Predicted versus observed ground behaviour in pilot bore in schist in northern Italy.

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ABSTRACT:

A comparison between preliminary hypothesis and the reality encountered in a 3-km pilot bore in Northern Italy is presented. Such a comparison is developed according to the geological and geomechanical aspects, and to the behaviour of the cavity, with particular attention being paid to the schist formations. In general the observed behaviour was found to be better than predicted, and this result is studied with respect to the rock mass classification schemes, characterization procedures and constitutive models used in the analyses.

1. INTRODUCTION

This paper deals with the "Armo-Cantarana" pilot bore between Liguria and Piedmont, in North West Italy.

The 3-km-long pilot bore was realised using a 3.6m diameter Atlas-Copco TBM (MK 12T) crossing Mesozoic schists and carbonatic rocks of different paleogeographic origin, under a maximum cover of 400m.

In the following sections a detailed description of this project is presented with the purpose of showing, above all, the main differences between the predicted conditions and the real situation encountered during construction of the pilot bore. Such a comparison is developed sequentially regarding the geology, the geomechanics, and the behaviour of the cavity, with emphasis on the schist formations.

2. GEOLOGY

2.1 Predicted geology

The geological knowledge about this project was essentially derived from:

- Bibliographic documentation
- Aerophotogeological study
- Geological-geomorphological survey (outcrops were lower than 5% on a 6km^2 surface).
- Four boreholes, for a total length of 410m (260m with core recovery).

Fig. 1 shows schematically the hypothesized geological profile: a southern flyschioid unit (mainly shales) is in tectonic contact with a northern mainly carbonatic unit (Calcschists), limestones and dolostones).

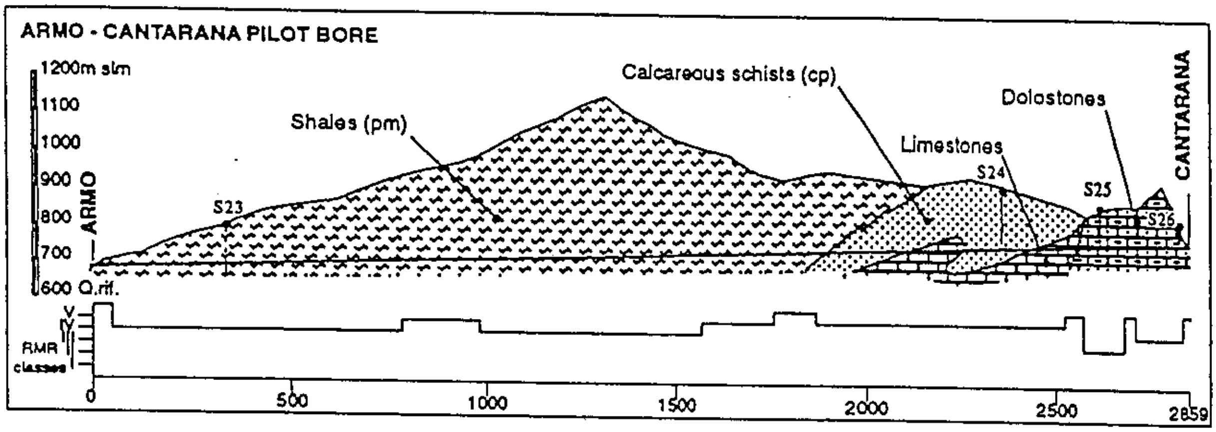

Fig.1 Predicted geology and geomechanical classification

2.2 Observed geology

Geostructural surveys were continuously carried out on the excavation walls during advancement of the pilot bore, and geolithological, structural and geomechanical information were collected.

Fig. 2 shows the observed geology: after a short distance in shales, pelitic-calcareous rocks with crenulation schistosity were encountered and extended until reaching the tectonic contact with the massive carbonatic formations.

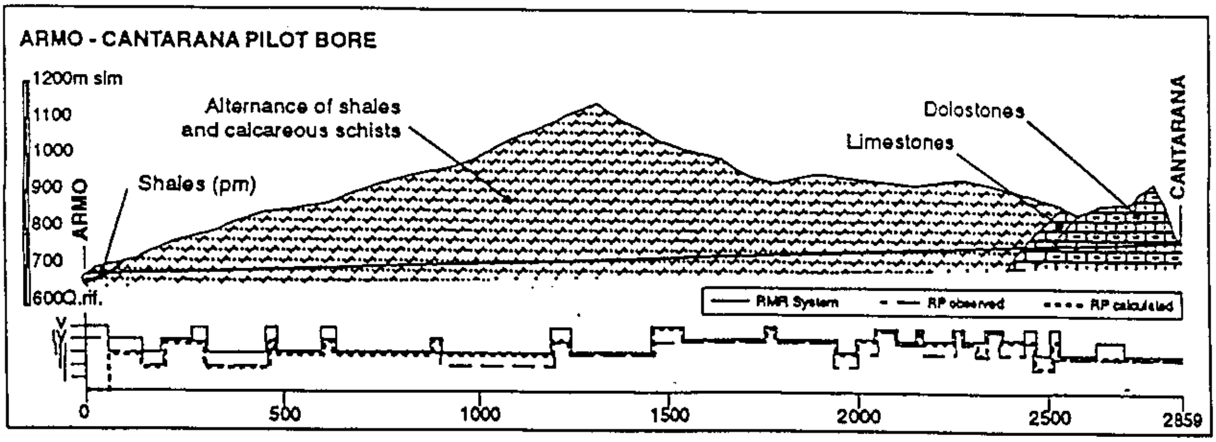

Fig.2 Observed geology and geomechanical classification; comparison between behaviour observed (RP classes) and calculated with the method of Grasso et al. (see 4.1)

3. GEOMECHANICS

3.1 Preliminary geomechanical characterization

The geomechanical study considered, in particular:

- geomechanical characterizations of the rock masses penetrated by boreholes with determination of the quality indices of the rock mass
- six geostructural surveys along significant outcrops with in situ index tests like sclerometric test, tilt test, etc.
- laboratory tests as reported in Tab. 3.1.

	Unit weight	Uniaxial compression	Triaxial compression	Brasilian	Tilt test	Point load	Schmidt hammer	Measure of P wave
Schists	19	11	4	4	2	14	13	19
Total	54	25	10	18	7	78	33	54

Table 3.1: Summary of laboratory tests (the total includes all rock type).

In Tab. 3.2 are summarized the mean characteristics of the schists; according to the preliminary hypothesis the following two rock types were distinguished:

Rock type "A": mainly shales

Rock type "B": mainly calcareous schists

By commenting on and integration of Tab. 3.2 one can observe:

- in spite of the structure of the rock and its anisotropy, it was often not possible to define, with certainty, the angle between the loading direction and schistosity, due to the pervasive crenulation of the rock at sample scale;
- the statistical distribution of the parameters is not mainly of the normal type: for example, "C_0" and "E_t" show a lognormal distribution (Fig. 3).

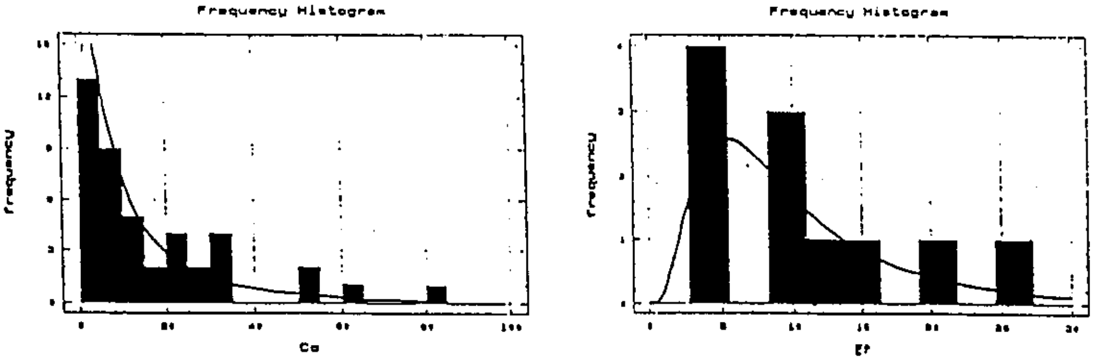

Fig.3 Frequency histograms of Co and Et derived from Uniaxial Compression test. (Co also from Point Load test)

Rock type/borehole		A/S23			B/S24			A+B		
		n°	$\bar{x}$	σ_n	n°	$\bar{x}$	σ_n	n°	$\bar{x}$	σ_n
Unit weight	γ (kN/m³)	5	26.9	0.2	14	26.9	0.9	19	26.9	0.8
Uniaxial compress. strength	C_0 (MPa)	3	15.4	10.6	8	24.4	16.7	11	21.9	16.6
Elastic modulus	E_t (GPa)	3	5.7	2.4	8	12.4	7.1	11	10.6	7.2
P wave	V_p (km/s)	5	4.0	1.5	14	4.8	0.6	19	4.6	1.0
Point Load Index	I_s (50) (MPa)	14	1.1	1.4				14	1.1	1.4
Brasilian tensile strength	T_0 (MPa)	2	3.3	1.2	2	6.6	0.1	4	5.0	1.9
Rebound value	R	11	32.2	7.7	2	49.3	0.2	13	36.3	10
Friction angle of schistosity	phi_p (°)				2	36.3	2.0	2	36.3	2.0

Table 3.2: Geomechanical parameters of schists from laboratory tests (n°=number of measurements, $\bar{x}$ = mean value, σ_n= standard deviation)

- The following relationships were derived from sclerometric and Point Load tests through correlation analysis:

$$C_o = \text{Exp}\,(1.918 + 0.037\,R)$$

$$C_o = 15\, I_{s(50)}$$

- More than 80% of the samples tested in uniaxial compression exhibited after failure a strain-softening behaviour (Fig. 4).
- The triaxial test results, in general, do not permit the establishment of failure envelopes with good correlation with either the Mohr-Coulomb or the Hoek & Brown failure criteria. This is probably due to difficulties in finding samples of similar structure for testing.

With reference to the RMR system of Bieniawski (1973), the rock mass conditions along the pilot bore were classified and the results are shown in Tab. 3.3, while and the distribution of classes is shown in Fig. 1. The Priest and Brown (1983) formulations were used for the definition of the m and s values of the failure criterion of Hoek and Brown (1980). In agreement with Hoek and Brown (1988), the Basic Mass Rating (BMR) was used excluding the correction for the orientation of discontinuities.

The corrective factor proposed by Bieniawski (1978) was used for the estimation of the deformability modulus of the rock masses.

The original in situ stress was hypothesized as of the hydrostatic type.

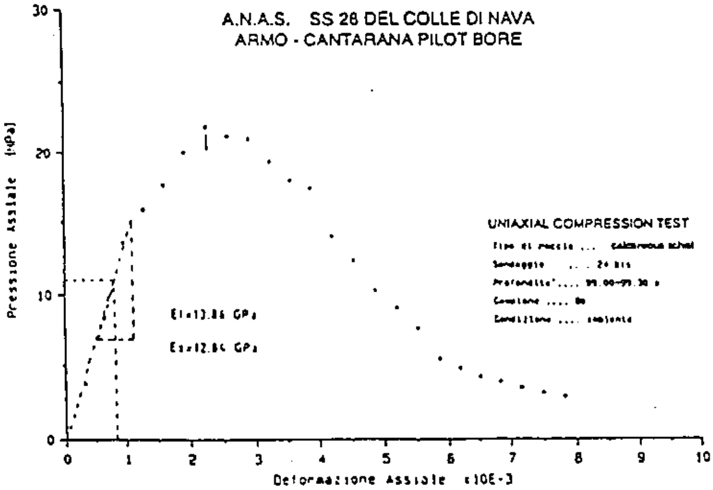

Fig.4 Typical stress - strain curve for a calcareous schist.

Rock type	A	A-B	B
RMR	49	28	9
Class	III	IV	V
BMR	54	33	14
C_o (MPa)	20	20	20
Ed (GPa)	5	4	2
c (kPa)	300	200	100
φ (°)	35	30	25
m	2.3221	1.0964	0.5563
s	0.0060	0.0006	0.00007

Table 3.3 Rock mass classification and characteristic parameter values

3.2 Geomechanical characterization during excavation

As already mentioned in 2.2, a continuous geomechanical characterization was carried out during the TBM's advancement, applying systematically the RMR system and the Q system of Barton (1974).

For the parametrical determinations carried out, it should be noted that:

- the sclerometric tests were carried out for estimating C_0,
- the value of RQD was derived from fracture frequency using the correlation proposed by Farmer (in Sheorey et Al., 1989).
- the adjustment for the orientation of discontinuity was carried out using Fig. 5, which permits an objective evaluation of the intermediated conditions.

The percentual distribution of the observed Q values and a comparison between predicted vs. observed RMR are given in Figs. 6 and 7, respectively. With respect to the preliminary hypothesis, one can note from Fig. 7, that the percentage of relatively good classes encountered is higher than predicted.

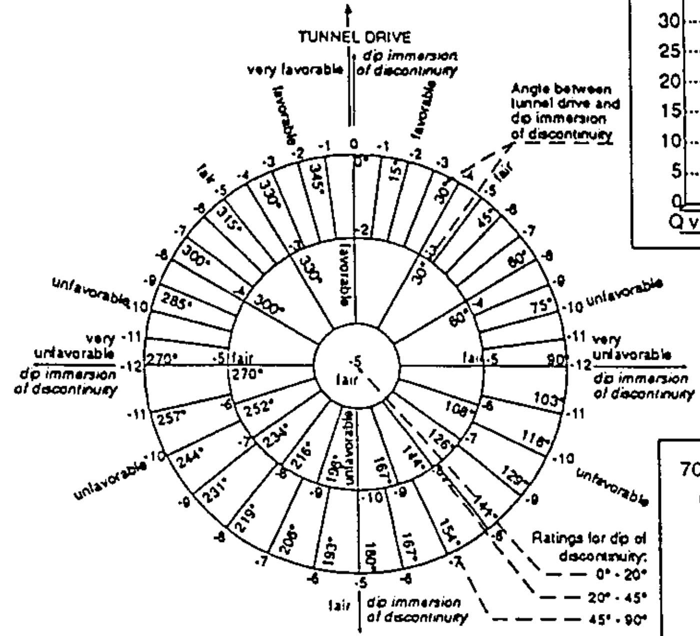

Fig.5 Circular diagram for determining the rating adjustement for orientation of discontinuities (little arrows) with respect to tunnel drive (great arrow).

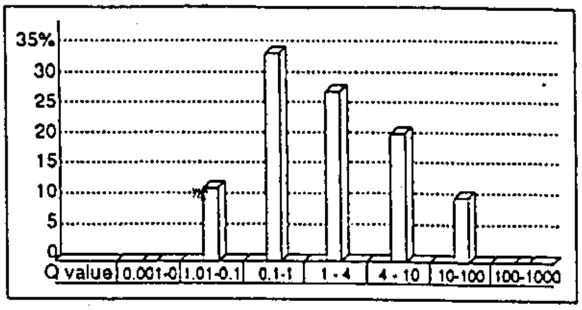

Fig.6 Observed Q values in pilot bore (Morrel, 1992)

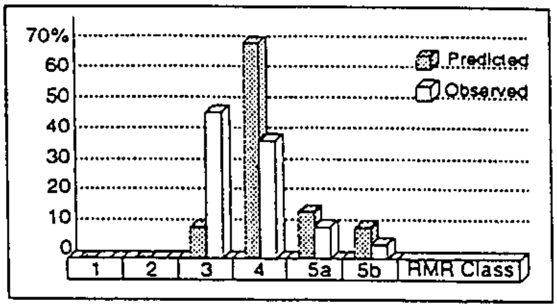

Fig.7 Predicted vs Observed RMR classes (Morrel, 1992)

By analysing the classification parameters it seems that the improvement is due mainly to the structural characteristics of the rock mass, rather than rock strength (Figs. 8-9).

It is interesting to note how such a determination can be differently conditioned by the TBM cutting action: on the one hand as an improvement in determining the rock mass integrity due to a possibly "masking" of the discontinuities; on the other hand, worsening in determining rock strength due to possible presence of coating on the excavation walls.

The two classification systems used are compared in Fig. 10 and the correlation range as reported in the literature (Bieniawski, 1984) is given: though it is not considered correct to generalize a correlation between these two systems that are conceptually different for the

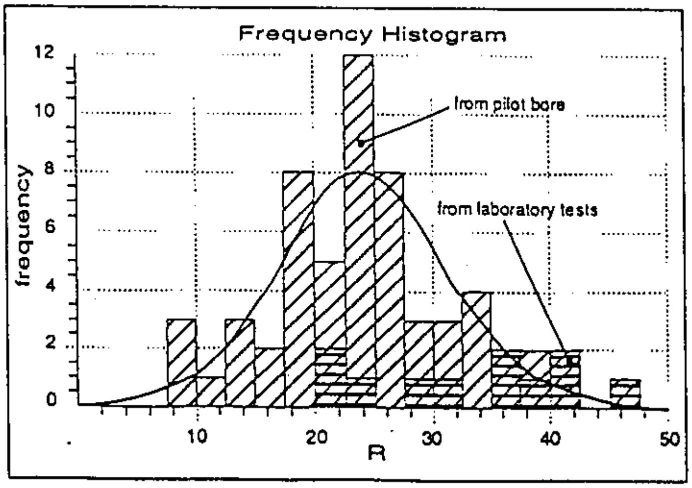

Fig.8 Comparison between sclerometric index R from laboratory tests and from pilot bore

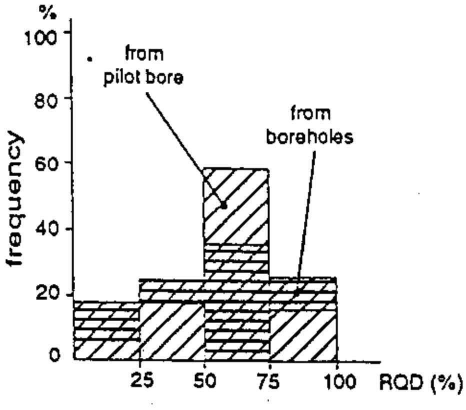

Fig.9 Comparison between RQD frequency in schists from boreholes (130 m) and pilote bore (2430 m)

"Stress Reduction Factor" is present only in the Q system, one can observe:

- 75% of the cases fall within the confidence limits shown in Fig. 10.
- about 80% of the cases are positioned below the "RMR = 9lnQ+ 44" line, thus showing that the estimated rock mass quality is generally higher with the Q system than with the RMR system.

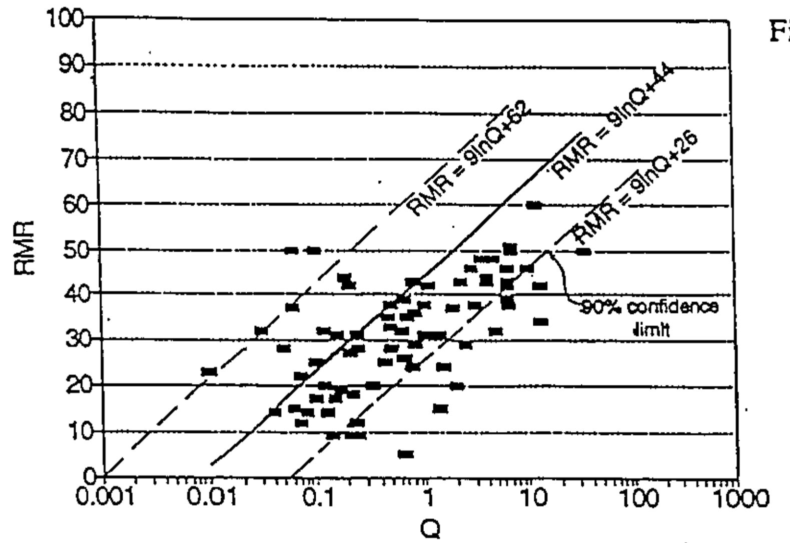

Fig.10 RMR vs Q from pilot bore (Morrel, 1992, modified)

4. BEHAVIOUR OF THE CAVITY

4.1 Predicted behaviour of the cavity

The "convergence-confinement" method was used for evaluating the behaviour of the cavity, applying the "Amberg-Lombardi" solution (1974) based on the Mohr-Coulomb failure criteria and a "elastic-brittle-plastic" constitutive law.

The support types and their theoretical support capacity so obtained are shown in Tab. 3.4.

The relationship between the predicted rock class, overburden and support type is reported in Fig. 11.

Support type	Shot-crete (*) (cm)	Bolts (**)		Steel ribs type/spacing	Support capacity (MPa)
		l (m)	pattern		
A	3	2	1.0x1.0 (S)		0.3
B	8	2	1.0x1.0 (S)		0.6
C	10	3	1.0x1.0 (SS)	UPN 100/1m	1.0
D	12	3	0.5x1.0 (SS)	UPN 100/0.5m	1.6

Table 3.4 Types of support and theoretical support capacity (Hoek & Brown, 1980); (*) steel-fibre reinforced; (**) friction anchored rock bolt (S=Swellex SS=Superswellex)

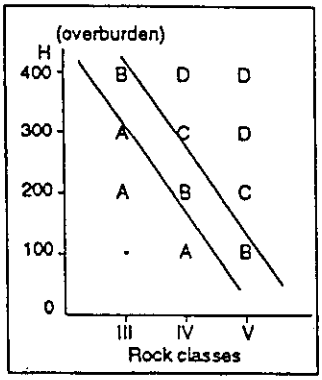

Fig.11 Rock classes, overburden and types of support

Several empirical relations are also available in the literature for the quantification of the behaviour of the cavity, in particular under squeezing conditions as reported by Barla (1994).

* In Fig. 12 are given the predictions of squeezing using the methods of Singh et al. (1992) and Verman et Al. (1990); the Q values used are those encountered in the pilot bore. As can be

noted, the "squeezing" condition concerns about 15% of the data with the first method, and about 10% with the second.

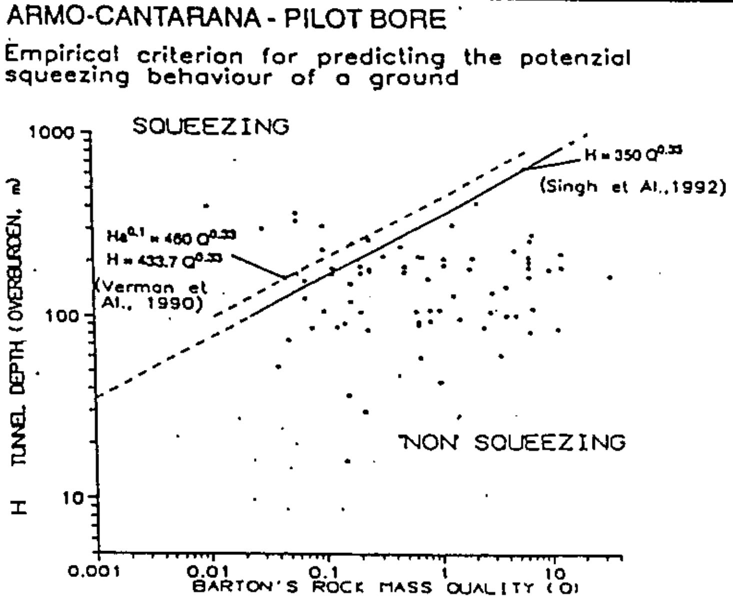

Fig.12 Predictions of squeezing using the methods of Singh and Verman

* With the method of Jethwa et Al. (1984) squeezing conditions (with different degrees) are predicted in all cases examined; the uniaxial compressive strength of rock mass (C_{om}) was calculated from the Hoek & Brown failure criteria (Fig. 13).

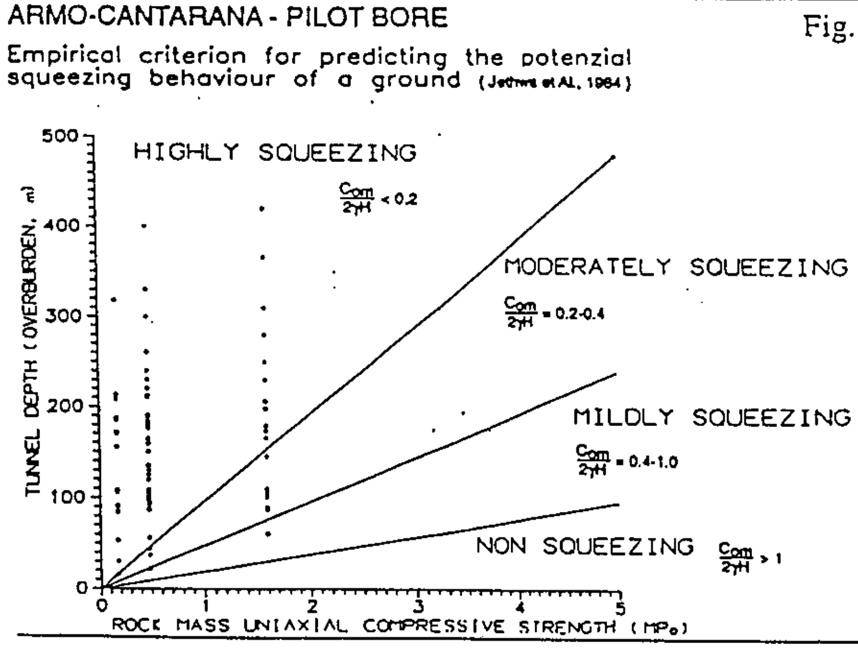

Fig.13 Prediction of squeezing with the method of Jethwa et al.

* With the method of Aydan et Al. (1991), the "squeezing" condition occurs if $C_0/\gamma H<2$. Using the mean value of C_0 (20MPa), the critical overburden is 385m and then only 3% of the cases would show "squeezing" behaviour. If using the lower quartile (C_0=10MPa), the critical overburden is halfed and 30% of the cases will show squeezing.

* The predictions of cavity behaviour according to the method proposed by Grasso et Al. (1993) are given in Fig. 2. This method uses the BMR value in the stress analysis of the excavation profile, defining the behaviour classes according to Pacher-Rabcewicz (RP), (1974). According to this method the rock mass strength on the excavation profile is reached and exceeded (Classes 3-4-5 of RP) in about 90% of the cases.

4.2 Observed behaviour

An estimation of the types of support placed during the advancement is summarized in Tab. 3.5, and a possible reference to the RP behaviour classes is also shown, on the basis of the observed deformative phenomena and places of installation of support (Note that convergence measurements were not available).

The distribution of such classes along the tunnel route is given in Fig. 2: the incidence of the 3-4-5 RP classes is about 80%.

Class RP	Type of support	Shotcrete (*) (cm)	Bolts l (m)	(S) pattern	Steel ribs Type/spacing	Support capacity (MPa)	Place of installation
(1)-2	A_0	-	1.5	(2-3)		-	Platform
3	A_1	0-2	1.5	1x1.5		0.1-0.2	Platform
	B_1	3-5	2.1	1x1		0.3-0.4	Platform
4	C_1	5-10	2.1	1x1	UPN 100/1m	0.6-0.9	Behind cutterhead-platform
5	D_1	10-12	2.1	1x1	UPN 100/1m (**)	0.9-1.2	Before & behind cutterhead

Table 3.5: Types of support installed and theoretical support capacity (Hoek & Brown, 1980); (*) with wire mesh; (**) with eventual integration of liner plates

The correspondence between the geomechanical classes of Bieniawski and the observed behaviour classes (RP) can be seen from the frequency histograms show in Fig. 14. From this figure one can observe:

- Rock class III: For the majority of cases there is a direct agreement between the rock class and the behaviour class; for almost 40% of the cases a better behaviour (RP class 2) was observed .
- Rock class IV: Direct correspondence was observed for more than 60% of the cases, with a better behaviour observed in the remaining part (40%).
- Rock class V: a correct agreement was observed for about 55% of the cases, with a better behaviour observed in the remaining part.

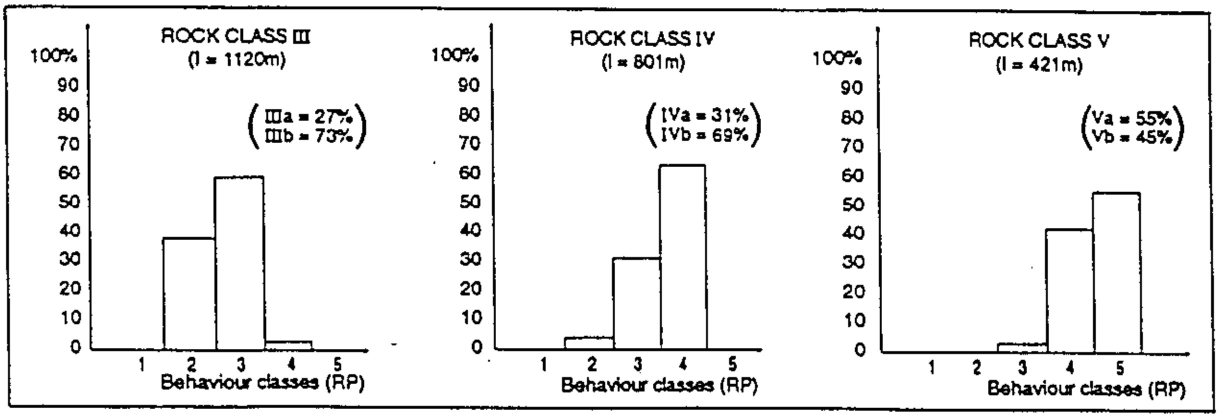

Fig.14 Effective behaviour classes observed for each rock class

Following the suggestion of Laubscher and Taylor (1976) to divide a RMR class into "a" and "b" subclasses, it is interesting to note that the percentage of direct agreement roughly coincides with the percentage of the lower subclass "b" and the percentage of the better behaviour corresponds to that of the upper subclass "a".

The relationship between the observed rock class, overburden and support type is shown in Fig. 15. In order to obtain a qualitative comparison with Fig. 11, the same letters have been used for the support types, even though the section type and the support capacity are changed. With such a premise, one might note essentially:

- a less marked influence of the overburden on support requirements,
- a tendency towards less serious load conditions on a given support type than predicted.

It is necessary to mention that the actual safety factor of support is unknown and therefore it is possible that a significant variation of such margins can exist between the hypothesized conditions and those encountered.

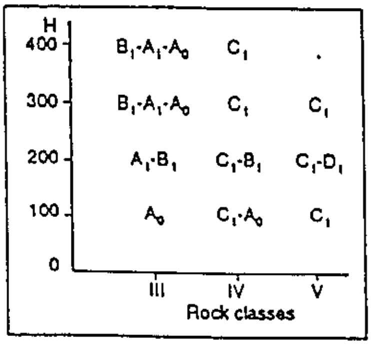

Fig.15 Rock classes, overburden and type of support installed

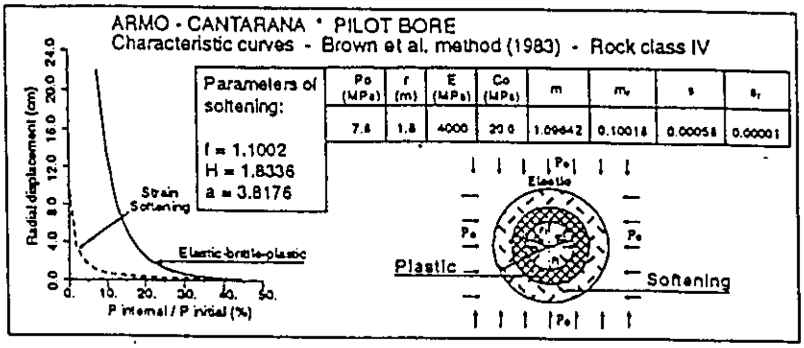

Fig.16 Comparison between characteristic curves for rock class IV, with different constitutive models.

However, considering the absence of monitoring data, it is necessary to further study the following principal arguments before generalizing the above observations:

- the evaluation of discontinuities of the rock mass, taking into account particular structures like crenulation schistosity;
- the original in-situ stress;
- the constitutive law.

With respect to the last point, for instance, it would seem to be more adequate to adopt a "strain-softening" model considering the laboratory results .

Two characteristic curves for rock class IV are compared in Fig. 16, one with an elastic-brittle-plastic model and the other with an elastic strain-softening model (Brown et Al., 1983). One can see that the shift towards the axis of radial displacement in the second case is evident, confirming a more favourable condition for reaching equilibrium as observed.

Similarly, for the intrinsic characteristics of a rock class V, it seems preferable to use an elastic perfect plastic model without a parametric decay from peak to residual values.

5. CONCLUSIONS

An attempt was made to compare the preliminary hypothesis and the reality encountered during the excavation of a pilot bore of about 3km long, dealing sequentially with the geological and geomechanical aspects and the behaviour of the cavity, with particular attention paid to the section in schists.

As far as the geology is concerned, the data collected showed a geological structure set-up which is different from that predicted in agreement with the bibliographical interpretation.

Regarding the geomechanical aspect, a relatively better quality of rock mass, a sufficient agreement between predicted vs. observed rock classes, but a much more marked variability of the same classes along the tunnel, were encountered.

The observed ground behaviour is found to be generally better than expected, with the rock mass along 20% of the pilot bore remaining presumably in elastic conditions. The influence of the overburden was less pronounced than predicted. A good agreement between RMR system and RP class was observed for about 60% of the pilot bore, with a better behaviour (RP class) in the remaining part. These proportions approximately coincide with the proportions of RMR subclasses "a" and "b", showing a tendencial upward shift of half a class in the RP system.

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