

STABILITY STUDY FOR A LARGE CAVERN IN SALT ROCK FROM SLANIC PRAHOVA*

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The design of a large detection infrastructure for measurements of very rare events like proton decay, or neutrino astrophysics, is in preparation and has a strong support from EU by the FP7 project LAGUNA (Design of a pan-European Infrastructure for Large Apparatus studying Grand Unification and Neutrino Astrophysics). The project has the main goal to establish the best location for a huge underground detector. The location will be selected from 7 underground laboratories from Great Britain, France, Spain, Finland, Italy, Poland and Romania. The LAGUNA site will be chosen taking into account different criteria, *e.g.* the depth of the site *i.e.* the ability to absorb and shield against high energy muons, the available space and possibility to install a large volume detector inside (larger than $70\,000\text{ m}^3$), and the natural radiation background. The site proposed in Romania is located in the salt mine Slanic Prahova. Three detector types are investigated based on different active detection media: MEMPHYS-water, LENA-liquid scintillator and GLACIER-liquid argon.

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1. Introduction

LAGUNA [1] (Design of a pan-European Infrastructure for Large Apparatus studying Grand Unification and Neutrino Astrophysics) is the research project, supported by the European Union to setup the infrastructure for

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a large underground laboratory with a first step to explore adequate locations for looking for extremely rare events like proton decay in order to investigate the Grand Unification Theory, or for studies of low energy neutrinos of astrophysical origin.

The present situation in the neutrino physics and proton decay research is a challenging one, since, for example by improving the experimental data and the theoretical calculations the future neutrino-less double beta decay experiments may be able to check the actual limits of the neutrino masses. On the other side, it would be meaningful to know the real mechanism by which the neutrino-less double beta decay occurs, because, in this way, one could verify some of the fundamental hypotheses advanced by the GUT theories: non-conservation of the lepton number, existence of the SUSY particles, existence of the right handed components of the weak currents *etc.*

The study of neutrino properties is one of the hottest physics research fields since, on one side, fundamental properties of neutrinos, as its absolute mass and its nature (is it a Dirac or a Majorana particle?) are still unknown and, on the other side, because the neutrinos are key particles in many physical processes from nuclear and elementary particle physics, astrophysics and cosmology. That is why the detailed knowledge of the neutrino properties would give us essential information about the evolution of the universe and of the astrophysical objects as stars, planets, Supernovae, galaxies *etc.* There are three types of experiments (with the corresponding theoretical calculations) for the determination of the neutrino properties:

- direct measurements of the neutrino mass: tritium beta decay and double beta decay (from the neutrino-less double beta decay experiments the only claim for observing the signal gave the neutrino mass value of 0.39 eV [2] (result not yet confirmed). This value also shows the present experimental and technological limits for the measurement of the neutrino-less double beta decay half-lives);
- neutrino oscillation experiments: these brought compelling evidence in favor of massive neutrinos [3, 4];
- cosmological data: from which one can derive lower limits for the sum of the neutrino masses; presently this value is around 1 eV [5].

Seven underground laboratories of Great Britain, France, Spain, Finland, Italy, Poland and Romania are involved in the LAGUNA project. Three detector types are considered based on different active detection media: MEMPHYS with water [6], LENA, a liquid scintillator detector [7] and GLACIER using liquid argon [8]. The final location for LAGUNA will be established

taking into account various criteria. The possibility to excavate large stable caverns to install large volume detectors inside (GLACIER — $77\,000\text{ m}^3$, LENA — $71\,000\text{ m}^3$, MEMPHYS — $600\,000\text{ m}^3$) has to be demonstrated.

The Romanian site is located in Slanic Prahova. Its salt rock is a lens 500 m thick and a few kilometers long and wide. Based on the muon flux measurements, the water equivalent depth of Unirea mine in Slanic underground site was determined. Based on that, we can say that the Slanic Unirea mine is a feasible location for GLACIER detector, considering the determined depth of 600 m.w.e. (meter water equivalent), and the huge cavity already excavated (about $3\,000\,000\text{ m}^3$). In parallel, another possible solution for LAGUNA at Slanic site has also been investigated.

It appeared that a new cavern 100 m bellow the Cantacuzino mine could be excavated in a reasonable time. In this case a depth of about 1000 m.w.e. could be achieved for the experiment. In this work we report a study of the stability of a new cavern at about 100 m bellow the Unirea mine in Slanic.

2. Geomechanical characterization of the rocks from Slanic Prahova salt mine

2.1. Geomechanical characterization of rock salt

The geomechanical characteristics of salt samples, taken from the layers between 200 m and 300 m below the surface in the Slanic Prahova salt mine, have been determined according to accepted standards. The measurements have been performed in the Geomechanical Laboratory of the University of Petrosani [9, 10] (see Figs. 1 and 2).



Fig. 1. Samples of salt for testing.



Fig. 2. Device for determining the mechanical properties of salt.

In the Geomechanical Laboratory, the salt physical, mechanical and rheological properties have been determined for all samples [11, 12]. The average values obtained for the samples collected at the deepest layer are presented in Table I. The important physical properties for the stability analysis of the underground excavations are: specific weight (γ), bulk density (γ_a) and porosity (n), while the relevant mechanical properties to be taken into account are: uniaxial compressive strength σ_{rc} , tensile strength σ_{rt} , shear strength, cohesion C and internal friction angle ϕ [13]. Also, the rheological properties of salt, (long term compressive strength, σ_{lldc} and long term traction strength, σ_{lldt}) have been determined.

TABLE I

Geomechanical parameters of rock salt.

Parameters	Unit	Average value
Bulk density, γ_a	N/m ³	2.125×10^4
Specific weight, γ	N/m ³	2.200×10^4
Porosity	%	3.319
Uniaxial compressive strength, σ_{rc}	MPa	28.876
Tensile strength, σ_{rt}	MPa	2.689
Cohesion, C	MPa	5.2
Internal friction angle, ϕ	[°]	30
Young's modulus, E	MPa	3000
Poisson ratio, μ	—	0.250
Long term compressive strength, σ_{lldc}	MPa	19.8
Long term traction strength, σ_{lldt}	MPa	11.46

2.2. Geomechanical characteristics of rock masses

Along with direct measurement of parameters by laboratory tests of the salt samples, the analytical methods are used to estimate the rock mass behavior around excavations in Slanic Prahova salt mine. The Hoek–Brown failure criterion is widely accepted and has been applied. Hoek and Brown [14,15] introduced their failure criterion in an attempt to provide input data for the analyses required for the design of underground excavations in hard rock. The Hoek–Brown failure criterion introduces sometimes uncertainties and inaccuracies in the obtained results; for this reason numerical models and limit equilibrium programs are complementary used. The criterion uses the properties of intact rock as a starting point and then introduces factors to reduce these properties on the basis of the characteristics of joints in the rock mass.

It became necessary to re-examine the relationships and to introduce new elements to account for the wide range of practical problems to which the criterion was being applied. Typical examples of improvements are the introduction of the idea of “undisturbed” and “disturbed” rock masses by Hoek and Brown [14], and the introduction of a modified criterion to force the rock mass tensile strength to zero for very poor quality rock masses. One of the major obstacles, which are encountered in the field of the numerical modeling for rock mechanics, is the problem of input data for rock mass properties. The RockLab program [16] provides a simple and intuitive implementation of the Hoek–Brown failure criterion, allowing users to easily obtain reliable estimates of rock mass properties and to visualize the effects of changing rock mass parameters, on the failure envelopes. In Fig. 3 the results of simulation are presented for two sets of parameter values, see Table II, case 1 and 2.

The Hoek–Brown failure criterion was modified by the authors [17], who proposed a relationship for principal stresses characterizing the intact rock (Eq. (1)), which is more suitable than the relation between shear and normal stresses

$$\sigma_1 = \sigma_3 + \sigma_{rc} \left(m \frac{\sigma_3}{\sigma_{rc}} + S \right)^a. \quad (1)$$

In this equation σ_1 and σ_3 are the major and minor effective principal stresses at failure, σ_{rc} is the uniaxial compressive strength of the intact rock material, and m and S are material constants, where $S = 1$ for intact rock.

In the generalized Hoek–Brown criterion the constant m was substituted with m_b , which is a reduced value of the material constant m_i . The Geological Strength Index GSI and factor D depend upon the degree of disturbance of rock masses.

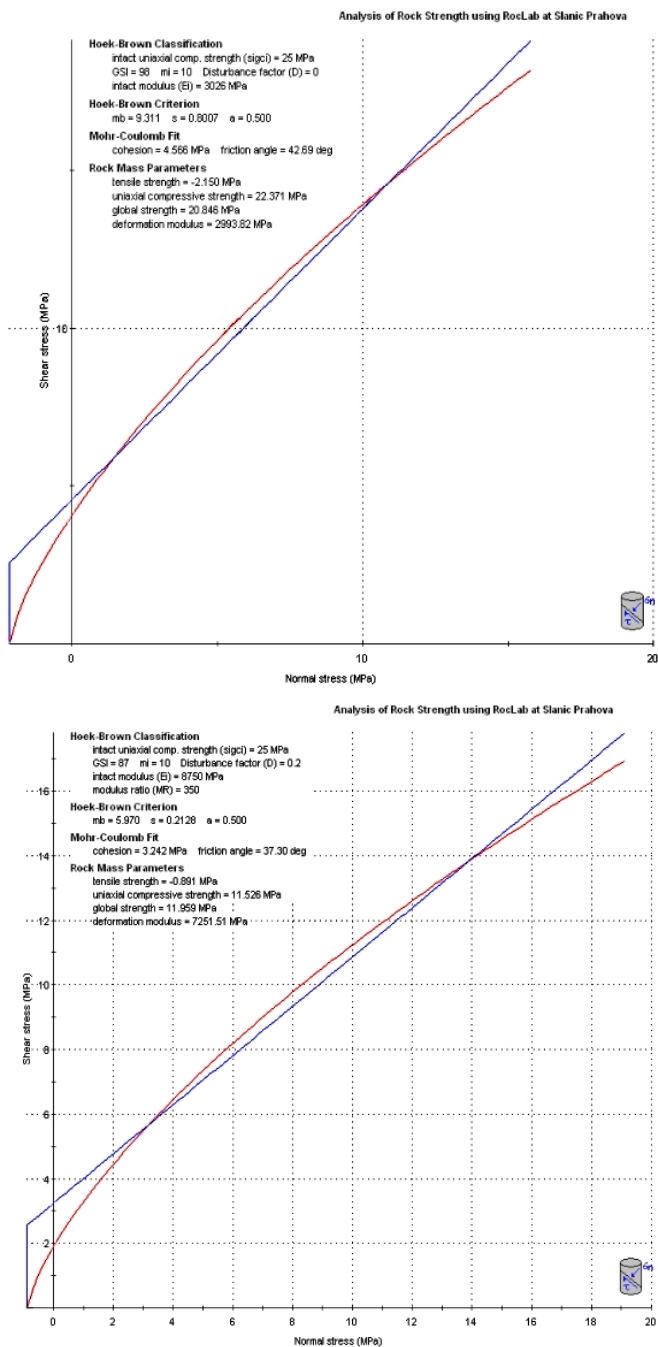


Fig.3. Relationship between normal and share stress for GSI = 98 (top) and GSI = 87 (bottom).

TABLE II

Rock mass parameters.

Parameters for Hoek–Brown classification	Case 1	Case 2
Compressive strength of intact rock σ_{ci} [MPa]	25	25
Geological Strength Index, GSI	98	87
Disturbance factor, D	0	0.2
Intact modulus E_i [MPa]	3026	8750
Hoek–Brown constant, m_i	10	10
Hoek–Brown criterion		
Constant a	0.50003	0.500292
Constant m_b	9.31063	5.96979
Hoek–Brown constant, s	0.800737	0.212754
Mohr–Coulomb fit		
Cohesion, C [MPa]	4.5657	3.2418
Internal friction angle, ϕ (degree)	42.6888	37.3002
Rock mass parameters		
Tensile strength σ_t [MPa]	−2.15006	−0.89096
Uniaxial compressive strength σ_c [MPa]	22.3708	11.5261
Global rock mass compressive strength σ_{cm} [MPa]	20.8458	11.959
Deformation modulus E_{rm} [MPa]	2993.82	7251.51

Engineering practice enables us to choose, for the salt massive in Slanic Prahova, GSI in the range (98–87) that is intact or massive structure, with few areas of discontinuity. Simulations were made for the value of GSI = 98, *i.e.* salt block almost intact, well structured by internal links, consisting of cubic blocks formed along three axes intersected by discontinuities and for GSI = 87, the worst possible case.

For the salt rock masses from the Slanic salt mine, knowing the geomorphological characteristics and the literature recommendations, the structural index m_i has been chosen as (12 ± 2) . In the Hoek and Brown failure criterion the m value reflects the frictional characteristics of the component minerals and grains of the intact rock.

The differences between disturbed and undisturbed rock mass are known from engineering experience and practice. Hoek and Brown [14] estimate them by a disturbance factor D , which could range from 0 to 0.5. The simulations have been performed for $D = 0$ and for $D = 0.2$. The results are close to those given in Table II. For $D = 0$ we got a deformation module almost equal to that obtained from the laboratory tests and, in the case of $D = 0.2$, we estimated the Young's modulus using modulus ratio $MR = 350$ for rock salt.

Since most geotechnical software is still written in terms of the Mohr–Coulomb failure criterion, it is necessary to determine equivalent angles of friction and cohesive strengths for each rock mass and stress range. For each rock sample and for each set of strength values, the friction angle ϕ and cohesion strength C can be determined using RockLab (Table II), by fitting the average values of the minor principal strength, given by the solutions of equation (2).

Note that the value of $\sigma_{\max,rc}$, the upper limit of confining stress over which the relationship between the Hoek–Brown and the Mohr–Coulomb criteria is considered, has to be determined for each individual case. The Mohr–Coulomb shear strength τ , for a given normal stress σ , is found by introducing these values of C and ϕ into equation (2):

$$\tau = C + \sigma \operatorname{tg} \phi. \quad (2)$$

The uniaxial compressive strength of the rock mass σ_c is given by equation (3) and the tensile strength σ_t is given by equation (4)

$$\sigma_c = \sigma_{rc} S^a, \quad (3)$$

$$\sigma_t = -\frac{S \sigma_{rc}}{m_b}. \quad (4)$$

Failure initiates at the boundary of an excavation when σ_c is exceeded by the stress induced on that boundary. The failure propagates from this initiation point into a biaxial stress field and it eventually stabilizes when the local strength, defined by equation (1), is higher than the induced stresses σ_1 and σ_3 . Most numerical models can follow this process of fracture propagation and this level of detailed analysis is very important when considering the stability of excavations in rock and when designing support systems [17,18].

Based on these considerations, we estimated the rock mass properties of rock salt from Slanic Prahova and implemented them in the simulation program RockLab.

3. Analysis by FEM of stress and strains developed around cavern

3.1. General consideration

An acceptable design is achieved when numerical models indicate that the extent of failure has been controlled by installed support, that the support is not overstressed and that the displacements in the rock mass stabilize. Monitoring of the displacements is essential for confirming the design predictions.

Basic equations such as differential equation of equilibrium, relations defining environmental continuity (deformations arising from the displacement field), the law of behavior *etc.* are solved by means of numerical

approximations applied to every single element in a set of finite elements. This method is called Finite Element Method (FEM). Depending on the choice of the approximation made for each element, related to the available computer resources, one can obtain the results with desired accuracy. As the relations that describe environmental continuity and behavior law are directly involved in the formulation, it is possible to introduce large deformations in the computer programs (preserving higher order derivatives) and behaviors more complex than linear elasticity. Integral methods are aimed at determining the full field displacement and environmental state of tension, deducting them from the knowledge of forces distributed over a surface of studied domain. This surface can be an internal boundary, as in the boundary element method.

Forces distributed across this boundary are adjusted so that in each point the vectors known as stress vectors can be found after the integration of forces over the border.

Integration is realized using the border elements containing mesh nodal points. Thus, the problem is reduced, due to numerical approximations, to the knowledge of values of applied forces in nodal points. This approximation is one of the key points of the method. The second point is to use classical relations for determining stresses and displacements in an infinite or semi-infinite medium, isotropic and elastic, and subject to point forces (Kelvin's solution) or dispersed forces.

Several computer codes for analyzing complex structures by applying FEM have been developed. The CESAR-LCPC code (2D and 3D version 4 [19, 20]) is a general computer code based on FEM, which addresses the following areas: structures, soil and rock mechanics, heat transfer, hydrogeology.

3.2. The 3D model

The modeling, using the CESAR-LCPC 3D software, concerns the excavation located at an average depth of $H = 300$ m as measured from the surface. The excavation consists of a lower part in a form of vertical cylinder of diameter $d = 74$ m and of height $h_1 = 25$ m and of an upper part in a form of spherical vault of height $h = 20$ m and radius R of about 44 m (see Fig. 4).

Making 3D modeling required the following stages [21]:

- (i) establishing the geometry of the model (defining geometry, setting the limits and the model discretization);
- (ii) establishing calculation bases (determining areas — regions, type of the stress analysis, formulation of stress state, geomechanical characteristics and material properties);
- (iii) boundary conditions;

- (iv) loading sources (setting the initial conditions and restriction conditions);
- (v) solving the problem and analyzing solutions (performing calculations and storing results).

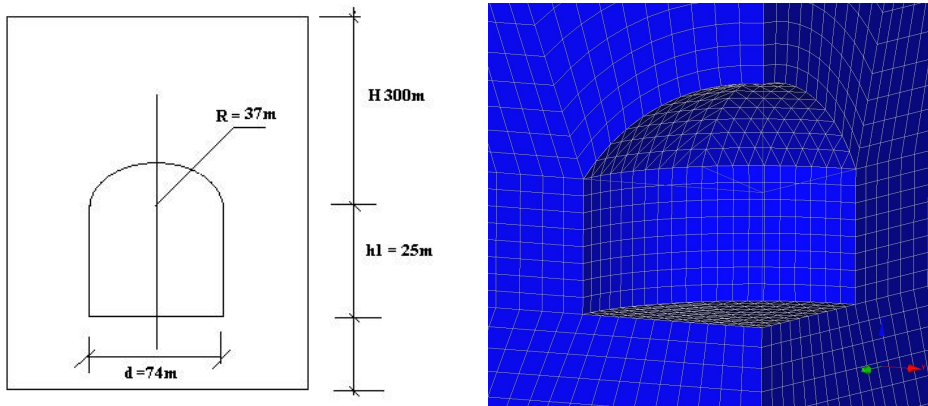


Fig. 4. Geometry model and the detail of excavation.

For the second stage, to simplify the model, two geomechanical regions with different characteristics were taken into account, assuming elastic–plastic behavior of Mohr–Coulomb type without hardening, defined as in Table I.

Boundary conditions of stage (iii) were defined on the walls of a cuboid with dimensions $(300 \times 300 \times 620) \text{ m}^3$. This is an approximation assuming that the deformations in the Earth crust outside this cuboid can be neglected. We denote by u , v and w , respectively, the displacements parallel to the horizontal (x , y) and the vertical (z) axes. The boundary conditions are $u \neq 0, v \neq 0$ and $w = 0$ at the lower surface of the model, $u = 0, v \neq 0, w \neq 0$ on the vertical surfaces orthogonal to the x axis and $u \neq 0, v = 0, w \neq 0$ on the other two vertical surfaces.

At stage (iv) the initial conditions of the model are loaded. The components of geostatics stress σ_0 are:

$$\sigma_{0z} = \rho g H, \quad (5)$$

$$\sigma_{0x} = \sigma_{0y} = \frac{\mu}{1 - \mu} \sigma_{0z}, \quad (6)$$

where the depth $H = 300 \text{ m}$ and

$$\frac{\mu}{1 - \mu} = K_{0x} = K_{0y} = 0.33. \quad (7)$$

Stresses induced by the presence of the excavation are σ_e , represented by horizontal stress $\sigma_{ex} = \sigma_{ey}$ and vertical stress σ_{ez} . In the model, the total stress is given by the matrix equation:

$$\sigma_{\text{Total}} = \sigma_0 + \sigma_e. \quad (8)$$

The calculations (stage (v)) were made considering 60 iterations per increment and a tolerance of 1%, using “the method of initial stress”. Results of processing have been stored in graphical form on the model surface (vectors and tensors) and in pre-defined sections. An example is given in Fig. 5.

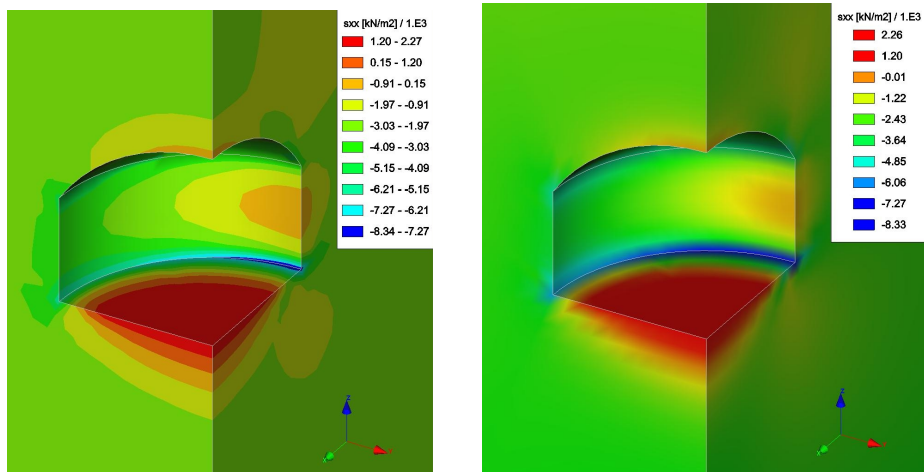


Fig. 5. Horizontal stress σ_{xx} .

Comparing the values of share strength and compressive strength with the corresponding values of stresses, one sees that these are not exceeded. Maximum vertical displacement at floor level is +115 mm, which produces “floor swelling”, and at vault is −78 mm, which produces “convergence”. The walls have horizontal displacements up to 12.7 mm.

4. Excavation stability analysis

The assessment of stability of the salt masses, in which an underground excavation is performed, is the subject of stability analysis. In this case, ratios of stress concentrations, in general, and the major and minor principal stresses σ_1/σ_3 , in particular, describe the disequilibrium of stress and thus the possibility of failure occurrence. Also, to asses stability, the study of tensile and shear stresses is important. The rock salt has comparatively low traction and shear resistances and often break occurs because certain known

limits are exceeded. The classical approach used in designing engineering structures is to consider the relationship between the capacity C (strength or resisting force) of the element and the demand D (stress or disturbing force). The factor of safety of the structure is defined as $F = C/D$ and failure is assumed to occur when $F < 1$.

If the tensile strength and compressive strength values are compared with the corresponding values of stresses, one sees that these are not exceeded, but are close ($\sigma_{rt} = 1.17\sigma_t$; $\sigma_{rc} = 1.48\sigma_c$). There is a risk that, locally, in some parts of the cavern, where the salt rock is less resistant or broken, cracks due to tensile stress could occur

$$\sigma_t = +2.29 < \sigma_{rt} = +2.689, \quad (9)$$

$$\sigma_c = -19.500 < \sigma_{rc} = -28.876. \quad (10)$$

The safety factors for a number of significant points on the outline of excavation for a set of significant values on the excavation outline were calculated. Three of them are shown in Table III. The results are presented in Cartesian coordinates in the x - z plane.

TABLE III

Safety factors of excavation.

Coordinates of point ($y = 0$)	σ_1 [MPa]	σ_3 [MPa]	Safety factor F
Roof vault: $x = 0, z = 20$	-0.2427	-0.485	1.17
Wall: $x = -37, z = 0$	-2.608	-14.819	1.48
Floor: $x = 0, z = -25$	2.285	0.291	1.17

Horizontal deformations are between 0.0000343% and 0.000254% while vertical deformations are between 0.0021% and 0.000254%. Floor swelling occurs under the effect of tensile stress that does not exceed 2 285 MPa. Excavation walls are subjected to compression stresses which increase from the vault (-14 819 MPa) to the floor (-19 308 MPa). The compression strength value of salt rock is 28 876 MPa, with a safety factor from 1.48 for the wall to 1.17 for the floor and vault. In the walls major shear stresses develop with the highest values near the corners of the vault and floor. They range from 7 130 MPa to 7 930 MPa, which is less than the breaking strength of shear stress equal 10 500 MPa.

Note that the maximum compressive stress (-19 308 MPa) is lower than the long term strength limit, which is 19 800 MPa. Also maximum tensile stress of 7 930 MPa is lower than the long-term traction strength limit of 11 460 MPa.

5. Conclusions

An analysis of the geomechanical characteristics of salt from Slanic shows that an underground cavern could be excavated at a depth of 300 m below surface.

Comparing the values of tensile strength and compression strength with the corresponding values of stresses one sees that the cavern would be stable.

Maximum vertical displacement is at the floor level (+115 mm), which produces “floor swelling”, and at the vault (−78 mm) corresponding to “convergence”. The walls have horizontal displacements up to 12.7 mm.

An analysis of safety factor values at nodes on the excavation contour indicates that the highest values are to the center of the floor and towards the center of the vault. This explains vertical displacements along the excavation axis and the relaxation of stresses due to vault geometry. The floor is affected by traction stresses. In this area the safety factor value is reduced about three times.

Dangerous crack stress is 7.5% which corresponds to a deformation rate of 4.1×10^{-4} %/day and the calculated deformation rate is 1.397×10^{-7} %/day.

For all the cases examined, the safety factor value is bigger than one, which confirms that the designed excavation in the conditions considered, is stable for a period of more than 50 years.

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REFERENCES

- [1] <http://laguna.ethz.ch>
- [2] H.V. Klapdor-Kleingrothaus *et al.*, *Phys. Rev.* **D63**, 073005 (2001) [[hep-ph/0003219](#)]; *Mod. Phys. Lett.* **A16**, 2409 (2001).
- [3] S. Fukuda *et al.*, *Phys. Lett.* **B539**, 179 (2002).
- [4] Q.R. Ahmad *et al.*, *Phys. Rev. Lett.* **89**, 011301 (2002).
- [5] O. Elgaroy *et al.*, *Phys. Rev. Lett.* **89**, 061301 (2002).
- [6] A. Tonazzo *et al.*, *Nucl. Phys. Proc. Suppl.* **B168**, 363 (2007).
- [7] T. Marrodan Undagoitia *et al.*, *Prog. Part. Nucl. Phys.* **57**, 283 (2006).
- [8] A. Rubbia, [arXiv:hep-ph/0402110](#).
- [9] S. Arad, V. Arad, J. Veres, O. Stoicuta, Safety Excavation in Salt Rock Used for Underground Storage in Romania, Proceedings of 25th International Symposium on Automation and Robotics in Construction-ISARC-2008, pp. 265–271, ISBN 978-9955-28-304-1, IDS Number BID71, Vilnius Gediminas Technical Univ Press, 2008.

- [10] V. Arad, S. Arad, P. Chelaru, I. Ilie, D. Ciocan, D. Cotes, Rock Mechanics: Meeting Society's Challenges and Demands, Proc. and Monographs in Engineering, Water and Earth Sciences, Vancouver Canada, 2008, Eberhardt, Steed and Morrison Eds., published by Taylor and Francis Group, London/Balkema, Vol. 2, p. 1489, ISBN 978-0-415-44401-9.
- [11] V. Arad, Mechanics of Rock Salt, Focus Press, Petrosani 2008, in Romanian.
- [12] V. Arad, Mining Geotechnics Tehnical Press, Bucuresti 1995, in Romanian.
- [13] I. Bud, Stability of Underground Excavations, North University of Baia Mare, 2000, in Romanian.
- [14] E. Hoek, C. Carranzas-Torres, B. Corkum, Hoek–Brown Failure Criterion, Proc. North American Rock Mechanics Society meeting in Toronto, July 2002.
- [15] E. Hoek, E.T. Brown, The Hoek–Brown Failure Criterion — a 1988 Update, Proc. of 15th Canadian Rock Mech. Symp., Ed. J.C. Curran, Toronto, Dept. Civil Engineering, University of Toronto, 1988, p. 31.
- [16] Software Rocklab, www.rockscience.com
- [17] S. Arad, V. Arad, Gh. Chindris, Environmental Geotechnics, Ed. Polidava, Deva 2000, in Romanian.
- [18] N. Letu, C. Semen, Supporting Efficient Underground Excavations, Infomin Press Deva, 2005, in Romanian.
- [19] Lin Jifang, Modeling Anisotropic Media, CESAR-LCPC software application to study the stability of the slate mines of Angers–France Report DEA, Mechanics of Soil Laboratory, Ecole Supérieure de Mine de Nancy, Franta, 1990, in Romanian.
- [20] Software CESAR-LCPC 2D and 3D, v4.
- [21] I. Onica, Introduction to Numerical Methods Used in Mining Excavations Stability Analysis, Universitas Press, Petrosani 2001, in Romanian.