# On the use of Direct and Indirect Methods in Defining Geomechanical Properties of Weak and Complex Rock Masses - Application: Borzoli Cavern

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ABSTRACT: The determination of geomechanical properties of weak and complex rocks presents special difficulties due to the problems arizing in sampling, specimen preparation, and testing this type of materials. Questionable is also the representativeness of the results since the tests are conducted on small specimens that cannot depict the structure of a complex formation. One usually resolves these problems by using classifications and empirical relationships between rock mass properties and geomechanical indices. Nevertheless, such an exercise yields questionable results, since defining the state of the parameters consisting the geomechanics classification is difficult to say the least for complex and weak formations. A case is presented where an integrated approach was used, consisting of direct and indirect methods, aiming at overcoming these difficulties. The proposed approach combines geomechanical characterisation, with the results of in-situ testing, and back analysis. Especially for in situ testing, a new method is developed for obtaining in an efficient and economic manner specimens of weak rocks. The results of the different methods employed indicate a good agreement between expected and as monitored behavior. The paper is developed with reference to the design of the largest in cross-sectional area railway junction in Europe, the Borzoli Cavern.

## 1. INTRODUCTION

The determination of geomechanical properties of weak and complex rock masses presents special difficulties due to the problems arising in sampling, specimen preparation, and testing this type of materials. Questionable is also the representativeness of the results since the tests are usually conducted on small specimens that cannot depict the structure of a complex formation. Ideally, large in-situ tests shall be performed in order to determine the properties of the rock mass necessary for design analyses. Unfortunately, today's economic situation does not allow for such tests to be conducted. One usually resolves the issue of properties determination by using geomechanical classifications and empirical relationships that have been reported in the literature between rock mass properties and geomechanical indices. There are two shortcomings in applying this approach to weak and complex rock masses: (1) deriving parameters states is difficult, even impossible sometimes, due to the rock mass structure itself, making the derived index of questionable value; and (2) the proposed relationships are usually developed for the whole range of possible conditions (very good to very poor rock masses), therefore any particular relationships for the lower range representing weak rock masses (usually represented by very few data points) is sacrificed for a generally better correlation for the whole range.

In the following sections, an integrated approach is developed as applied in the design of the Borzoli Cavern consisting of indirect and direct methods. The following methods are to be discussed:

1. the empirical method where rock mass properties are determined by scaling down intact rock ones using empirical relationships function of rock mass quality indices;
2. the experimental method where tests on representative samples are conducted; and
3. the observational method where back analysis of the observed behavior is employed to derive rock mass properties;

The proposed approaches were subsequently applied in the characterization of the Borzoli Cavern, a railway junction that was to be excavated in a cataclastic zone.

## 2. DESCRIPTION OF THE PROJECT

The high-speed railway system under planning and construction in Italy involves, among the others, the construction of the link between Milan and Genoa, the principal harbour of Northern Italy.

The connection between the Valico line (Genoa-Milan) and the new Genoa-Voltri harbour, involves the Voltri tunnels with a 8km-long main line and 4km of connection tunnels. The construction of the Voltri tunnels began in 1992 with a first part of 7.5km to be accomplished by 1998.

The Borzoli Cavern belongs to the Voltri tunnels system; its function is to permit the connection of the main line with a pre-existing secondary line. The 101m-long Cavern is located in a weak rock and under an overburden of 220 m, and it has a cross-section area varying from 160 to more than 338m$^2$, the largest cavern in Europe. Positioning of the Cavern was determined by railway alignment constraints.

## 3. GEOLOGIC SETTING

The Cavern is located in a structurally complex zone, which has been interpreted as the tectonic boundary between the two different orogenic domains of the Alps and Apennines (Vanossi et al., 1984) and which is crossed by the Sestri-Voltaggio tectonic line.

It develops into an ophiolitic unit, composed of a main basaltic body which is crossed by thick lenses and slices (having a width of more than tens of meters) of serpentinites and talc-serpentinoschists. The unit has undergone metamorphism at pressure-temperature conditions of the lower part of the Greenschists facies which resulted in intense multiphase deformations and tectonisation causing the rock to become extremely fractured and very often finely cataclased. Normally the most fractured zones coincide with talc-serpentinoschists lenses. The most evident structural feature is the inhomogeneity of the rock mass with reference to the project scale, being characterized by large portions of altered cataclastic rocks in which are embodied relatively less altered, preserved solid serpentinite blocks. There is a complex, irregularly-developed network of interconnecting discontinuities, making difficult to describe them in terms of statistically clustered sets. Discontinuities are often associated with large aperture values (5 to 10mm) and are filled with clay minerals.

A brief review of the investigations conducted during the life of the project follows.

## 4. SITE CHARACTERIZATION

### *4.1 Investigations for the Executive Design*

During the site investigation phase (1987-1990) of the Genoa-Voltri project, a comprehensive study was conducted (Eusebio et al., 1994) with the aims of (1) reconstructing the geologic profile along the tunnels to be excavated, and (2) providing the necessary geomechanical information for the Executive design. These investigations included: study of the regional geology, mappings of surface outcrops (25km$^2$), drilling (2000m of core logs from 45 boreholes), geophysical surveys (refraction and reflection seismic lines, 12km), laboratory and in-situ tests on rock specimens obtained from the drill cores. Despite its extent, the site investigations did not reveal any specific information for the Cavern, since the boreholes did not reach the Cavern location. As consequence of this, the expected geomechanical conditions were based on the investigations conducted in adjacent tunnels. Based on these studies two possible scenario-models were identified for the conditions to be encountered during the Cavern excavation: (a) basalt-rock formation belonging to class IV of Bieniawski's (1989) Rock Mass Rating (RMR) system, and (b) serpentinite-rock of RMR class V. The latter proved to be the formation to be encountered during the excavation of the Cavern (see section 4.2). These inferences were based on information obtained from boreholes (5), geostructural surveys (2) and a number of direct and indirect tests including geophysical investigations. Table 1 summarizes the characterization indices of the second scenario, while the relative intact rock properties are shown in Table 2.

### *4.2 Site Characterization with Focus on the Cavern - An Introduction*

Excavation of the tunnel segments preceding the Cavern allowed for refinement of the results of the preliminary investigation. Specifically, a horizontal borehole was carried out in 1995 along the entire length of the Cavern to be excavated. Dilatometric tests (see Section 4.2.2) were conducted along the borehole, and specimens were obtained for laboratory testing. In addition, monitoring a section in a tunnel segment of ground characteristics similar to those expected for the Cavern provided an insight into the behavior of the rock mass with respect to the excavation.

TABLE 1: SELECTED ROCK MASS PARAMETERS AND PROPERTIES FOR THE SERPENTINITE-ROCK (EXECUTIVE DESIGN)

| Parameter | Values for Serpentinite-rock |
|---|---|
| RMR (representative value) | V (8) |
| RQD, % | <25 |
| $\phi_m$, $^o$ peak - residual | 20 - 10 |
| $c_m$, MPa ” | 0.020 - 0.005 |
| $E_m$, GPa ” | 1.00 - 0.50 |

Note: RMR: Rock Mass Rating (Bieniawski, 1989); RQD: rock quality designation, $\phi_m$: rock mass friction angle, $c_m$: rock mass cohesive strength, $E_m$: deformation modulus.

At first, it was realized that weak serpentinoschists are to be expected in greater proportions than initially expected. These formations were associated with cataclastic zones of very poor rock conditions consisting of rock pieces of few centimeters.

During the excavation of the main tunnels and of the Cavern (1996-97) an additional series of tests was conducted. This time, specimens were obtained with a special sampling technique (see Section 4.2.2), aiming at obtaining representative undisturbed samples of the rock mass.

In the following sections selective results from these investigations are given. The methods used in defining the properties of the rock mass correspond to two approaches, *indirect* which includes empirical and observational methods, and *direct* for the experimental method.

### *4.2.1 Indirect Approach - Empirical Method*

A preliminary geomechanical model of the rock mass was devised in which some basic simplifications were adopted. The rock mass was thought to be characterised by a complex structure built up of two components: a prevalent soil-like component of altered serpentinoschists and, a rock-like component of hard fractured serpentinite.

Following the scheme, it was assumed that the shear strength of the rock mass should be mainly controlled by the weakest domains, and that the presence of fractured blocks could have had a more important role in increasing the overall deformation modulus of the rock mass.

Considering that initially it was only possible to sample the rock-like component, the indirect empirical method was used to derive the representative parameters of the rock mass. In doing so, intact rock strength and deformability parameters were scaled-down to the rock mass values using geomechanical indices and/or empirical relationship.

A large number of tests were conducted on core samples obtained from the horizontal borehole, including uniaxial compression tests, triaxial tests, Brazil tests and direct shear tests on altered rock specimens and discontinuity surfaces (11 and 8, respectively). The derived intact rock properties are summarized in Table 2.

TABLE 2: INTACT ROCK PROPERTIES FOR THE EXECUTIVE DESIGN AND UPDATE INVESTIGATION PHASES

| Phase<br>Property | Executive Design | Update (from horizontal b/h[1]) | Cumulative |
|---|---|---|---|
| $\gamma$ (kN/m$^3$) | 27.2[2] (1.6) *[18]* | 25.5 (1.5) *[47]* | 26.0 (1.5) *[65]* |
| $\sigma_{ci}$ (MPa) | 27 (26) *[6]* | 44 (24) *[16]* | 39 (24) *[22]* |
| $E_t$ (GPa) | 11.7 (6.4) *[6]* | 18.5 (6.5) *[16]* | 16.6 (6.5) *[22]* |
| $\sigma_t$ (MPa) | 5.3 (2.6) *[10]* | 4.3 (2.3) *[12]* | 4.7 (2.4) *[22]* |
| $v_p$, (m/s) | - | 5399 (389) *[32]* | 5399 (389) *[32]* |

Note: [1]b/h: borehole; [2]Mean value, (standard deviation), *[number of tests]*; $\gamma$: unit weight; $\sigma_{ci}$ and $\sigma_t$ uniaxial compressive and tensile strength of intact rock, respectively; $E_t$: tangent modulus; and $v_p$: longitudinal wave velocity.

As it was mentioned earlier, the borehole confirmed the worst scenario anticipated in the previous investigation phase of the project (serpentinite-rock of RMR class V). Despite difficulties in applying the RMR system in characterizing rock masses of such type, classification of the borehole logs was attempted. In summary, two subclasses were identified: Va, and Vb, the latter corresponding to a completely broken and highly altered rock formation.

Following the derivation of the RMR, rock mass properties were established based on empirical relationships proposed in literature. Table 3 summarizes these derivations while Figures 4 and 5 present them graphically together with the ones obtained with direct tests (see Section 4.3.2); allowing for comparison between the different approaches employed.

TABLE 3: ROCK MASS PARAMETERS AND INDICES

| SERPENTINITIC UNIT | SUBCLASS | |
|---|---|---|
| Selected parameters-indices, properties | Va | Vb |
| RMR | 17 | 4 |
| BMR (Basic RMR) | 22 | 9 |
| RQD (%) | 10-30 | 0-10 |
| Discontinuity frequency $\lambda$, (Nb/m) | 20 | 30 |
| Deformation modulus, $E_m$, GPa - Reference & function of | | |
| Serafim & Pereira, 1983; *f*(RMR) | 2 | 0.9 |
| Bieniawski, 1978; *f*($E_t$,RQD) | 2.3 | 1.5 |
| Hobbs, 1981; *f*(RQD,$E_t$,J) | 3.2 | 3.2 |
| Rock mass cohesive strength $c_m$, MPa | | |
| Bieniawski, 1989; *f*(BMR) | 0.110 | 0.045 |
| Manev & Avr.-Tacheva, 1970; *f*(ci,$\lambda$) | 0.054 | 0.032 |
| Rock mass friction angle $\phi_m$, degrees | | |
| Bieniawski, 1989; *f*(BMR) | 16 | 10 |
| Trunk and Hönisch, 1989; *f*(BMR) | 19 | 13 |
| Hoek-Brown criterion parameter $m_b$ | | |
| Hoek-Brown, 1988; *f*($m_i$,BMR) | 1.160 | 0.729 |
| Hoek-Brown criterion parameter s ($\times 10^{-4}$) | | |
| Hoek-Brown, 1988; *f*(BMR) | 1.72 | 0.41 |

Note: factor J in the Hobbs formula describes rock mass degree of fracturing; GSI: Geological Strength Index.

The analysis of failure and post-failure behaviour of the specimens tested, showed an elastic-brittle-plastic behavior. For the rock mass a similar behavior was presumed. Estimation of the residual values of the rock mass strength was based on Hoek and Brown (1988) formulations.

### *4.2.2 Direct approach*

In the following sections the methods used to directly evaluate the actual geomechanical properties of the rock mass are discussed.

- *Rock Mass Deformability*

In order to determine the in situ deformability of the rock mass 9 dilatometric tests were performed along the horizontal borehole. Table 4 summarizes the results of these tests giving reference to the RMR classes defined for the correspondent borehole segments.

One can observe that the unloading modulus ($E_{mY}$) is approximately double the $E_{mD}$ one.

- *Rock Mass Shear Strength*

During the construction of the Cavern and the main tunnels**,** a program of sampling of the crushed serpentinoschists for laboratory testing was planned in co-operation with the Rock Mechanics Laboratory (LMR) of the Swiss Federal Institute of Technology Lausanne (EPFL). One of the main difficulties associated with performing laboratory tests in weak rocks is the acquisition of representative specimens because, in general, conventional drilling equipment tends to damage tectonized rocks. Besides the advances in core sampling that have been made in the last decades, e.g. triple tube barrels, use of such an equipment is still difficult and expensive.

TABLE 4: DILATOMETRIC TESTS[1] RESULTS

| RMR SUB-CLASS | No of Tests | Unloading Modulus $E_{mY}$ (GPa) | | Loading Modulus $E_{mD}$ (GPa) | |
|---|---|---|---|---|---|
| | | average | st.dev.[2] | average | st.dev.[2] |
| Va[3] | 4 | 3.1 | 1.2 | 1.5 | 0.8 |
| Vb | 4 | 2.1 | 1.7 | 0.9 | 0.6 |

Note: [1]Maximum pressure 4.3-5.8MPa; [2]The standard deviation is calculated according to the procedure suggested by Snedecor and Cochran cited at Lacasse and Nadim (1996) - it is a function of the range of values and the number of tests; [3]One of the tests resulted in high values ($E_{mY}$=12.4GPa and $E_{mD}$=6.3GPa) and for caution was disregarded.

To overcome these difficulties a *portable sampler* was developed in LMR aiming at minimizing disturbances on specimens caused by drilling. It consists of a hydraulic jack which pushes cylindrical metallic tubes ending in a sharp barrel shoe into the ground (Figure 1). This simple equipment has been applied heavily tectonised rocks in tunnels under construction in the Swiss Alps and it has been proven to be efficient and rapid in use. It is flexible and its capacity depends on the hydraulic jack and the stiffness of metallic tubes used.

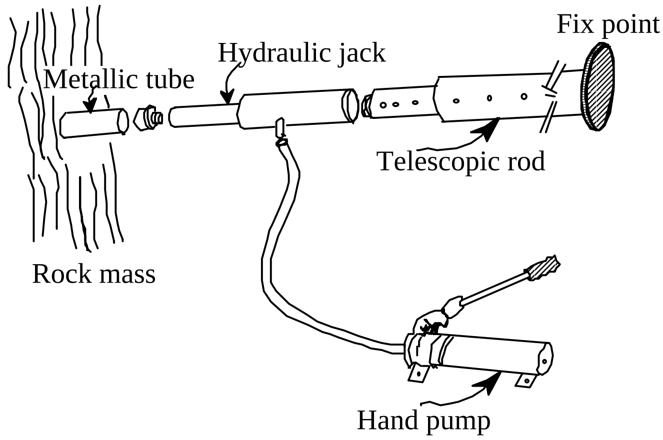


Figure 1: Sketch of the *portable sampler*

About 30 specimens were collected from the main tunnels and Cavern site. Then uniaxial and triaxial compression tests were carried out in LMR. Some direct shear tests were also performed in Geodata's rock mechanics laboratory. Table 5 summarizes the Coulomb criterion parameters as derived from the most representative triaxial test envelopes.

TABLE 5: RESULTS OF TRIAXIAL TESTS

| Envelope | c (MPa) | $\phi$ ( $^o$ ) | $\sigma_{c(m)}$ (MPa) | $\beta$ ( $^o$ ) |
|---|---|---|---|---|
| 1 | 0.3 | 15 | 0.8 | 30 |
| 2 | 0.5 | 12 | 1.2 | 45 |
| 3 | 0.7 | 11 | 2.3 | 90 |

Note: $\beta$ is the angle defined by the direction of the axial load and the schistosity planes.

The following comments can be made based on the results of the tests conducted:

- The Mohr envelopes appear to be linear for practical purposes. A non-linear behavior could be observed only for very low stress levels and/or in the ductile region (Hoek and Brown, 1980, see Figure 2).

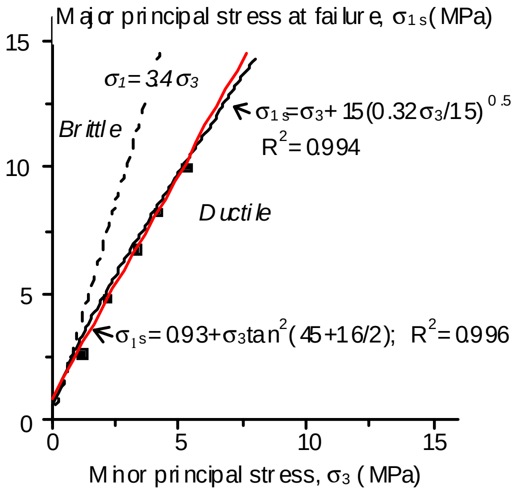


Figure 2: Typical result of triaxial tests

- The three uniaxial compression tests performed resulted in values between 0.26-0.39MPa. It should be said that these specimens were characterized by a chaotic structure and did not exhibit an anisotropic behavior. This structure was considered to be representative of the prevalent conditions observed on site. These results, in addition to the equivalent $\sigma_c$ values as derived from triaxial tests, suggest that the strength of the complex material tested lies in the range of soil-type material, and/or in the range of transition soil to weak rocks.
- For the cases where schistosity was well developed and associated with a distinct orientation an anisotropic behavior was observed (see Table 5).
- The stress-strain curves clearly showed the influence of the minor principal stress, $\sigma_3$, on the strength, and an elasto-perfectly plastic behavior was observed even for low confining pressure levels (Figure 3). It was presumed that this model was representative of the behavior of Vb sub-class. This is in accordance with the recent indications in literature (Hoek and Brown, 1997).
- In Figure 5 the typical direct shear tests results are reported. A linear envelope fits well this data with $c_m$=0.054MPa and $\phi_m$=29$^o$.
- Some problems arose in fitting the experimental data with the Hoek-Brown failure criterion, because of the questionable choice between the formulations for intact and broken rock. For the latter, it is difficult to define the correct input value for $\sigma_{ci}$ considering that the values obtained for the rock-like serpentinite could not be representative of this property (see relevant comment in 4.3.2).

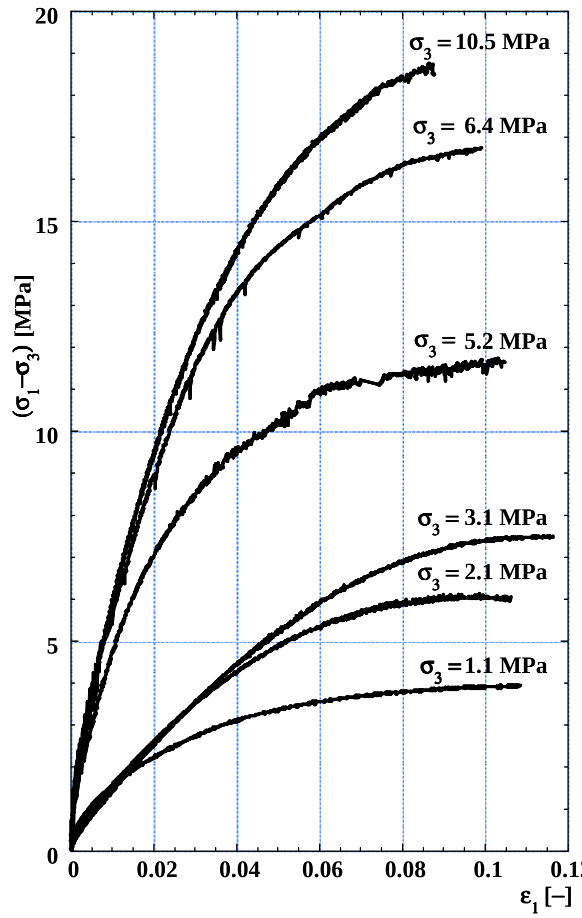


Figure 3: Typical stress-strain curves for broken samples of serpentine-rock

### *4.3.2 Comparison Between Direct and Indirect Methods*

*- Rock Mass Deformability*

In Figure 4 the dilatometric test results are compared with the results of the empirical indirect methods. It is possible observe from this figure that the modulus calculated by Hobb’s (1981) empirical formulation is equal to the $E_{mY}$ value for practical purposes, while the other empirical formulations result in values which are closer to the $E_{mD}$ one. The former value was assumed to be representative of the peak behavior, while the latter more representative of the residual one, taking into consideration the complex structure of the rock mass. Based on this assumption a relatively lower deformation modulus than in the Executive design phase was considered.

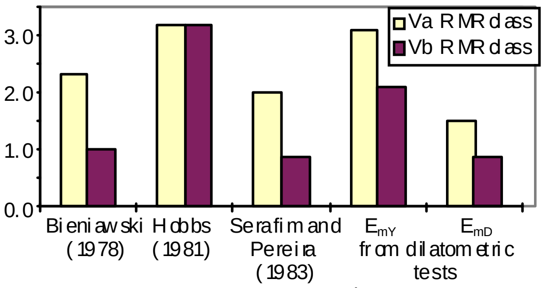


Figure 4: Deformation modulus as obtained by different approaches

*- Rock Mass Shear Strength*

In Figure 5 the results obtained with the different methods are compared (Vb sub-class), showing also the selected design range of values. The following comments can be made:

- For the stress conditions of interest (lithostatic pressure, $\sigma_o$=4.8MPa) there is a good agreement between the Coulomb linear envelopes as defined in the Executive design phase and the update characterization.
- The Hoek-Brown envelope for undisturbed rock mass obtained with the constant values given in Table 3, and with the average value ($\mu$) of $\sigma_{ci}$ (39MPa) appears to be quite different from the other ones. This is probably due to the different intact rock properties that are associated with the worst rock mass conditions (RMR class Vb). A better agreement is obtained by assigning a value of $\sigma_{ci}=\mu-\sigma$, where $\sigma$ is the standard deviation, and assigning to $\mu_i$ half of its original value. These reduced values are believed to be more adequate in representing the intact rock properties associated with class Vb.
- Despite the elasto-perfectly plastic behavior observed testing the specimens, a decrease from peak to residual values was considered in order to simulate the rheologic behavior of the rock mass with reference to the design of the final lining.

To verify the results of the project investigation phases back-analysis of the observed behavior was performed for a tunneling section in the vicinity of the Cavern. The main conclusions of this analysis are presented in the following section.

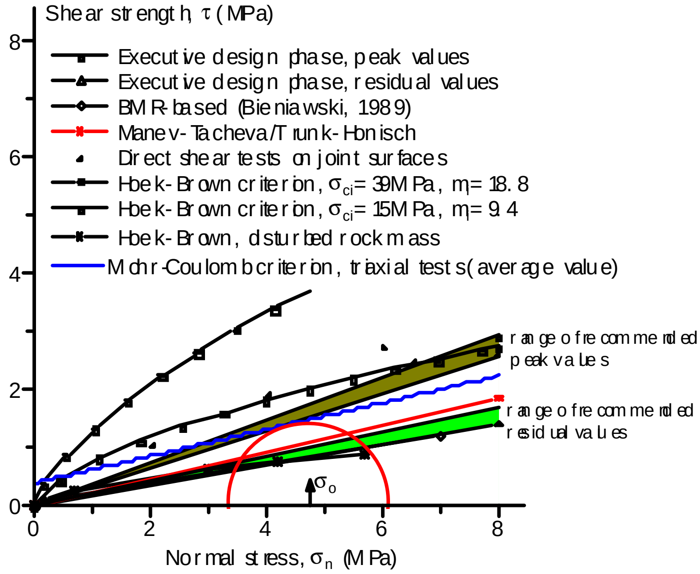


Figure 5: Rock Mass shear strength envelopes as resulted from indirect and direct approaches

*4.3.3 Indirect Approach - Observational Method*

To check the validity of the predicted parameters, a monitoring program was planned to verify on a real scale the accuracy of the model defined during the investigation phases. In doing so, along a tunnel segment characterized by very similar geological conditions (rock mass of Va class) as the ones that were expected to be encountered during the excavation of the Cavern, various monitoring instruments were installed including: three multiple-point extensometers (24m-long, with four anchors), five and nine load cells behind and at the base of steel ribs respectively, nine strain gages, and five reference studs for convergence measurements.

According to the applied approach in tunnel design, to limit the deformation of the cavity in this zone, the geomechanical properties of the rock mass were enhanced with cemented steel bars, dimensioned in accordance with the theory of *effective cohesion* (Grasso et al., 1989). The primary support system was composed by heavy steel ribs and fiber-reinforced shotcrete. A final concrete lining was also designed to bear the calculated loads.

The convergence-confinement method (solution of Amberg and Lombardi, 1974) was used to calculate the equilibrium conditions of the primary support using the selected properties: $c_m$=0.050MPa, $\phi_m$=25°, and $E_m$=3.1GPa. The residual values of the improved rock mass were defined according to the aforementioned theory. A comparison between the expected and observed values of equilibrium are given in Table 7. A reasonable good agreement was obtained confirming the validity of the geomechanical model.

TABLE 7: COMPARISON BETWEEN PREDICTED AND OBSERVED - AS MONITORED - INDICES

| Property | Predicted | Observed |
|---|---|---|
| $R_p/R_o$ | 2.5-3.0 | 1.5-2.0 |
| $P_e$ (MPa) | 0.5-0.6 | 0.2-0.6 |
| convergence (cm) | 15-20 | ~10[1] |

Note: Rp,Ro : Plastic and equivalent radius respectively; Pe: pressure at ground primary support equilibrium; [1]total value, including the "lost" convergence, according to the formulations of Guenet et al. (1985).

The detailed design of the cavern followed based on the results of the updated characterization, and the systematic data collection and model verification during the construction. As reported by Grasso et al. (1997) the approach followed for the design of the cavern was in particular based on this principal sequence: (a) detailed numerical analysis of each excavation and support phases, (b) continuous verification of the geomechanical and behavioral hypotheses by geostructural investigations and extensive monitoring, and (c) pre-defined counter-measures to optimize the design for the actual conditions encountered.

## 5. CONCLUSIONS

A case history is presented where an integrated approach was used, consisting of direct and indirect methods, aiming at sound definition of weak and complex rock mass properties. The proposed approach combines geomechanical characterisation, with the results of in-situ testing, and back-analysis.

The study suggests a good agreement between the different approaches used in defining the geomechanical model. With reference to the Executive design phase investigations, shear strength properties were confirmed by the update characterization, while a relatively lower rock mass deformability resulted from in-situ tests.

On the basis of the updated characterization the design of the Borzoli Cavern was realized. Systematic geomechanical investigations and monitoring during the construction of the Cavern confirmed the presumed model.

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