EXPERIENCES WITH THE USE OF SHOTCRETE
IN SOFT ROCKS

by
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INTRODUCTION

This paper refers to the application of shotcrete in tunnels driven through soft rocks which behave as squeezing ground. More than 15 km of tunnel have been successfully driven through those rocks using shotcrete as the main stabilization measure. The described experiences refer only to a small percentage of the total excavated length, where local problems and cracks were evident in the shotcrete lining. The observational program which included instrumentation and observation of shotcrete behavior, indicated that shotcrete failure in soft rocks could be due to:

a. Stress concentrations during the redistribution process around the tunnel that induce high tensile strains and cracking of the shotcrete.

b. Inability of the shotcrete lining to resist the squeezing pressure


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when acting as a continuous lining (compresión ring) with a compressive shear mode of failure.

To describe the experiences with the use of shotcrete in soft rocks, this paper will concentrate upon two projects, namely the Chivor and Chingaza, which are being built in the northeastern part of Colombia, South America. The tunnels at these projects are being driven partially through shales, siltstones and claystones of Tertiary age, which behave as squeezing ground. One of the interesting features with the use of shotcrete at the Chingaza project, has been the remining work of more than 3 km of tunnel that had been initially supported with steel ribs and timber lagging that were badly overstressed two years after excavation. Rock bolts and shotcrete were successfully used to replace this support.

1. The Chingaza Project

The Chingaza Project, situated 40 km East of Bogotá, is being built by EAAB (Empresa de Acueducto y Alcantarillado de Bogotá) and represents the newest addition to the water supply system of the city. The Guatiquia and Chuza Rivers, which flow to the East side of the Oriental Mountain range, will be diverted to Bogotá through a combined system of tunnel and pipelines (Marulanda and Cepeda, 1975). The main feature of the project is the 28 km-long Palacio-Rioblanco Tunnel (TPR) with a horseshoe cross-section and a nominal diameter of 3.70 m.

Folded and faulted sedimentary rocks of Lower Cretaceous and Higher Tertiary age form the bedrock at the project site.
Quaternary glaci-fluvial deposits cover the rocks in most of the area. A fault near the adit of the TPR placed the Tertiary Lower Guaduas formation on the upstream side, next to the middle Cretaceous Inferior Guadalupe formation on the downstream side of the fault. The Lower Guaduas formation is mainly composed of heavily slickensided claystone with coal lenses, interbeded with thin layers of siltstone and sandstone.

2. The Chivor Project

The Chivor Project, situated 160 km Northeast of Bogotá, is being built by ISA (Interconexión Eléctrica S.A.). The scheme consists of a 237 m high rockfill dam with an inclined core, an 815 hm³ reservoir, a conduit system comprising two 5.8 km-long tunnels and two 2.1 km-long penstocks, and a power station with eight 125 MW generating units. The penstock was initially designed as a conventional surface pressure pipeline but because of unusual geological conditions the design was changed to an underground steel-lined penstock. The 3.9 m inside diameter conduit comprises a vertical shaft 182 m deep, an upstream tunnel, a second vertical shaft 267-m deep and a downstream tunnel.

II REMINING WORK

During the first stage of construction of the TPR at the Chingaza project the main support used was steel sets and timber lagging, that in some places and after some time were badly overstressed and required replacement before excavation could proceed. The timber
deteriorated very fast and the rock material was intensively weathered. Concrete could not be placed since the steel sets were heavily deformed and it was not possible to obtain the required dimensions for the operation of the tunnel. Therefore, it was decided to replace them with shotcrete and rock bolts.

1. Typical Results

The most difficult problem in this type of work refers to the amount of material that fell down once the steel sets were removed. The first estimates were done using Terzaghi's (1946) criteria for rock loads for steel support. As the sets were overstressed and in some places heavily bended, it was not unreasonable to suppose, at least following the same line of thinking in which the criteria was developed, that the height of loose material above the crown could be the maximum envisaged for the different types of material.

This remining work was quite illustrative about the validity of the assumptions involved in the dimensioning of steel sets in tunnelling work, and over the great advantages that shotcrete can offer for tunnelling.

According to the daily field records the material loosened with time due to inadequate blocking, timber deterioration etc, which have been cited as common problems associated with the use of steel sets (Mahar, et al., 1972). The successful subsequent use of shotcrete showed the ability of this system to support ground which had been
considered suitable only for steel sets. It can be argued that the material that came down represented all loosening, up to the point when the section of the tunnel became stable. Nevertheless, the developed cross section were very different from those that could be considered a self-stabilized vault. The problems encountered during the stabilization with shotcrete and bolts after the steel sets were removed and the loose material came down, were very similar to the problems encountered during excavation. Figure 1 shows the way the remining work advanced through the tunnel, and Figures 2 and 3 illustrate how the tunnel looked before and after the steel sets were replaced.

2. Progressive Failure

One of the most interesting observations during construction was a collapse that involved about 15 meters of tunnel, interpreted as a progressive failure of shotcrete.

After removal of the steel sets two at a time, the tunnel in this particular area was supported with shotcrete 20 cm in thickness and with 2 m-long grouted rock bolts around the tunnel section. At a certain station, the steel sets that had to be removed where particularly overstressed. As the operations of removal began, a sudden collapse occurred in which the material at the crown filled up the tunnel, and cracks in the shotcrete developed immediately behind. In about three hours, the failure progressed backwards some 15 m, before it stopped.
FIG. I REMINING WORK THROUGH THE TUNNEL
FIG. 2 TUNNEL BEFORE REMINING WORK
at a point where steel sets embedded in concrete were placed. The way
the failure developed, due to loads acting vertically, and therefore
concentrated at the crown, is illustrated in Figures 4 and 5.

The failure has been interpreted as the result of a rearrange-
ment of stresses along the tunnel, instead of across the tunnel, that
develops tension stresses in the shotcrete. Therefore, a progressive
failure develops at the crown of the tunnel, which resembles the
movement of a zipper. The process stops at a point where an element
with the required rigidity to absorb the induced tensile stresses is
located.

The described behavior shows that in ground with very little
stand-up-time and where immediate loosening can develop, it is essential
to incorporate into the shotcrete some element able to absorb this
rearrangement of longitudinal stresses. For that purpose, the so called
reinforcement bar trusses or "ribs" (Figure 6) have been used during
construction at the Chingaza project, and they have proven to be an
effective measure to avoid the development of the progressive failure.
One advantage of these is that they can be adjusted rather easily to the
irregular perimeter of the excavation. Also, it does not take much
time to embed them in shotcrete.

III GENERAL BEHAVIOR

1. Theoretical Considerations

Several two dimensional analytical methods have been proposed
20 cm of shotcrete

2.0-m-long grouted rock bolts

Failure stages

6WF20 steel ribs

3.80m

FIG. 4 SKETCH OF PROGRESSIVE FAILURE (ZIPPER EFFECT)

See this figure with Figure 5
FIG. 5 SKETCH OF PROGRESSIVE FAILURE (ZIPPER EFFECT)

Cave

2.0m-long grouted rock bolts

20 cm of shotcrete

Overstressed 6WF20 steel ribs

A - A

B - B

See this figure with Figure 4
Excavation line

Steel bar rib

Shotcrete

N° 4 steel bars

N° 2 steel bars

A - A

FIG. 6 STEEL BAR "RIBS" AS SHOTCRETE REINFORCEMENT
for estimating the internal pressure on a circular lining in squeezing ground. These methods assume the development of a plastic continuum under a hydrostatic state of stress and recognize that the ground itself is the major load-carrying member through arching around the opening.

In the case of soft rocks, the knowledge of the stress rearrangement has a marginal value in the design of the support after the tunnel excavation, since the main question is if the displacement of the tunnel walls during this rearrangement process is compatible with the deformation that the support can sustain without failing, or the rock itself sustain without loosening. In this case, for the evaluation of the internal pressure produced by the shotcrete-bolts systems, the equations proposed by Rabcewicz (1973) based on the hypothesis of the lining acting as a compression ring with a compressive shear mode of failure, have been used. This internal pressure has been related with expected deformations using the solution for an elasto-plastic material with dilatancy as proposed by Aiyer and Hendron (1971). In this way, the magnitude of the plastic zone for a given internal pressure can be estimated and related to the deformations.

The validity of this solution has been tested using tape and single position extensometers, with readings comparing reasonably well with the expected displacements. For example, 6 cm radial displacements were expected and measurements showed 4 cm; a plastic zone of 1.5 m deep was computed, and the readings showed that it
extended 1.7 m behind the tunnel wall. The experiences obtained using the calculated values cannot be considered mathematically exact, and they have to be interpreted with considerable judgement, but they do seem to be quite helpful in developing a support design.

It should be mentioned that tape extensometers indicated diametral displacements of 7 cm without any cracking of the shotcrete. Since the first readings were done about 3 hours after excavations, the total allowable displacements for the shotcrete could be somewhat larger.

2. Typical Cracks

Within the aforementioned frame of thinking, it is important to analyze the cracks that have been observed in the shotcrete lining used in tunnels driven through soft rocks. The cases that will be discussed correspond to zones where shotcrete was more than 7 cm thick, and where, in spite of being cracked, it was enough to support the excavation.

a. Stress Concentration

Figures 7 and 8 illustrate a typical crack in a zone where appreciable irregularities of the tunnel surface occurred. Figure 9 illustrates a case where an overbreak resulted after removing the steel sets during the remining work, and cracks developed in the vertical sidewall. Cracks of similar pattern have been observed at the corners of niches constructed to install pumps, transformers, etc, as shown on Figure 10.
FIG. 7 TENSION CRACK AT A ROCK PROTRUSION
FIG. 8  TENSION CRACK AT A ROCK PROTRUSION

See this figure with Figure 7
FIG. 9 DEVELOPMENT OF TENSION CRACK AT A ROCK PROTRUSION
FIG. IO TENSION CRACKS AT SECTION ENLARGEMENT
These cracks have a different origin to those related to blocky ground as reported in the literature (Mahar et al. 1972). They are caused by high stress concentrations at a change in shape of the opening. The yielding process that starts when the material strength is exceeded in an excavation will have a larger effect in those places where the stress concentration has a larger value. Under these circumstances larger displacements are produced, causing the shotcrete cracks during the rearrangement process.

The displacements generate tensile stresses on the shotcrete which can be sustained, depending on their magnitude. In some cases, cracks developed in places where the shotcrete was only 10 cm thick, and an additional layer (5 cm) was enough to stabilize the excavation. In other places, cracks developed in spite of thickness of shotcrete as large as 20 cm, there, it was necessary to use additional bolts which partially absorbed the stress concentrations.

b. Construction in Stages

The cases that are reported here refer to the penstock tunnel of the Chivor Project, where the Contractor excavated first the upper part through the total length of the tunnel, and later the lower portion. Steel ribs were used as main support, with shotcrete applied for protection of the rock (soft shales) against slaking.

Cracking occurred, as shown on Figure 11, in places where the upper part was supported with steel ribs and shotcrete, and
FIG. II CRACKING DUE TO CONSTRUCTION IN STAGES

1st stage

2nd stage

HE-140B (6WF 20) steel ribs @ 0.90 m

2.40 m

1.60 m

3.00 m

0.10 m

0.80 m

Thick shotcrete

Crack

Crack
the lower part with shotcrete only. This behavior has been interpreted as a consequence of the additional rearrangement of stresses after excavating the lower part of the section.

Loads that developed on the steel ribs at the upper part showed that there was movement during excavation of that part. As the lower section was excavated, an additional rearrangement occurred that caused more movements and load on the support. Figure 11 shows a sharp change of the excavated surface at junction of the two stages of excavation. Also, with the steel ribs ending at this point, a larger concentration of stresses developed in addition to that due to the sharp change in the line of excavation.

It is interesting to mention that in zones where only shotcrete was used as support (this means the support requirement were low, because at the project, shotcrete was used mainly as protection of the rock surface), some small cracks were observed at the junction of the two stages, indicating that steel ribs accentuated the problems, but were not the only cause. This means that when excavating in two stages, and where ground pressures are of some importance, it is not advisable to use shotcrete as the only support, since cracking can develop at the junction of the two stages. It is certain that after excavation of the second stage, an additional rearrangement of stresses will be produced, inducing tensile stresses due to the difference in rigidity of the support (i.e. shotcrete of different
ages), with a larger effect if the change in surface geometry is not smooth.

IV. BEHAVIOR IN HIGHLY OVERSTRESSED GROUND

The preceding examples were related to the behavior of overstressed ground that was supported without too much difficulty since the degree of overstressing was moderate. The following examples are taken from the highly overstressed ground encountered during the excavation for a vertical shaft and a particular stretch of the TPR at the Chingaza Project.

1. Anisotropic Stress Conditions

A vertical shaft, 2.8 m in diameter, was sunk through a low strength material (approx. compressive strength, 5 kg/cm²; modulus of deformation, 1000 kg/cm²) that showed an anisotropic virgin state of stress.

After 60 m of shaft excavation the high ground pressures became evident through cracking and shotcrete falls. It was intended to control the ground pressures with circular 4WF20 steel ribs, welded together two-and-two, but the support system failed completely. Heavier steel ribs were not available. A support system with shotcrete 15 cm in thickness and 2 m-long grouted rock bolts in rings of 12 bolts per 0.5 m, was used.

The analytical design of the total support had shown that the
combined support system could quite well stabilize the shaft excavation without deformations beyond the shotcrete capabilities. But, instrumentation data indicated larger movements in a preferential direction, which were in agreement with the presence of vertical cracks on the walls. These moved the most, creating a continuous cracking and the falling apart of the shotcrete.

The observations indicated that failure of the shotcrete-bolts support system occurred under an anisotropic in-situ state of stress that seems to be related with tectonic activity in the area. No stress measurements were made, but apparently, the two principal horizontal stresses were quite different in magnitude, causing a higher stress concentration when the shaft was excavated. This led to the large deformations which induced tensile stresses that the shotcrete was unable to sustain.

A rigid permanent support system less able to deform would have had huge dimensions in order to carry the loads at hand. It was therefore desirable to place a temporary support system that would permit some deformations in order to reduce the ground pressures acting on the support, inducing at the same time a non-isotropic state of stress that would allow the high shear strength of the applied shotcrete to develop.

Initially, rock bolts with shotcrete ranging in thickness from 7 to 10 cm was placed, allowing two days for deformation prior to
installing subsequent rock bolts and an additional 5 to 10 cm layer of shotcrete. Final support 25 cm in thickness of conventional concrete was placed after the shaft excavation had been completed.

2. Shotcrete - Steel Rib Interaction

A stretch of the TPR was excavated through heavily slickensided claystones with thin coal lenses, having an estimated rock-mass strength of the order of 10 kg/cm² (130 m of overburden). Shotcrete-encased light steel ribs (4WF13 welded together two-and-two) was initially placed 0.80 m between centers. Rock pressures were evident and the support system failed four days after excavation. Shotcrete 25 cm in thickness was heavily cracked, and the steel ribs were twisted and distorted, in particular the invert struts.

Figure 12 illustrates the failure process. It began with the popping of the shotcrete that covered the ribs, leaving them uncovered. Apparently, the steel ribs took a larger initial load due to their larger stiffness compared with that of the shotcrete, but the load exceeded the capacity of the ribs and the inward deflection caused the shotcrete to detach. Afterwards, the shotcrete took the loads, and failure occurred along the shotcrete-steel rib interface, causing the observed vertical cracking.

Generally speaking, it was difficult to develop a good quality shotcrete around the steel ribs. When steel ribs are too closely spaced, shotcrete applied close to the flanges is mainly composed of material
FIG. 12  FAILURE OF THE SHOTCRETE-STEEL RIB SUPPORT SYSTEM
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that rebound between the ribs.

When the shotcrete failed, the load was again passed over to the steel ribs which under these load condition could carry only a unit load of about 0.5 kg/cm². Failure first occurred at the crown and next close to the joint with the invert strut; then the ribs moved inward and the invert heaved.

An analysis of this stretch using the criteria mentioned before indicated that the tunnel could have been supported with shotcrete and rock bolts. It is thought that the steel ribs had interfered with the working system of the shotcrete, avoiding the development of its shear capacity. Therefore, all the failed steel ribs were taken out and all the cracked shotcrete was ripped off; shotcrete 15 cm in thickness was then applied with rings of 2 m-long grouted rock bolts, per meter of tunnel length. Figure 13 illustrates how the tunnel looked before and after the repair work.

It is important to notice that the described behavior does not indicate that the use of shotcrete in combination with steel ribs is always unsatisfactory. The observations have suggested the following behavior:

- Light, closely spaced steel ribs placed where shotcrete can control the rock pressures, reduce the shotcrete working capacity in changing its basic behavior. The steel ribs effect can be reduced using light steel supports spaced enough to avoid stress concentrations. In this way the ribs
Fig. 13 Repair work after failure for shotcrete - steel rib interaction
would act by reinforcing the shotcrete, which becomes the principal mean of support.

In overstressed ground, where yielding would not be desirable to reduce ground pressure (the loads carried by the ground become larger), heavy steel ribs should be used. The capacity of the steel ribs would be increased by encasing them in shotcrete, and shotcrete would act by reinforcing the principal support system (the steel ribs).

3. Heavily Squeezing Ground

Ahead of the described zone, the ground became even less competent and the shotcrete-rock bolts support system was reinforced by light steel ribs, placed 10 cm inside the tunnel section to avoid the shotcrete-steel rib interaction. The steel ribs acted mainly as protection against rockfalls.

Spiling reinforcement (fully grouted rock bolts) was placed ahead of the face at 30° in order to preserve the tunnel section ahead of the excavation. The Contractor advanced successully about 7 meters with this support system, but suddenly shear cracks were evident on the shotcrete. Figure 14 illustrates the type of cracks that were initially detected, which agree with the expected behavior when the shear strength of shotcrete is exceeded. The cracks increased without a distinct pattern, and diametral deformations of up to 30 cm were observed.
FIG. 14  SHEAR FAILURE
Additional rock samples were tested in order to review the designed support system. The material had an average compressive strength of 3 kg/cm$^2$, a friction angle of 17$^\circ$, and a very low deformation modulus of 100 kg/cm$^2$. An analysis with these new data indicated:

- The low strength and deformation modulus cause large strains when stress redistribution occurs.

- Shotcrete and rock bolts producing an internal pressure of the order of 12 kg/cm$^2$ would allow for radial deformation of about 25 cm, assuming an ideal working condition of the support system.

- In order to have no deformation the applied support system should be able to develop a load carrying capacity of the order 19 kg/cm$^2$, which is certainly a large pressure.

There were two ways to solve the problem: First, to call for a support system having compatibility with the allowed deformations; that meant to allow a certain deformation before placing the permanent support. Second, to have a support system with a load carrying capacity of about 19 kg/cm$^2$ that would impede deformations; this would have been possible with 8WF48 circular encased steel ribs, spaced 0.40 m between centers.

The first method was adopted with the following construction stages:
- Excavation by hand of 0.80 m rounds, with an increased diameter of the order of 50 cm, to allow for the controlled deformations.

- Placing of shotcrete ranging in thickness from 5 to 7 cm on the walls and on the excavation face for early support in order to minimize loosening and deterioration.

- Installing of circular 6WF20 steel ribs welded together two-and-two or 4WF13, and compressible blocking.

- Full circle spiling reinforcement.

Working personnel was kept at the excavation in order to remove partially the blocking and avoid the full load being carried by the steel ribs. When blocking was not removed on time, the steel ribs were twisted and bent. Total thickness of shotcrete was applied to encase the twisted steel ribs when the deformations were as large as expected. This was done a week after excavation, getting up to 1.5 to 2.0 m of daily progress.

Figure 15 illustrates the measurements taken with the tape extensometer and Figure 16 illustrates the measurements taken with the one position borehole extensometer at a typical station. The readings indicate large displacements in spite of a small plastic zone, due to the low deformation modulus of the material.
Figure 15: Readings with Tape Extensometer
FIG. 16 READINGS WITH ONE POSITION BOREHOLE EXTENSOMETERS
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VI REFERENCES


