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ABSTRACT

The paper illustrates the design criteria and the construction phases of a generating and pumped-storage power plant built at Edolo in the Val Camonica. Three large caverns have been excavated with parallel longitudinal axes to house the rotary valves room, the machine hall and transformer hall respectively. Considering the size of excavation, particularly for the machine hall, a careful investigation of the rock mass was carried out for the determination of the design parameters. In particular the values of the peak and residual cohesion have been estimated by means of an original procedure which is based on convergence measurements, field test and calculation through semplified analytical model. On the basis of the studies carried out during the excavation of the upper parts of the cave it was possible to construct a previsional model on which the stability verification of the successive excavation phases has been founded.
The monitoring system based on multibase borehole extensometers installed for the control of the deformations of the rock mass around the excavation is also presented. The reliability of the design was checked during the excavation phases by the comparison between the observed deformations and those expected on the base of the mathematical model.

FOREWORD

The Lago d'Avio - Edolo pumped-storage hydroelectric plant exploits, as upper reservoir, the existing Lago d'Avio, of 21,24 x 10^6 m^3 capacity. Through 8126 m-long, 5.40 m-diam power tunnel and two penstocks having a diam varying between 3.40 and 2.85 m, water is conveyed to the underground powerhouse, which consists of 3 caverns with longitudinal axes parallel to each other.

The central cavern, housing the machine hall, is sided by valve and transformer caverns. Max head is about 1265 m. Eight reversible units are installed in the machine hall. They have an overall generating capacity of some 1000 MW. Through a 685 m long, 5.5 m diam tunnel water flows in a lower reservoir, constructed by means of a cohesionless material embankment, having a capacity of 1.344×10^6 m^3.

The main dimensions of the three cavities (width, length and height) are the following (Fig. 1):- valve hall 9.50x134x15 m

- machine hall 16x175x47 m
- transformer hall 14x164x20 m.

The size of the caverns, and expecially of the machine hall, called for detailed geological and geomechanical analysis of the rock mass concerned with the work, in order to face both the problem of the overall stability of cavities and local stability problems connected to the presence of rock wedges delimited by joints.

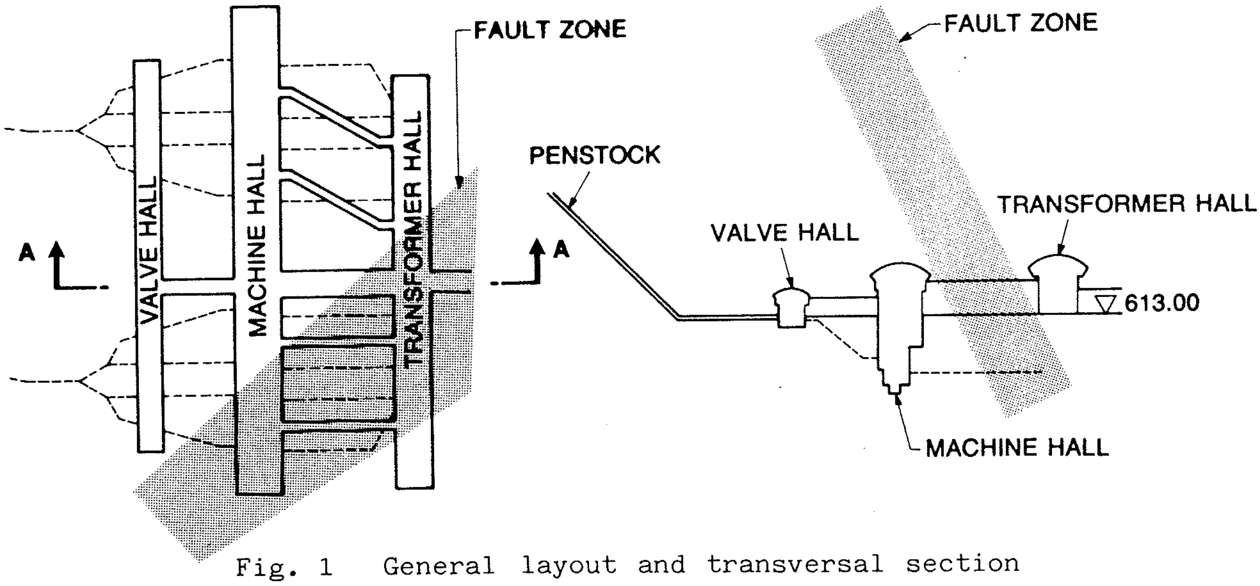

Fig. 1 General layout and transversal section of underground powerhouse with the indication of the fault zone

DESIGN CRITERIA OF LARGE UNDERGROUND CAVERNS

The design of a large underground opening consists generally in the study concerning the global and local stability of cavern to be excavated and is developed through the following synthetically listed activities:

- geological and structural survey of the site;
- determination of geomechanical parameters by means of "in situ" and laboratory tests;
- calibration of geomechanical parameters by "back analysis" carried out with finite element or analytic mathematical models referred to excavations with simple geometry (exploratory adits);

- determination of global and local stability of excavations by means of numerical models;
- determination, by means of numerical models, of the active or passive support, if any, of excavation walls;
- check during excavation progress, of the geomechanical parameters assumed and of models used, through the comparison between the results obtained by the numerical model and displacement measurements recorded by means of an appropriate instrumentation.

The aforementioned first three activities provide the appraisal of some geomechanical parameters, a part of which characterize the behaviour of the large scale rock mass, while others, expecially those relating to the discontinuities, provide indications on possible instability of rock wedges.

A decisive importance for a correct design is assumed by the check, in the excavation stage, of the design parameters. To this purpose it is necessary to plan an appropriate instrumentation, placed in representative sections. The placing of the check system has to be made through proper adits carried out before the excavation was started. Measurements chiefly concern displacements and convergencies, since, according to interpretative models of elasto-plastic type, the parameters affecting the mass stability have also a great influence on the deformations. Having at disposal reliable displacement measurements, it is possible to check, on real scale, the reliability of the design model used, and the validity of the choice of geomechanical parameters.

GEOLOGICAL AND STRUCTURAL STUDY

The rock mass involved in powerhouse excavation may be referred to the formation which is called, in geological literature, "Schists of Edolo" and belongs to the cristalline basement of the Southern Alps. Many layers of cataclasis and mylonite are present inside the formation, their laying being in accordance with the general tectonic direction (ENE-WSW). However, in the proximity of Edolo, they tend to rotate towards NW, and they represent in some cases the convergence of tectonic lines which, towards the west of the zone in question, are separated and independent.

The first investigation stage was set up on the photointerpretation paying special attention to the identification of major structural lines. This study, made on air photos taken from high and lower elevation indicated that the rock mass, from the powerhouse to the surrounding mountain is concerned with nine faults, all of them well identified by quite sharp valley cuts whereas the zone strictly relating to the powerhouse is concerned with a single fault.

The in situ investigations born out that the area strictly relating to powerhouse excavation was directly concerned with a North dipping single fault accompanied by a layer of 20 m thick laminated material. The expected position of this fault at the powerhouse elevation is schematized in Fig. 1.

The powerhouse location on the basis of this study of general tectonics was therefore modified in relation to a previous design.
A first structural survey, carried out when excavating the access tunnel, indicated the presence of 5 joints sets. This survey made it possible to process a statistical model able to foresee the rock mass cracking condition and to define the geometry of the elements into which the rock mass itself could be subdivided. The analysis of the model indicated that large wedges could be isolated both at the upstream and downstream walls of machine hall cavern. The various assumptions formulated at the initial design stage, were subsequently ascertained by systematically surveying the joints which appeared during the excavation of cavern arch sections (Fig. 2). The detailed structural survey made it possible to study the possible instability of rock wedges and to define the stabilizing works.
During the excavation of machine and transformer halls arch sections, two different lithologic types of "Schists of Edolo" formation were observed: the first one, called rock type 1, is a mycaschist composed by the following principal minerals: quartz, muscovite, chlorite and biotite. The second one (rock type 2) is a very fractured laminated schist locally present inside the rock type 1.

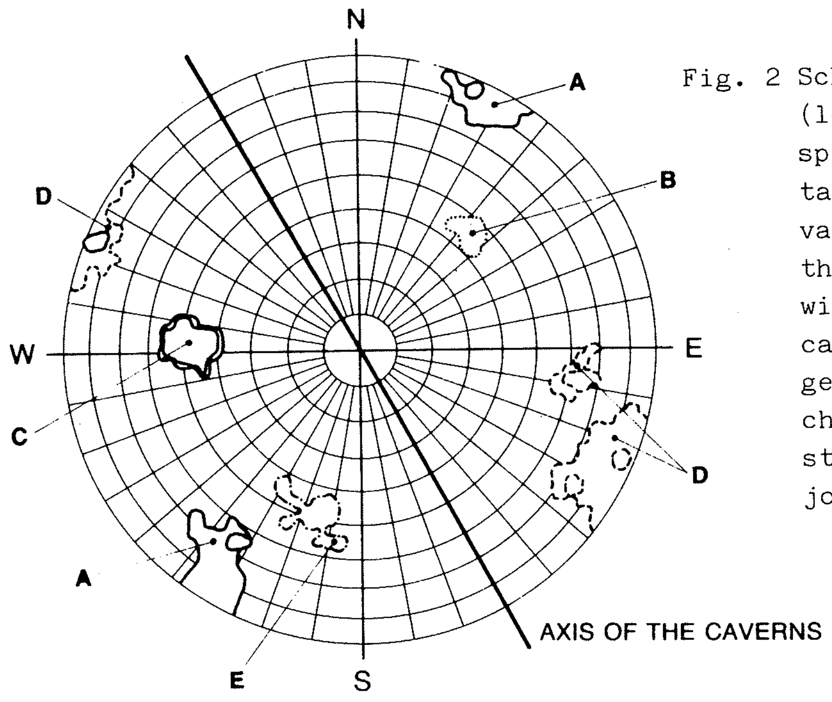

Fig. 2 Schmidt's net (lower hemisphere) obtained in the vaults of the three caverns with the indication of the geomechanical characteristics of the joint systems.

Joint sets	Dip direction [o]	Dip [o]	J.R.C.	J.C.S. [Mpa]	Spacing [m]
A	32	88	9.8	60	10
B	222	48	9.0	60	30
C	92	50	10.0	60	50
D	300	85	9.6	40	10
E	20	50	9.3	60	20

IN SITU AND LABORATORY INVESTIGATIONS AND DEFINITION OF GEOMECHANICAL PARAMETERS

The geomechanical characterization of the rock mass was carried out both at laboratory, and "in situ" by tests and observations on the behaviour of exploratory adit excavation, in order to determine stress and strain parameters relating to the mass itself. Geomechanical parameters may indeed be subdivided into two groups:

a) parameters needed to characterize the lithotypes forming the rock mass (obtained by means of laboratory tests);

b) parameters relevant to the rock mass behaviour on the whole (obtained by means of in situ tests and field measurements)

Among the parameters of the second group, a particular attention is devoted to the study of cohesion parameters (peak and residual), since these parameters, being not so far determinable by direct measurements, called for the set up and the use of a determination method, based on convergency measurements and their interpretation by means of a simplified analityc model.

LABORATORY TESTS

Laboratory tests were carried out on samples taken from exploratory adits to determine the characteristics of two types of rock involved in cavern excavation. In addition to physical and mechanical tests of conventional type, special attention has been devoted to the performance of strain-controlled triaxial tests to determine the peak $(\emptyset_p, C_p)$ and residual $(\emptyset_r, C_r)$ strength parameters of the two rock types (Fig. 3).

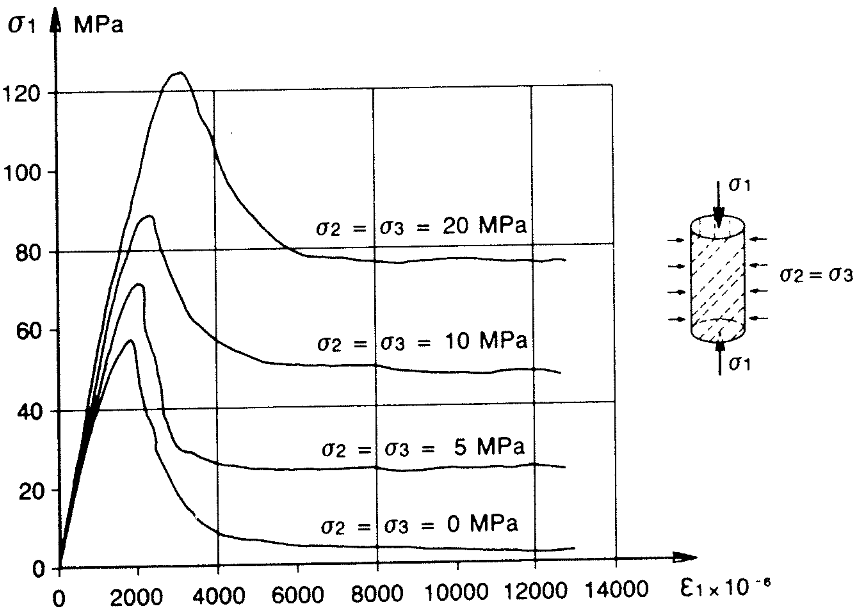

Fig. 3 Strain-controlled triaxial tests on lithotype 2. Stress-strain diagrams for different values of confining pressure

The peak strength parameters of the two types of rock are shown in the following table:

Rock type	Uniaxial compr. strenght σc [Mpa]	Friction angle ø p[°]	Cohesion Cp [Mpa]
1	64	43°	15
2	45	43°	10

TABLE 1 Average values of strength parameters determined by laboratory tests.

The value of the residual friction angle goes down to about 38° and that of residual choesion is included between 0.5 and 1.5 MPa.

IN SITU INVESTIGATIONS

Measurements of the natural state of stess

To perform preliminary stability analysis the undisturbed rock mass stress state should be known. To this end, the "CSIR doorstopper" technique was used inside 76 mm boreholes which were differently oriented in the space.A disagreement was observed between stress values derived from consideration connected to the thickness of rock cover and those obtained by in situ measurements. The most important reason of this disagreement is most probably due to the great tectonization supported by the rock mass.
The plane of the main stresses σ_1 and σ_2 practically concided with the plane of the machine hall cross section. The following values were determined: σ_1 = 24.9 MPa, σ_2 = 15.6 MPa, with angle of deviation of $_1$, in relation to the vertical, of about 6°.

Deformability tests

Deformability characteristics of the rock mass were determined by means of plate bearing and flat jack tests with the aid of geophysical investigations. The plate bearing technique made it possible to determine, by means of special multibase borehole-extensometers, the deformability characteristics of the undisturbed rock mass below the released layer present at the surface of the exploratory adits. Two plate bearing tests were carried out in each section, in normal and parallel direction to schistosity planes. Fig. 4 shows typical diagrams of deformations measured at different depths from the loaded surface. A marked anisotropy of deformations was observed on the superficial released layer; the anisotropy is less marked in the undisturbed rock mass.

Flat-jack test made it possible to determine the deformability characteristics of the released rock layer, as well as the tangential stress existing on the adit surfaces.

The average values of peak deformability modulus determined in situ on the two types of rock were the following:

E = 40000 MPa for type 1

E = 25000 MPa for type 2

The values of deformability modulus in residual condition (relative to the plastic zone around the excavations) were assumed equal to:

E_r = 20000 MPa for type 1

E_r = 12500 MPa for type 2

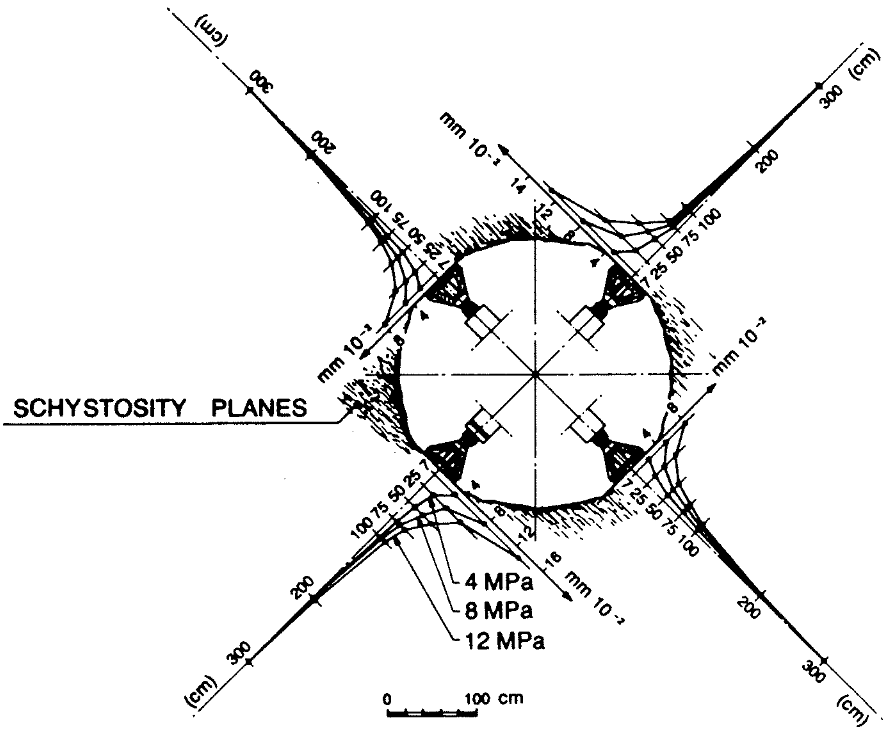

Fig. 4 Plate loading test in directions normal and parallel to schistosity. Diagrams of deformations versus depth measured at the center of the loading plate by means of borehole multibase extensometers

DETERMINATION OF PEAK AND RESIDUAL COHESION OF THE ROCK MASS

As mentioned above, there is not the possibility of a direct determination, by "in situ" tests, of the cohesion parameters of the rock mass. A deductive method was then used based on in situ tests, on convergency measurements and on the calculation by means of a simplified

analytic geomechanical model. The method of the characteristic lines theory according to Amberg-Lombardi formulation (1974) was used with a bidimensional axialsymmetric model.
The state of stress, given the high isotropy coefficient recorded (0.70), was assumed to be of hydrostatic type. The rock mass was assumed as isotropic and homogeneous with Mohr-Coulomb elasto-plastic strain-softening behaviour. The convergence measurements and the analytical model allowed the deformation of the possible couples of peak and residual cohesion values consistent with the convergence measurements (Fig. 5).

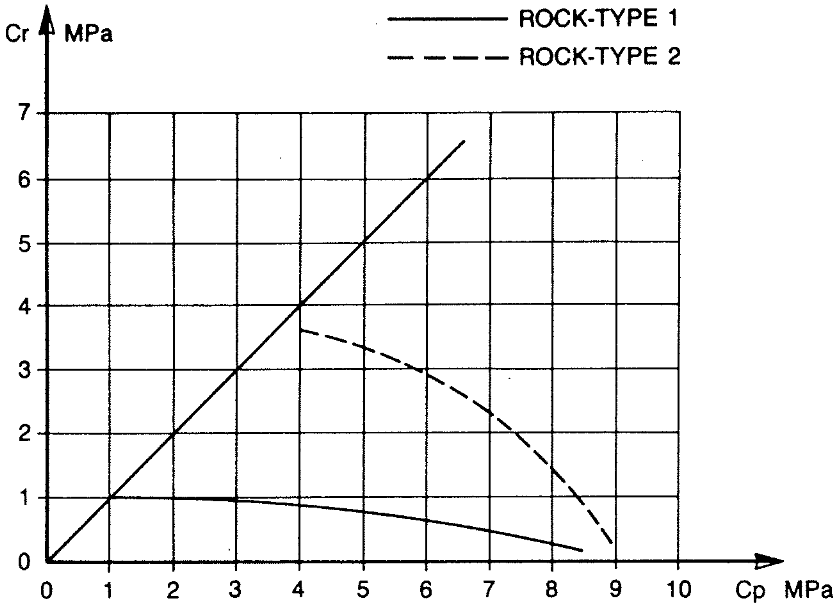

Fig. 5 Parametrical curves Cp = f(Cr) obtained by means of a simplified geomechanical model

Measurements of tangential stress by flat jack carried out in the plastic zone, made it possible to individuate the curve characterized by the following values of the peak cohesion (Cp) and of the residual one (Cr) (Fig. 6).

Cp = 3.0 MPa
Cr = 1.5 MPa for rock type 1

Cp = 1.5 MPa
Cr = 1.1 MPa for rock type 2

STABILITY ANALYSIS OF EXCAVATIONS AND DESIGN OF SUPPORTING WORKS

The analysis of the stability of the whole excavations was carried out by means of a bidimensional finite element numerical model set up at ISMES through the NOLIVP computation program, already experimented for the study of others underground powerhouses designed by ENEL. The behaviour of the rock mass was simulated by means of a viscous elasto-plastic strain-softening model.

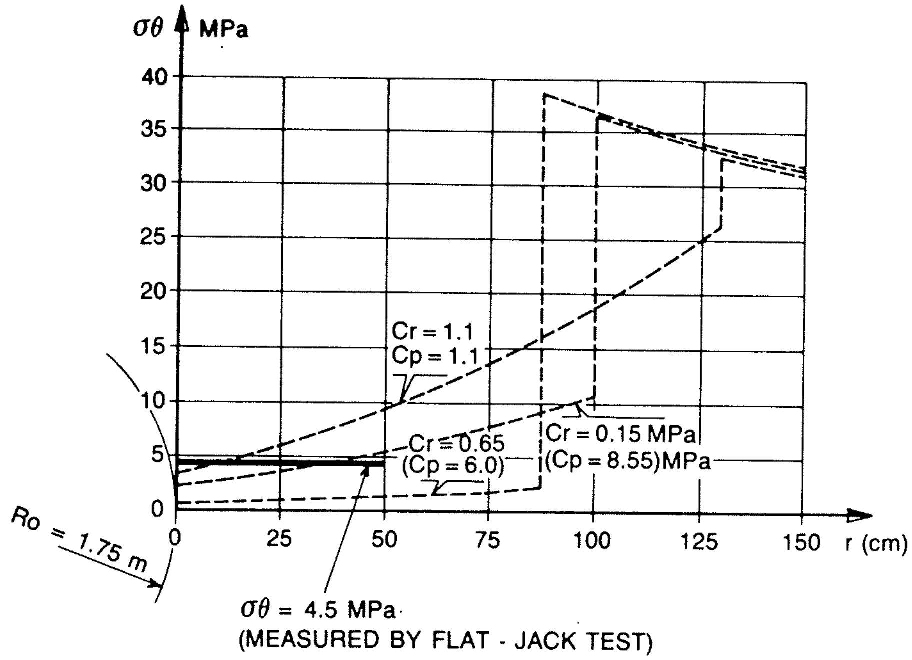

Fig. 6 Tangential stress in the plasticized layer of rock calculated by the simplified geomechanical model as a function of the residual cohesion

The choice of the viscous-plastic algorithm was suggested by the following reasons:

- most mechanism usually described by elasto-plastic models actually develop through a fast viscous-plastic phenomenon;
- the viscous-plastic algorithm provides in the iterative calculation a sure check of the convergency of the calculation itself
- it is possible to simulate a load history consistent with the actual excavation stages.

The numerical computation was conducted by using a finite element mesh which made it possible to reproduce the excavation geometry of the three caverns and the main stages planned for excavation progress (fig. 7). The analysis of mass stability was completed by means of a second finite element bidimensional numerical model, based on the known principles of characteristic lines, referred to the machine hall. This model, which made it possible to appraise the state of stress in the plastic rock zone near the vertical walls of the excavation, provided the value of support pressure required on the excavation walls in order to reduce to acceptable values (less than 1 MPa) the tensile stress . The amount of support pressure calculated for the machine hall was about 0.01 MPa, except for an area in the north section of the cavern where the mechanical characteristics of the rock were lower and for which a value of 0.015 MPa was calculated. It was observed that the areas involved in slight tensile stresses were sited within a band of 3 - 4 m from excavation face.

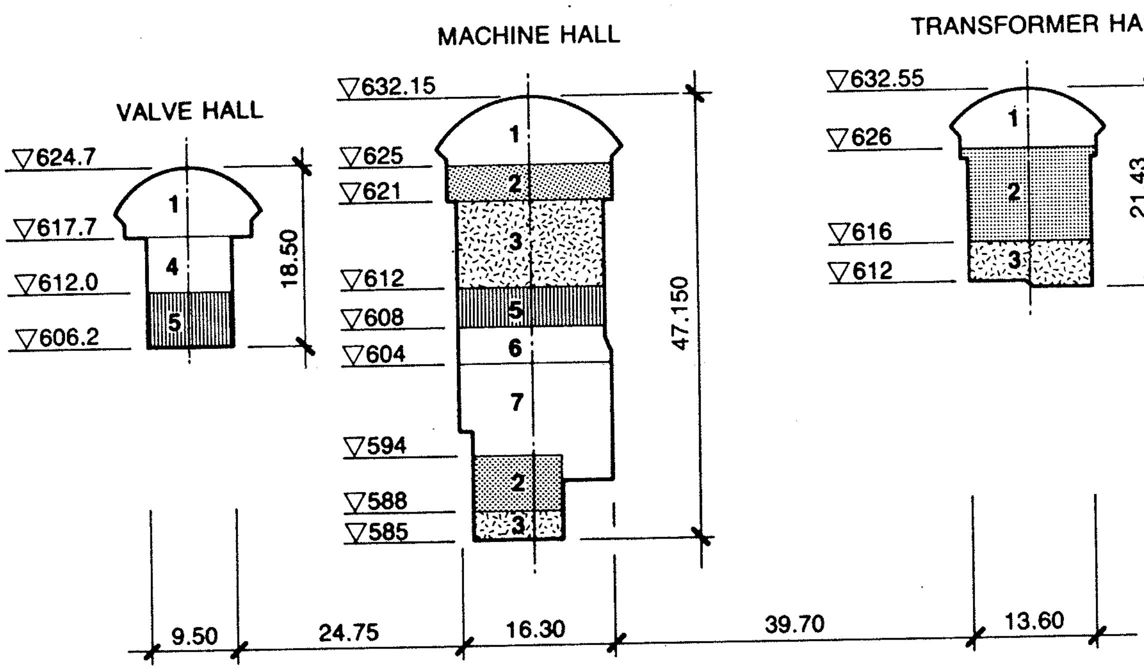

Fig. 7 General scheme of excavation phases

The analysis of potential instability of rock wedges was carried out on the basis of geostructural investigation in the exploratory adits and during the excavation progress.
The strength characteristics along the joint surfaces were evaluated on the base of the method proposed by Barton 1977.
It was found that the maximum size of the potential unstable wedges was about 4 m and the maximum value of the stabilizing pressure at the free surface of excavation faces, was about 0.02 MPa.
Following the aforementioned indications, the excavation support system was planned with 6 m-long carbon steel prestressed anchors, fixed by means of epoxy resin for length of 3 m, placed at a rectangular mesh of 1.50 x 3.00 m and uniformely distributed along the walls.A stress of 120 KN was applied to each anchor. The rock anchors were placed soon after each excavation stage of 4 m height in order to obtain safety conditions of the wall before the performance on the following excavation stage.
In a limited portion of north cavern walls, owing to the poorer characteristics of the rock mass, the rectangular mesh of anchors was reduced to 1.50x1.50 m, and a steel net with spritz beton was applied.
At the overhead travelling crane in the machine hall, 20 m long anchors were placed in double row and spaced 4.0 m.
The scheme of supporting works is shown in Fig. 8.

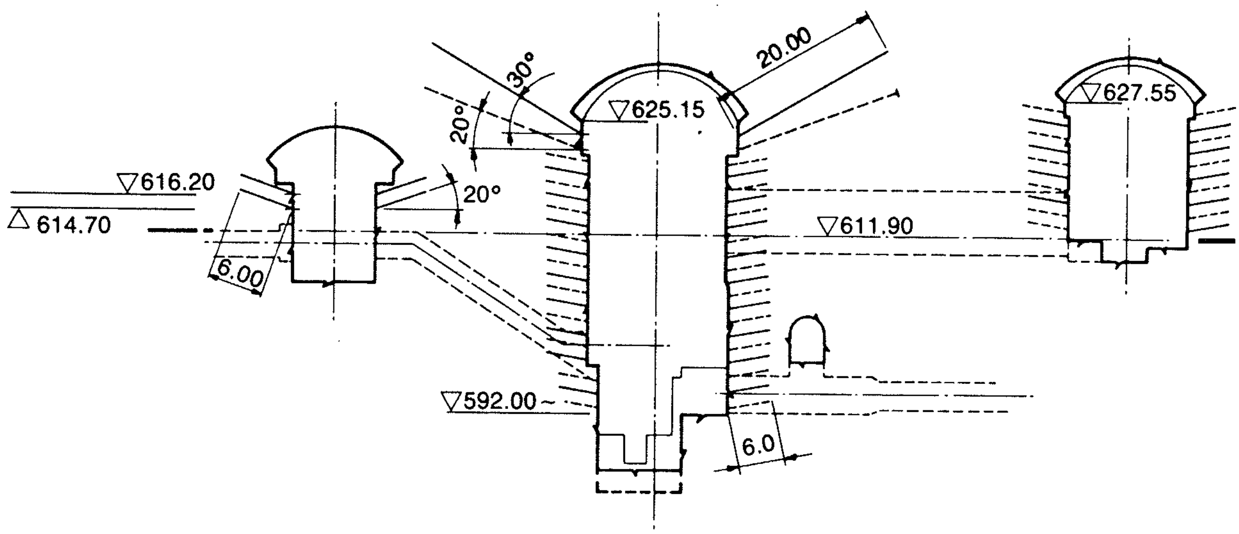

Fig. 8 Scheme of rock anchors

MONITORING

The monitoring instrumentation of a large underground excavations is an integrant part of the design process, as it provide elements which either confirm the validity of design criteria or indicate the need of some modifications. The monitoring system also provides prompt indications on the excavation safety.
Displacements have to be measured both at the excavation face and at different depths in the rock mass, and the instrumented sections must the most representative of the whole excavation. Safety controls must be so arranged as to monitor the whole area of excavation faces. Of course, the most difficult problem to be solved consists in the interference with the excavation works.
The excavations of Edolo powerhouse were instrumented at two typical sections. The monitoring system planned for each section was formed by:

- five multi-base borehole extensometers (70 m length)
- two single base extensometers between the machine hall cavern and each of lateral ones;
- distometric bases for measuring convergency at three different elevations.

An additional instrumentation was installed:
- piezometric cells for checking the natural water table;
- optic collimation and levelling along four alignments.

Displacement measurements were planned in order to promptly follow excavation progress in each of the devised stages. It was thus possible to compare, at each excavation stage, the measured deformation with that determined by the numerical model. As an example, Figs. 9; 10, show comparison between the measurements carried out by multi-base extensometers for subsequent excavation stages and those provided by the numerical model.

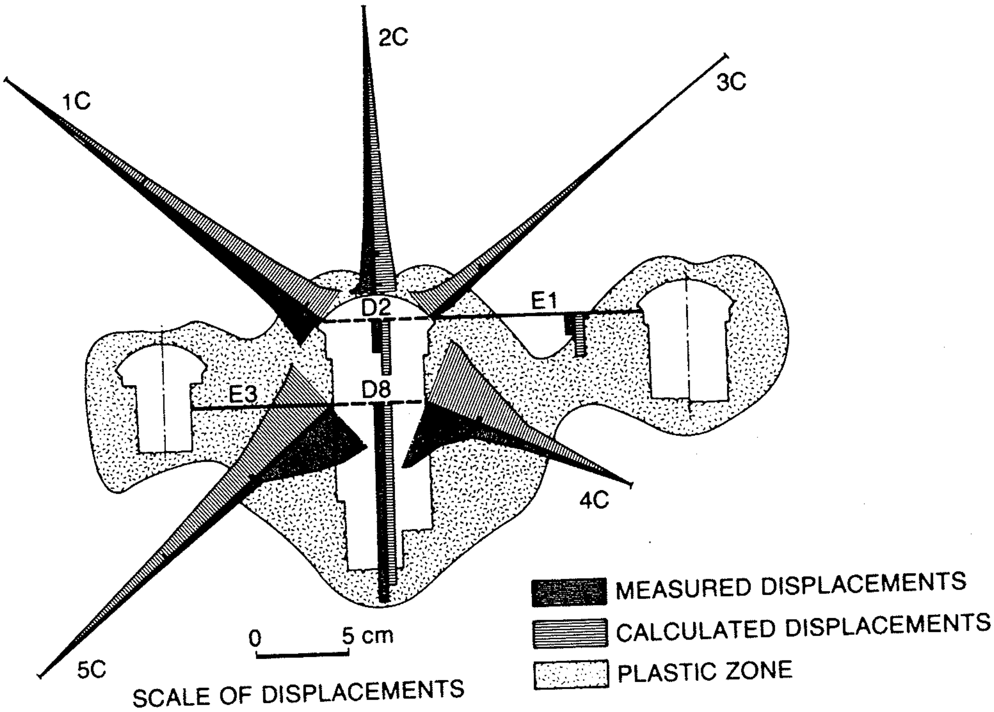

Fig. 9 Comparison between displacements measured by extensometers and those calculated by FEM model

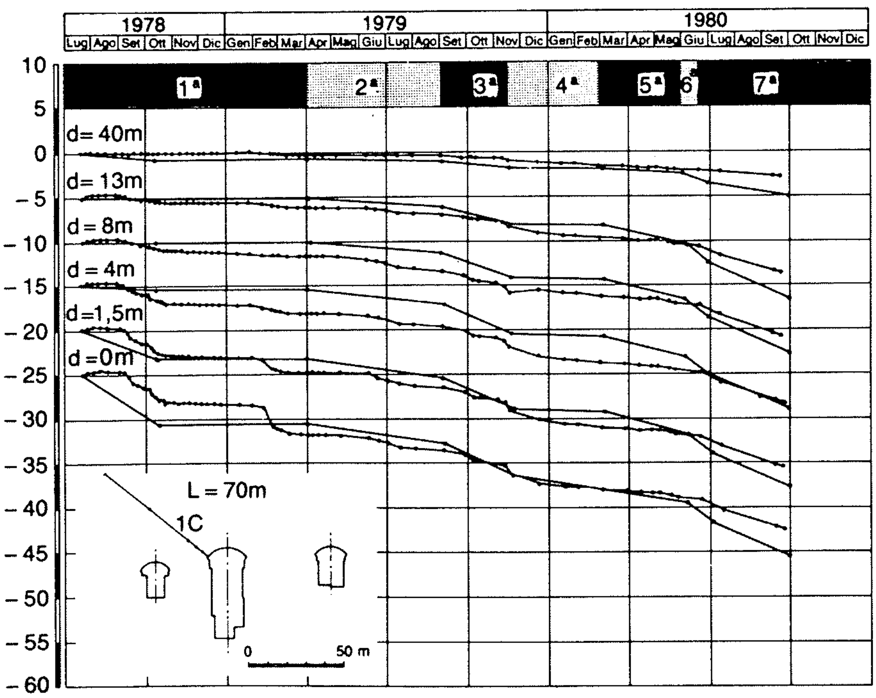

Fig. 10 Diagrams of the displacements measured by a multibase borehole extensometer during the excavation phases

As may be noted, a good agreement has been found between forecast and measured values for all excavation phases.
With reference to borehole extensometer 4C, Fig. 11 shows three different diagrams of displacements:

a) measured values;
b) calculated by means of a viscous-elasto-plastic model
c) calculated by means of a linear-elastic model.

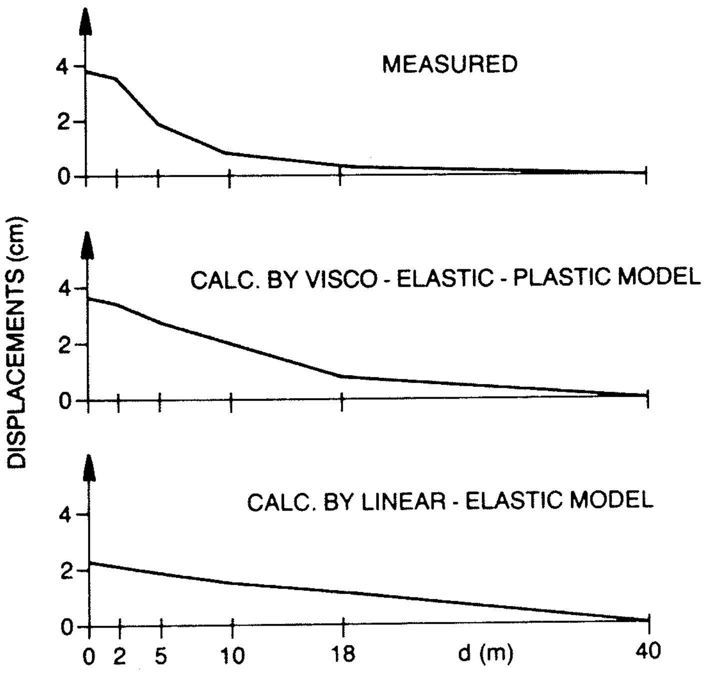

Fig. 11. Multibase extensometer 4C. Comparison between measured and calculated displacements

It may be observed that the use of a linear-elastic model would have involved a subestimate of about 40% of the displacement values.
Finally Fig. 12 shows the displacement diagrams recorded by means of the

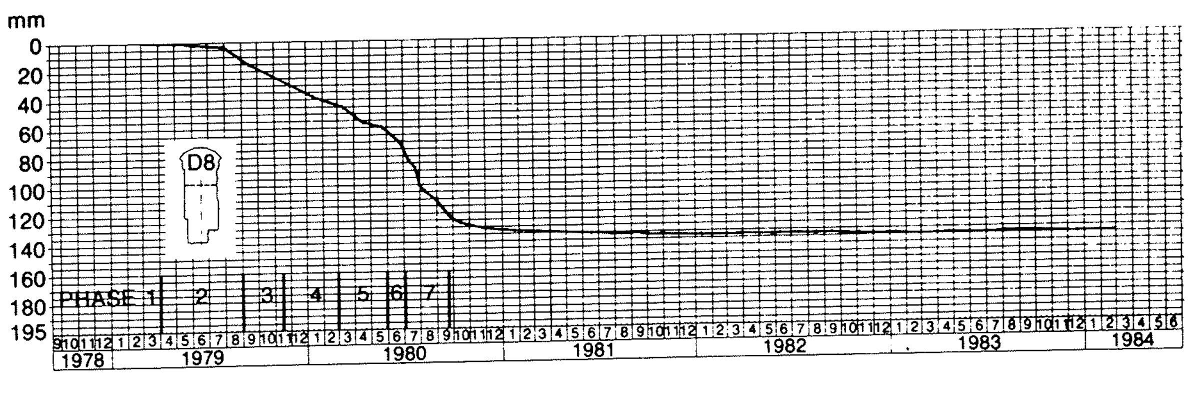

Fig. 12 Diagram of convergency measurements of distometer D8 during and after excavation works

distometer D8 during the excavation stages and for two-years monitoring period after the excavation was completed. The analysis of the aforementioned figure indicates the significant rate of displacement connected to creep phenomena (equal to about 20%) of the overall displacement measured after the excavation was completed.

CONCLUSIONS

The designer of large underground excavations has today at disposal mathematical tools able to simulate, with good approximation, the structural behaviour of the work. He also disposes reliable "in situ" and laboratory testing techniques to appraise the geomechanical parameters of the rock mass. This makes it possible to optimize the design of supporting structures, acting with safety margines which may be clearly estimated. During the excavation of the caverns a constant check of design assumptions throught the accurate control of excavation deformative behaviour is required. It is important that the check be carried out during all the partial excavation phases which therefore must be simulated by numerical model. The monitoring system has also to be designed so as to reduce to max the interferences with the excavation itself in order to facilitate the cooperation with the contractor and thus making easier the acquisition of the sensitiveness needed by this type of design philosophy. In the construction of Edolo powerhouse, the aforementioned objectives have satisfactory been achieved.

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