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# Cost-effective pre-injection with rapid hardening microcement and colloidal silica for water ingress reduction and stabilisation of adverse conditions in a headrace tunnel

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ABSTRACT: During the drill-and-blast excavation of the headrace tunnel, difficult rock conditions were encountered in a zone caused by a regional fault. Tunnelling through the zone was initially done without pre-treatment of the ground, resulting in severe difficulties and a lengthy delay. Therefore it was decided to excavate a bypass tunnel utilizing pre-excavation grouting from the tunnel face. A two-stage pre-injection scheme was laid out in order to achieve the desired penetration of grout into the ground. First stage grouting was done with rapid hardening microcement, in which a limited grout take was achieved. The second stage of grouting was done with liquid colloidal silica, thus achieving penetration into the finest discontinuities. The result was an acceptably fast advance, excellent ground stability and virtually dry ground conditions. This enabled the safe installation of rock support with steel sets combined with sprayed concrete.

## 1 BACKGROUND

### 1.1 *The project*

During the drill-and-blast excavation of the 16 km long headrace tunnel of 4 × 76 MW Maneri Bhali Hydro Electric Project, difficult ground conditions were encountered in the stretch passing below a stream. The ground cover mostly consists of river born material (RBM).

Tunneling through this section was initially done without any pre-treatment of the ground, resulting in the puncturing of the small rock cover over the tunnel crown, causing severe leakages from the water saturated RBM zone and chimney formations. The difficulties caused lengthy delays, so much so that a 56 m long problem zone in the tunnel took nine months to excavate.

Moreover, due to problems faced in negotiating this sitaution, the tunnel could not be excavated to full required cross section. As a result the designers of the tunnel lining advised the reduction of the tunnel section and the installation of steel lining.

The work on the project was stopped in early nineties. When it was resumed in 2002, the project management did not feel comfortable with the idea of having a reduced tunnel section with steel lining in this portion due to the of extra time required for the steel lining, as well as the increased friction losses associated with this arrangement.

### 1.2 *Geological conditions*

The challenge in this situation was to ‘again’ cross this fault zone in the bypass tunnel, without ground collapse and in dry conditions, thereby allowing lining construction in dry and stable conditions. The rock mass consisted of heavily jointed quartzites and metabasics on each side of a regional fault zone.

A river valley at surface followed the fault zone alignment with subsequent thick deposits of river born materials over lying the quartzite and metabasics. The fault zone exhibited highly crushed material with associated high water seepages that resulted in significantly reduced stability of the excavated rock-mass. Joint fillings consisted of fine grained quartzite material of clay and silt fractions.

### 1.3 *The problem*

During the initial phase of the project, the tunnel was excavated through this zone without any pre-treatment

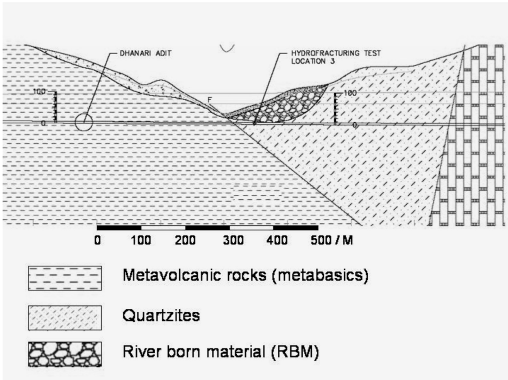


Figure 1. Longitudinal geological section with the tunnel alignment passing under the weak zone. Note that the rock overburden is indicated as uncertain. The vertical lines indicate exploratory holes.

of rock mass. Large water ingress and cave-ins were experienced. A very irregular tunnel contour with less than required excavation diameter was the result. The tunnel was supported with steel sets and bakfill concrete. The tunnel has suffered from major seepages in this section since the excavation in the eighties.

After the work was restarted in 2002, it was decided to reconsider the design of the tunnel lining in this section, with a intent of reducing the lining construction time and friction losses. The first option was to re-excavate this part of tunnel to the fully required cross section. This was not implemented since this would have caused stoppage of all excavation/lining activities further downstream.

Moreover, with the rate of water seepage being observed it was considered uncertain if a structurally sufficient support of the tunnel could be achieved in the already excavated tunnel. For these reasons, it was decided to excavate a by-pass tunnel around the problem area. Pre-treatment ahead of the excavation was decided in order to improve the properties of the ground as well reduce water ingress to a minimum.

The by-pass tunnel was located at approximately 25 meters distance running parallel to old Head Race Tunnel alignment. Limited relief in the form of already happening seepage in the original Head Race Tunnel alignment was anticipated, though it could not be considered adequate to create desirable dry excavation conditions.

When excavating through the zones, the exploratory holes drilled ahead of the tunnel phase always showed a significant amount of water seepage. Hence, a pre-injection scheme was considered necessary, to consolidate the ground and reduce the seepage to a minimum.

## 2 PRE-INJECTION SCHEME

### 2.1 *Required results of the pre-injections*

The main goal of the pre-injection works was to achieve penetration of the ground with a grout with a significant mechanical strength. The extent of the penetration had to be sufficient in order to reduce the water seepage to a virtually dry rock mass.

The densely fractured competent quartzitic schist which was the rock mass to be encountered, would require an improvement of the joint characteristics in order to achieve a general improvement of the rock mass. General weathering of the rock mass or voids in the rock were not encountered. Hence, the pre-injection scheme was a pure issue of achieving good penetration into fine joints with fillings of in-situ crushed rock material.

### 2.2 *Pre-injection strategy*

The first attempts with pre-injections with ordinary Portland cement (OPC) were not convincing. Penetration of the ground was not achieved. In most cases, one only achieved filling of the drill holes with the grout. It was therefore obvious that one had to employ a more sophisticated injection system, including a well laid out method in order to achieve the desired result.
The strategy for the injection works consisted of the following main elements:

a) Injection with pressure up to 60 bars to achieve sufficient penetration
b) Injection rounds up to 15 m ahead of the tunnel face, covering the full tunnel circumference
c) A method for the establishment of drill holes to desired length in the fractured rock
d) Injection materials which would penetrate into the rock mass to the necessary extent to provide the desired improvement result
e) A second stage of injections (if necessary) with low viscous grout
f) A simple method to verify the achieved result of the injections, and hence, decide further advance of the tunnel face

### 2.3 *Method considerations*

After the initial attempts with injection with OPC as well as microcements, it was obvious that a two-stage injection scheme would be required. Basically this consisted of a first stage injection with microcements with a limit to the injection pressure.

Following the first stage of microcement injections, one drilled a few holes ahead of the tunnel face to verify the achieved water seepage reduction and improved properties of the rock mass (by drillability). In the case the result were unsatisfactory, one would decide the second stage injection.

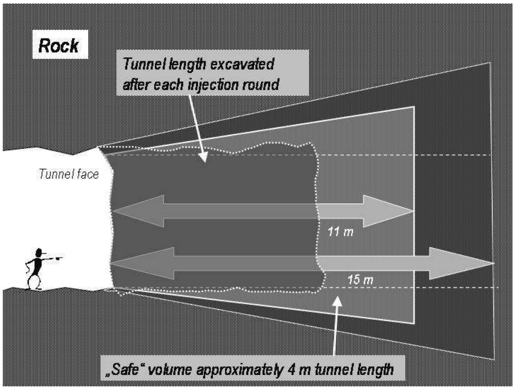


Figure 2. Principal sketch showing coverage of the two injection stages ahead of the tunnel face. The dark area indicates the coverage of the first stage with microcement injections. The lighter area shows the second stage with colloidal silica injections.

The second stage of injections was designed as an inner injection fan which was entirely covered by the rock mass volume treated under the first stage of injections. Figure 2 shows the layout of the two stages of injections.

With this method it was possible to advance the tunnel face approximately 7–8 m before a new injection cycle had to be undertaken. This provided a 'safe' volume of treated ground of minimum 3–4 m ahead of the tunnel face which had been injected during both injection stages.

The jointing of the rock mass imposed a limit to the feasible drilling length with normal percussive drilling. In untreated rock the holes would partially collapse and create difficulties with the retracting of the drilling rods. It was also difficult to achieve good and tight placements of the injection packers in open drill holes under such high extent of jointing and unstable rock.

For this reason it was decided to establish the drillholes through grouted steel pipes. Furthermore, the drilling and injection through the steel pipes was done in advancing steps with lengths of approximately 3 m. Hence, one would re-drill and inject through the same steel pipe several times, advancing longer for each time. In this way one would be re-drilling through a stabilized length and hence, one could drill further into un-treated rock. This method allows making long holes but limiting the drilling length in untreated ground to sections of 2–5 m.

This particular method was applied for injections in adverse ground with rapid hardening microcements during the construction of the Bjoröy subsea road tunnel in Norway in 1994. A controlled improvement of the ground was achieved and safe drill-and-blast excavation through the zone could be realized. (Holter et al. 1995, Holter et al. 1996)

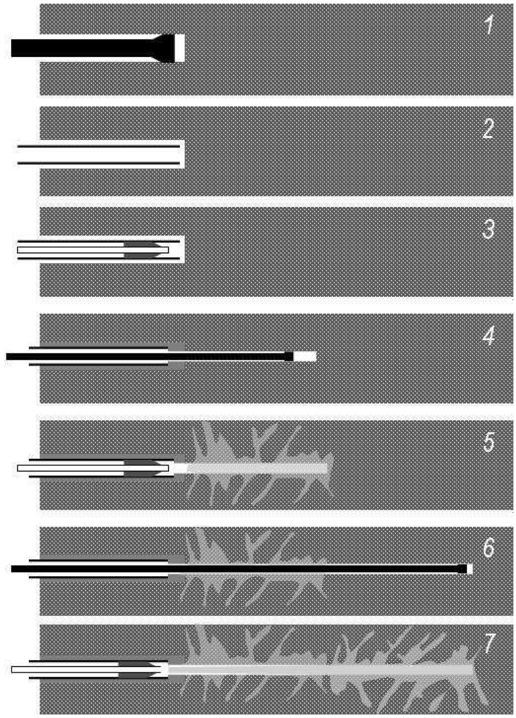


Figure 3. Sketch showing the main steps in the method of establishing drillholes through steel pipes for injection in difficult rock. Repeated drilling and stepwise advance through grouted steel pipes.

The method of stepwise advancing repeated drilling and injection through grouted steel pipes was one of the key issues to achieve any success with pre-injections under these ground conditions. This method is therefore explained in detail below. Figure 3 below illustrates the method graphically.

– *Step 1:* Drilling of 65–70 mm diameter hole to 3 m length
– *Step 2:* Installation of a steel pipe with internal diameter 50 mm (or diameter to fit the expandable packers)
– *Step 3:* Placement of packer (diameter 48 mm) at the very end of the steel pipe and injection of a stiff (very viscous) cement based grout which fills the annular space between the rock and the steel pipe. Hardening for approximately 12 hours
– *Step 4:* When the grout is hardened, drilling through the steel pipe to feasible length (approximately 3 m beyond the steel pipe)
– *Step 5:* Placement of packer and pressure injection with microcement grout for penetration into the rock mass in the drilled length of the hole.

Termination criteria for the injection set to 60 bars injection pressure, or 300 kg injected cement per meter drillhole
– *Step 6:* After hardening of the injected grout, re-drilling through the pipe and injected area to approximately 3 m beyond last drilled length
– *Step 7:* Placement of the packer in the pipe and inject (repetition of step 5)

The length of the drilling steps was adjusted to the encountered rock conditions.

Normally it was possible to advance the drilling to a depth 4–5 m beyond the preceding injection depth. The same method was used when injecting the liquid colloidal silica. However, drilling lengths were normally 5–6 m in untreated rock due to the improvement effect in the rock of the preceding microcement injections.

### 2.4 *Injection materials*

Bearing in mind the relatively low rock cover and the poor ground, it was necessary to inject with moderate pressures. In order to achieve penetration of the different joint features with moderate injection pressures, one recognized the need for two different grouts. One grout was designed to penetrate into the largest fissures and the other grout type to penetrate into the fine joint features which were partially filled with the in-situ crushed rock material (silt and clay fractions).

#### 2.4.1 *Microcement grout*

In order to achieve the best possible penetration in the first stage of injection, a microcement with particularly good penetrability properties was chosen. Furthermore the chosen microcement had rapid hardening properties. Initial set of the grout mix took place after approximately one hour with water/cement ratio 1:1 at 20°C (typical temperature in the tunnel). The grain size characteristics of the microcement was a $d_{95}$ of 16 $\mu$m.

The standard utilized mix design of the microcement grout had a water/cement ratio 1:1 with the dosage of a special dispersing agent. This gave an exceptionally good stability combined with low viscosity of the grout (Marsh cone time 32 sec) which was considered essential for the penetrability properties. The rapid hardening properties of the microcement grout enabled the continuous work without any delay between the different steps in the injection operation. For the microcement a maximum injection pressure of 60 bars was experimentally established, based on when hydrofracturing with a grout would with water/cement ratio 1:1 would occur. Furthermore, at 60 bars injection pressure with this mix a relatively limited grout take was experienced. Hence, a risk of a significant pressure build-up in the ground fed by an injected volume of grout could be neglected.

#### 2.4.2 *Colloidal silica grout*

For the second stage injection an injection gel based on liquid colloid silica was chosen. The reason for this was the penetrability properties that this gel offered. The chosen gel had a viscosity of 5 mPas in liquid state (during injection) and a particle size of 0.016 $\mu$m. This allowed for extremely good penetration under difficult ground conditions.

The workability properties of this grout were exceptionally favourable under these conditions. One could easily utilize the same injection equipment for the microcement and the colloidal silica grout, a cement injection plant with a high pressure single component plunger pump.

The open time of the colloidal silica grout could be easily and precisely controlled from approximately 10 minutes up to 2 ½ hours.

Colloidal silica is a pure mineral grout. The gelling process takes place by a physical reaction between particles of silica ($SiO_2$). The accelerator for this grout is a solution of sodium chloride. Hence, this grout is very user and environmentally friendly, imposing no health risks due to chemical reactivity or toxicity during the injection works.

Furthermore, colloidal silica offers long term chemical stability and durability (resistance to washout or leaching). For this reason colloidal silica was preferred to silicate (waterglass) based gels.

## 3 ACHIEVED RESULTS

During injection it was necessary to limit the maximum injection pressure to approximately 60 bars. The reasons for this were occurrence of backflow of grout into the tunnel and sliding of the packers in the steel pipes.

The first stage injections with rapid hardening microcement showed a relatively limited grout take of only 100–150 kg per m drillhole when the termination pressure of 60 bars was reached. Bearing in mind the seepage which was encountered in the drillholes one would expect a higher grout take.

The reason for the relatively low grout take was joint fillings which consisted of the silt and clay particles, which in turn limited the penetration of the grout created by filtration.

The first stage injection fan with microcement was always completed in the full circumference of the tunnel before the secondary fan was attempted. The reason for this was that the first stage injection provided penetration of grout into the joints with the largest apertures. Hence, the second injection stage with an extremely low viscous grout could be targeted for the finer joints and the joints which were partially filled with clay and silt.

Figure 4. A typical situation during ongoing injection works. The steel pipes for injection are seen along the perimeter of the tunnel face.

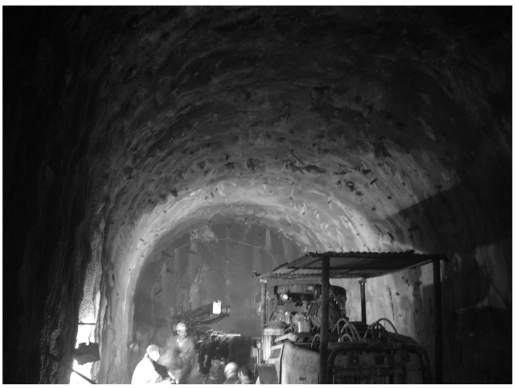

Figure 5. Tunnel under excavation through successfully treated ground in the weakness zone. Litterally dry and stable conditions at the tunnel face, as well as a favorable shape of the tunnel contour have been achieved.

The secondary fan was drilled and injected with liquid colloidal silica with a termination pressure of 25 bars, or approximately 100 kg per m drillhole length.

Injection beyond a pressure of 25 bars usually showed signs of hydrofracturing. The control of the achieved result was done in two ways. Firstly, the water seepage situation after the injection of the two stages was controlled in drillholes. Secondly, the result was observed in the tunnel contour after the excavation of the first round starting from the injection location. In this way the detailed criteria for termination of the injection were fine tuned and continuously adjusted.

The result was a literally dry and stable tunnel contour. No excessive breakouts of rock or cave-ins occurred during excavation through the weakness zone. (Figure 5).

## 4 CONCLUDING REMARKS

This proactive approach to pre-grouting fault zones ahead of the excavation face resulted in the tunnel being excavated without a single incidence of collapse and with virtually dry conditions. The effects could be directly compared to the poor conditions and heavy water flows observed in the adjacent tunnel where pre-injections were not used. A direct comparison of the effectiveness of microfine cements and colloidal silica gels versus OPC was also clearly evident.

This exercise also showed that a proactive approach to drilling and grouting with a clearly defined method statement and utilizing materials with the proper penetration, viscosity and rapid hardening characteristics can significantly reduce the construction time through such highly faulted and unstable zones (6 months in by-pass tunnel versus 18 months in original tunnel). The low cost of pre-injection in comparison with post-injection techniques can also be clearly seen. The cost for materials on this project came to a material cost of approximately Euro 1,200/m versus a similar condition in another tunnel in India where polyurethane post-injections were utilized at a cost of approximately Euro 12,000/m and currently almost 2 years behind schedule.

The post-injection then cost about 10 times more just in materials. Considering that materials are mostly less than 10% of the total cost, post-injection as a method may easily cost 20 to 30 times more than pre-excavation injection. (Stenstad, 1998)

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