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Mechanical properties of pervasively fractured carbonate rock mass

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Mechanical properties of a pervasively fractured carbonate rock mass

Les caractéristiques mécaniques d'un rocher carbonatique intensement fissure

Mechanische Eigenschaften einer sehr zerklüfteten Kalkformation

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SUMMARY - The cavern of the Vomano hydroelectric plant is being excavated in the "Bisciaro" formation, a fine grained limestone which is characterized by intense and pervasive fissuring, even at the scale of laboratory specimens. The results of the laboratory and in situ tests are presented; it was concluded that strength values obtained in laboratory tests could be reliably used for the static analysis of the cavern because of the prevailing influence of the pervasive fissuring of the rock. The variations of the strength parameters with the increase of fissuring are compared with the values suggested by Hoek for a rock mass as a function of its quality index.

RESUME - La caverne de la centrale électrique de Vomano, est excavée dans la formation "Bisciaro", formée d'un calcaire à grain fin, totalement fissuré, même à l'échelle des échantillons de laboratoire. Ici l'on présente les résultats obtenus en laboratoire et en situ, et l'on peut conclure que les valeurs de résistance de laboratoire pourraient être utilisées avec confiance pour l'analyse statique de la caverne grâce à l'influence dominante de la fissuration répandue du rocher. Les variations des paramètres de résistance par rapport à l'augmentation de la fissuration sont comparées avec les valeurs suggérées par Hoek pour un rocher en fonction de son indice de qualité.

ZUSAMMENFASSUNG - Die Kaverne für die Wasserkraftanlage Vomano wird gerade in der "Bisciaro" Formation gegraben. Das ist ein feinkörniger Kalkstein, der auch im Labor eine starke und verbreitete Zerklüftung aufweist. Die Ergbenisse von Labor - und in situ - Versuchen werden hier wiedergegeben; daraus hat man geschlossen, dass die im Labor bestimmten Festigkeitswerte auch für die statische Analyse der Kavernen benutzt werden konnen, da die verbreitete Felszerklüftung den uberwiegenden Einflüss darauf ausubt. Die von der Felszerklüftung abhangigen Veränderungen der Festigkeitsparameter werden mit den Werten verglichen, die Hoek für eine Felsmasse in Funktion ihrer Qualitätsindex vorgeschlagen hat.

INTRODUCTION

In order to increase the installed capacity of the Vomano hydroelectric plant (Abruzzo, Central Italy) and to introduce pumped-storage facilities, it was decided to build a new underground power station consisting of 2 groups (a Pelton turbine and a reversible unit), with a total installed power of 375 MVA.

The new plant is located at a depth of about 650m below ground surface, at the foothills of the Gran Sasso, the highest peak of the Apennine range. The machine hall, which is now in the process of being excavated, is about 72m long, 30m wide and 35m high; at the foot of the cavern, a 50m deep and 18m diam. shaft will house the reversible unit.

In the design stage a detailed study of the rock mass was performed by means of exploratory galleries (Fig. 1) from which 45 boreholes for a total length of 2050m were drilled. These galleries were also utilized to carry out in situ plate loading tests, flat jack tests, seismic velocity investigations and state of stress determinations with the CSIR doorstopper cell, CSIRO triaxial cell and hydraulic fracturing.

In this paper the geomechanical properties of the rock material and of the rock mass, which present some peculiar features, will be discussed; the problem of state of stress determinations was dealt with in ref. 1.

GEOLOGICAL AMD STRUCTURAL CONDITIONS

The machine cavern is located in the "Bisciaro" formation of Miocene age. The rock is a fine-grained limestone composed mainly of calcite crystals (65-80%), the remaining being quartz grains (15-20%) and phyllosilicates (5-15%).

The structural pattern of the region is characterized by many important overthrusts generated during the Alpine orogenesis (Late Miocene-Pliocene). The formation was subjected

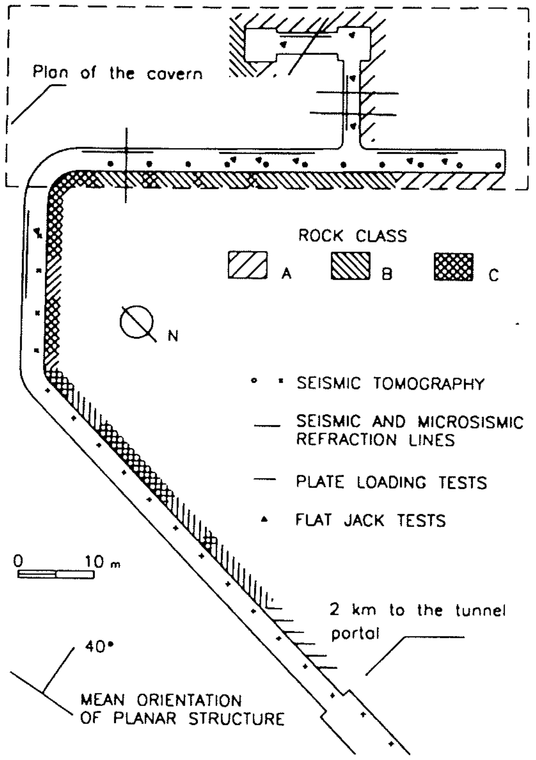

Fig. 1 - Plan of the investigation galleries

to intense shear deformation both along the bedding planes and along other surfaces.

At the large scale, the formation can therefore be considered as being composed of "lithons", which are wedge-shaped or lenticular domains of rock, delimited by layers of more concentrated shear deformations.

A pervasive fissuring is frequently present also throughout the rock material, so that even laboratory samples are often affected by weakly bonded fractures and weakness surfaces. Cataclastic flow and pressure solution phenomena caused mobilization of the calcite, originating a network of calcite veinlets, many of which were subsequently sheared.

The samples of more intensely sheared material can often be easily disgregated into scales with polished surfaces and sharp edges.

The general structure of the rock mass, although very complex in detail, can be approximately represented by a polar symmetry where the equatorial plane dips W or WSW and has an inclination of 35-45°.

Notwithstanding its intense and diffuse fissuring, the joints or the weakness surfaces are irregular and often undulated; on the whole, the rock mass appears to be closely packed and very tight, as is also confirmed by the very low permeability observed in all the boreholes and by the negligible inflows into the excavations.

On the basis of these observations it was felt that the mechanical behaviour of the laboratory samples of the rock could be representative of that of large volumes of rock mass. For this reasons the laboratory investigations performed were more intense and more detailed than would normally be necessary for this type of rock engineering project.

LABORATORY TESTS

Before the testing, the laboratory samples were classified into four classes on the basis of a visual examination of their fissuring conditions.

- IN seemingly intact samples
- IF samples containing a few healed fractures and/or weakness surfaces
- FR samples containing many partially healed and often slickensided fractures or weakness surfaces
- SH samples intensely fractured and sheared, often formed by weakly healed slickensided scales

The mean values of the mechanical parameters for the various classes are summarized in Table 1. We notice that the rough visual classification described above is matched by the mechanical behaviour, and there is a gradual worsening of the mechanical parameters from the IN class to the SH. For instance, although scattered, the P-wave velocity distributions are clearly different for the various rock types (Fig. 2).

Table 1 - Characteristics of the rock material

Class	IN	IF	FR	SH
Density (Mg/m3)	2.63	2.63	2.63	2.61
Sonic velocity (km/s)	4.81	4.42	4.23	3.36
Tensile strength (MPa)	2.12	1.34	0.43	0.26
Uniax. compr. strength (peak value) (MPa)	57.2	12.0	14.2	8.4
Uniax. compr. strength (residual value) (MPa)	10.5	3.1	2.5	1.9
Young modulus in uniaxial tests (GPa)	34.5	20.9	20.4	8.1
Young modulus in triaxial tests (GPa)	38.5	31.5	32.[illegible]	[illegible].0

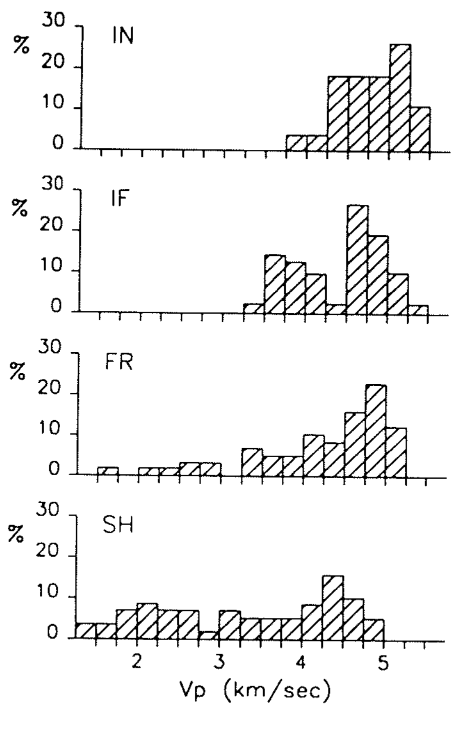

Fig. 2 - Histograms of sonic velocity for the various rock types (unloaded samples)

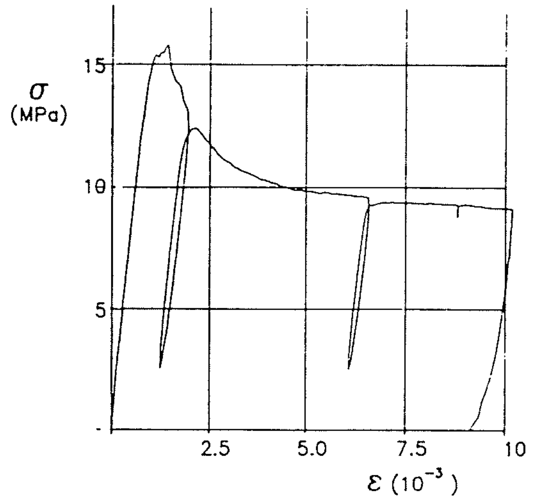

Fig 3 - Typical load-displacement curve in triaxial compression

Most of the specimens were subjected to uniaxial or triaxial compression (at a confining stress varying from 2.5 to 20 MPa) utilizing a servocontrolled stiff testing machine, so that the complete load displacement curve could be determined (Fig. 3). The stress-strain behaviour is generally almost linear up to load levels near peak load; beyond the peak, the rate of strengh decrease is initially large and then it gradually tails off; unloading-reloading cycles in the post peak region do not show a marked modulus degradation with the plastic strain.

The results of the uniaxial and triaxial tests on the various lithotypes are shown in Fig. 4. A sharp decrease of peak strength with fissuring intensity can be noticed; the residual strength is however less affected, so that the difference between peak and residual conditions becomes less important when the rock is intensely fractured, especially for the higher confining pressures.
The shape of strength versus confining stress diagrams is always strongly concave downwards, so that a non linear Hoek-Brown failure criterium appears best suited for analysing the data

$$\sigma_1 = \sigma_3 + (m_i\, \sigma_c\, \sigma_3 + s\, \sigma_c^2)^{1/2} \qquad (1)$$

where σ_c and m_i are two parameters (the former corresponding to the uniaxial strength of the intact rock) and s=1 for the laboratory specimens.

It can be noticed that the peak strength values of the specimens of the IN class are strongly scattered (especially in uniaxial load conditions) and some of them are very low. In fact, upon examining these specimens, it was found that the rupture often occurred along preexisting weakness surfaces which were not apparent before the test; probably no really intact specimen of the size utilized for the tests can be extracted from the formation.

Given this situation, we should really adopt a value s_d<1 for our laboratory samples (even for the "intact" lithotype) and utilize a relation

$$\sigma_1 = \sigma_3 + (m_d\, \sigma_c\, \sigma_3 + s_d\, \sigma_c^2)^{1/2} \qquad (2)$$

As the true value of σ_c is not known, it was deemed better to fit the experimental data according to a relation

$$\sigma_1 = \sigma_3 + (m_d'\, \sigma_{cd}\, \sigma_3 + \sigma_{cd}^2)^{1/2} \qquad (3)$$

in which σ_{cd} corresponds to the uniaxial strength of the "damaged" laboratory samples, either for the peak or residual conditions. The fitted curves are obviously identical to those which could be obtained by rel. (2) assuming an arbitrary value of σ_c. The parameters in rel. (2) and (3) are related by

$$m_d' = m_d\, (\sigma_c/\sigma_{cd}) \qquad s_d = (\sigma_{cd}/\sigma_c)^2 \qquad (4)$$

The fitted curves and the strength parameters m_d' and σ_{cd} obtained by means of a non linear least square analysis are shown in Fig. 4 and in Table 2. Making the (reasonable) guess that σ_c = 100 MPa, values of m_d can be calculated; they are also shown in Table 2.

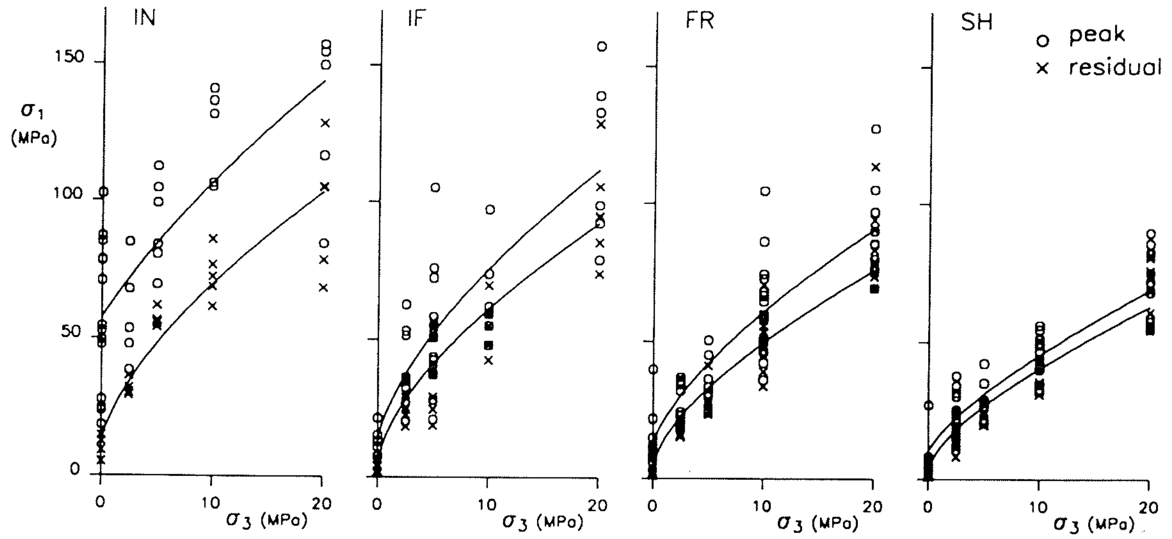

Fig. 4 - Triaxial strength of the various rock material classes and best fitting Hoek-Brown curves for peak and residual conditions

Table 2 - Analysis of the triaxial tests according to a Hoek-Brown strength law

Rock type	IN	IF	FR	SH
Number of tests	35	37	50	53
Peak strength				
σ_{cd} (MPa)	57.3	14.1	12.9	8.4
m'_d	10.6	29.3	18.7	15.5
m_d	6.07	4.13	2.41	1.29
Residual strength				
σ_{cd} (MPa)	12.6	2.9	1.9	1.5
m'_d	26.9	90.1	90.0	63.8
m_d	3.22	2.61	1.72	0.95

Given the conditions of pervasive fissuring, it was felt that the behaviour of the investigated lithotipes may reveal general indications about the strength of a fractured rock mass. An instance of the application of this technique was presented by Hoek (ref. 2-3), who used the strength obtained for laboratory specimens in post-peak conditions; on the basis of these results and of other tests in fractured rocks, Hoek proposed the following relationships between rock mass quality (expressed by the RMR index) and its strength parameters m_a and s_a.

peak strength

$$m_a/m_i = \exp\,[(RMR-100)/28]$$
$$s_a = (\sigma_{ca}/\sigma_c)^2 = \exp\,[(RMR-100)/9]$$

residual strength (5)

$$m_a/m_i = \exp\,[(RMR-100)/14]$$
$$s_a = (\sigma_{ca}/\sigma_c)^2 = \exp\,[(RMR-100)/6]$$

If the behaviour of the damaged rock material is representative of that of a rock mass, and if the relation (5) is a good estimate of the rock mass behaviour, the following relations should hold for the parameters obtained in our laboratory tests.

$$m_d/m_i = (\sigma_{cd}/\sigma_c)^{9/14} \qquad \text{peak conditions}$$
$$m_d/m_i = (\sigma_{cd}/\sigma_c)^{6/7} \qquad \text{residual conditions} \qquad (6)$$

These relationships are compared in Fig.5 with the experimental data from the tests on

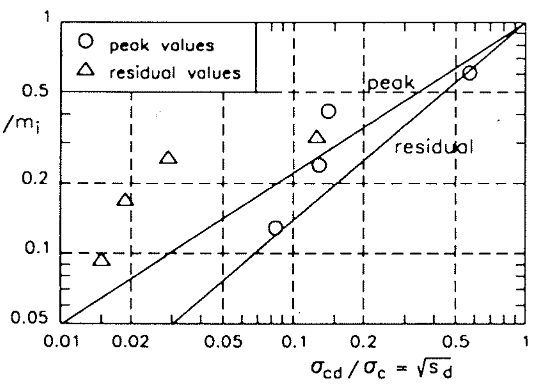

Fig 5 - Comparison between the triaxial strength parameters for a fractured rock mass according to the relationships proposed by Hoek (full lines) and those provided by the tests on the "Bisciaro" limestone

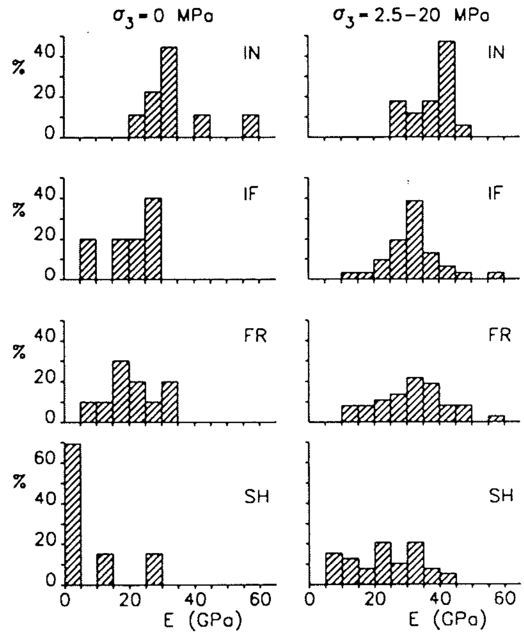

Fig. 6 - Histograms of the Young moduli in uniaxial and triaxial conditions

the Bisciaro limestone of the Vomano site, assuming, as a reasonable guess, that m_i=10 and σ_c=100 MPa.

The data relative to the peak conditions, though highly scattered, are not in contrast with Hoek's relationships; instead the values of m_d in residual conditions appear to be considerably higher; this indicates that the residual strength curves, when scaled down with respect to the uniaxial strength are steeper than the peak strength curves, and that the strenth drop between peak and residual conditions is smaller than predicted by rel. (5).

The Young moduli are clearly influenced by fracturing conditions and by confining stress (Fig. 6). In triaxial conditions moduli are not significantly influenced by the confining pressure (in the range 2.5 to 20 MPa) and so they have been considered en bloc; they are markedly higher than those obtained in uniaxial load conditions, which means than even a small confining pressure is sufficient to bring about

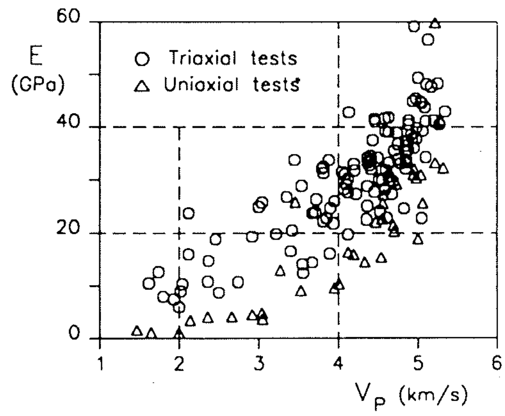

Fig. 7 - Young moduli in uniaxial and triaxial conditions versus sonic velocity (in unloaded specimens)

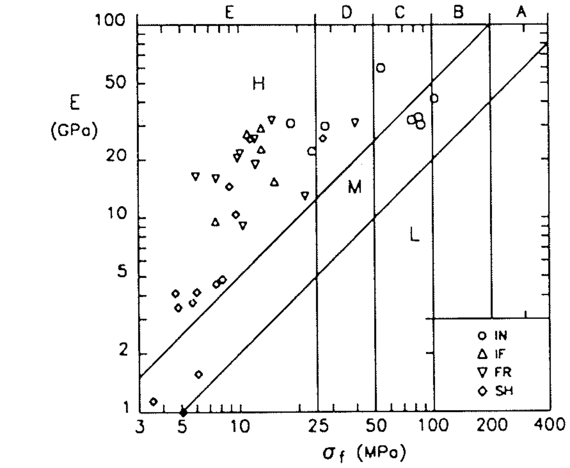

Fig. 8 - Deere's diagram for the "Bisciaro" limestone of the Vomano site

a marked increase in stiffness.

The Young moduli, both for uniaxial and triaxial conditions, are strongly correlated with the seismic velocities (which were measured on unloaded samples in laboratory conditions); the velocity appears therefore to be a sensitive indicator of fissuring conditions (Fig. 7).

The strength-modulus relationship is shown in the Deere diagram (Fig. 8). The trend of the data shows that increasing fracturing affects the strength more than the stiffness does, and so the modulus ratio may be as high as 2000-3000; only in the intensely sheared lithotype (SH) the decrease in the strength is paralleled by a marked drop in stiffness values. This behaviour, which was also observed in other brittle carbonate rock types, departs somewhat from what has been described for igneous or metamorphic rocks and for artificially cracked rocks (ref. 4-5); in these cases microcracking causes a sharp drop in the moduli and only a slight decrease in strength.

The peculiar behaviour of the carbonate rocks is very likely caused by the fact that the great mobility of their constituent minerals induces a partial healing of the fractures existing in the rock material: as long as the cohesive bonds along these fractures are not overcome, the stiffness of the material remains high.

CHARACTERISTICS OF THE ROCK MASS

Rock mass quality

The rock mass crossed by the exploratory drifts and by the boreholes was classified at the mesoscale (that is at size of 2-5 m) according to three quality classes.

A - rock mass composed mainly of IN and IF rock types

B - rock mass composed mainly of FR rock type

C - rock mass composed mainly of FR and SH rock types

An example of the zoning along the walls of the drifts is shown in Fig. 1.

Except for class C, the RQD values are generally high (Fig. 9 and Table 3) and the average RQD is 75%; moreover it must be pointed out that many of the fractures observed in the cores can be caused by the action of the coring bit on weakness surfaces which in situ were instead partially healed.

Geophysical seismic methods were extensively used to characterize the rock mass. In most of the boreholes sonic logs were carried out using a 1.0m long probe.

The seismic characterization of the rock mass for larger volumes was accomplished by means of crosshole techniques over distances of 20-40m. Similar indications are obtained measuring the transit time along travel paths in the rock mass between different exploratory drifts (Fig. 1). Finally seismic and microseismic refraction techniques were utilized on the walls of the drift, where receiver spacings were respectively 5 and 0.75-1m; the latter technique was adopted in particular to evaluate the thickness of the loosened zone around the drifts, especially at the sites of the plate loading tests and of the flat jack tests (Fig. 1).

Table 3 - Properties of the rock mass (number of tests in parenthesis)

Rock class		A	B	C
RQD (%)		89	70	52
V_p (km/s)		5.02	4.81	4.72
E_d (PLT)	(GPa)	30.4 (11)	28.7 (2)	-
E_e (PLT	(GPa)	41.7 (11)	34.0 (2)	-
E_d (PLT, superf. layer)	(GPa)	10.9 (11)	8.8 (2)	-
E_e (PLT, superf. layer)	(GPa)	20.9 (11)	15.8 (2)	-
E_e (FJ)	(GPa)	18.2 (13)	17.1 (13)	8.8 (2)

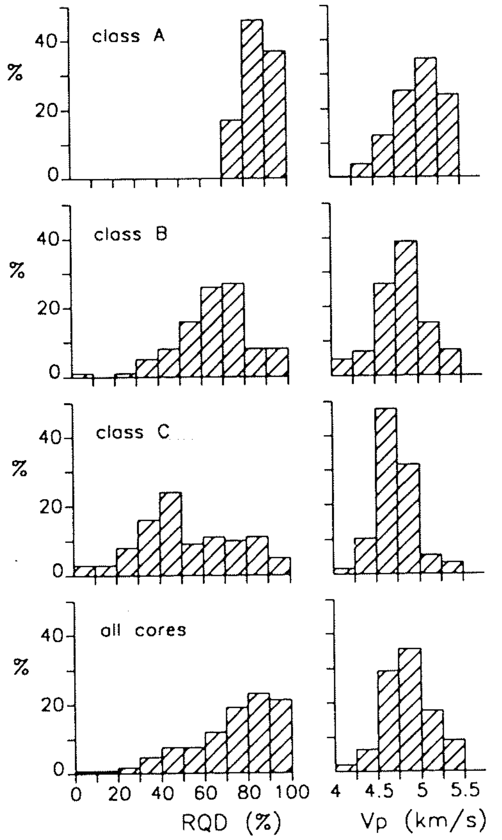

Fig. 9 - Histograms of RQD and sonic velocity for the various rock classes

Fig. 9 shows the distribution of the P wave seismic velocities obtained from the sonic logs. The mean and modal values decrease with the quality of the rock mass, but the differences in velocities between classes and the scatter of the data within each class are smaller in comparison with laboratory specimens. This fact can be attributed to the influence of confining pressure for in situ tests, which is considerable when the rock is fissured, and probably also to the effects of different saturation conditions.

Some of the scatter observed in the sonic log diagrams can be attributed also to the effect of an anisotropy of the formation, even though in most of the boreholes the axis is inclined between 40° and 60° with respect to the planar structure.

For instance the analysis of the transit times in the tomographic net (Fig. 1) shows that in the investigated zone (where B and C rock classes are prevailing) the velocities are related to the angle θ between the path direction and the normal to the planar structure of the rock mass (Fig. 10). Assuming that the V_p versus path angle relation is

$$V_p^2 = V_1^2 \sin^2 \theta + V_3^2 \cos^2 \theta$$

it is possible to evaluate that the maximum velocity V_1 along the structural plane is equal to 5.4 km/s and the minimum velocity to 4.2 km/s.

Deformability of the rock mass

The moduli of the rock mass have been determined by means of 13 plate loading tests, with a plate diameter of 0.5m, using the technique commonly adopted in Italy which is described in detail in ref. 6. The load on the rock surface was applied by means of a flat jack to assure a uniform pressure distribution; the displacements were measured at 3 load levels (4, 8 e 12 MPa), both at the wall surface and at various depths along the loading axis (up to 3.0m).
The moduli of the undisturbed rock mass were calculated on the basis of the relative displacements of the points located at 0.25 and 3.0m; the relative displacements of the point at 0.25m and at the surface give the moduli of a loosened rock layer. For each load level both the deformation modulus, E_d, corresponding to the total displacement up to that load level and the "elasticity" modulus, E_e, corresponding to the unloading curve, were determined.

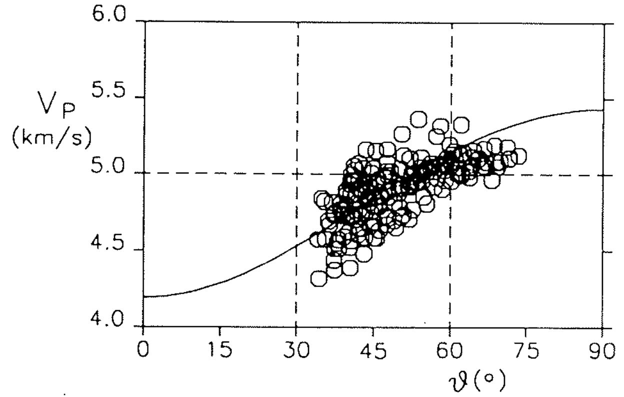

Fig. 10 - Seismic velocity versus path angle in the tomographic net

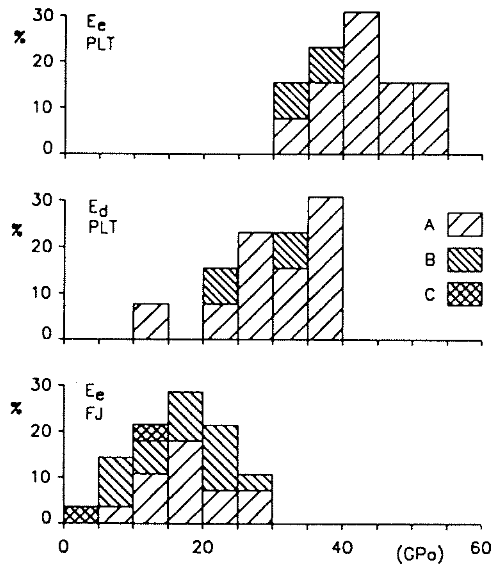

Fig. 11 - Histograms of the in situ deformation (E_d) and elastic (E_e) moduli determined by means of plate loading tests (PLT) and flat jack tests (FJ)

For a load level of 8 MPa the distribution of the moduli is shown in the histogram of Fig. 11 and the mean values are summarized in Table 3. It was found that the moduli are only slightly influenced by the load level. Unfortunately most of the tests were carried out in class A rock, so that class B is poorly characterized and class C not at all.

The moduli are clearly related to the quality of the rock, as is shown by the difference of the mean values in classes A and B and also by the correlation between the static and the dynamic moduli (Fig. 12); the latter were obtained from the sonic log velocities in the hole underneath the axis of the plate. A comparison with the data collected in site investigations in other fractured

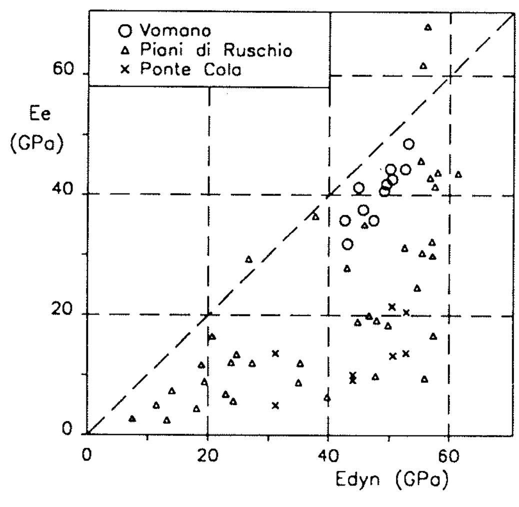

Fig. 12 - Correlation between in situ moduli E_e (from PLT) and dynamic moduli E_{dyn} (from sonic logs) for carbonatic formations

carbonate formations (also plotted in Fig. 12) shows that the data of the Vomano site are in agreement with the general trend, but are characterized by comparatively higher values of the ratio between static and dynamic moduli.

A comparison between the in situ moduli and those determined in laboratory specimens shows that the latter are on average lower, even for the specimens tested in triaxial conditions; this is very likely due to the disturbance caused by coring.

To evaluate the in situ moduli use was also made of the results of the flat jacks tests (the aim of which was mainly that of determining the tangential stress on the walls of the drifts).

For these tests, a 0.6m wide and 0.3m deep slot was used; the displacements were measured using anchors placed along the axis of the flat jack with spacings of 0.2 and 0.4m. The readings were interpreted on the basis of load-displacement theoretical relationships obtained by means of a finite element code (ref. 7); these relationships provide moduli which are only 60% of those provided by the approximate relations proposed by Jaeger and Cook (ref. 8).

Fig. 11 clearly shows that the moduli obtained from the flat jack tests are similar to those obtained from the plate loading tests in the loosened layer and markedly lower than those of the undisturbed underlying rock mass.

Strength of the rock mass

Flat jack tests provide some indications about the strength of the in situ rock mass. The cancellation pressure of a flat jack is in fact a good estimate of the tangential stress on the walls of the test gallery. If the rock has an elastic behaviour, we should expect that the tangential stress is 2-2.5 times the initial vertical stress; on the contrary, if the rock around the excavation yields plastically, the tangential stress should corrispond to the uniaxial strength of the rock mass in residual conditions.

The mean value of cancellation pressures was about 8 MPa for the flat jacks located in of class A and B rock masses and 3.5 MPa for class C rocks. These values are somewhat higher than the residual strength of laboratory specimens respectively in IF-FR and SH lithotypes; this discrepancy could be explained assuming that the rock cores are damaged by the vibrations and stresses induced by the coring bit.

CONCLUSIONS

The cavern of the Vomano power plant is excavated in a limestone formation in which pervasive fissuring is present throughout the rock material, so that even laboratory samples are affected by many weakly healed fractures.

Laboratory testing of the rock material shows that the strength envelopes are markedly concave downwards and that the strength parameters (according to a Hoek-Brown strength law) are clearly correlated with the cracking conditions of the specimens. The strength drop between peak and residual conditions decreases with the degree of initial cracking and becomes percentually less important when the confining pressure increases.

Pervasive fracturing of the specimens affects their strength more than their stiffness, so that the specimens are characterized by high modulus ratios. Very likely the network of partially healed fractures markedly reduces their strength, but until the cohesive bonds along these fractures are not overcome, the stiffness of the material remains high. It is the high mobility of the calcite which probably accounts for the healing, and consequently the behaviour of other types of rock masses can be quite different.

The seismic velocity and the Young moduli in situ are higher than those determined in laboratory specimens (which are probably somewhat disturbed by the coring); this indicates that the rock mass is very tight and closely packed, as is confirmed also by its very low permeability. Main joints and faults very likely have a subordinate influence compared with the pervasive fracturing of the rock materials and, therefore, the choice of the strength parameters for the static analysis of the cavern was mainly based on the results of the laboratory tests.

Finally, the results of this study may be of general interest, because they contribute to a better understanding of the behaviour of rock masses, of which a pervasively fractured rock material can be considered a good model. It was found that the results obtained are only partially in agreement with the well known relationships, proposed by Hoek (ref. 2-3), between rock strength parameters and quality indices for the rock masses; in particular these relationships appear to underestimate the residual strength of a yielded rock mass.

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