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# The Anapo pumped-storage power station: geomechanical investigations and design criteria

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# The Anapo pumped-storage power station: geomechanical investigations and design criteria

L' usine électrique de l' Anapo avec aménagement de pompage: recherches géomécaniques et critères de projet

Das Anapo Pumpspeicherwerk: geomechanische Forschungen und Entwurfskriterien

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A pumped-storage power plant (500 MW) is being constructed by ENEL (Italian State Electricity Board) in the eastern coast of Sicily, in the province of Syracuse. The paper presents the various activities carried out for the design of caverns. These activities include geological, hydrogeological and geophysical investigations, "in situ" and laboratory tests, mathematical modelling. Various alternaties examined during the design are briefly presented.

L' ENEL (société nationale italieene pour l' énergie électrique) est en train de construire une usine électrique (500 MW) avec aménagement de pompage sur la côte orientale de la Sicile, dans la province de Syracuse. Ce rapport présente les différentes activités déployées pour l' élaboration du projet des cavernes. Ces activités concernent les recherches géologiques, hydrogéologiques et géophysiques, les essais "in situ" et au laboratoire, les modèles mathématiques. On présente brièvement différentes alternatives examinées pendant l' élaboration du projet.

An der östlichen Küste Siziliens, in der Provinz Syrakus, baut ENEL (Italienische staatliche Elektrizitätsgesellschaft) ein Pumpspeicherwerk (500 MW). Dieser Bericht erläutert die verschiedenen Verfahren die für den Entwurf der Kavernen angewendet worden sind. Diese Verfahren betreffen geologische, hydrogeologische und geophysiche Forschungen, "in situ" und Laborversuche, mathematische Modelle. Es werden kurz verschiedene Alternativen dargelegt, die während dem Entwurt geprüft worden sind.

## 1. INTRODUCTION

ENEL's Design and Construction Department of Venice is currently constructing a pumped-storage hydroelectric plant in Sicily, in the vicinity of Solarino, which is located in the province of Syracuse. The purpose is to supply in a satisfactory manner the Sicilian electrical network, which for the greater part is fed by thermoelectrical groups. The first design, drawn up in 1976, provided for the setting up four 125 MW groups located in shaft, at a depth of 70 meters and with a diameter of 18 meters, in the alluvial plane of the Anapo River.

The results of the in situ geotechnical investigations, combined with landscaping and environmental difficulties of situating the structures, made it advisable to locate the station underground. The main features of the plant are the following:

a) upper reservoir, with a $5.6\times10^6$ $m^3$ capacity, situated in the plateau of the Climiti mountains at an altitude of 400 m;

b) no. 2 underground penstocks, with an inclination of 48°, having length of 475 m and a diameter of 5 m;

c) underground power station made up of 155x20x40 m machine hall and a transformer hall measuring 150x15x22 m;

d) lower reservoir having a capacity of $7.3\times10^6$ $m^3$, situated in the Anapo River plain at an altitude of 98 m. This reservoir is connected to the power station via two spillway tunnels having a diameter of 6.5 m and length of 800 m.

A geological study of the surface allowed the identification of main faults, the main joints systems, as well as the various lithotypes present in the area involved by the structures.

For purposes of deeper geological and geomechanical investigations, an exploratory adit was bored, in order to reach the expected position of the power station. In this point, the rock was found to be highly fractured, owing to which it was deemed advisable to move back the position of cavern by 50 meters. With the aim of providing the designers the necessary parameters for the study of structures, in situ geomechanical tests were carried out in the exploratory adit as well as sampling for laboratory tests. Furthermore, the water-table was intercepted during the excavation of the exploratory adit and as such it was possible to effect an in-depth analysis of the permeability characteristics of the rock mass.

The design criteria were verified by means of mathematical models. The results of a preliminary FEM model and the presence of the water-table made it advisable to re-examine the geometry of the excavations of the machine cavern. It was thus decided that the turbines were to be installed in separate shafts (D = 20 m), in order to limit the excavations under the water table (Fig. 1). Further calculations confirmed the structural and economic advantages of this solution.

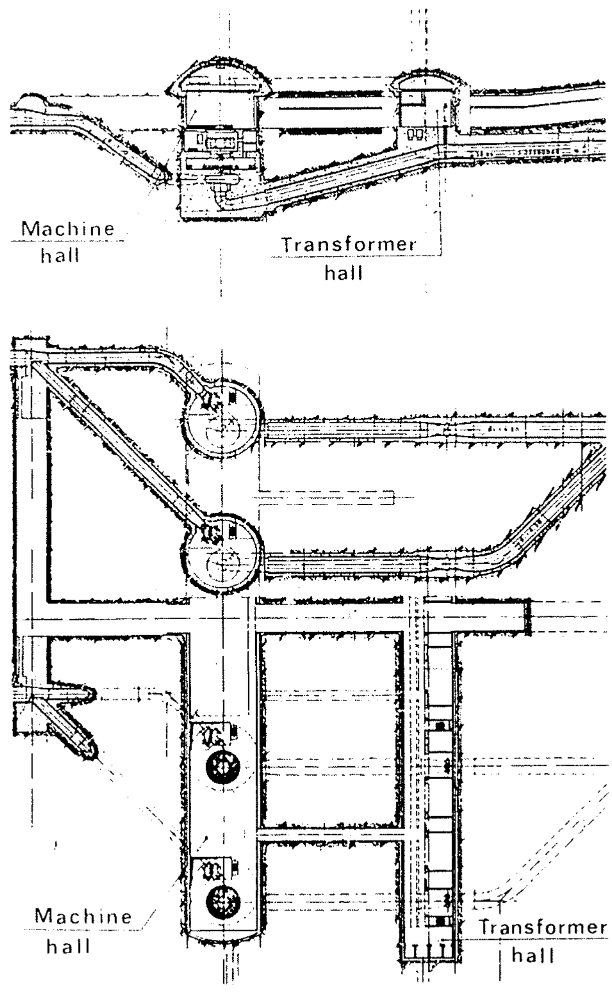


FIG. 1 General scheme of the plant.

## 2. GEOLOGICAL INVESTIGATION

The plant lies between floor of the valley of the Anapo River and the Climiti mountains which, from morphological and geological viewpoints, make up a part of Ibleo plateau. The Climiti mountains present calcareous series that are almost continuous from Cretaceous to Miocene and only locally interrupted by small scale lava effusions. Miocenic formation of "Palazzolo Limestones", composed of stratified variable fine to medium-grained limestones, are to be met with in the area involved by the excavations. The environment holding deposits of rocks typical of this formation are characterized by a detrial-organogenic sedimentation. Rough detrial materials (limestone breccia) are locally and occasionally to be encountered. The relationships between the various lithological types are difficult to evaluate inasmuch that vertical and lateral passages are rather frequent and random.

A geological investigation carried out on the surface made it possible to identify in the rock mass involved by the excavation the presence of three lithotypes characterized by the following parameters:

Lithotype 1

Dry density = 1.5-1.8 $t/m^3$

Sound velocity = 3,000-3,400 m/sec

Uniaxial compression strength = 10.0-25.0 MPa

Lithotype 2

Dry density = 1.8-2.2 $t/m^3$

Sound velocity 3,400-4,400 m/sec

Uniaxial compression strength = 20.0-40.0 Mpa

Lithotype 3

Dry density = 2.2-2.5 $t/m^3$

Sound velocity = 4,400-5,000 m/sec

Uniaxial compression strength = 40.0-50.0 MPa

From surveys of the surface and photointerpretation, it was further found that the area is affected by at least two fault systems with E-W and N-S directions respectively.

As stated in the introduction, for purposes of a deeper knowledge of the geology of the rock mass in which the power station was to be located and also for carrying out the investigations for the mechanical characteristization of the rock mass, an exploratory adit was bored, having an 18 $m^2$ cross-section and a length of 750 m. In the exploratory adit a structural-geological survey was carried out in order to identify the lithotypes present in the excavation, the faults and the joints systems. The survey facilitated in defining a geological model of the mass, in the immediate surroundings of the excavations of the machine hall, and in particular from the top downwards (see Fig. 2):

a) medium grained biocalcarenites above the water-table

b) medium grained biocalcarenites below the water-table

c) vuggy limestones

d) stratified fine-grained calcarenites.

In the area where the power station was to be located, the rock mass was found to be involved with only a single systems of faults whose planes are characterized by dip angle equal to 80° - 85° and by dip direction equal to 190° - 210°. This group belongs to the first of the two systems identified by means of the surface surveys. In particular, two tectonic disturbance bands at a distance of about 170 m one from the other were verified, the innermost one having a width of approx. 15 m. The rock mass between the two bands cited above is subdivided into tabular shaped blocks by joints of XX type, sub-parallel to the main faults having a spacing ranging between 50 - 300 cm and of the closed type. In the tectonic disturbance areas, the fractures have a spacing between 10 and 60 cm and are open.

## 3. LABORATORY TESTS

To classify the various lithotypes of which Mts Climiti mass is composed, somewhat interesting were the results of laboratory simple determinations performed on integral samples: dry density ($\gamma_d$), sound velocity (V) and point-load index ($I_s$) (Figg. 3, 4).

A wider laboratory investigation was subsequently carried out on lithotypes involving the power station excavations. Since from outstart it was obvious that the mechanical strength parameters were directly influenced by the porosity of the rock (which presents variable characteristics within each lithotype), the execution of a large number of laboratory tests was found to be necessary to allow a statistical interpretation of the results.

During the surveys, the behaviour of the medium grained biocalcarenites above and below the water-table was thoroughly investigated, inasmuch that the greater part of the excavation operations was to take place within these lithotypes. Fig. 5 shows, in a diagram $\sigma_1 - \sigma_3$,

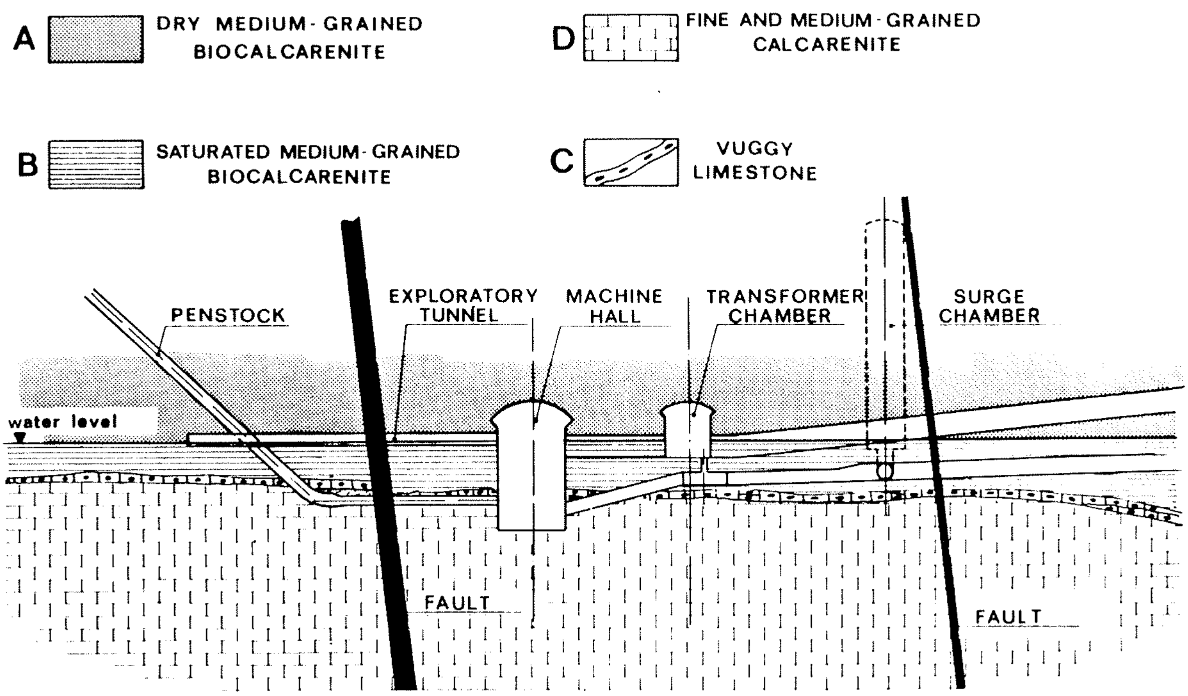


FIG. 2 Geologic scheme.

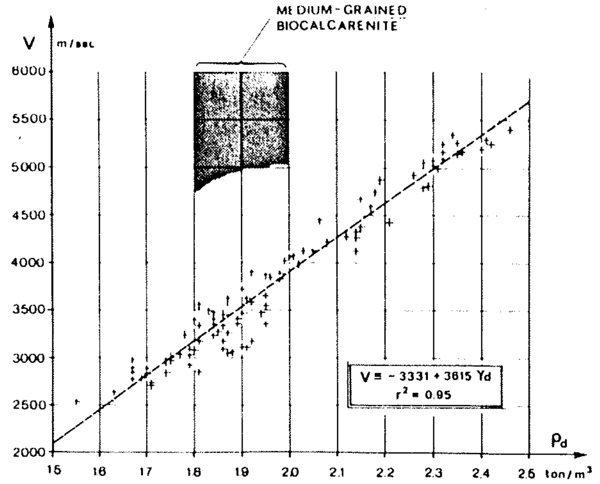


FIG. 3 Plot of sound velocity versus dry density.

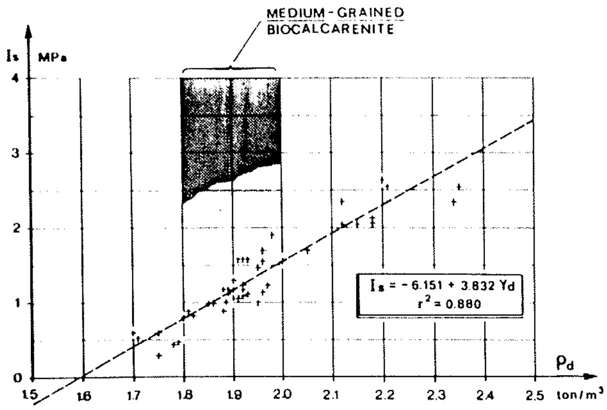


FIG. 4 Plot of point load index versus dry density.

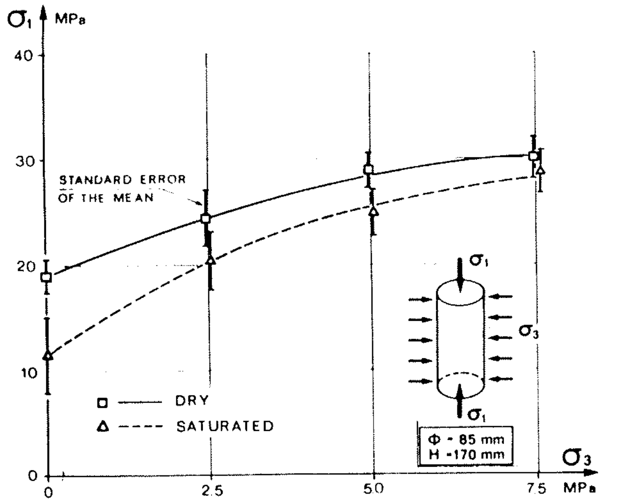


FIG. 5 Triaxial tests on dry and saturated biocalcarenite. Failure stress ($\sigma_1$) as a function of confining pressure ($\sigma_3$).

peak strength curves for lateral confining pressures of up to 7.5 MPa, while Fig. 6 shows

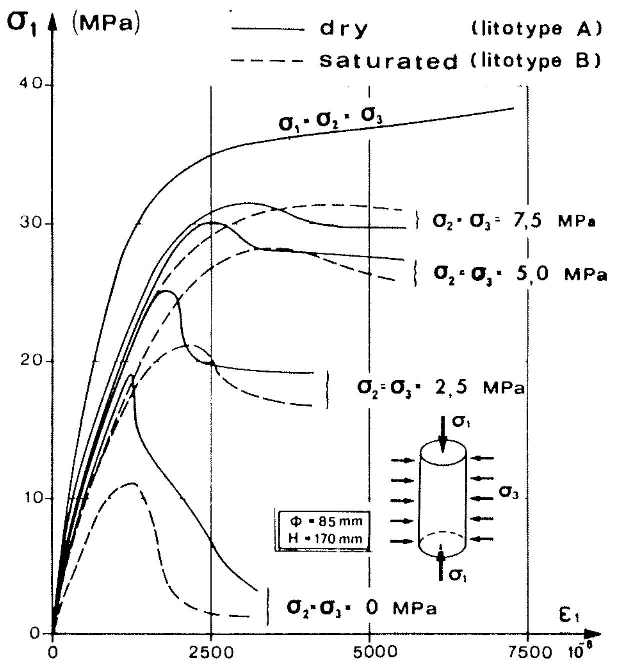


FIG. 6 Complete stress-strain curves for unconfined and confined tests on biocalcarenite samples.

typical complete stress-strain curves. It may be pointed out that, as the confining pressure increases, differences between strength and deformability properties of the material under dry and saturated conditions tend to decrease. Althought direct measurements of pore-pressures arising during the failure processes were not perfomed in this study stage, this phenomenon may likely be ascribed to the different dissipation extent of pore-pressure during the isotropic compression phase. It is also interesting to point out that the transition from the brittle to the plastic behaviour of the material is already attained for a confining pressure of about 5 MPa. For further increases of confining pressures, the increase in ultimate axial stress tends to become negligible. The transition from a rock-like behaviour to a soil-like behaviour - typical of porous rocks of the type in question -, checked by isotropic compression tests ( $\sigma_1 = \sigma_2 = \sigma_3$), is determined for a stress level of 35 - 40 MPa (Fig. 6).

## 4. IN SITU TESTS

The following investigations and determinations were carried out in the exploratory adit (Fig. 7):

a) convergence measurements

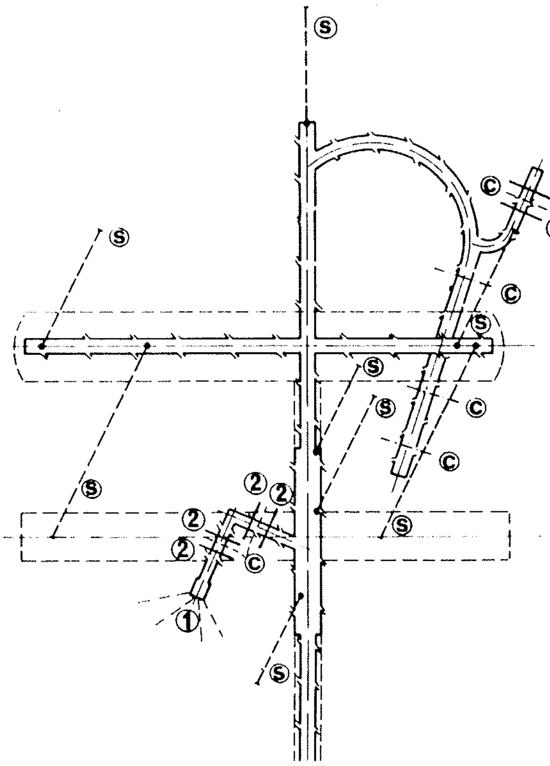


FIG. 7 General layout of the testing points:
s - drillholes;
c - convergence measurements;
1 - measure of the state stress;
2 - plate loading tests and flat jack tests.

b) determination of the natural state of stress by means of overcoring technique

c) determination of the tangential stress " $\sigma_{tang}$ " on the walls of the adit by flat jack tests

d) plate loading tests for the determination of the deformability modulus, carried out with a technique which makes it possible to evaluate the deformability characteristics of the undisturbed rock mass

e) seismic measurements along the walls of the exploratory adit.

The determination of the natural state of stress was carried out inside four bore-holes with different orientations and provided the following values of the principal stresses:

$\sigma_1$ = 7.3 MPa; $\sigma_2$ = 4.2 MPa; $\sigma_3$ = 1.9 MPa

The direction of the principal stresses is shown in Fig. 8.

The average values of deformability modulus of the various lithotype, determined by in situ tests, are set out in the following table:

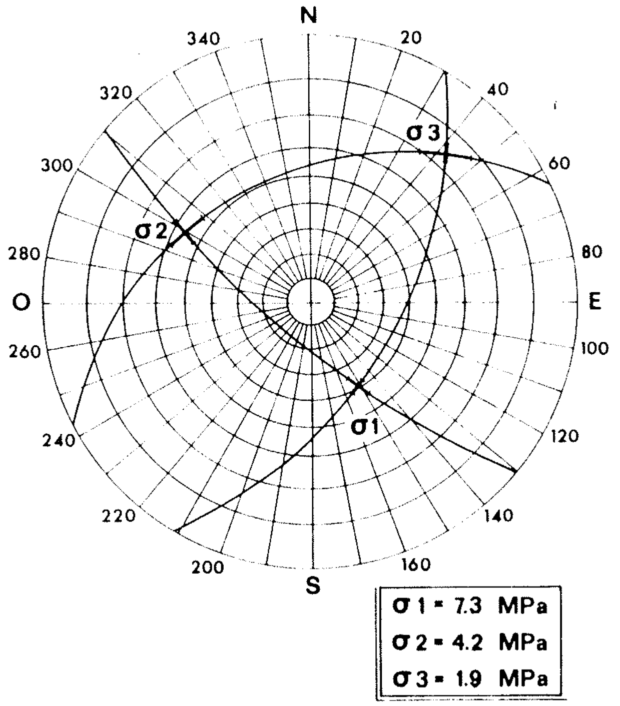


FIG. 8 Directions of principal stress and their confidence intervals.

| Lithotype | Deformability Modulus (MPa) |
|---|---|
| A. Medium grained biocalcarenites above water-table | 18,000 |
| B. Medium grained biocalcarenites below water-table | 10,000 |
| C. Vuggy limestones | 18,000 |
| D. Fine grained calcarenites | 25,000 |

The convergence measured in exploratory adit analysed on the basis of the above mentioned stresses and modulus, turned out almost compatible with an elastic behaviour. Also the stresses measured by the jacks were on the same line, allowing for a quite small thickness for the loosened layer at the wall. The reference model for the design was assumed of the elasto-plastic type with fragile softening. Peak and residual intrinsic curves (Fig. 9) were mainly derived from triaxial test results: a conservative interpretation of those last ones was made in order to be compatible with a small yielding at the scale of the exploratory tunnels.

## 5. STUDY OF THE WATER-TABLE

At the end of the excavation of the exploratory adit the surface of the water-table was intercepted at an elevation of 50 - 52 m. A series

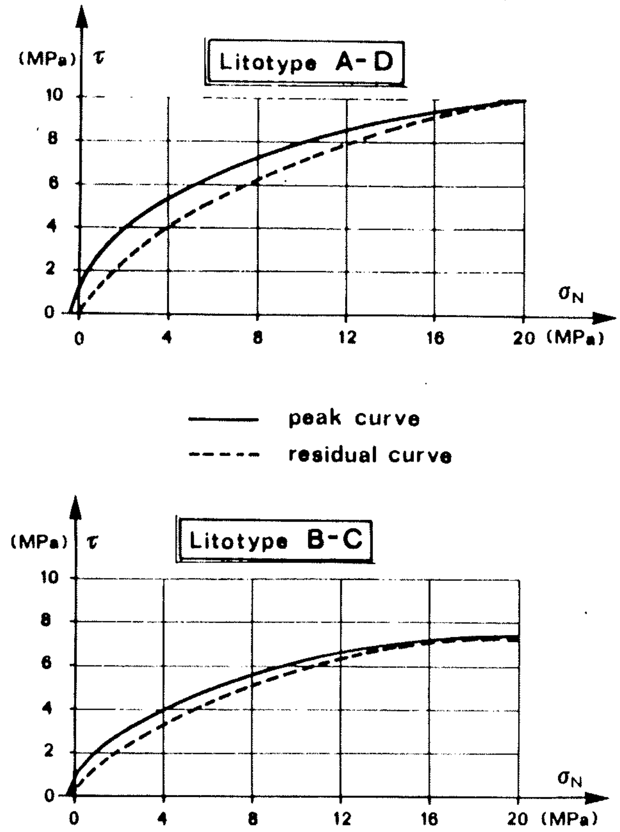


FIG. 9 Intrinsic curves of the various lithotypes adopted for the design.

of 10 bore-holes were executed and equipped with Casagrande type piezometric cells, which were installed at various depths in correspondence to the different lithological types. Pumping tests were effected inside wells (D= = 200 mm), attempt being made to involve the three lithotypes separately. The flow rate was found to be modest, below 0.1 l/sec, until the well was deepened into the biocalcarenite; an abrupt increase was then registered, about 5 l/sec when the test involved vuggy limestone level; while a further increase in the rate at the third and deepest level of cemented calcarenite was found to be only perceptible. The simultaneous measurements effected on the piezometers registered head variations only when the pumping tests involved the bank of the vuggy limestone. The height recorded by the instruments installed in this bank in fact varied rapidly and became stabilized in some ten minutes, while in the other lithotypes the stabilization required more than 48 hours. The piezometric head variations were recorded up to a maximum distance of 20 - 25 meters from the pumping well. On the basis of the structural characteristics of the rock mass, of the results of the pumping tests and of a number of preliminary permeability measurements in the laboratory the following values of the permeability coefficient were calculated:

- Biocalcarenites: $K = 10^{-5}$ cm/sec
- Vuggy limestone: $K = 10^{-2}$ cm/sec

## 6. DESIGN AND MATHEMATICAL MODELLING

Following the choice of an underground plant, the design of the mechanical and electrical apparatus was started. A conventional design with a machine and transformer chambers was then adopted. The preliminary location of the caverns was established on the basis of the superficial geological survey results and hydraulic optimisation requirements.

In the ENEL's experience a detailed and early recognition of the geomechanical situation at depth proves to lead to more economical and reliable design. An exploratory tunnel was then excavated along the crown of the foreseen access tunnel (an access to main excavations will be then obtained quickly by enlargement). The inspection suggested, as previously outlined, a new location of the main caverns. Some calculations were performed on the basis of a preliminary assesment of the geomechanical design parameters. The results of an elastic FEM calculation of the excavations are resumed in Fig. 10.

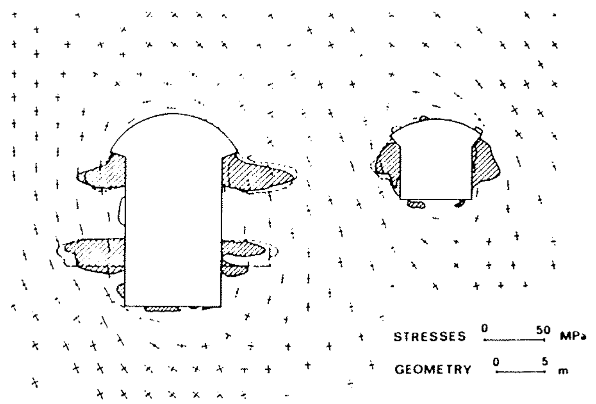


FIG. 10 Elastic FEM calculation results. Tensile zones are dashed. Padded lines identify an unit safety factor on peak strength.

A yielding zone was found, not very deep endeed. In addition tensile zones were evidenced, the tension being mainly due to the different compressibility of the strata. However, the main problem pointed out from exploratory adit information was the elevation of the water-table and the presence of a high permeability in the vuggy limestone. The idea of a permanent drainage system was discarted because of the high water take which, beside other consideration, could perturb the equilibrium of the table, very important in the region. Obviously without drainage the stability of the wall is lowered; in addition any lining must carry the water load too. In conclusion it seemed advisable to discart the hypothesis of a fully excavated section; in the alternative solution the turbines were placed in separate shafts 18 meters deep and 20 meters in diameter. With this disposition the fully excavated section was submerged by few meters only, the most permeable layer being in the shafts. The distance between the machine axes resulted of 40 meters somewhat higher than in the original design. This dimensioning was also checked by another elastic computation.

The 3-D analysis was performed, via FIESTA program, by means of the mesh shown in Fig. 11. The complete initial state of stress was

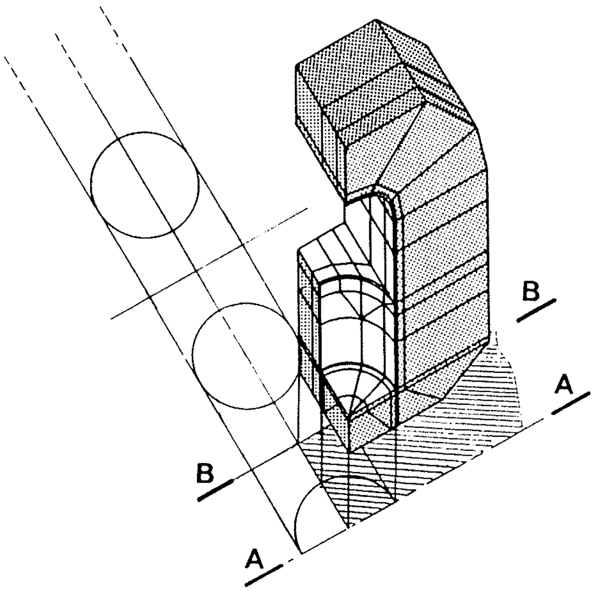


FIG. 11 Detail of 3-D mesh.

taken into account by superposition of simmetric and antisimmetric solutions. Results are summarized in Fig. 12. The rock sets, between shafts resulted favourably stressed; taking into account that a mechanical excavation method was planned, it was assumed that the rock in the previously said sets could maintain its full strength and consequently provide a significant contribution to the stability of the walls. It is to be noticed that the tensile zones in Fig. 12 are smaller then those of Fig. 10. In the lower part of the cavern the effect is clearly due to the new design;

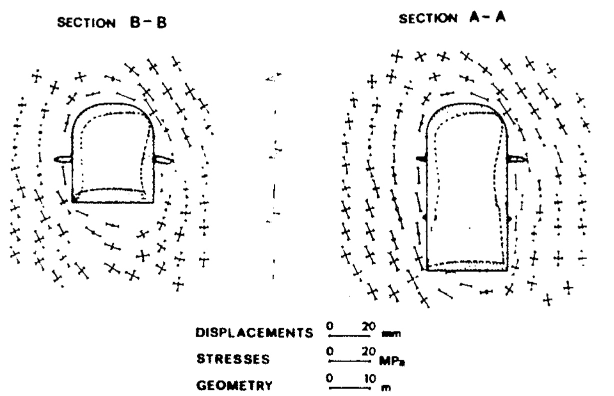


FIG. 12 Results of an elastic computation for the main chamber. Tensile zones are deshed.

in the upper part the effect is due to the smooth profile of the arch (introduced for sake of simplicity in the computation). A horse shaped profile of the upper part of the cavern was studied in some detail. Usually a solution alike is discarted because of the additional cost of the excavation but in the present case, the difference was low because of the excavation method to be used. The comparison from the statical point of view required two F.E. elastoplastic analyses. The calculations were performed by NOLIVP program with the parameters given in par. 4. Results are resumed in Fig. 13. The structural advantage obtained was not judged decisive (probably it would have been if the rock strength was a bit higher). Consequently the standard profile prevailed, mainly because it permits to have two travelling cranes, 160 tons each, working very early and helping the mouvement of machinary and of the excavated material. After this decision was taken, the control system was designed.

In particular the excavation of mounting drifts is foreseen in order to reach the crown and springlines of the arch so the instruments (see Fig. 14) will be placed before the start of the main excavation. Subsequent measurements will be compared with the prediction of mathematical models in order to confirm the geomechanical parameters assumed for the design and consequently the stabilization works planned.

The excavation schedule can be resumed as follows:

In a first phase the roof of the machine hall will be excavated down to the springline (elevation 55.7); the casting of the concrete arch and of the beams will follow the excavation front. The stabilization of the arch, at the

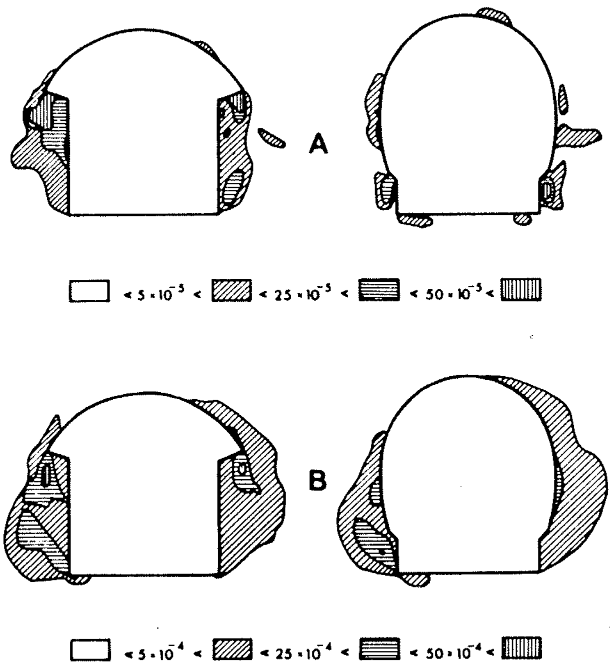


FIG. 13 Elastoplastic analysis comparison between two shapes of the upper part of the machine hall:
a) Unrevocable volumetric strains due to tensile failures
b) Unrevocable shear strains due to tensile shear failures.

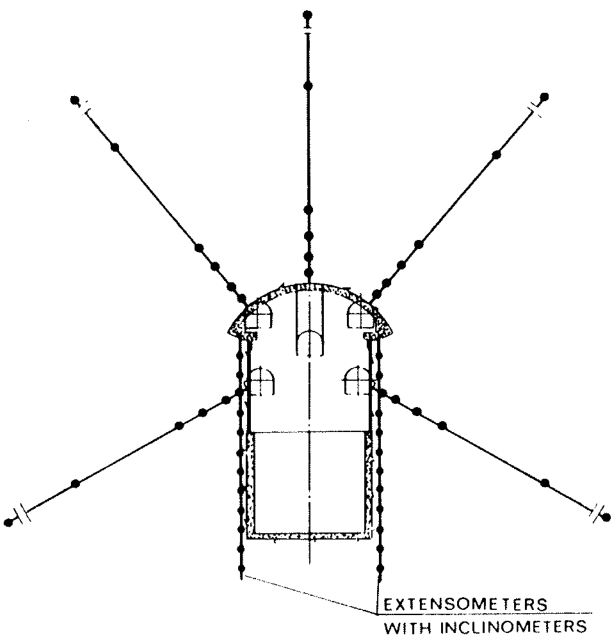


FIG. 14 Scheme of multi-base extensometers to be installed around the excavation of the machine hall.

springline, will be obtained by placing two rows of anchor cables (120 tons, every 4 meters). Benching down to the elevation 40.3 (46 in the central mounting bay zone), the walls will be stabilized by means of shotcrete (0.15 m thick and reinforced by steel mesh) and rockbolts (Ø 24 mm) having a variable length of 8 to 6 meters. This length should ensure a safe anchorage where the rock still mantain sufficient strength and work under compression. The excavation of the transformer chamber will follow the same criteria. In preparation to the subsequent phases the exploratory adit will be deepened to the level 20 m where a drainage tunnel is fore by side to the bottom of the main chamber. Secondary drifts will be then excavated toward the axes of the shafts where bore-holes (Ø 300 mm) will collect the water infiltrations. During the excavation of the shafts short rockbolts and shotcrete will be used and draining tubes placed against the rock walls. The prelining will be then covered by a waterproof PVC sheet and a final lining of reinforced concrete,0.9 meters thick, placed.

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