# The Stability of the Underground Structures Achieved in Salt Massif and Their Monitoring

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*Abstract:* In mining, one of the methods of exploitation which is still the most used in rock salt deposits of potassium and magnesium salts is the method with abandoned rooms and pillars. The correct dimensioning of rooms and pillars (safety pillars) supporting is particularly important from the point of view of security exploitation but also the stability of the entire underground structures, protection and rational exploitation of the deposit, especially in terms of stability and surface protection updated when measures to avoid the occurrence of lands surface movements are imposed. Ensuring the stability of underground voids created by the exploitation of rock salt by solid way, requires the complex studies of correct knowledge of the geomechanical characteristics, the micro- and macroscopic behaviour of salt, but also for the rocks where the salt deposit is quartered; a particular importance in this direction, is the study of the phenomena of dilatancy and especially the involvement of the time factor to determine the characteristics and rheological parameters, features which can be assessed through the period of maintaining the stability of underground structure during exploitation, and after those, exhaustion.

*Key words*: salt, stability, pillars, underground voids, displacement, stress, dilatancy, dissolution, creep, rheological behaviour

## 1 Necessity of knowing the geomechanical characteristics of salt

Given the fact that currently, the exploitation of the salt deposits is taking place mainly in the depth, the thicknesses being appreciable, the extraction of the salt is done in several floors with the coaxial arrangement of the rooms so that the pillars that are left between them to be situated exactly in prolongation, in order that the lithostatical pressure and the stresses which manifest upon them, to be transmitted coaxial. Just as in the horizontal plane between rooms' pillars - pillars of protection - are left, then on vertical is imposed to leave some floors that have both role of delimitation the height, especially the role of increase the safety in exploitation and the stability of the pillars. The thickness of a floor between two rooms is determined by the way that the exploitation is carried, namely ascending (method very rarely used in practice) or down frequently used. The pillars have the role to take over the static loads given by the corresponding weight of the deposits itself and the weight of the overburden rocks, preventing them to collapse into voids created by the carrying rooms and lead the appearance of the displacements that would cause production the settlements and the subsidences of ground surface. However, because of this, the pillars must be properly sized, the establishment of the optimal dimensions being a very delicate problem that involves taking into account several factors, such as: geological and petrographycal characterization of salt, training mode, micro- and macroscopic characterization of salt, the presence of fluid inclusions and pores, physical - mechanical characteristics of salt, behaviour of deformation and the phenomenon of dilatancy, time factor through rheological characteristics that have an important role in assessing the time in which the underground structure is maintain stable, etc. In terms of safety and stability, the safety pillars must satisfy the strength requirements but also the ones of deformation. Knowing the elastic limit and plastic limit of small deformations of rock salt - the material making up the pillar – the determinations being based on the laboratory tests, checks of the pillars by measuring at time intervals of deformation or by analytical calculations based on different computing hypotheses existing in the literature could be performed. If the measured deformations do not exceed the deformations appropriate to the limit of elasticity or to the limit of small plastic deformation, then the pillars have enough security and stability, otherwise there is a potential possibility that at any time the fissures, fractures or cracks appear, and sometimes can trigger a partial or total collapse of pillars. Regarding the rock salt from Romania, based on laboratory tests (Stamatiu, 1962) a mean value 25 daN/cm$^2$, was established for the limit of elasticity and for the limit of plastic deformation as 100 daN/cm$^2$ [1]. If the salt in pillar is located in the field of plastic deformation, another important factor occurs, which we have already mentioned, namely the time that makes the deformations to increase even if the external loads acting on the pillar remain constant, i.e. the creep phenomenon of the salt occurs. In fact, this phenomenon has been studied since 1992 and by the author too, in laboratory on different kinds of salt, the results proving that the load samples beyond the limit of elasticity and through the limit of plastic deformation an increasing in time of the deformations of samples takes place, Figure 1, [5], [12]; this variation of the deformation is most often accompanied by the phenomenon of dilatancy of salt and for the rock salt from Romania the data based on the laboratory tests demonstrated the dilatant character of salt behaviour under load.

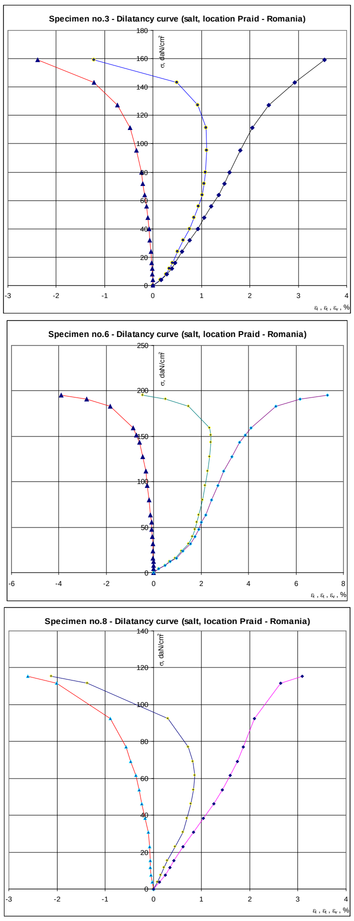


Fig.1- Dilatancy curves for the rock salt from Praid (after Todera ,2003)

The same dilatant character of salt was also mentioned by Penkova in his studies, concluding that, during the fast triaxial tests on rock salt, the transversal deformations may exceed the axial deformations, so the salt is characterized by a pronounced phenomenon of dilatancy. The same dilatancy phenomenon of rock salt were described by Stavrogin (1967), showing that, for prismatic specimens of 15 x 15 x 30 cm and a load equal to 0.7$\sigma_{rc}$ ($\sigma_{rc}$ is the uniaxial compressive breaking strength), the volume of specimens increased by 17 % over a period of 800 x 24 hours. Studies have shown that for rock salt from Romania the maximum depth up to which can exploit on the solid way by rooms and pillars method is 1000 m; over this depth, the risks and technical difficulties and also safety occur, so it is recommended that the exploitation is effected by dissolution, with the achievement of caverns of which the brine is then extracted at surface.

## 2 Problem of the stability of underground structures in rheological context

In the calculations of stability of salt massif its essential features must be evaluated, namely the rheological properties, its nonlinear character of behaviour $\sigma$ - $\varepsilon$, the phenomenon of dilatancy under the action of shear stress. In the case when the state equation – behaviour equation – don't describe the three stages of creep, then to may resolve mathematically the stability's problem, initially it must establish the stress – deformation state, and then is imposed the long-term strength condition. As a long-term strength condition, the criterion which characterized satisfactorily the strength of salt under an instantaneous load can be used. In this case, in the condition of the salt's strength, it will introduce the long-term strength value instead of instantaneous characteristics (t = 0) [5], [10], [11], [15], [16], [17].

Knowing correctly that the behaviour in time of underground structures, and implicitly of galleries and rooms, is not possible without accurate knowledge of the constitutive law behaviour of rock and in the same time of salt in which these mining workings are executed.

Establishing an appropriate law behaviour of salt which corresponds to the real situation, involves the analysis of certain factors that may influence the rheological behaviour of salt, e.g. the existence of hardening deformation, the way to influence of main stress, the existence and the form of viscous-plastic potential, the influence of temperature, etc. [8], [12],

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[13]. Establishing law of behaviour, its validity and calculation the parameters involved in the constitutive behaviour equation, depends largely on the results achieved experimentally and / or in situ [12], [15], Figure 2.

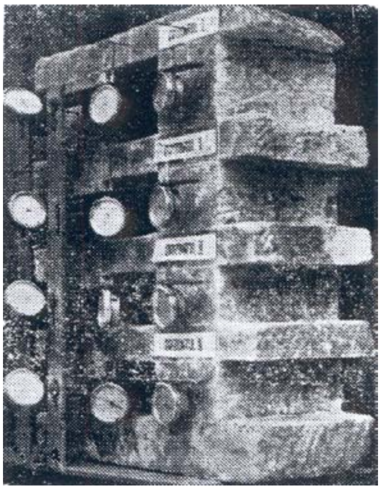

Fig.2- Room and pillars model - example of creep test under Praid salt

To understand the rheological behaviour of salt better and for a complete analysis of the stability of an underground structure achieved in salt, both in its exploitation by solid way and dissolution, I proposed the following scheme that shows the main stages of study that must be respected or taken in such an analysis, Figure 3, [12].

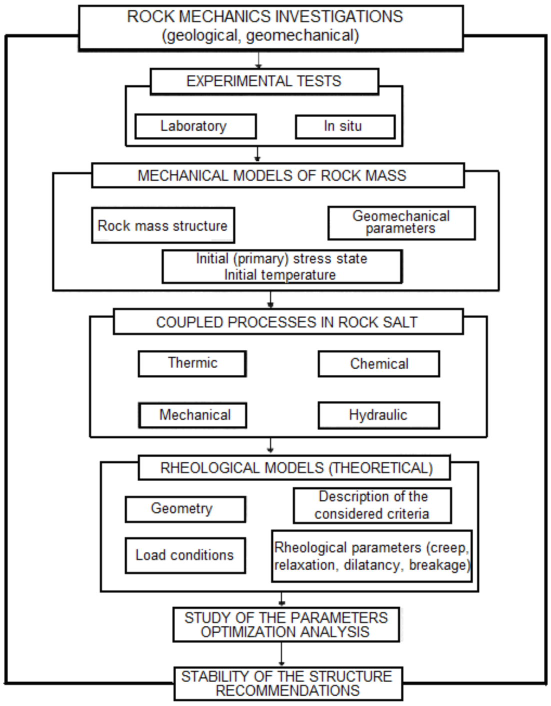


Fig.3- Stages of stability analysis of an underground structure and a cavern with brine (after Toderas, 2003)

Depending on floors which belong to geological salt deposits, the main mineral components are present in different formation, which have determined the appearance of a certain number of lithofacieses with various texture and composition from dominant halite (phenoblastic salt and phenoblasts of halite in clayey - sulphate - carbonate matrix and intermediate salt or mixed salt) and lithofacieses associated (clastic - clay and marl and sulphate - carbonate).

The presence in abundance of the fluid inclusions can influence the behaviour of salt by:

- Mechanical interaction that can lead to plastic deformations, but most often to the fragmentation of rock by hydraulic fracturing;
- Dissolution – crystallisation when upon contact with the solid, the fluid acts by a physical – chemical interaction.

In a global and very general way, it was found that the phenoblastic salt has somewhat a pronounced isotropy, but it's heterogeneous, unlike white salt characterized by an anisotropy marked by the presence of intercrystalline defects, while its overall composition is homogeneous; for mixed salt a slight anisotropy due to ensemble of some characteristic components resulting in dissolution zones filled with halite in directions perpendicular to the stratification was observed [8].

To analyze the stability of underground structures made in salt massive, first you must know very well the behaviour of salt, both micro- and macro-structural scale; microscopic analyzes allow the identification of neo-formations mineral.

## 3 Behaviour of rock salt

Salt has its particularities: solubility and viscous-plastic behaviour. Elastic - viscous - plastic behaviour of rock salt can be interpreted in terms of different fundamental mechanisms of deformation. Understanding the behaviour of at great depths of salt requires knowledge of the deformation processes that occur in ordinary conditions. Halite (NaCl) and potassium (KCl) present different processes of deformation that depend on the stress state ($\sigma'_m$), temperature (T) and the limit of plasticity ($\tau$) but in accessible conditions of laboratory ($\sigma'_m < 70$ MPa ; $\tau < 40$ MPa ; T < 150 $^{o}$C). Rock salt behaviour can be described and treated by analogy with the behaviour of rocks located at great depths [2], [9], [12].

Often the salt has a viscous - ductile brittle type behaviour for the strain like $d\varepsilon / dt = 10^{-2} – 10^{-12}$ $s^{-1}$: the failure and dilatancy field for the small stress σm, ideal elastic-plastic behaviour and without dilatancy to the high values $\sigma_m$, a transition from viscous-elastic creep to viscous-plastic creep for high shear values (τ), secondary creep stage, strain dependent of the plasticity criteria and primary creep with recovery. In rock mechanics the elastic-plastic field (EP) is considered as a field of conventional brittle (Y). For example, the results achieved by Brazilian method and triaxial tests performed on salt samples [3], Figure 4, show that for a low value of the stress $\sigma_3$ the brittle behaviour with the development of dilatancy occurs and the brittle deformation, ε is small; the conventional brittle field depends by the value of the stress $\sigma_3$, the uniaxial compressive strength being of 12 times greater than the tensile strength ($T_0$). For a high value of stress $\sigma_3$, a cataclastic flow occurs accompanied by dilatancy and rupture of salt crystals, and to lateral stress by intermediate values (for triaxial test) the transient behaviour is present. For halite, the transition from fully cataclastic plastic behaviour appears for stresses $\sigma_3$ = 10 – 20 MPa; $\sigma_1 - \sigma_3 |_{EP} \approx 35 – 40$ MPa.

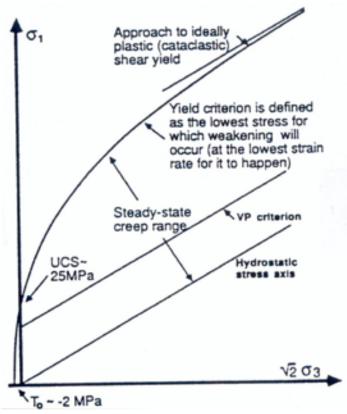


Fig.4- Failure field for a halite dome

Generally, the elastic-plastic failure field (EP) of rock salts for low values of $\sigma_3$ is defined as follows: if tangential stress does not exceed the limit of conventional fracture (τ < Y), then the secondary creep occurs, but does not exclude the situation that a tangential stress τ < Y to be applied for brief periods of time before the deformation begins to attenuate; the major conceptual elements of elastic-plastic failure of rock salt are shown in Figure 5.

The area of viscous-plastic field (VP) is the surface limit between two types of viscous-plastic behaviour, Figure 6. VP failure criterion depends on the value of $\sigma_m$ and actually is plasticity criterion of type Tresca, i.e. $2K = (\sigma_1 - \sigma_3)_{VP}$ [17]. Typical values for salt and potassium are 2K = 15 – 18 MPa and $d\varepsilon/dt]_{2K} = 10^{-11} – 10^{-10}\ s^{-1}$.

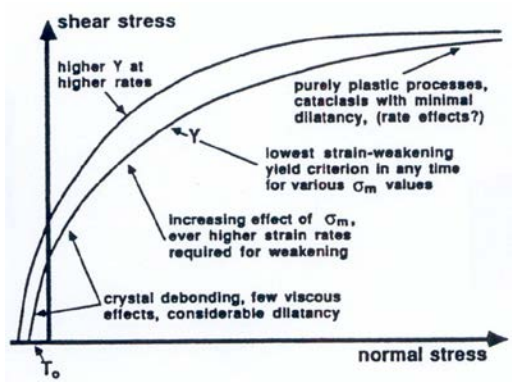


Fig.5- Elastic-plastic failure criterion of rock salts

For any values of stresses situated below the limit of conventional failure field, the salt manifests the creep phenomenon.

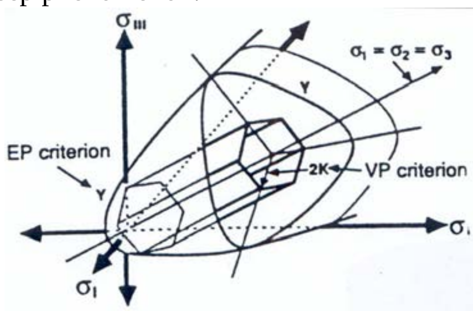


Fig.6- Elastic-plastic and viscous-plastic failure criterion in stresses space

Interest presents two areas, namely once located below 2K representing viscous-elastic area (VE) where the relevant are solution - precipitation and slide of dislocations mechanisms; area between 2K and Y, which is viscous-plastic area (VP). Basic physical equations are the form [4]:

$$\dot{\varepsilon}_{ss} = A\left(\frac{\sigma_1 - \sigma_3}{G}\right)^n \cdot e^{-\frac{Q_1}{Rt}} \qquad (1)$$

where: A, Q si G – constants of material; Q – intensifying energy for each mechanism; G – shear modulus; for normalization, the authors used size 2K. For the precipitation and slide of dislocations mechanisms n = 1, respectively n = 3, [6], [9].

Physically, such mechanisms are not coupled, don’t appear simultaneously, they are additive; the increasing of the velocity of deformation for different temperatures depends on the intensifying energy, grain size, moisture, mineralogy, defects density and texture. These mechanisms, however, don’t depend on the amount by stress values $\sigma_m$, and for the values below 2K the strain curves are parallel to the octahedral axis. The two mechanisms can be studied in the laboratory for the same conditions, but the temperatures encountered in mine it seems that the processes are very weakly observed [9].

In the viscous-plastic area, the exact nature of the deformation mechanism is still not clear, fracture or failure occurs along the grains surface, but can be found for the rock salts under the stress $\sigma_m$ relatively reduced, due to the presence of water and high solubility of NaCl and KCl.

The continuity of secondary creep stage, stable without development of tertiary creep field observed during in situ and laboratory testing confirms a balance between fracturing and the character of a normal or normalized fracture (as it's called in the literature), but not for rock salt. Probably for such rocks, the concept of critical deformation related to the tertiary creep is not valid [14]. In the viscous-plastic area the strains related to secondary fracturing stage, are not the power functions type; the fracturing starts for reduced values of tangential stress $\tau$ (= K) and developing rapidly with its increasing but is prevented by the stress $\sigma_m$. For secondary fracturing state, the strain's law proposed by some researchers is:

$$\dot{\varepsilon} = 0 \quad if \quad \sigma_1 - \sigma_3 = 2K \quad (other\ processes\ exist)$$

$$\dot{\varepsilon} = C \cdot \sinh\left(\frac{\sigma_1 - \sigma_3 - 2K}{\sigma_0}\right) \cdot e^{-\frac{Q_3}{RT}} \cdot f(\sigma_m) \qquad (2)$$

where: C and $\sigma_0$ – specific characteristics of the material; $f(\sigma_m)$ – function that takes into account the effect of normal stress on the strain.

Relating to the primary creep, macroscopically speaking there are three sources that determine its manifestation: stress redistribution in the rock mass, reestablishment and small changes in texture. Neither this inherent transient creep is not known too well as regards the mechanisms that occur on a microscopic scale and determines the macroscopic effects. Kornelson (1988) assumed that in the polycrystalline rocks, the transient creep is virtually the result of unavoidable redistribution of the stresses to crystal scale. In order to validate numerically such a concept, the author studied a viscous-plastic element (Figure 7) which he considered it as a consolidated ensemble; the element was divided into irregular networks of triangular elements that describe transient response.

From the quantitative point of view, the deformation behaviour of with recovery obtained was similar to laboratory test results, the transient response being dependent on the distributions of deformation characteristics E, and $\mu$. For the transient creep the exponent of stress is n > 1.0, a high value of this coefficient n is associated predominantly with a pronounced transient response.

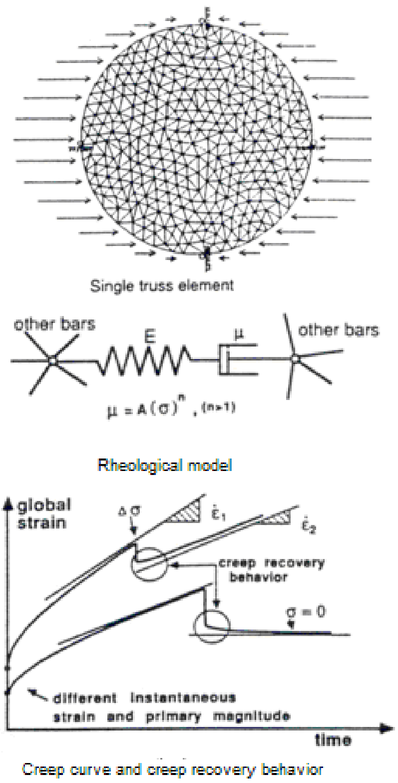


Fig.7- Typical response for a meshed element (after Kornelson, 1988)

Around an excavation carried out in a potassium deposit there can be identified 3 zones, Figure 8: elastic-plastic zone (EP) in which the salt has a very low resistance (you might say that the salt has lost its strength); viscous-plastic zone (VP) where is a permanent balance between $\tau$ and K; viscous-elastic zone (VE) for the rest of the rock mass, where $n \leq 3$.

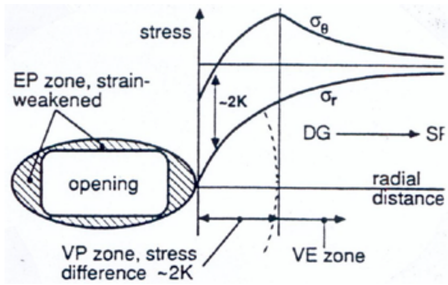


Fig.8- Stress state on the profile of an excavation – potassium mine (after Kornelson, 1988)

Stress distribution and strain are essentially stable on the profile of excavation, and a very low strain is typical and probably associated with small changes in geometry and in the evolution of landslides at the level of texture. The deformation on the contour depends on the dilatancy phenomenon that occurs for stresses of several MPa. Based on knowledge of stationary creep law we can obtain a very good approximation of the long-term evolution of strain

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on the profile of excavation, even of mines where the rooms and pillars is a method of exploitation. Considering the microscopic and macroscopic creep laws allows understand and knowledge of all structural responses in the case of underground structures from a mine and can be easily extended for the cavities or even underground storages [12].

# 4. Salt behaviour at great depth and the safety factor

Rock mechanics allows determine with some precision the mechanical parameters of rocks, but doesn't take into account the phenomenon or the factor "time" to real scale. To resolve such a problem, it calls for rheology, but have clearly specified that it is no possible to study in the laboratory the behaviour of a large number of samples, which would involve a period of years, and therefore the problem can be solved using different numerical programs that allow modelling not only the rock samples, but also the behaviour of underground structures over an extended period of time.

The execution at great depth of an underground working in a salt deposit made up of several layers of salt with different characteristics requires from the beginning a detailed analysis of deposit geometry with parallel achievement of the laboratory tests in order to establish the parameters that characterize the behaviour of each type of salt (compressive strength, tensile strength, shear strength in triaxial stress state; deformation parameters - elastic modulus, Poisson's ratio, characteristic curve stress - deformation and dilatancy curve; parameters of viscous character of the deformation behaviour by creep tests under constant load).

Dudek (1989) has studied the stress - deformation state on the profile of a rectangular excavation and horseshoe shape, excavated into a salt massive of 1300 m at a depth [2]. The laboratory tests have concluded that most samples have not reached its steady (stationary) creep, but in some cases it was possible to distinguish this phase. To interpretation the results, the author applied the rheological model of Nakamura that expresses the variation axial deformation respect to time, Figure 9, described by the equations:

$$\varepsilon(t) = \frac{\sigma}{G} + \sigma \sum_{i=1}^{n} \frac{1}{G_i}\left[1 - \exp\left(-\frac{t_i}{T_{rel_i}}\right)\right] \qquad (3)$$

$$\frac{t_i}{T_{rel_i}} = \text{const.} \quad ; \quad \varepsilon_{pl} << \varepsilon_{el} + \varepsilon_{neel}$$

$$\varepsilon(t) = \frac{2}{9\,K} + \frac{1}{3\,G_2} + \frac{1 - \exp\left(-\frac{G_1}{\eta}\,t\right)}{3\,G_1} \qquad (4)$$

where: $K = \frac{E}{3(1-2\nu)}$ - bulk modulus, admitted as independent of time, MPa; parameters $G_1$ and $G_2$ determine the changing shape, MPa; $\eta$ - salt viscosity, MPa·days.

The results obtained have clearly demonstrated a pronounced heterogeneity massif studied, due to the intrusion clay - marl and of course, because of the composition of each layer.

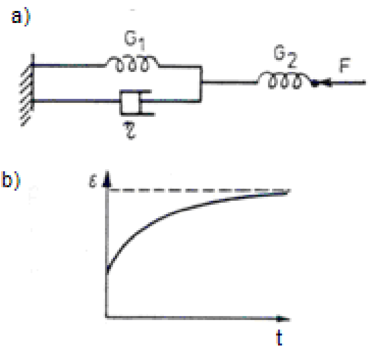


Fig.9- Rheological model of Nakamura: a) elements of a model; b) response respect to time – creep curve

Modelling the behaviour of rocks salt the profile or of the underground working has been done based on the finite elements taking into account the factor *time* and the elastic - viscous properties of the environment, with the possibility to establish areas where the resistance of massive was overcome. Like failure criterion is admitted the tensile strength compared with reduced stress, whose computation involves the transformation of three-dimensional stress state in a single value.

On the profile of excavation the displacements reach certain values (at the time of excavation t = 0, which corresponds to elastic behaviour), after which they increase in time with a tendency of amortization obtained even from rheological model of Nakamura (period after which the displacements were balanced was 60 days for both studied geometries). Just as it was natural, the bigger movements have been registered for rectangular profile compared to the horseshoe shape; in the middle of the floor the bulking deformations were recorded too. The stress state shows the presence of stress concentrations in the corners of the floor and tensile stresses occurring in the middle of the side walls; in the vaulted side walls the tensile stress were not developed, Figure 10.

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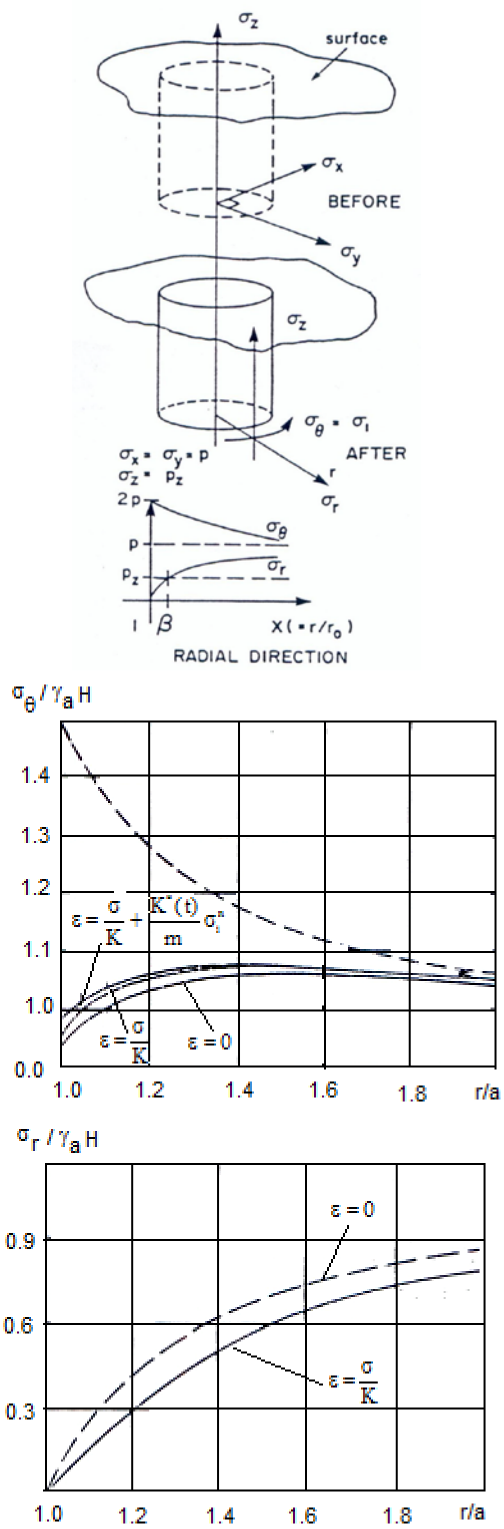


Fig.10- Stress state before and after excavation boring and experimental stress state variation around the excavation

Based on the data, the influence of the different factors on the underground structures stability and deposit could be analysed; characteristic curves of salt massif offers the possibility to establish with sufficiently high precision the profile displacement of the working respect to time.

The stability of rocks and therefore of salt on the profile of the underground workings is strongly influenced by the mechanisms of deformation at different temperatures. One of these very important mechanisms is the fracturation either one pre-existing, either a fracturation that can be created, mechanism that changes the rock permeability. In rock mechanics, to study the stability of rocks is used the safety factor, like a ratio between the principal stress of failure and the applied principal stress ($\sigma_{1f}$ / $\sigma_1$). Wilkins (1974) shows that the variation of this safety factor is given by an exponential relationship, on the form:

$$\frac{dY}{dt} = \frac{1}{n-2} Y^{3-n} Q^{2-n} \tag{5}$$

in which: $Y = \sigma_{1f} / \sigma_1$; Q – constant that depends on the stress state: for traction Q = 1 and for compression Q > 1, (seconds)$^{1/(n-2)}$.

Using the numerical modelling and of course considering the influence of the time too [19], the displacement of rock on the profile of the excavation and the creep deformations of them for a given distribution was establish, Y, on the contour of the cavity and for n and Q known. For a cylindrical excavation, Figure 8, the stress state in cylindrical coordinates after its excavating, is the following:

$$\sigma_r = -p\left[1 - \left(\frac{r_0}{r}\right)^2\right]$$

$$\sigma_\theta = -p\left[1 + \left(\frac{r_0}{r}\right)^2\right] \tag{6}$$

$$\sigma_z = -p_z$$

where: r – distance in radial direction; $r_0$ – radius for t = 0; it's note that the compression is consider negative.

To failure, the principal stress [7] is:

$$\sigma_{1f} = |\sigma_3| + \sigma_c \left(m\frac{\sigma_3}{\sigma_c} + s\right)^{\frac{1}{2}} \tag{7}$$

where: $\sigma_c$ – compressive strength; m and s – constants.

The principal stresses respect the conditions:

$$\begin{array}{ll} \sigma_1 = \sigma_\theta & \text{ever} \\ \sigma_3 = \sigma_f & \text{for } 1 < X < \beta \\ \sigma_3 = \sigma_z & \text{for } \beta < X < \infty \end{array} \tag{8}$$

with: $X = r / r_0$ and β defined in Figure 10.

The safety factor is given by the expression:

$$Y = \frac{\sigma_{1f}}{\sigma_\theta} = g(X) \quad \text{for} \quad 1 < X < \infty \tag{9}$$

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The fracturing will determine decreasing of the compressive strength, $\sigma_c$, and of the safety factor, Y, in time; it's suppose that the elastic modulus of the rock and the stress state remain the same for Y > 1. In this model it's supposed that for Y = 1 the rocks are fracturated and the radius of excavation shall be reduced. By integration the dY/dt, the following is obtained:

$$t = Q^{n-2}\left(Y_2^{n-2} - Y_1 n - 2\right) \qquad (10)$$

where: $Y_2 > Y_1 > 0$ and t is the time (seconds) required for $Y_2$ decrease to $Y_1$. In Figure 11 is represented the distribution of the safety factor Y for t = 0, Y = g(X), respectively Y = $f(X)_1$, with a function $f(X)_1$ obtained starting from the equation resulted after integration:

$$f(X)_1 = \left[g(X)^{n-2} - g(a)^{n-2} + b_1^{n-2}\right]^{\frac{1}{n-2}} \qquad (11)$$

with (a, $b_1$) – coordinates of X and Y for each point $f(X)_1$. If $b_1$ = 1, then the radius will be ($r_0$a).

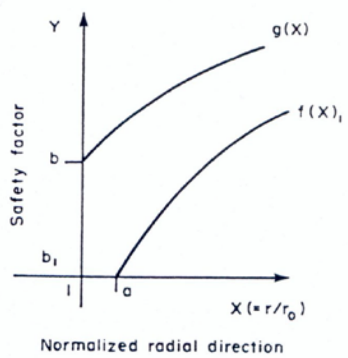


Fig.11- Safety factor depending on the normalized radial direction (after Wilkins and Rigby, 1989)

If X and Y take a negative value, then the function $f(X)_1$ doesn't exist, and the safety factor has no physical signification because its negative values are no possible and the radius must be greater then 0. Time equation becomes:

$$t(1) = g(a)^{n-2} - b^{n-2} \qquad (12)$$

The relative displacements in the horizontal plane can be obtained based on the Y distribution for different values of the time; for t = 0, the relative position is describe by the corresponding value X, respectively Y, and for t > 0 and for the same value of Y, the new positions and displacements are defined and thus we can calculate the deformation along the radial direction, figure 12.

The radial deformation varies from (1 – $a_1$) to 0, if X varies from 1 to $\infty$, being given by the following expression:

$$\varepsilon_r = \frac{a_1(a_3 - a_{11}) - (a_5 - a_{44})}{a_5 - a_{44}} \qquad (13)$$

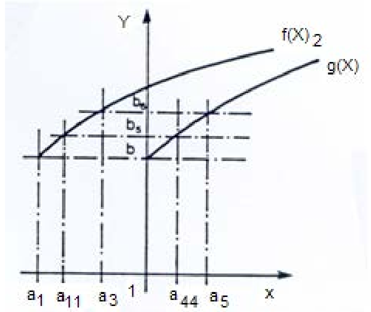


Fig.12- Safety factor in relation with normalized radial direction after transformation

In order that the radial deformation be small, the difference ($a_3$ – $a_{11}$) must be small; assuming that $f(X)_2$ is parallel with g(X), then in the range 0 < X < $\infty$ the radial deformation is $\varepsilon_r$ = 0. This model establishes on one hand the velocity of manifestation of the undamaged (intact) rock creep, and on the other hand the deformation on the contour of underground workings taking into account the dependence time – fracturing; these sizes depend on the rock strength, stress state and especially on the fracturing's exponent n. In depth the in situ stress increases and Y decreases, and the micro fracturing is significant. One of the disadvantage of the model is the fact that ignoring the variation of elastic modulus as an effect of fracturing; the breaking of rocks occurs when Y = 1, but in practice, the rocks are located in a triaxial compressive stress, which shows that in fact $\sigma_3$ can not be reduced to zero for Y = 0, at any point the decreasing of Y can be small. For example, if we speak about an underground cavity where it's brine, the pressure of brine will reduce the existent micro fracturing and consequently $\sigma_3$ > 0 [12].

# 5 Monitoring of underground structures carried out in salt massive

Underground working's monitoring is aimed at the acquisition of data and information necessary to improve the knowledge about the behaviour of these cavities and their evolution. Depending on the available means and opportunities at their disposal for monitoring the underground structures can be used the seismic-acoustic methods and geophysical methods - seismic method of high resolution, electromagnetic methods, electrical methods, micro-gravimetric method – through which can be made primarily detected the underground old voids, abandoned and actual, and based on the results achieved by these methods can be studied on the one hand the behaviour of underground openings and can be made associated modelling studies, and on the other hand can be performed fundamental studies of laboratory regarding the fracture mechanism and the possibility of its modelling. Geophysical measurements involve making measurements of deformation fields in depth and on the surface from which we can identify and characterize the previous stage of subsidence or settlements, one can study the stage of subsidence or proper settlements and also can determine their consequences. Scientific researches performed on geophysical methods for detection, localization and characterization of cavities / the underground structures located at depths ranging from a few tens of meters to several hundred meters are those topical interest. Actually, for this range of depths, most methods present limits on their use it and needs to be adapted to target scope, that is detection and characterization of abandoned underground workings and rock covers. However, some methods seem to be promising on condition to develop and obtain the appropriate means of investigation and interpretation of results. The difficulty in applying the geophysical methods consists in holding the necessary equipment and its calibration, respectively the correct use of processing methods of the signal. In the first stage, the monitoring of underground workings involves carrying out the studies concerning the methods of prediction and prevention of ground movements related to the underground cavities, which involves the study of factors which influence the behaviour in long-term of the underground structures and the material left unexploited, namely pillars of safety; these studies involve laboratory tests and in situ measurements. It is necessary to analyze the propagation of consecutive instability phenomena from underground to surface which may occur and taking into account the rock - structure interaction of the different mechanisms involved in the phenomenon of terrain movement. Based on these considerations, the monitoring of the underground salt cavities involves the following studies, presented in Figure 13. It must be noted that this studies, refer both to the underground voids created through the exploitation by solid method as well as the exploitation by dissolution method of rock salt.

To register the movements of terrains on the surface during both the initial phase of realization of underground cavities and the subsidence process can be done by tachometric auscultation method, using high precision tachometers coupled for example to GEOMOSS code of Leica's company. Acquisition of these data could allow the comparative analysis with the available geophysical (micro seism) and geotechnical (extensometers, inclinometers) measurements; tachometer can be piloted from distance with transferring the information automatically to a computer.

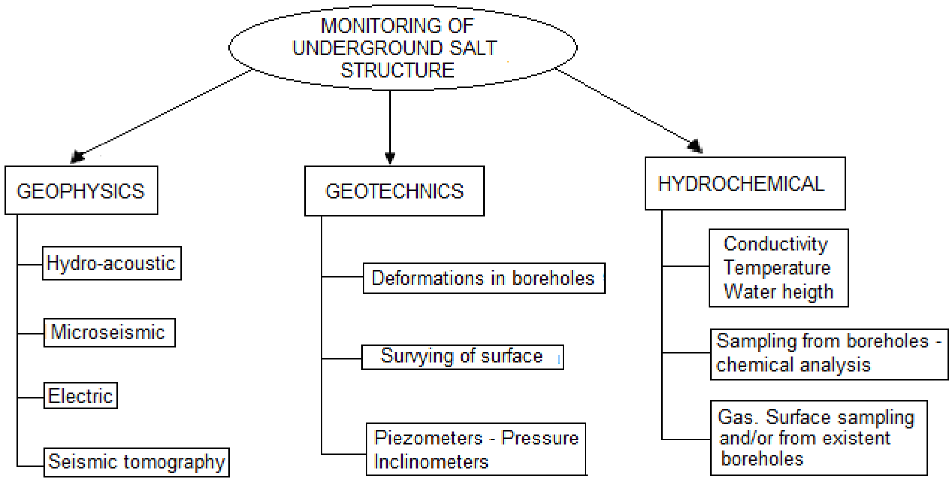


Fig.13- Monitoring methodology of an underground salt structure

Therefore, the monitoring of underground workings must include the probability that the mine could be flooded with water (or brine). However, we shouldn't avoid the fact that sometimes cases when the mine is not correctly dimensioned are encountered and then inevitably, the phenomenon of subsidence occurs. If we were to take into account the numerous salt mines that are active, which have evolved rapidly and the previously specifications too, then frequently concludes that one of two events - subsidence or water invasion in mine - that may occur is the consequence of the other one. Thus, it should be previous or conceive a system of alert that prevents of early warning the possibility of occurrence of flood water since they involve other events; in the moment of their manifestation in

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surface the cracks occur immediately. These are the first signs of the movements and instability terrain. There may be situations when the top wall collapses suddenly (here I should remember the case of Jefferson Island mine when she was flooded in a few hours or Retsof mine [18], when the first only the cracks appeared causing a descend of the terrain with a few tens of cm and after 21 months, the mine was completely flooded and the ground surface moved vertically by 18 m; there are also situations when the events occurred gradually and was enough time to make decisions appropriate to the conditions). In such situations, the question that arises is: what to do after finding a flood? Must discharged what is found on the surface should be injected in saturated brine to voluntarily limit water effects, or what to do? These are several aspects which should be prepared in advance and known the answers.

If the exploitation of salt is by dissolution and the mine is flooded with saturated brine unable to dissolve the salt, then, there will be no important changes of mine geometry. But, if the flow has high values, then erosion may occur in the proximity of the access of water or brine, which entrains the other materials in mining voids, causing the appearance of a localized subsidence or voids which, may extend up to the surface. Certainly, there are very rare the situations when the mine could be flooded by an important flow of saturated brine, but should not be excluded. Conversely, if invasion of freshwater takes place, then this could lead to setting up of spectacular voids on the surface, mainly due to dissolution that occurs on the access route, even to the mine, preferentially in the point of penetration of the water in mine.

## 6 Conclusions

Since the rooms and pillars method set up a unique spatial system of dry exploitation of salt, then it is certainly imperative to establish an efficient correlation in terms of stability - reliability between geometrical parameters of this system and massif surrounding.

The opening of mine determines a redistribution of stress; when the mine is correct dimensioned, this redistribution is not a negative factor that could compromise or endanger the stability, but through it can create new discontinuities or open the pre-existing discontinuities that will be a way of access for groundwater, even if their opening is reduced. If the mine is worked on several horizons by rooms and pillars of safety method in the salt floor the bending phenomenon may occur, which entrains the manifestation of tensile stress having the effect to setting up the discontinuities in salt mass and in the some cases even in the surround rocks. Unexpected occurrence of these phenomena can be solved only by the improvement of mechanical stability.

Establishing of natural stress – deformation state is related to making evidence the limits of two areas under a triaxial context, taking into account the anisotropy determinative (of strength and deformation) and of course the in situ rheological behaviour of salt. Researches made, should be based primarily on the results of laboratory and in situ experiments, but we should not forget that the octahedral stress finding that occurs in salt determines generally the structural stability of excavations in a salt mine. Depending on the octahedral concept, deformation properties as well as the characteristics and rheological parameters of salt, based on triaxial cubic experiments, analytically, from the viewpoint of the value of natural stress state, the salt massif can be characterized by three areas: stable area, transition area and the unstable area.

Throughout the exploitation as well as after its using up or the passage to other horizons of exploitation will follow the evolution in time of the resistance structures - the behaviour of the room - pillar – floor system to avoid the occurrence of any events that could have negative effects on the stability of the underground structures.

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