting process with desired prestressing effects. The grouting process must be actively managed and the grouting performance must be controlled by continuous measurements of the tunnel lining deformation during injection.

The pressure grouting can be performed using cheaper drilling equipment (percussion drilling instead of the rotary one), and lower requirements as to the size of boreholes (even ϕ 33 instead of ϕ 56 mm, with borehole length 1.0 m instead of the designed 2.0 m).

It is always possible to realize additional prestressing effects by increasing the grouting pressure or by thinning the slurry. The effect of “material memory" is apparent in Figure 4a, since the lining started to deform only after the previous grouting pressure had been exceeded.

The prestressing cannot be achieved unless the concrete lining is completely sealed and the grout prevented from moving laterally through the rock along the tunnel (by consolidation grouting). The grout movement along the lining-rock contact, although with some marginal positive effects through the additional action upon the neighbouring sections, was undesirable because of "spotting", closing of circulation ways within the rock, leakage into the tunnel and greater grout consumption.

The grouting regime should be continuous, based upon quick initial grouting pressure increase using thin suspensions C:W = 1:4, and then thickening them up to the C:W = 1:1 in consolidation grouting, or to W:C = 2:1 in pressure grouting. The completion criterion should be defined as 2 hours of maximum pressure maintenance without grout consumption.

6. PRESSURE GROUTING FINAL DESIGN

The test section showed that, with an adequate control and managing of the grouting process, the tunnel lining prestressing can be performed in two ways:

- with previous consolidation grouting of the rock mass, and

- without previous consolidation, if the uncontrolled loss of grout could be prevented (when the neighbouring sections have been previously consolidation grouted).

The second option offered significant reduction of the quantity of works proposed by the original design (consolidation grouting could be reduced to a half of the proposed scope). The following final grouting methodology was therefore formulated:

- the stretch of the tunnel to be prestressed, consisting of 31 grouting sections in total, was divided into lines of "even" and "odd" sections;

- odd sections were to be grouted two times: the first grouting for the rock consolidation, the second one for the lining prestressing;

- even sections were to be grouted once, to prestress the lining;

- the pressure grouting had to be performed under strict control of the tunnel lining prestressing effects by measuring the deformations using the ACS method;

- the designer had to define, for each section, the target value of ACS deformation at the end of the injection, according to the actual quality of the rock and concrete, the real lining geometry, etc. If the ACS target value had not been achieved, the grouting had to be repeated.

7. EVALUATION OF IMMEDIATE PRESSTRESSING EFFECTS

The grouting success coefficient GSC was defined as a ratio of the achieved permanent prestressing deformation in the lining axis ε_{meas} and the required permanent deformation ε_{req}, calculated after Nikolic and Markovic (1992) on the basis of real data, separately for each grouting section. The value ε_{meas} has been converted into a required ACS measurement at the intrados at the end of injection, using two factors.

The decrease of prestressing deformations with time due to rheological characteristics of the grouting mass, rock and concrete was accounted for using the stress relaxation factor K1 = 0.65. This was an estimated value: the ratio of the mean VWE strain after about a month after grouting and at the end of injection (Figure 4d).

The conversion of the ACS data from the intrados to the lining axis was enabled using another factor K2 = 0.60, estimated as a ratio of the ACS and the average VWE measurements during grouting. This factor accounted for heterogeneous distribution of tangential deformation in the lining as well.

Figure 5 presents the GSC values along the tunnel, with highly variable distribution and mostly much higher than required. A few isolated "unsuccessful" results with GSC<1 were accepted within the whole, as localized “weak zones” of more permeable lining.

8. REMAINING STRESSES AFTER 12 YEARS

An investigation of the remaining stresses in the tunnel lining was performed 12 years after the grouting, in September 2003 (Andjelkovic and Petrovic, 2005). It should be noted that the tunnel has never been put under pressure, as the construction ceased in the mid 1990’s. Two stress-deformation release methods were used at three locations along the grouted section (Figure 1):

- Deformation state determination at the tunnel intrados by overcoring (SRO), with two measurements on each side of the tunnel (left and right), and

- Tangential stress measurement at the tunnel intrados by the Tincelin-Mayer method, with one measurement in each location.

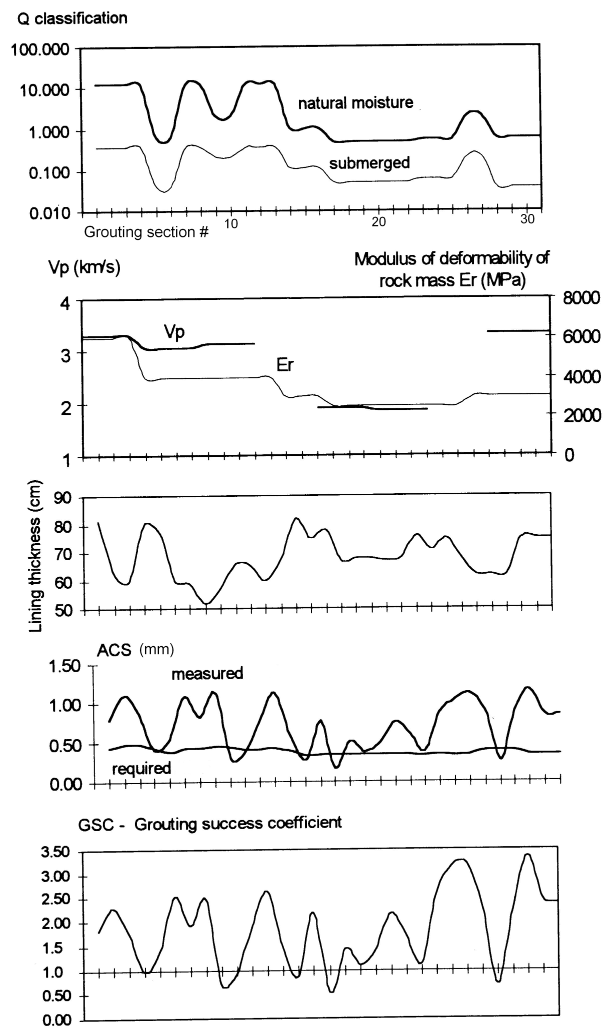

Figure 5. Grouting success evaluation

One measurement location at 0+350 m was set in the middle of the central test section TS from 1991.

8.1 Overcoring

The overcoring was conducted using a portable drilling rig with 180 mm diameter diamond bit. The deformations were measured using a rosette with 60 mm long strain gauges at 0-45-90-135 degrees. The measurements were taken each 10 mm up to 50 mm of coring depth, and then each 50 mm to the final depth of 250 mm.

The four measured strains allowed determination of the strain state at the lining surface, and the principal stresses were obtained from the principal strains using the modulus or elasticity E = 25,000 MPa and the Poisson’s ratio ν = 0.17 for the concrete, respectively. Eventually, the horizontal and vertical stresses in the lining were calculated to allow comparison with the Tincelin-Mayer method.

Table 2 presents a summary of calculated vertical and horizontal stresses in the lining; the stresses shown are the average values within the depth interval 50 mm to 250 mm. The vertical stress is the tangential stress; the horizontal stress is the axial stress in the lining. Positive stresses are compressive. It should be noted here that the value σ_h = 0.20 MPa at 0+321 m (left) was measured near a transverse concrete joint where stress relaxation probably occurred quickly after grouting.

8.2 Tincelin-Mayer

This method was originally developed for stress measurements in the walls of adits in iron mines in France (Tincelin, 1952). The method described here is a slight modification regarding the equipment and procedure used. This is the first application of the Tincelin-Mayer method to the tunnel linings in Serbia.

The measurement of deformations was performed using a Hugenberger mechanical deformeter with the base of 500 mm and the accuracy of 0.001 mm. The deformeter markers were set up before slot drilling. A horizontal slot 21 cm deep and 42 cm wide was then cut in the lining by drilling a series of overlapping 15-mm diameter holes. The resulting lining deformations were registered several times during drilling. The slot was then filled with a fine sand cement mortar and a flexible hydraulic flat jack 40 cm by 20 cm and 6 mm thick, was inserted in the slot. After the concrete had reached the required strength, the flat jack was filled with water and connected to a Glötzl hydraulic pump with a manometer. The jack was then pressurized in conjunction with the deformation measurement. The loading was performed in two loading-unloading cycles, to the pressure that was sufficient to cancel the deformation released during slot cutting.

The pressures needed to compensate the released deformations are presented in Table 2; they correspond to the vertical (tangential) lining stresses obtained by overcoring.

Table 2. Stresses in the lining obtained by overcoring (SRO) and Tincelin-Mayer method

Station (m)	Side	SRO Strains (10^{-6})		SRO Stresses (MPa)		Tincelin-Mayer (MPa)
		ε_v	ε_h	σ_v	σ_h	σ_v
0 + 321	left	46	0	1.18	0.20	1.86
	right	93	78	2.74	2.42	
0 + 350	left	65	54	1.91	1.67	3.54
	right	34	29	1.00	0.90	
0 + 431	left	101	114	3.10	3.38	
	right	84	85	2.53	2.56	1.98
Average		70	60	2.07	1.85	2.45

The absolute values of measured strains and stresses are rather low with respect to the values recorded at the end of grouting, with stress relaxation of about 50% on average, and with the strains measured at the test section suggesting perhaps even higher stress relaxation. The average SRO tangential strain of $70*10^{-6}$ is a little below the value of $75*10^{-6}$ measured a month after grouting, showing that the majority of relaxation occurs immediately after the injection. The Tincelin-Mayer tangential stress is about 20% higher than the SRO average stress, indicating possibly higher elasticity modulus for the concrete than adopted in the calculation.

Apparently, the SRO method (overcoring) seems superior to the Tincelin-Mayer method both technically and economically as it provides more data of higher quality - a complete strain and stress state instead of a single component, and its application is technically easier and less time consuming. The interpretation, though, depends on the reliability of material parameters for the lining.

9. CONCLUDING REMARKS

The investigations performed have proven the possibility of prestressing a very stiff tunnel lining in a soft rock mass using pressure grouting.

Measurements performed at test sections remain as an essential part of the pressure grouting design, due to modest capabilities of the existing numerical methods.

Certain aspects of the described grouting methodology (extreme sensitivity to variations of grouting pressure and slurry thickness, heterogeneous deformation distribution, difficulties in preventing the grout loss, etc.) are unavoidable; they cannot be foreseen nor prevented at the present level of knowledge. In other words, pressure grouting cannot be "designed", and its success cannot be a priori guaranteed by the strict following of a prescribed working procedure. The injection procedure must be actively managed and a continuous control of prestressing effects must always be conducted by measurements of tunnel deformations.

Prestressing effects rapidly decrease in the first several days after the injection, but then decline at a much lower rate. An estimate of the proper stress relaxation factor is essential for the design, but difficult to predict accurately, which calls for some conservatism in the design. In this case, the relaxation factor had been a little underestimated, as the measurements of the remaining prestressing 12 years after the grouting showed. Certain hidden reserves, though, exist in the consolidation effects – increased stiffness and reduced permeability of the rock mass, which were not considered in the analysis.

This case history can be assessed, with modest optimism, as a success since the prestressing has been performed under very relaxed requirements for the drilling and grouting equipment, and the grouting layout as well. It could possibly open a wider prospective for application of this procedure as an alternative to steel lining in unfavourable working conditions.

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