Stability of Caverns Created in Rock Salt by Solution Mining

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ABSTRACT

A solution mining operation is carried on by injecting water in the lower part of a salt deposit through boreholes that are lined up approximately according to the greatest dip of the seam. After wells are connected by hydrofracing, water is injected into several wells and brine goes through only one well, whose bottom lies downwards in the seam. Around each injection well the cavity grows upwards. The process results in files of lengthened cavities with rock salt pillars remaining between them. We have determined the mechanical characteristics of the rock salt and of the layers above the salt in the laboratory. The tests show that the rock salt has an elastoplastic behavior which may be defined by its cohesion and internal friction angle. From those data, we built a mathematical model based upon the finite element method in elastoplasticity. Using this model we determined the maximum width and height that a cavity must have to prevent collapse up to the surface. We also determined the minimum distance between two boreholes files required to ensure that the pillar which has not been dissolved between them, will remain stable.

INTRODUCTION

The following study was carried out on a solution mining project in a Keuper rock salt deposit located near Nancy (Lorraine, France). It is a sedimentary deposit, with a very light dip.

As mining progresses, water is simultaneously injected through several wells. The brine flows up through only one extraction well. The wells are located on a triangular mesh pattern. They were connected by hydrofracturing. Around each water injection well, a cavity develops upwards to the roof of the layer by the dissolution of salt. The development of each cavern is carefully monitored, especially by "sonar". In such a shallow deposit (the salt roof is about 200 meters under the surface) the final stability of each cavity as well as the stability of the rock salt "pillars" that remain between the cavities are all the more necessary as the overlying ground is to be maintained for agricultural uses.

With the collaboration of the Centre D'Etudes De Mecanique des Roches de L'Ecole des Mines de Paris, the mining company, Compagnie des Salins du Midi et des Salines de l'Est, therefore investigated the working standard which guarantees the stability of the surface grounds.

EXPERIMENTAL STUDY—MEASURE OF THE MECHANICAL CHARACTERISTICS OF THE DIFFERENT LAYERS

Compressive and triaxial tests were performed on cylindrical samples from drill cores. Rock salt and the rocks from walls and roofs were tested and the stress-strain curves recorded from strain gauges. It was established that, for rock salt, the transition from brittle to plastic behavior occurs at a low confining pressure (<50 bars). Under those conditions, when the stresses exceed the yield limit, there is no failure of the material, but plastic strains occur that may be very large.

Plotting the Mohr circles corresponding to the yield strains we have established that the yield limit is defined by a Coulomb straight line $\tau = C + n \, tg\phi$. The angle of internal friction of rock salt is equal to zero.

One can find in Table 1 the main results of the tests.

We did not try to measure the cohesion and angle of internal friction of dolomite because the few tests done on this material show that its yield limit in compression is close to 30 Mpa. Under those conditions, at the depth of this layer and its distance to the caverns, it is obvious that the

TABLE 1

Results of Compressive and Triaxial Tests

Depth	Layer	Young Modulus Mpa	Poisson's Ratio	Cohesion Mpa	Angle of Internal Friction
−130 m to −140 m	Dolomite	32600	0.25	—	—
−140 m to −145 m	Marls	26800	0.24	4.5	17°
−145 m to −170 m	Sandstone	5400	0.18	4.6	2°–12°
−170 m to −220 m	Marls	26800	0.24	4.6	7°–17°
−220 m to −285 m	Salt	20000	0.30	3.5	0°
Below −285 m	Marls	1000	0.10	4.6	17°

stresses in the dolomite will be low enough to ensure an elastic behavior.

In the calculus we did assume that rock salt was perfectly elasto-plastic. That means that in the plastic zones, the Mohr circles of the stresses are tangent to the Coulomb straight line defined in Table 1. In fact, this means that we disregard the viscosity of the rock, and thus assume that the asymptotical state of stresses takes place instantaneously.

METHOD OF CALCULUS

Computation of the stresses and displacements produced by mining has been done with the finite element method in plane strains, the displacements being a linear function of the coordinates inside of each element. Plasticity was dealt with by the method of the initial stress, using the classical iterative processes (cf. Zienkiewicz, The Finite Elements Method, 2nd edition Chapter 18).

STUDY OF THE STABILITY OF A UNIQUE CAVERN—BASIC ASSUMPTIONS

Strata section. Although the thickness of the layers varies slightly from one place to another we disregarded this variation and have assumed that the strata section defined in Table 1 was available for the whole district. We have assumed that the overburden which lays above the dolomite act only by its weight.

Shape of the caverns. According to the measurements made by sonar devices during solution mining we assumed that the caverns grew upwards from the floor and preferentially updip with a constant width, except in their upper part where their shape is that of a dome.

Initial stresses. It appears that in the district studied there were no large tectonic stresses in the rocks, thus it is correct to assume that the vertical direction is a principal direction of the initial stress tensor. This initial vertical stress is simply equal to the weight of the overburden that is to say: $\sigma_v = 0.023$ H, σ_v expressed in MPa, H in meters. Because we knew nothing about the horizontal stresses we stated $\sigma_H = \kappa\, \sigma_v$ and tried the two assumptions:

$$\kappa = 0.5 \text{ and } \kappa = 1$$

Permeability of the ground—effective stresses. Because of the great permeability of the marls of the roof we have considered two situations for which the analyses are basically different.

1. The cavern does not reach the marls of the roof. One assumes here that rock salt is impervious. Thus the cavern is subjected to an internal pressure $P_i = 0.012$ H (H in meters, P_i in MPa) H is the height of the brine column the density of which is 1.2 g/cm^3.
2. The cavern grows up to the marls of the roof: then the pressurized brine seeps through the marls and their tensile strength decreases to zero. This has been proved by laboratory experiments. In addition, the pressurized brine, when it seeps into the cracks, decreases the normal stresses acting upon the cracks and has no influence upon the shear stresses, thus the risk of failure increases.

This mechanical influence of the seepage is taken into account by introducing effective stresses which are those one must check with the failure criterion of the rock. To obtain effective stresses in a pervious rock, it is necessary to add to the stress tensor computed, the stress tensor $\sigma'_x = \sigma'_y = P\ \tau'_{xy} = 0$ where P is the pressure of the brine.

In the case that we study, because of the great permeability of the marls, the pressure of the brine that seeps into the ground is the hydrostatic pressure.

$$P = 0.012 \text{ H (P in MPa, H in meters).}$$

This pressure must be cut off from the initial stresses and our problem is equivalent to the problem of the stability of an empty cavern mined in a ground subjected to the initial stresses:

$$\sigma_v = -(0.023 - 0.12)\ H = -0.11\ H$$
$$\sigma_H = -(0.023\,\kappa - 0.12)\ H$$

One can see that when κ is close to 0.5, where the brine seeps through the ground, the horizontal effective stress is close to zero.

MATHEMATICAL MODEL—RESULTS

The calculus has been done using the mesh of the Figure 1, which represents a unique cavern 60 m wide and the main layers in its vicinity.

The first calculus, using the more pessimistic assumptions $\left(\frac{\sigma_H}{\sigma_v} = 0.5 \text{ and the cavern has reached the marls}\right)$ leads to the conclusion that the roof of the cavern is not stable. As a matter of fact the tensile stresses that appear in the roof create cracks inclined in such a way that a collapse is kinematically possible.

In addition we have checked that it is an unstable process and that the collapse can reach to the dolomite. When $\frac{\sigma_H}{\sigma_v} = 1$ no tensile stress occurs in the vicinity of the cavern and the compressive stresses are low enough to prevent any plastic strain to occur in rock salt or shear failure in the marls.

If a layer of salt is left above the roof of the cavern to prevent any seepage of the brine, the cavity is stable even if $\frac{\sigma_H}{\sigma_v} = 0.5$.

In conclusion, inasmuch as we do not know the actual value of $\frac{\sigma_H}{\sigma_v}$, it is necessary to abandon a layer about 10 m thick above the roof of the caverns. This layer, by preventing any seepage of the brine in the marls, allows a cavern 60 m wide to remain stable whatever the initial stresses may be.

STUDY OF THE STABILITY OF A GROUP OF CAVERNS

It has been proven that the caverns created by solution mining grow upwards parallel to the steepest line of the rock salt layer. As the injection wells are located at the nodes of a regular mesh, one obtains rows of caverns with "pillars" left between two rows.

The second part of this study deals with the stability of those pillars. According to the shape of the caverns, the plane strain conditions have been assumed in their cross section. In addition the symmetry in the location of the boreholes introduces some planes of symmetry in such a way that it is only necessary to study the zone between two vertical planes containing the axis of a pillar and the axis of a cavern.

According to the earlier results, a layer 10 m thick of rock salt has been left above the caverns. Their width is 60 m. The initial horizontal stress is assumed to be the half of the initial vertical stress which is equal to 0.023 H (σ_v in MPa, H in meters). The internal pressure is 0.012 H. The thickness of the overburden is 285 m from the floor of the rock salt layer.

A very simple calculus based upon the tributary area theory shows that, if one desires that the stresses in the pillar not exceed the yield limit of the rock salt, the width of the pillar must be close to 50 m. This result is useful because it allows us to get sensible dimensions for the finite element mesh used to do a more sophisticated calculus (Fig. 2). Actually it can be seen that with a width of 52 m, plastic zones occur on the side walls in the vicinity of the marls on the floor and in the beginning of the vault. When the width of the pillar is decreased the plastic zones increase towards the center of the pillar. Figure 3 represents the case of a pillar 40 m wide.

Theoretically, even in this last case, the pillar is not unstable since the plastic zones are surrounded by elastic ones. Nevertheless, because of our limited knowledge of the initial state of stresses and because it is not easy to predict the actual shape of the caverns, we think it is best to limit to 50 m and more the width of the pillars so as to ensure the stability of the whole field.

Consequently, water injection wells must be located at the nodes of a regular mesh built from equilateral triangles, one side of which being parallel to the steepest line of the rock salt layer. Each borehole must be 127 m far from its neighbors.

CONCLUSION

Although in the last twenty years, knowledge in rock mechanics has made great progress in the field of the rheological behavior of rocks and in the techniques of in situ measurement of stresses, it is often still impossible to know the value of all parameters which have an influence on the stability of an underground structure. Frequently one can only give limits to the possible variations of those parameters.

Nevertheless, with the modern methods of calculus it is possible to represent without too much simplification the actual situations. These methods permit an increase in the value of the measured parameters, allow one to check the influence of the variation of the unknown parameters on the stability of the studied structure and allow one to test what changes in the shape of this structure would improve its stability. This study is an example of using the method. It shows the practical information provided by the use of the finite element method in elastoplasticity, especially to those who care for mining salt in the best way without taking any unnecessary risks.

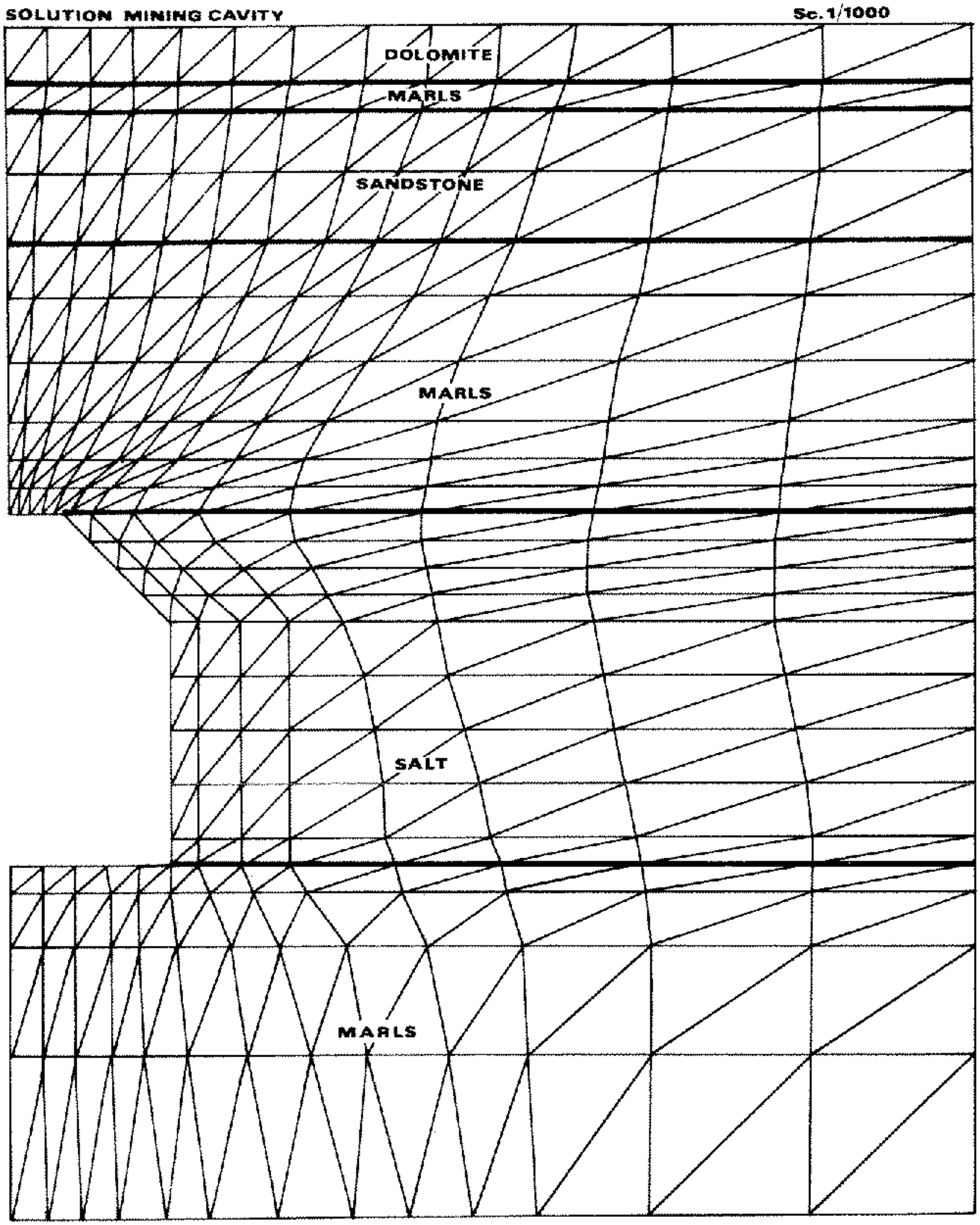

Figure 1. Solution mining cavity.

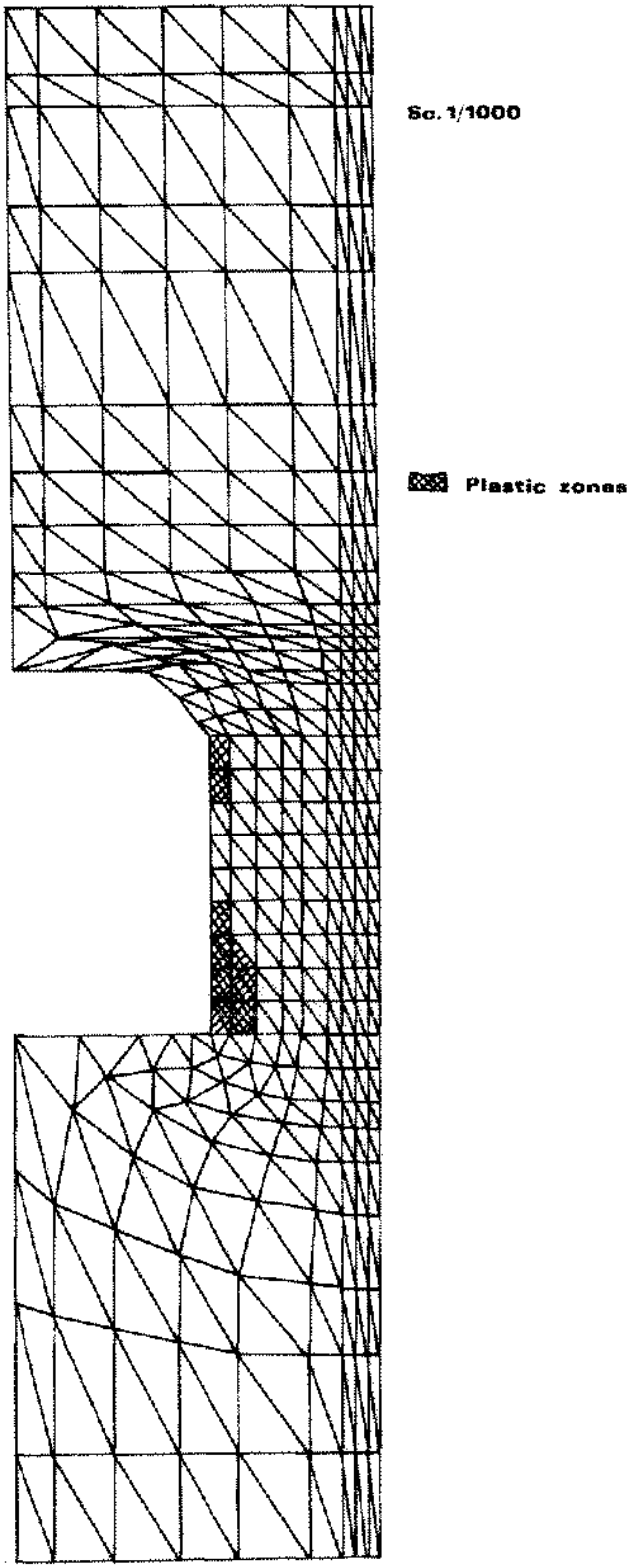

Figure 2. Stability of a group of solution cavities.

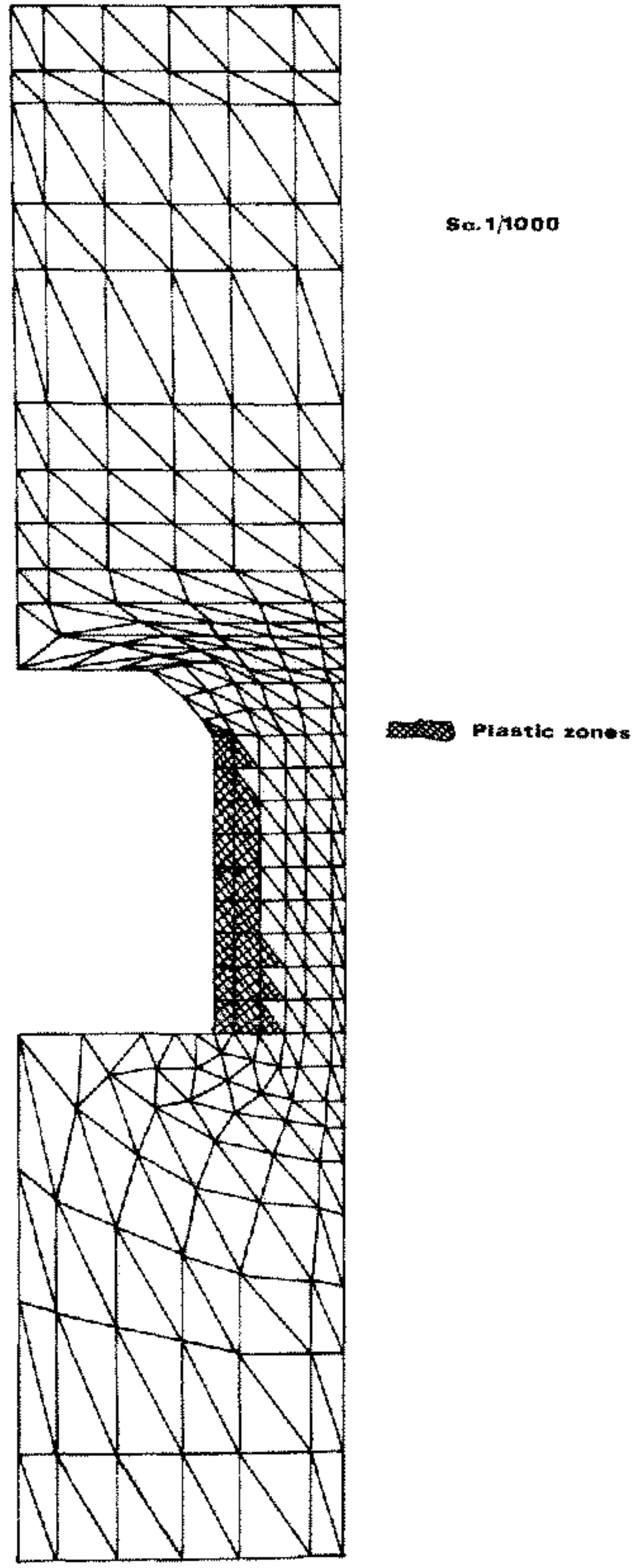

Figure 3. Stability of a group of solution cavities.