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Technical Conference Paper

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Mechanical Behaviour of a Marly Floor in Two Salt Mines with Abundant Brine or Water

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INTRODUCTION

Salt, Brine and Marls — A Preliminary Comment

Bedded salt formations often remain approximately horizontal after they are deposited. For this paper, extraction occurs in a layer that is several meters thick (dry mines) or several dozens of meters thick (solution-mined caverns). The mine or cavern is located in a series of strata that includes less soluble layers. These layers exhibit diverse mechanical properties, ranging from stiff and brittle anhydrite layers to the weak and alterable marls or argillite layers. Such configurations are found, for instance, in the Permian Wellington formation in Kansas and in the Upper Silurian Salina Group in New York (United States) or in the Lower Keuper formation in Lorraine (eastern France).

Argillite or marly layers often play a significant mechanical role. On one hand, these layers, or the interfaces between these layers and salt layers, are weakness planes. Consider a large volume of rock including both salt layers and thin marly layers. When submitted to compressive stresses perpendicular to the bedding plane, the interbedded rock mass exhibits the same mechanical properties (stiffness and strength) as a homogeneous rock-salt mass. This is not true when the interbedded rock mass is submitted to tensile stresses perpendicular to the bedding plane or to shear stresses parallel to the bedding plane: the debonding of layers may take place, and both strength and stiffness may be reduced considerably. When thin salt layers at the roof or on the floor of a mine gallery (or salt cavern) are submitted to high compressive horizontal stresses, buckling can take place, as often is observed when a thin salt slab is left on the mine floor.

On the other hand, marly or clayey layers are known to weather considerably if they are in contact with water or brine. Such weathering takes place when a cavern roof reaches the claystone or marly layers overlying a salt formation. The cavity migrates upward because of the collapse of roof layers. Such a migration, or stopping, is observed in the Hengelo Brine Field in the Netherlands. Bekendam et al. (2000) lists the various physico-chemical effects that, together with stress development in the roof, contribute to claystone collapse. Similar phenomena, including slow roof rise through non-halitic strata, are described by Buffet (1998) and Jeanneau (2005) for Lorraine caverns. Boidin (2007) performed laboratory tests to assess the behaviour of roof rocks in various chemical environments.

It is often difficult to assess the relative significance of these two factors. As a general rule, purely mechanical effects are especially important when cavern span or mine extent (and extraction ratio) are large. Conversely, the chemico-physical effects (marls or argillite weathering when in contact with water or brine) depend on various factors (such as whether the brine is saturated, a condition that is not yet understood fully).

In the following, two mines are described in which these two effects played significant roles.

Dissimilar Fates of Two Salt Mines

In the 19th Century, two room-and-pillar mines, separated by a few dozen kilometres, were created in the same layer of the Lower Keuper salt formation in Lorraine (East of France).

The Saint-Maximilien panel of the Varangéville Mine was created in 1859. It suddenly collapsed in 1873, when pillars punched the marly floor of the mine. In hindsight, it was recognized that this marly floor had been weathered considerably by water and brine, which were abundant in the mine, as water was used to pre-cut gallery face before blasting.

The Dieuze Mine was created in 1826. It was flooded with saturated brine in 1864, when a shallow gallery connected to a mine shaft collapsed, allowing saturated brine to rush down to the mine. The mine was converted into a "brine-filled" — although not "solution-mined"! — salt cavern. No mine collapse or even significant surface subsidence was observed: pillars did not punch the floor, despite the marls and brine being in contact for more than a century.

This sharp contrast between the behaviours of two otherwise similar mines raised a puzzling problem. Because ground-level stability above these two mines must be assessed, GEODERIS, a state agency in charge of post-mining issues in France, performed an extensive research program to gain better insight into the mechanical behaviour of the two mines. It was decided to drill a cored borehole 10 m below the Dieuze Mine floor and to perform a sonar survey of the mine (Feuga, 2002); the latter proved that the mine was intact. Numerical computations were performed to assess the stability of the two mines.

It was proved that two factors played significant roles.

Chemical versus Mechanical Effects

Generally speaking, marls are extremely sensitive to the weathering effect of water or brine. However it is suspected that, in the case of the floors in Lorraine mines, the deeper "chocolate brown" marls, which rapidly weather when in contact with saturated brine, are protected by a stiffer and stronger marl crust that composes the upper part of the mine floor; in this crust, salt and anhydrite contents are high. However, this crust, because of its salt content, also weathers when in contact with soft water. Soft water was abundant in the Saint-Maximilien panel, and numerous slots, dug out at the mine floor, gave direct access to the chocolate brown marls. It is likely that, in the Saint-Maximilien panel, the crust did not protect the underlying layers effectively, as it probably did in the Dieuze Mine, which had been flooded by saturated brine.

Mine extension and mechanical loading also are instrumental. The Saint-Maximilien panel, which collapsed suddenly, was deeper than the Dieuze Mine, and its horizontal dimensions were larger, allowing a larger degree of roof sagging. After flooding, mine brine pressure bore a significant part of the overburden weight, resulting in small deviatoric stresses in the pillars. Numerical computations proved that, by 1873, a large dilatant zone developed above the edge of the Saint-Maximilien panel. No such zone was found in the case of the Dieuze Mine, even when the marly floor was assumed to be weathered — as it definitely was at Saint-Maximilien.

At Saint-Maximilien, both marls weathering and mine extent were necessary conditions for the mine to collapse. At Dieuze, shallower depth and smaller mine extent made the mine perennially stable.

1. THE 1873 COLLAPSE OF THE SAINT-MAXIMILIEN PANEL AT THE VARANGEVILLE SALT MINE

On October 31, 1873, the Saint-Maximilien panel of the Varangéville salt mine suddenly collapsed. The resulting quake was felt in the town of Nancy, ten kilometers from the mine. Prompted by loud cracks and a fissure that developed in a building at ground level, the panel had been evacuated in the morning, and, because it was payday, there were few workers in the mine, and the number of casualties were relatively low.

1.1 Observations Made after Panel Collapse

A subsidence bowl had formed at ground level (Figure 1). Shallow vertical fractures were visible along an outer circle with a radius of 160 m and whose centre was at the head of shaft n°1. This presented a clear sign of the effects of high-tensile, horizontal stresses in this area. Along a concentric inner circle, approximately 80 m in radius, folds had formed, generated by high, compressive horizontal stresses. Inside the inner circle, the ground remained flat and horizontal; it had subsided by 3.3 m. Between the inner and the outer circles, the ground had taken a uniform inward slope. Shaft n°1 was intact, but its bottom was filled to a height of 18 m with various types of debris.

On November 1 and 2, miners and a mine inspector, M. A. Braconnier, who left detailed descriptions of the accident,* came down shaft n°2 to visit the collapsed panel. The panel was mined according to the room-and-pillar method. Roof and floor depths were 150.5 and 156 m, respectively; square pillars were 6-m x 6-m wide, and the extraction ratio was 82%, a ratio that was considered reasonable at that time. A 29×40-m pillar was left at the centre of the panel to protect shaft n°1. The panel floor was a 25-m thick layer mainly composed of marls. The upper part, 1-m to 2-m thick, is a strong and stiff layer in which clay, salt and anhydrite are intermingled; below are the much softer "chocolate brown" marls. In fact, by 1873, the floor had been weathered considerably by water, which had been used extensively to pre-cut gallery faces.

The three reports prepared by Braconnier in the days following the collapse, together with observations that were made later, strongly suggest that a rock cylinder with its axis along shaft n°1, of height 160 m and radius 80 m or so, subsided as a monolithic block by 3.3 m in a couple of seconds, experiencing no or little deformation. It punctured the panel floor, which had been weathered by seeping water. Conversely, the rock annulus delimited by this cylinder (whose radius was $\approx 80\ \mathrm{m}$) and by the cylindrical surface above the mine contour (whose radius was $\approx 160\ \mathrm{m}$) subsided unevenly and experienced significant deformation. The detail of this deformation probably is complex, as the rock mass is stratified. However, it can be described roughly as "bending" of vertical planes.

* See Braconnier (1873a,b,c).

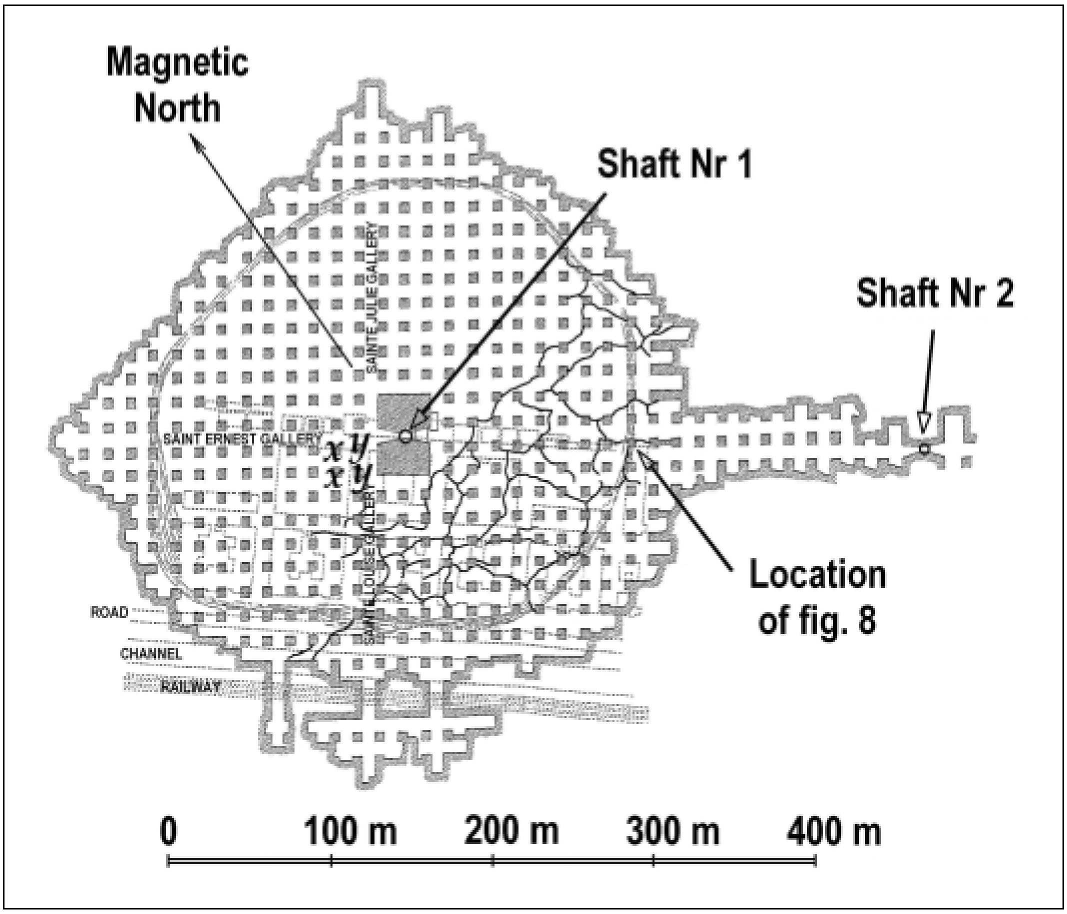


Figure 1 - Subsidence trough above the Saint-Maximilien panel as observed on November 1, 1873 (after Braconnier, 1873a,b,c.): Cracks along the outer ellipse, building locations at ground level, pillars and the irregular pattern of vertical mud-cracks at panel level also are drawn.

1.2 Collapse Scenario

This scenario is supported by several pieces of evidence (see Figure 2).

1. Inside the inner circle, the ground level remained flat.
2. Shaft n°1 was intact.
3. The same can be said of a small panel that had been mined out in 1859 in the 4th layer, 70 m above the 11th layer panel.
4. At ground level, along an inner circle whose radius was 80 m, folds were formed.
5. A couple of fissures, from some of which brine flowed, appeared near the external contour of the 4th layer panel.
6. Between this circle and an outer circle, whose radius was 160 m, the ground level gently dipped toward the shaft head.
7. Along this outer circle, shallow vertical fractures opened.
8. In the outmost galleries adjacent to the panel abutment, the roof rose by 3 m as salt blocks fell to the floor (Figure 3) under the effect of very high compressive stresses.

9. The rest of the panel roof, where it can be observed, gently dips downward to the panel shaft, and only a small number of fissures were visible (Figure 4).

10. In a small 3-m x 3-m gallery excavated several years after the 1873 collapse above the collapsed panel, salt is intact except in a 10-m long zone located 10 m above the outmost gallery: in this zone, small fractures, through which some air is exchanged with the outermost gallery, can be observed.

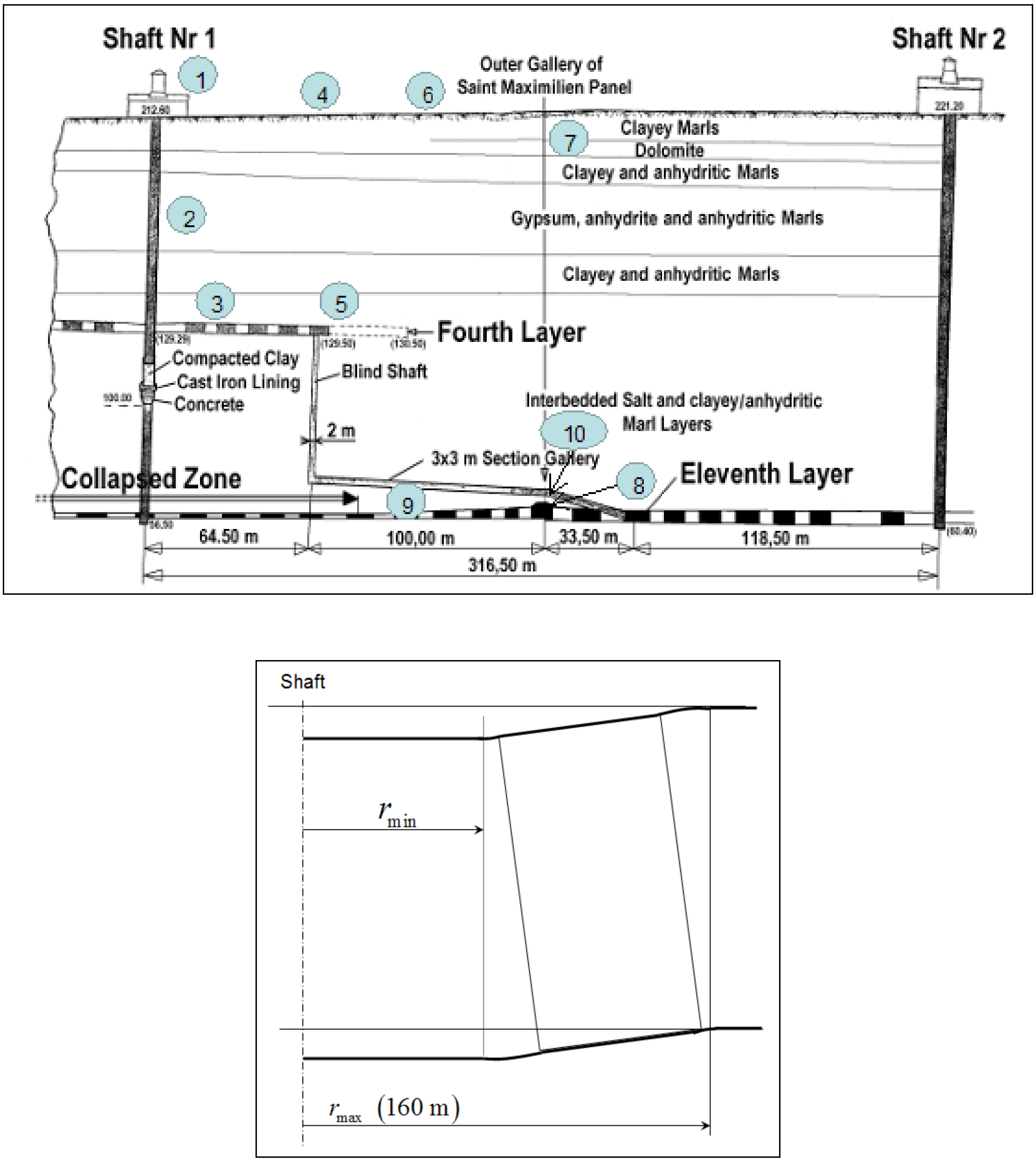


Figure 2 – Rock mass cross-section showing the 11th and 4th layer panels and the 3-m × 3-m gallery excavated after the accident and conceptual sketch of the collapse.

Figure 3 - Roof and pillar close to the external abutment in the collapsed Saint-Maximilien panel (CSME Archives).

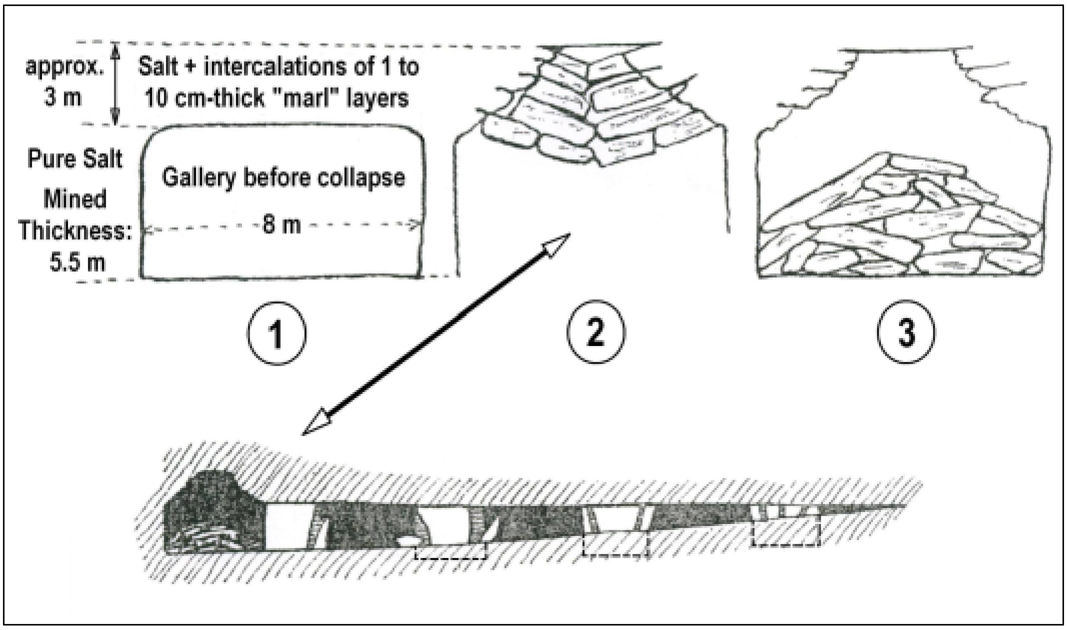


Figure 4 - Vertical cross-section of the four outermost galleries and the sequence of events in the outermost gallery (after Braconnier, 1873b).

It was noted that the salt pillars were almost intact. Minor damage was observed before the collapse. A large reduction in pillar height, which would have been expected after 14 years, was not observed and, before the mine collapse, Braconnier asserted that “Rock salt is so competent and strong, so bereft of any fissure that pillar width could be made twice smaller, with no risk of roof fall.” However, before the accident and *a fortiori* after the accident, vertical slots, 5 cm wide, could be seen in the pillars; these had developed along vertical discontinuities in the salt (“mud-cracks” shown on Figure 1) that were filled with marls and salt. After collapse, the pillars had not broken out, even at the panel edge where the roof fell (see Figures 3 and 4).

The absence of any water flow to the panel following panel collapse — a fact that remains true 130 years after the accident — also is remarkable: even small disorders in a mine roof are known to have led to flooding of dozens of salt mines worldwide (Bérest et al., 2003). Together with the absence of any visible vertical discontinuity in the ground-level or mine-roof profile after the collapse, this suggests that no large-scale open fracture was created inside the rock mass.

1.3 Sudden Collapses in Salt and Potash Mines

Several sudden collapses of salt and potash mines have been described in the literature (e.g., Minkley et al., 1996), but we know of no case involving salt pillars punching the floor, a common accident in coal mines. (However, Rothenburg et al. (2007) describe the case of a 500-m deep sylvinite panel. A halite or sylvinite slab is left at panel floor, below which a generally massive soft and creep prone tachyhydrite layer can be found. The stiff roof/pillar system pushes pillars into soft tachyhydrite and floor heaves).

Hundreds of caverns have been solution-mined in the upper part of the Lower Keuper salt formation in which the Varangéville Mine was opened. Cratering takes place when the diameter of such caverns enlarges to become slightly smaller (say, 130 m) than cavern depth (say, 200 m). Two such cases are described by Buffet (1998) and Jeanneau (2005). When leaching of these caverns was completed, the caverns had reached the top of the salt formation, stripping a marly layer that progressively weathered and fell to the cavern bottom. After several years, the immediate roofs of these caverns were a 6-m to 8-m thick, 145-m deep competent layer, the Beaumont Dolomite, whose stiffness and strength are high. This layer bore the full overburden weight, and was supported partially by cavern brine pressure. Bending occurred, and severe tensile stresses developed both at the centre and at the periphery of the Dolomite plate, ultimately leading to roof failure, cavern collapse and formation of a sinkhole. However, even when the diameter is large, depressurising the cavern often is necessary to provoke pre-emptive collapse. In the Saint-Maximilien panel case, the Beaumont Dolomite layer is shallow (12 m below ground level, or 140 m above the mine). At such depth, the dolomite often is weathered by groundwater and cannot be considered as a competent layer. Significantly, no discontinuous vertical displacement ("step") was observed at ground level above the Saint-Maximilien panel, in sharp contrast to the case of crater formation.

Comparison with neighbouring mines or panels can be more instructive. After 1873, mining methods in the Varangéville Mine changed drastically. A 20-cm thick salt slab was left at the floor, soft water was no longer used to pre-cut the gallery face, and the extraction ratio was decreased progressively from 75% at the end of the 19th Century to ultimately 52% for currently mined panels. Few or no disorders were observed in these panels, although some of them are more than a century old. At Dieuze, a few dozen kilometres from Varangéville, a salt mine had operated in the same salt formation from 1826 to 1864, when it was flooded. The behaviour of this mine is of special interest and will be discussed later.

1.4 Tributary Load versus Pillar-Bearing Capacity

The behaviour of a mine operated according to the room-and-pillar method often is interpreted through the "tributary load" theory, which results from simple equilibrium considerations. Consider a panel of circular shape with radius *R* and depth *H*. The weight of the rock cylinder above the panel is $\pi R^2 H\gamma$, where γ is the volumetric weight of the rock mass (typically, $\gamma = 2.3\cdot 10^4\,\mathrm{N/m^3} = 0.023$ MPa/m). One part of the cylinder weight is borne by the pillars: $\pi R^2 \sigma_{zz}(1-\tau)$, where τ is the extraction ratio, and σ_{zz} is the average vertical stress in the pillars. The other part is borne by vertical shear stresses, or $2\pi RH\sigma_{rz}$, where σ_{rz} is the average shear stress that applies on the cylinder lateral surface:

$$\pi R^2 \gamma H = \pi R^2 \left(1-\tau\right)\sigma_{zz} + 2\pi RH\sigma_{rz}$$

When the mine extent is very large ($R/H >> 1$), the contribution of shear stresses to equilibrium is small, and the load borne by a pillar is close to $\sigma_{zz}^{trib} = \gamma H / (1-\tau)$, which is the definition of tributary load.

When the mine extent is smaller, things are different. The roof is still stiff and able to transfer a large part of the overburden weight to the abutment. A simple model explains this notion. The roof is assumed to be a competent layer clamped on the abutment — i.e., a stiff plate whose bending stiffness is *EI*, where *E* is the elastic modulus, and *I* is the plate-bending inertia. The plate bears the weight of the overburden, or γH . When no pillar exists in the panel, the vertical displacement of the roof centre is u_0, and roof stiffness (in MPa/m) can be defined as the ratio between the weight of the overburden and the vertical displacement of the plate centre: $\gamma H / u_0 = 64 EI / R^4$. The roof is much stiffer when *R* is smaller, vertical displacements of the roof also are small, and, as some pillars are left in an actual mine, the load the roof applies on the pillars is only a part of the overburden weight, which mostly is transferred to the abutment.

The Saint-Maximilien panel is quite different. When soaked with water, floor marls have a low cohesion, C , and their friction angle is zero (for simplicity, floor viscosity is neglected). The floor-bearing capacity (i.e., the maximum load that a rigid pillar can apply on the floor without punching it) is ωC, where ω depends on pillar shape (but not pillar width). In the long term, pillars cannot bear more than $\pi R^2 (1-\tau)\omega C$. However, when the mine extent is small, the roof still is stiff: when it sags, it is not able to efficiently push on the pillars; floor punching remains limited; and equilibrium occurs such that the vertical stress on the pillars exactly equals the floor-bearing capacity. The pillars bear a fraction of the overburden weight, which is $\pi R^2 \omega C (1-\tau)$, and the average vertical shear stress that applies above the panel edge is $\sigma_{rz} = R\left[\gamma - (1-\tau)\omega C / H\right]/2$: shear stresses are proportional to the panel radius; and the dilatancy criterion is met in a larger zone above the panel edge when the panel extent increases. Large strains can localize in this zone and, ultimately, roof stiffness, which hinders floor punching by the pillar, suddenly decreases.

1.5 Mechanical Behaviour of the Floor and Roof

The Varangéville Mine operated for more than a century, and miners consistently reported that floor marls dramatically weather when in contact with water. The Varangéville marls first were studied from a geomechanical perspective by Gérard Vouille (1986), and mechanical properties were studied in the laboratory through different techniques. Test results were scattered, and defining a short-term failure criterion was not straightforward. A Tresca criterion, C = 2 MPa, ϕ = 0° for the *dry* and *shallow* (i.e., sampled in the immediate floor) material, was considered to be "conservative". Tests also proved that the behaviour of marls is viscoplastic and strongly influenced by the presence of (saturated) brine.

Water was abundant in the Saint-Maximilien panel. Pressurized water jets were used to pre-cut the gallery face before blasting, and unsaturated brine seeped through the floor. Because of the general layer dip, which is 12 mm by meter toward N-NW in the eleventh layer, slots in the floor were dug down to 2.5 m to collect water and brine. They were deeper and more frequent in the neighbourhood of the shaft. Furthermore, the water column device and its basin had been set in a room excavated in the marly floor, partly below the large pillars protecting shaft n°1. Water could penetrate deep into the floor, under the upper anhydritic marl crust, down to the much softer chocolate-brown marls, which are especially sensitive to the action of saturated brine. Starting from the central pillar, a weathered zone progressively developed in the panel floor. The cohesion of the marls in this zone was considerably lessened.

Because the salt-pillar creep rate is slow, a pillar can be considered to be a stiff body compared to the soft marly floor on which it rests. When an increasing vertical load, q, is applied on a pillar, a visco-plastic zone appears beneath the pillar. When it is large enough to isolate the pillar from the floor main body, unconfined flow becomes possible, and the pillar punches the floor. The critical ratio $\omega_c = q_c / C$ was discussed by Salençon and Matar (1982), who proved that ω_c = 6 for a cylindrical pillar. The database (of the cohesion and viscosity of marls when impregnated with brine) was small, but some evidence was available. On October 20, ten days before the panel collapse, it was observed that the floor in the Sainte-Julie gallery, which runs along one side of the central pillar (see Figure 1), had risen by 0.8 m. A similar figure can be found through numerical computations when the floor cohesion is assumed to be C = 0.75 MPa — i.e., four times less than the "dry" figure. The viscosity of the marls was more difficult to determine. Salt viscosity, when small loadings are applied (Bérest et al., 2005), typically is $\mu = 10^{17}$ Pa.s. It was decided to adopt a smaller figure, as it was known that the creep flow of marls is much faster than pillar salt creep; thus, $\mu = 10^{15}$ Pa.s was selected. Smaller values were tried, but they do not change the main results significantly, and they make numerical computations tedious. When this study had been completed, it appeared that the selected figure was not inconsistent with laboratory test results recently described by Boidin (2007).

The floor-bearing capacity is $q_c = \omega_c C \approx 4.5$ MPa when the floor cohesion is $C = 0.75$ MPa. The geostatic pressure at mine depth (156 m) is $\gamma H \approx 3.5$ MPa, and the extraction ratio is larger than $\tau = 80\%$, making the tributary load equal to $\gamma H / (1-\tau) \approx 17.5$ MPa, i.e., much larger than floor bearing capacity. When the extent of the mine progressively increases, the floor-bearing capacity is exceeded early on: witness the closure of the Sainte-Julie gallery and of the water-column device room, which, for this reason, was planned to be displaced before the collapse. However, two factors contributed to prevent generalized punching: (1) marls floor-softening was progressive, even though it is impossible to describe its evolution with respect to time and space; and (2) the panel roof remained stiff as long as the panel extent was small. Punching began early after the mine opening; it was not effectively slowed by the viscosity of the marls, which was small; however, punching stopped as soon as roof sagging was sufficient to make the load applied by the roof to the pillar close to the pillar-bearing capacity. As this capacity was small, an increasing part of the overburden load was transferred to the abutment when mine extent increased. Roof stiffness, therefore, is a key parameter for panel evolution.

The elastic modulus of Varangéville salt (as measured in the laboratory) is estimated to be $E = 26$ GPa, and the thickness of the salt overburden is η = 70 m. When this overburden is considered as an elastic plate, numerical computations suggest that, in 1873, roof sagging should have been 3 cm even when it is assumed that pillars apply no stress to the roof — a very pessimistic assumption. This figure, of course, is unrealistic: in the actual mine — including its pillars — ground-level subsidence was detected easily before the collapse, as fissures appeared in some buildings and cast iron tubes broke. The salt roof definitely cannot be the idealized elastic plate described above. In fact, Braconnier's reports (1873) describe the depth and thickness of ten marly layers separating the eleven salt layers that can be found from the salt top to the mine level. (They range from 0.5 m to 3 m in thickness.) However, at a smaller scale, many additional horizontal discontinuities can be observed. These discontinuities clearly are weakness surfaces. In those galleries close to the panel abutment, where the salt roof rose by 3 m as a consequence of the panel collapse, thin horizontal interbeds opened, and it could be observed that a dozen salt layers were separated from each other (see Figure 3). Less frequent in a bedded-salt formation, many vertical discontinuities also are present, both in the 4th and 11th layers. These vertical fractures, which are filled with mudstone ("marls"), salt and anhydrite, are organized according to a polygonal pattern, which is reported on the Saint-Maximilien panel map (see Figure 1). These vertical fractures also play a mechanical role, as many of them were opened before the collapse — and still more after the collapse.

A comprehensive geometrical and mechanical description of all these discontinuities cannot be obtained, and a simpler method was needed. To lessen roof stiffness, one can select a smaller elastic modulus (E) or decrease the plate inertia (I). For example, when a unique plate with thickness η is substituted by N elastic plates with thickness η/N, the inertia of the plate stack is divided by N^2. This second option is more satisfying, as it is consistent with the geological description of the rock mass. However, meshing a large number of thin layers leads to a prohibitive number of elements and a compromise must be found. Three 3-m thick horizontal layers were intercalated inside the salt roof, and the elastic moduli of salt and marls were reduced significantly. It cannot be claimed that this model provides a faithful picture of the actual roof, but it can be credited with reducing the bending inertia of the 70-m thick salt overburden by a factor of 20, and it results in credible values of ground subsidence and gallery height loss in the days before the collapse.

Salt is known to behave as a viscous fluid; in panels mined at the end of the 19th Century, when the extraction ratio was 75%, salt pillars of height is 4.7 m were monitored for 30 years, and the creep rate was from 2 mm to 5 mm/yr. Numerical computations take into account the viscoplastic behaviour of salt, but, in fact, its influence is small. On one hand, marls viscosity is smaller than salt viscosity by 2 orders of magnitude; in this context, salt can be portrayed as a stiff rock. On the other hand, the average vertical stress in the pillar cannot exceed the pillar-bearing capacity ($\omega_c C \approx 4.5$ MPa , a figure for which the salt creep rate would be exceedingly slow) by a large amount. However, even if at panel scale, pillars behave as stiff cubes punching the viscoplastic soft floor; at a smaller scale, the behaviour of the salt pillars is complex. When pillars punch the floor, horizontal marls flow toward the neighbouring free surfaces (room floors), resulting in large horizontal stresses at the base of the salt pillar.

1.6 Numerical Computations

The actual panel contained a 29-m x 40-m central protecting pillar and smaller 6-m x 6-m square pillars. The approximate external contour of the panel is a circle with a radius of 160 m (Figure 1). Because we were not interested in the detailed mechanical behaviour of each pillar, but, rather, in the global behaviour of the panel, we adopted a simplified axisymmetric description of the panel. One important constraint was to respect the overall extraction ratio (i.e., $\tau = 82\%$). In the model, the hundreds of actual square pillars were replaced by 8 toric "pillars" separating 9 toric "rooms", and the rectangular protecting pillar was replaced by a circular pillar with a radius of 21.37 m. Galleries were mined progressively from 1859 to 1873 in such a way that yearly salt production is constant. (This description of an "equivalent" axisymmetric mine and of the opening chronology of the galleries are due to Gérard Vouille, to whom the authors are indebted.) The panel floor is a 25-m thick marly layer with assumed homogenous properties. The mine roof is described as a succession of salt and marls layers. Several values successively were given to the most important mechanical parameters: marls cohesion and roof stiffness. As mentioned above, a marl cohesion of C = 0.75 MPa and a "soft roof" assumption provided the best fit against field observations. More details on these numerical computations can be found in Bérest et al. (2008).

A typical result is given on Figure 5. Displacements in the vicinity of a gallery adjacent to the central pillar are displayed. Displacements are computed from panel creation in 1859 to the eve of collapse in 1873. Radii are from the shaft axis. At the centre of the first gallery, the roof sagged by 40 cm, and the floor heaved by 50 cm. The floor heave is fed by large displacements of the marls beneath the central pillar, which are pushed toward free surfaces (room floor) when punching takes place. Ground-level subsidence (32 cm) and gallery-height loss (40 + 50 = 90 cm) are consistent with field observations. Floor heave is smaller in the other galleries, and large compressive stresses develop in the outermost gallery, as anticipated, because severe roof failures were observed in this gallery.

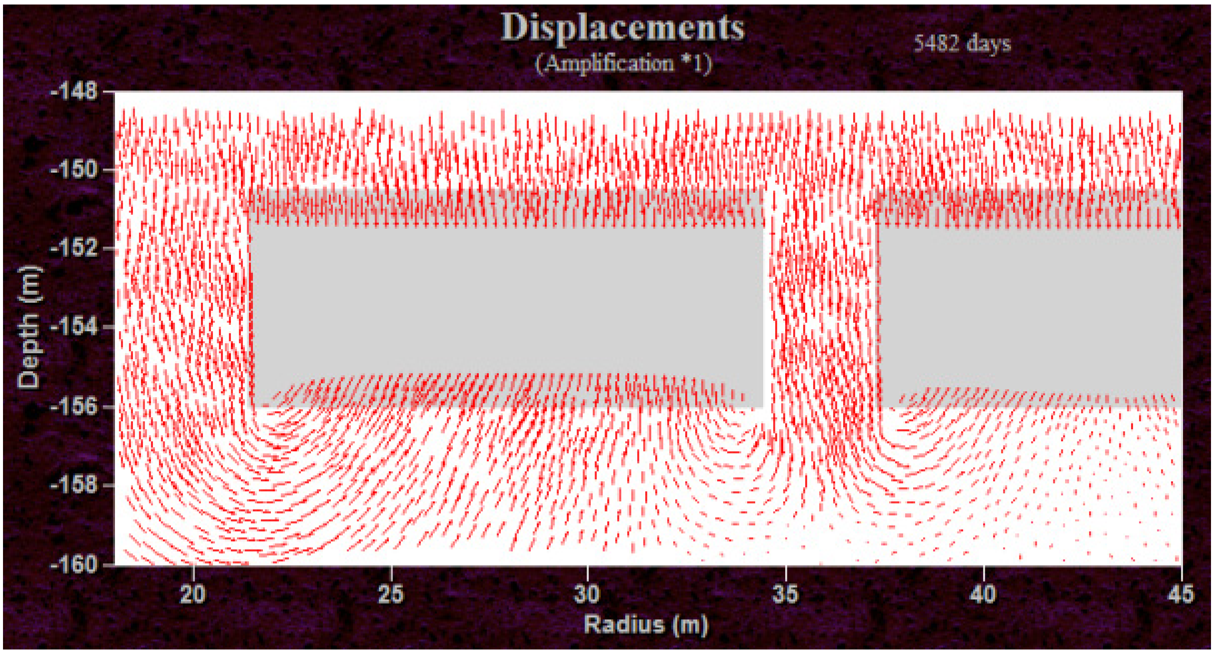


Figure 5 - Displacement field in the first and second galleries after 15 years.

1.7 Panel Collapse

Why the mine suddenly collapsed still remains to be explained. Sudden pillar failure is consistent neither with the mechanical model, which predicts that the load on the pillars is small, nor with field observations: the pillars experienced large vertical fracturing but did not burst see Figure 4). Sudden softening of the floor is consistent neither with the weathering process, which is progressive, nor with the mechanical model, which predicts that punching remains moderate as long as the roof remains stiff. Sudden collapse must originate in a rapid loss of roof stiffness. One good candidate seems to be the 3-m high roof failure in the outermost gallery (Figure 3), but such a failure would not lead to a drastic change in the bending inertia of the salt roof, which is 70-m thick. Roof fall in the outermost gallery is a consequence, rather than a cause, of the collapse. A less local mechanism still must be found. The mechanical model predicts that a large part of the overburden weight (still larger when the mine extent is greater) is transferred to the abutment, which is the seat of increasing shear stresses when the horizontal dimensions of the mine increase. When salt is submitted to high shear stresses, dilatancy develops. We did not attempt to describe fully dilatancy onset and progressive development, but we did draw the contour of the zone where the dilatancy criterion is met (see Figure 6). The dilatancy criterion is slightly more severe than that suggested by De Vries et al. (2005) for Cayuta Mine salt. It can be observed that the dilatant zone extends from the outermost gallery panel to the salt top in 1873, when the outermost gallery was dug out. This is consistent with the mechanism suggested above, which predicts that this zone is the seat of severe localized strains that allow the rock crown located between the un-deformed central cylinder and the abutment to lean toward the panel centre without experiencing brittle failure.

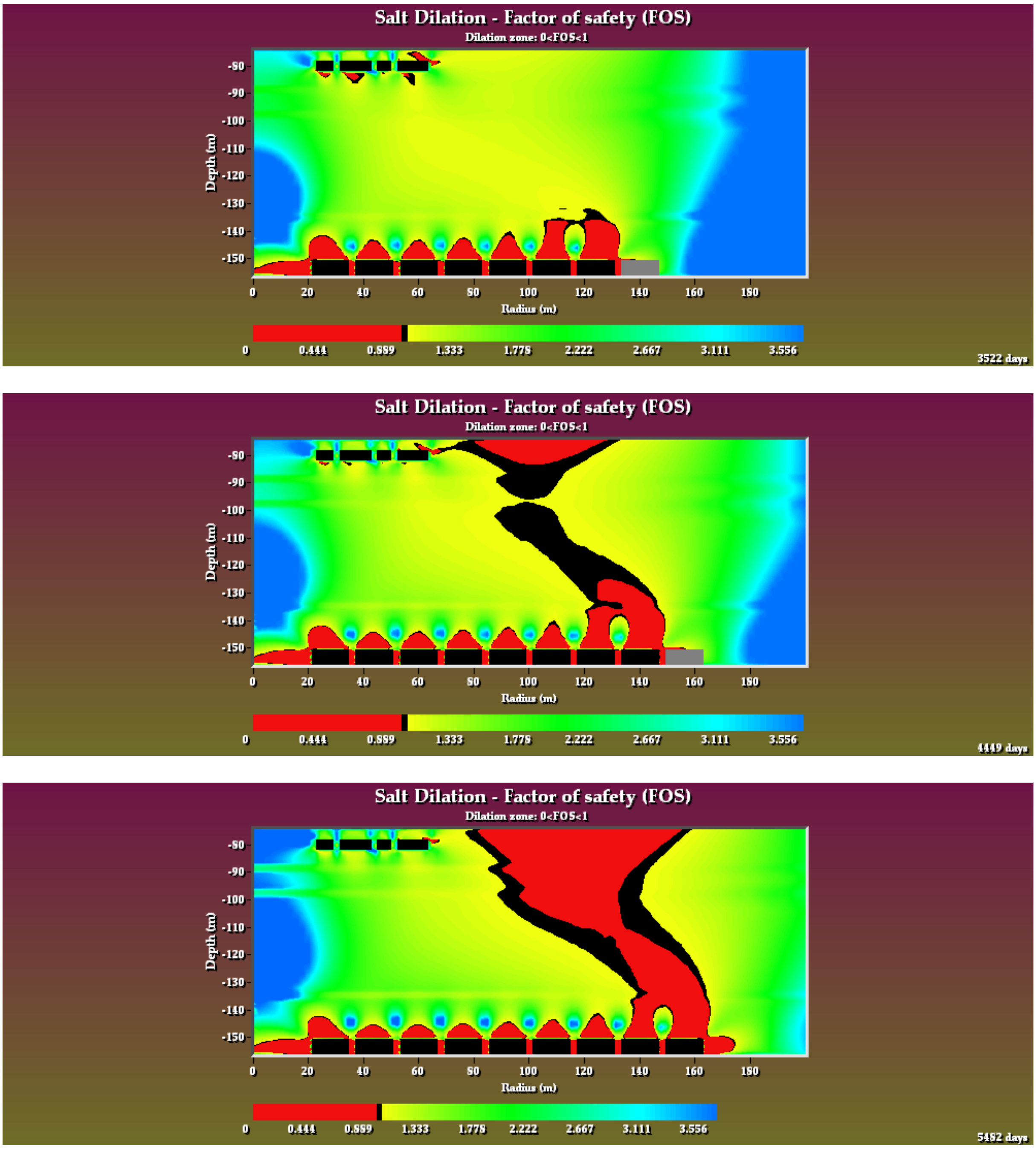


Figure 6 - Progression of the zone in which the dilatancy criterion is met during excavation of galleries n°7, 8 and 9. Radii are from the axis of symmetry.

2. THE 1864 FLOODING OF THE DIEUZE SALT MINE

2.1 Introduction

At Dieuze, a few dozen kilometres from Varangéville, a room-and-pillar mine was opened in 1826 (Feuga, 2002). Access to the mine was through two 2-m^2 shafts 8.50 m apart, dug within the walls of the royal saltworks, at the north-eastern edge of the mine. At a depth of 120 m, a 5-m-thick salt layer was mined at the base of the 11th layer — i.e., exactly where the Saint-Maximilien panel was mined. The mine contour was approximately rectangular, 550-m long and 130-m wide (Figure 7). The extraction ratio was high, varying from 80 to 90%. Pillars were square (4.5 to 5-m wide), and galleries were 6 to 9 m-wide and 4-m or so high. A 1-m-thick protective salt slab was left at the mine roof. No salt appears to have been left at the floor: the pillars rested directly on the underlying marly layers, as they did in the Saint-Maximilien panel.

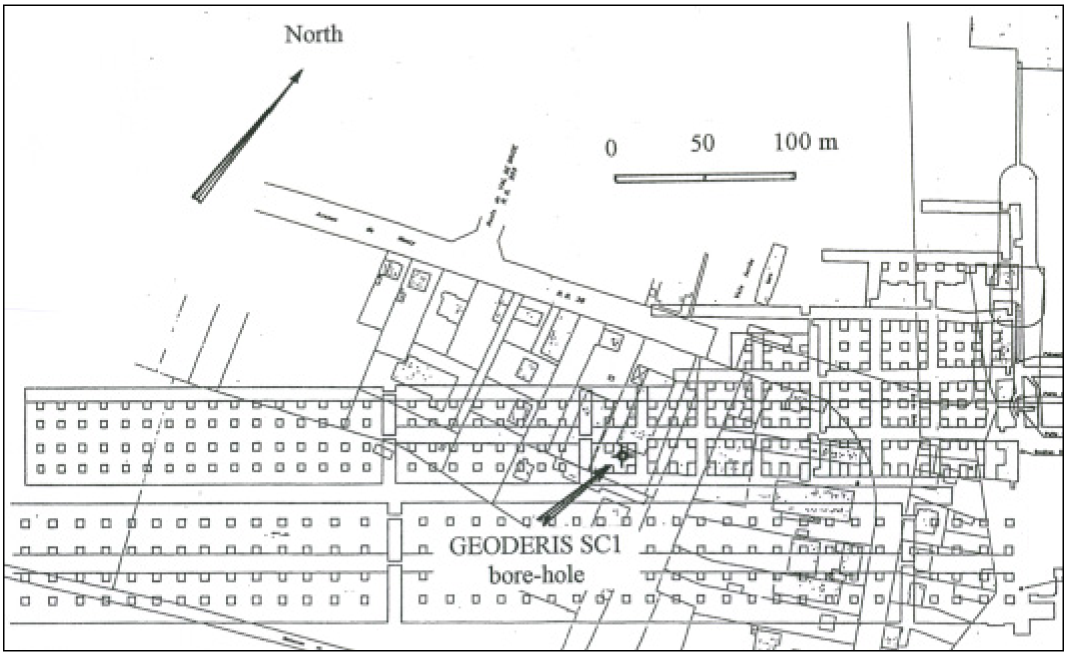


Figure 7 - Map of the Dieuze Mine and location of the GEODERIS SC1 borehole.

A total of 450,000 metric tons of salt were extracted during the 38-year period when the mine was open. No mechanical instability was reported. The mine was flooded in 1864. In the upper part of the salt-bearing formation, about 50 m above the mine at its north-eastern edge, a cavern had been solution-mined since 1860. It was connected to one of the two shafts by an access gallery. By 1864, this cavern was large (7-m high, 32-m wide and 90-m long), and it ultimately collapsed on February 8, 1864. A flood of brine invaded the access gallery and rushed down the shaft to the salt mine.

The collapse of the cavern created connections between the mine and the overlying aquifer levels. On March 7, 1864, the water flow-rate was estimated to be 432 m^3/hour, and it was decided to abandon the mine. The concentration of the brine flowing to the mine was measured daily during flooding: it was saturated almost all the time, except in the very last days before mine abandonment.

Ten years ago, the Dieuze town council decided to restore the historical salt-works buildings. However, because the buildings are located near the horizontal contour of the flooded mine, mining authorities requested that studies be performed to assess mine stability.

2.2 First Comments on Mine Stability

Field observations and archives suggested that no significant subsidence had taken place since the 1864 collapse. The mine depth was H = 120 m, and the extraction ratio was $\tau \approx 85\%$. Before mine flooding, the tributary load was: $\sigma^{trib} = \gamma_R H / (1-\tau) \approx 18$ MPa, a relatively high figure for which failure or the shortening of pillars caused by rapid creep could have been expected. However, no such event was reported. In fact, preliminary computations performed by Vouille and Humbert (2001) suggested that, because the mine span was small, most of the overburden weight was transferred to abutments on either side of the mine.

The extraction ratio did not change as a consequence of mine flooding, because the mine was flooded by (almost) saturated brine. A hydrogeological study proved that, after mine flooding, the hydrodynamic conditions would not allow soft water to displace the heavier brine which fills the old mine. Brine pressure in the mine is halmostatic, and the tributary "effective" load is significantly lower than it was before flooding, $\sigma^{trib} = (\gamma_R - \gamma_b) H / (1-\tau) \approx 8$ MPa. The stability of the mine has improved, as was suggested by Van Sambeek and Thoms (2000), who proved that the Belle Isle, Retsof and Jefferson Island mines experienced slower subsidence after they were flooded and by Rolfs and Crotogino (2000) who performed numerical analysis of flooded salt mines in Cheshire.

However, concerns remained, as the restored buildings were to be open to the public. It was decided to drill a cored borehole 10 m below the mine floor and to perform a sonar survey of the mine (Feuga, 2002). Borehole location is shown on Figure 7.

The salt top was found at a 51.5-m depth. The salt formation above the mine was cored and consisted mainly of hard, strong rocks, including rock salt (67%) and claystone considerably strengthened by large anhydrite content. Pure claystone never appears in thick beds. At 115.40 m, the borehole entered a large void. Boring rods touched void bottom at 119.14 m. The void height was 3.74 m, close to the initial height of the Dieuze rooms. A sonar survey of the cavity was performed, and cores were sampled for laboratory tests. The immediate mine floor consists of claystone reinforced by a variable amount of anhydrite and intersected by thin veins of salt or gypsum — in other words, it is similar to Saint-Maximilien floor. It was not weathered, and its mechanical strength was relatively high despite being in close contact to saturated brine for over 140 years. An attempt was made to sample deeper cores (16 cm below the mine floor), but core quality was poor. The map drawn after the sonar survey measurement matched almost exactly the 140-year-old map drawn by the miners. The measured mine height had not changed, and the floor remained perfectly flat. Weathering of the marly floor and punching into the floor by over-weighted pillars, an event feared after the 1873 Varangéville collapse, clearly did not take place in the Dieuze Mine, raising a puzzling question.

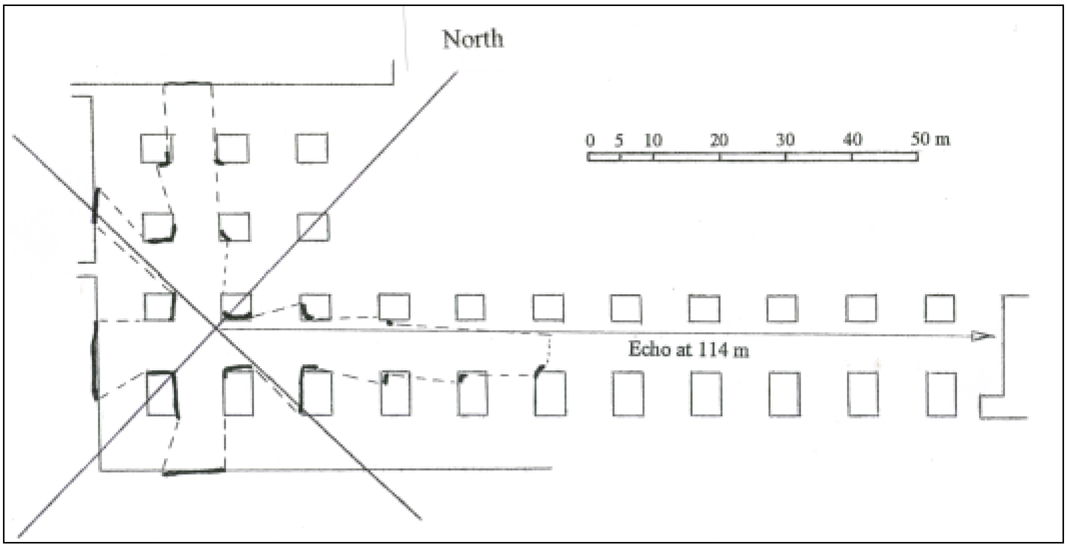


Figure 8 - Sonar picture of the Dieuze Mine (Bold features show sonar echoes; fine features show the 140-yr old map of the mine.).

2.4 Dieuze Mine and Saint-Maximilien Panel Behaviour

Extraction ratios were quite similar at Dieuze and Saint-Maximilien, and the floor composition is likely to be identical. Why did the panels exhibit such different behaviour? Two sets of explanations can be found.

1. The depth of the Dieuze Mine was smaller (120 m instead of 160 m). Furthermore, after mine flooding, the “effective” load on mine pillars was made significantly smaller as the halmostatic pressure of the brine bore approximately half of the overburden weight: as a whole, the effective overburden weight above the flooded Dieuze Mine was 1/3 of what it was above the Saint-Maximilien panel. Dieuze Mine was 130-m wide, instead of 320-m wide at Saint-Maximilien. It reasonably can be assumed that the Dieuze roof was much stiffer (see Section 1.4), roof sagging much smaller, and a much larger part of the overburden weight was transferred to the abutment, making the average vertical stress on pillars smaller than 1/3 of what it was at Saint-Maximilien.

2. Boidin (2007) points out that the upper crust of the marly layer at mine floor (which is the same at Dieuze as at Saint-Maximilien) is stiff and strong; it contains salt and anhydrite. Its strength significantly reduces when it is in contact with soft water, as soft water leaches out salt veins.

Conversely, saturated brine apparently has no, or minor, effect. Below this upper crust, which is 1-m to 2-m thick or less, are the softer chocolate brown marls that drastically weather when in contact with saturated brine. One difference between Saint-Maximilien and Dieuze is the fact that the chocolate brown marls of Dieuze were protected from the detrimental effects of saturated brine by the anhydritic upper crust. At Saint-Maximilien, soft water, instead of saturated brine, laid on the mine floor and was able to weather the upper crust. Furthermore, a large number of deep slots were dug out in the panel floor, allowing direct access of brine or water to the chocolate brown marls (see Section 1.1.)

It was decided to assess the first assumption and determine whether the same computations, which had led to the prediction that the Saint-Maximilien panel would collapse, would lead to similar conclusions at Dieuze.

2.5 Numerical Computations

Because the ratio length/width at Dieuze was approximately 4, two-dimensional plane-strain computations were performed (instead of axisymmetric computations, as at Saint-Maximilien). The salt roof is essentially the same (except for roof thickness) as for Saint-Maximilien: a low salt elastic modulus was selected, and three 3-m thick marls layers were intercalated inside the salt roof to lessen roof thickness. Twelve rooms and eleven pillars were represented (see Figure 9, pillars are numbered from left to right). Pillar width was made smaller than actual pillar width to respect the overall extraction ratio; however, a large pillar (pillar n°6) was left at panel mid-span. Because it existed in the actual Dieuze Mine, it was suspected that this pillar played a significant mechanical role. Floor marls cohesion was $C = 0.75$ MPa — i.e., the figure selected to describe the mechanical strength of the weathered Saint-Maximilien floor (see Section 1.4). The pillar-bearing capacity was taken as approximately $\omega_c C = 4.5$ MPa. The galleries were mined over one year, from 1844 to 1845, and mining was completed at day 365 after mine opening. (Because we mainly were interested in long-term (more than one century) behaviour of the mine, which had operated from 1826 to 1864, we did not try to describe its 38-year long history in detail). In 1864 (i.e., day 7300), mine flooding began: the internal pressure of the mine, which was atmospheric before flooding, was increased to halmostatic pressure in 30 days. Computations were performed until day 36500 — i.e., to 1944.

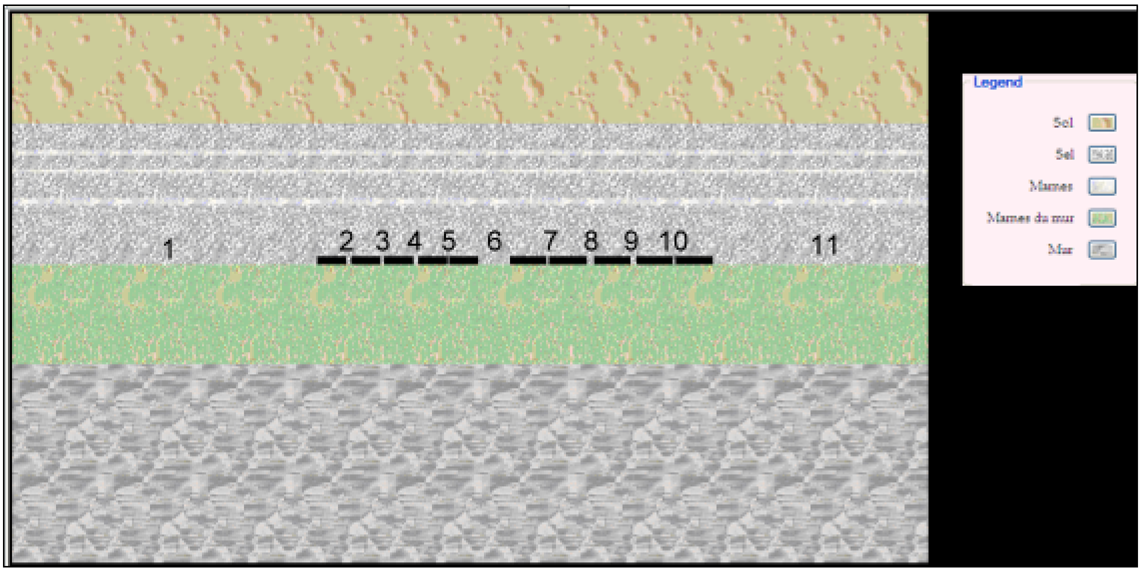


Figure 9 - Vertical cross-section of the Dieuze Mine.

The geostatic pressure at mine roof depth, or 122 m, is 2.8 MPa. Additional average vertical stresses on the eleven pillars (as a function of time) is displayed on Figure 1a. Most pillars exhibit the same behaviour except for the two outermost pillars, n°1 and n°11, and for the widest central pillar, n°6. When the mine is opened progressively (from day 0 to day 365), the pillar load increases by ≈ 11 MPa, making the (total) average vertical stress on the pillar $\sigma_{zz} \approx 11 + 2.8 = 13.8$ MPa. The mechanical response of the rock mass essentially is elastic during this period, but the average vertical stress ($\sigma_{zz} \approx$ 13.8 MPa) is smaller than the "tributary" load, which is $\sigma^{trib} = 18$ MPa, because (1) the mine span is not very large, and a significant part of the overburden load is transferred to abutments, and (2) pillar punching starts as soon as pillar-bearing capacity ($\omega_c C = 4.5$ MPa) is exceeded. (In this model, marls cohesion is low even before mine flooding takes place.) For 20 years, the average vertical stress on each of the eleven pillars slowly converges to pillar-bearing capacity, exactly as it did in the Saint-Maximilien panel. In year 1864 (or 20 years after the fictitious mine creation), mine flooding takes place, and the pressure at an average mine depth (124 m) abruptly increases by 1.5 MPa. During this 30-day period, the rock mass behaviour is elastic. The vertical load on most pillars decreases by $\approx$ 8 MPa (less on pillars 1, 6 and 11). The actual load on most pillars is a tensile stress — a surprising but logical consequence of the floor behaviour, which is plastic. No, or very slow, additional evolution can be observed, as both the pillar creep rate and the roof creep rate are slow or nil: the overburden weight is transferred fully to the abutments. Of course, this description is somewhat theoretical, as it was assumed that marls cohesion was small as soon as the mine was created — not only after mine flooding.

For this reason the case when floor marls cohesion is C = 2 MPa ("dry" marls) was also considered (Figure 10b) as it can be assumed that, before flooding, the marly floor was intact. Here again, pillars punch the marly floor after mine creation and for 20 years the average vertical stress slowly converges to pillar-bearing capacity – which is much higher than when floor cohesion was assumed to be C = 0.75 MPa. During flooding, the vertical load on most pillars decreases by $\approx$ 8 MPa. The actual load on most pillars is a compressive stress – a more reasonable result.

Displacements were analysed assuming that the floor marls cohesion is C = 0.75 MPa, a kind of a "worst-case" assumption.

Evolution of the displacements in the gallery closest to the central pillar is displayed on Figure 11 (a,b,c,d). After one year (in 1827), displacements remain small [Figure 11 (a)]. They are much larger 20 years later, as pillars had punched the mine floor [Figure 11 (b)]. When the mine is fully flooded, 30 days later, vertical displacements slightly reduce, an effect of the elastic behaviour of the rock mass when submitted to smaller deviatoric stresses. It is remarkable that, 100 years later, the displacements did not change. Large roof sagging is prevented by roof stiffness (The Saint-Maximilien roof was much less stiff, because the panel span was larger by a factor of 3), and the vertical loads on salt pillars are small, leading to very slow creep shortening.

The extent of the dilatant zones is drawn on Figure 12 (a,b,c,d). One year after mine creation [Figure 12 (a)], the stress distribution in the rock mass essentially was elastic, and dilatant zones appeared inside salt pillars and at the mine edge. (Large deviatoric stresses generated by the mine opening had not yet had time to release.) Twenty years later [Figure12 (b)], dilatant zones were smaller (Remember that no post-dilatant computation is performed.), because stress distribution slowly evolved from quasi-elastic distribution (at day 365) to steady-state visco-plastic distribution (at day 7300). After mine flooding [Figure12 (c)], the dilatant zones were smaller still smaller, as expected, because the mine was pressurized to halmostatic pressure. One century later [Figure12 (d)], dilatant zones were only present in some pillars. Comparing Figure 12 to Figure 6, due to its smaller depth, smaller mine span and higher internal pressure (after mine flooding), the extent of the dilatant zone at Dieuze is much smaller than at Varangéville, which explains why mine collapse would be feared at Saint-Maximilien but not at Dieuze.

It was proved that even very pessimistic assumptions (floor marls cohesion is lessened to $C = 0.75$ MPa as soon as the mine is created, and not after the mine is flood, 20 years later) lead to the conclusion that the mine is stable.

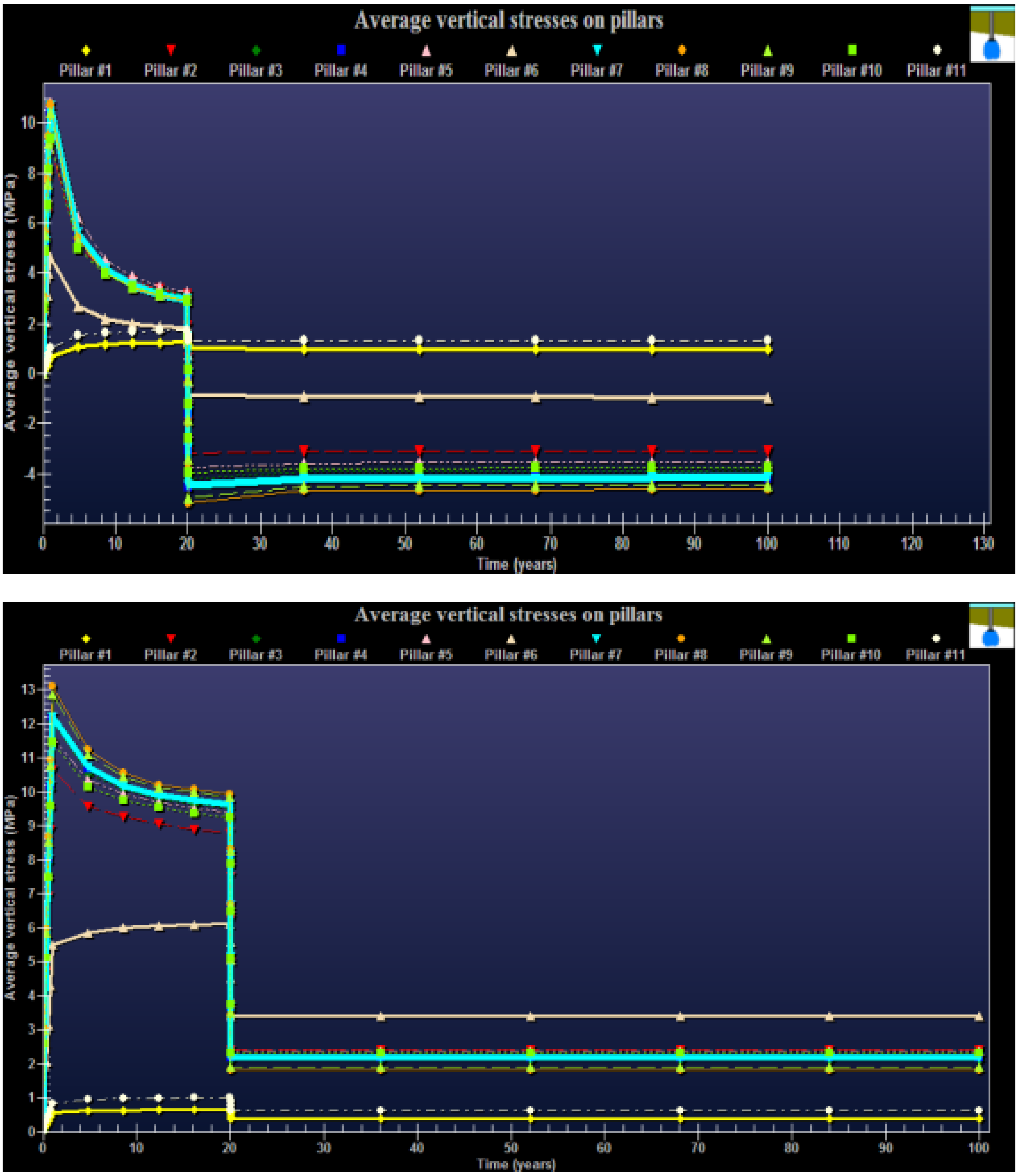
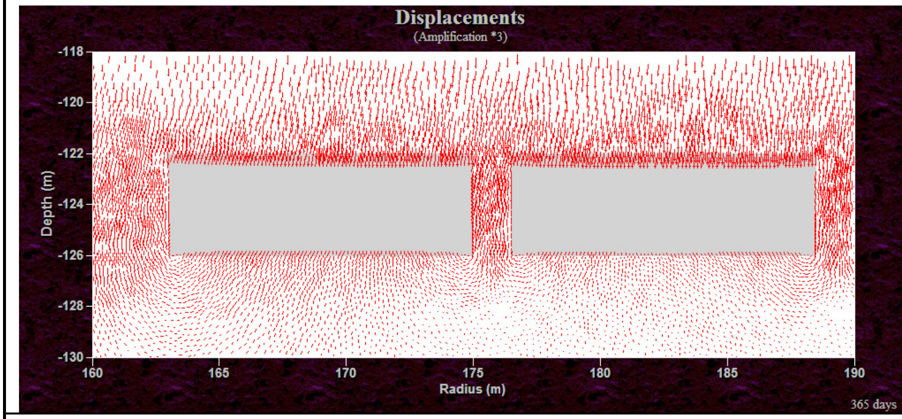
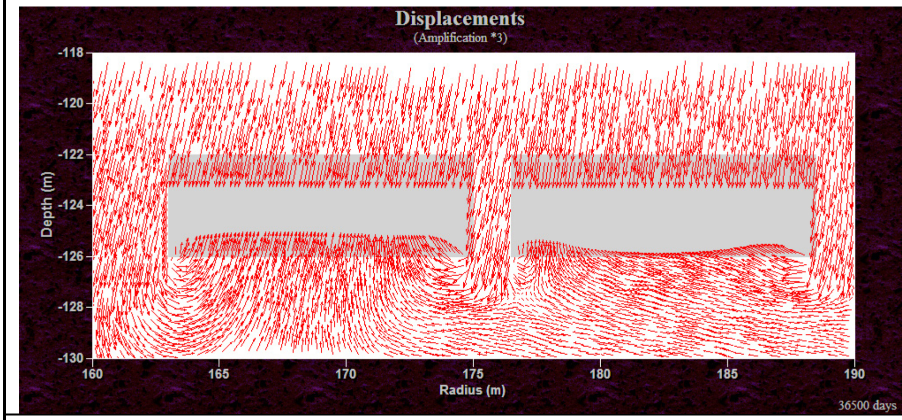


Figure 1. Average additional vertical stress on Dieuze Mine pillars. (a) Floor marls cohesion is $C = 0.75$ MPa **(b) Floor marls cohesion is** $C = 2$ MPa.



(a)



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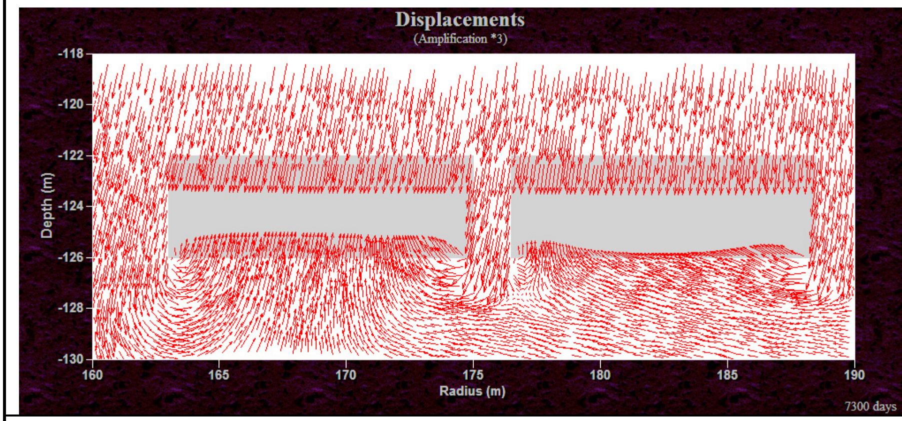
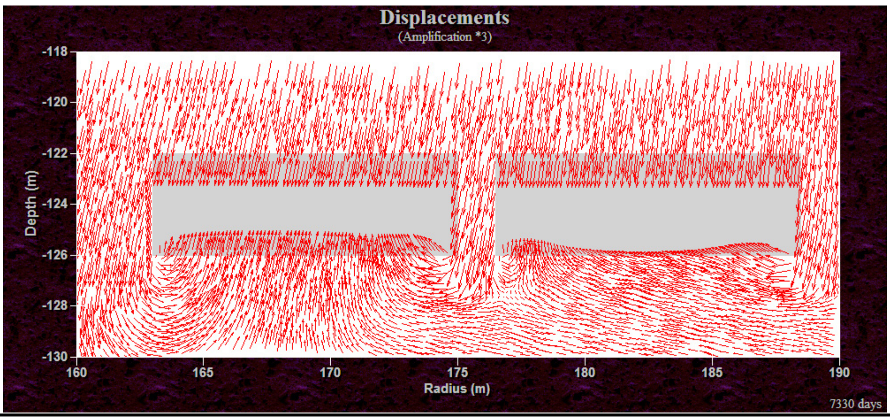
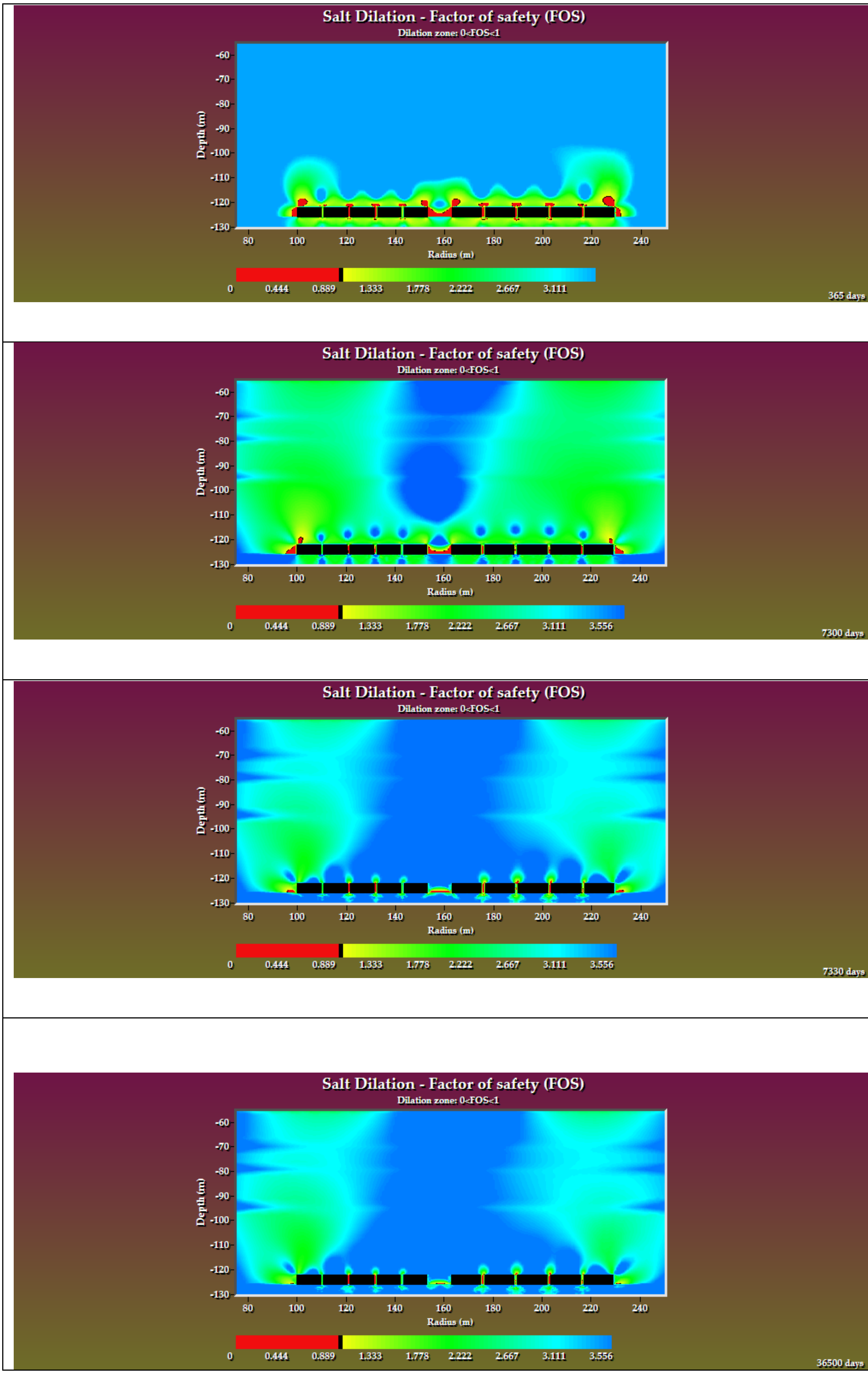


(c)



(d)

Figure 11 - Displacements in galleries n°7 and n°8 in (a) 1844, (b) 1864 (before mine flooding), (c) 1864 (after mine flooding), and (d) 1944.



(a)

(b)

(c)

(d)

Figure 12 - Zones in which the dilatant criterion is met in (a) 1844, (b) 1864 (before mine flooding), (c) 1864 (after mine flooding), and (d) 1944.

CONCLUSIONS

The behaviour of two mines in which water or brine was abundant has been discussed. Weathering of the floor marls led to sudden collapse of one of the two mines; the other mine was flooded by saturated brine and did not collapse. Two factors can explain the difference in the behaviour of these otherwise almost identical mines: brine concentration, and mine span. It has been proven that mine span alone can explain the perennial stability of the second mine.

REFERENCES

Bekendam R., Oldenziel C. and Paar W. (2000) Subsidence Potential of the Hengelo Brine Field (Part I). Physico-Chemical Deterioration and Mechanical Failure of Salt Cavern Roof Layers. *Proc. Technical Class and Technical Session, SMRI Fall Meeting, San Antonio,* p.103-118.

Bérest P., Brouard B. and Feuga B. (2003) Dry Mine Abandonment. Proc. SMRI fall meeting, Wichita, p.9-37.

Bérest P, Blum PA, Charpentier JP, Gharbi H. and Valès F. (2005) Very slow creep tests on rock samples. *Int J Rock Mech Min Sci 2005;42*:569-76.

Bérest P., Brouard B., Feuga B. and Karimi-Jafari M. (2008) The 1873 Collapse of the Saint-Maximilien Panel at the Varangéville Salt Mine. *To be published in: Int J Rock Mech Min Sci.*

Boidin E. (2007) Interactions between rocks and brines in the context of mines and caverns abandonment. (Interactions roches/saumures en contexte d'abandon d'exploitations souterraines de sel). *Ph.D Thesis, Institut National Polytechnique de Lorraine, Nancy, France, February 2007* [in French].

Braconnier M.A. (1873a) Mine inspector first report on the collapse of the Varangéville-Saint Nicolas salt mine, Nancy (1er rapport de l'ingénieur des mines sur l'effondrement de la mine de sel gemme de Varangéville-Saint Nicolas, 2 novembre 1873), *unpublished* [in French].

Braconnier M.A. (1873b) Mine inspector second report on the collapse of the Varangéville-Saint Nicolas salt mine, Nancy (2ème rapport de l'ingénieur des mines sur l'effondrement de la mine de sel gemme de Varangéville-Saint Nicolas, 3 novembre 1873), *unpublished* [in French].

Braconnier M.A. (1873c) Mine inspector third report on the collapse of the Varangéville-Saint Nicolas salt mine, Nancy (3ème rapport de l'ingénieur des mines sur l'effondrement de la mine de sel gemme de Varangéville-Saint Nicolas, 4 novembre 1873) *unpublished* [in French].

Buffet A. (1998) The collapse of Compagnie des Salins SG4 and SG5 drillings. *Proc. SMRI Fall Meeting*, Roma, p.79-105.

Rolfs O., Crotogino F., (2000) Rock mechanical problems of shallow salt mines in Cheshire, UK. *Proc. SMRI Technical Class and Technical Session, Fall Meeting, San Antonio*, p. 304-312

DeVries K.L., Mellegard K.D. and Callahan G.D. (2003) Cavern Design Using a Salt Damage Criterion: Proof-of-Concept Research Final Report. *Proc.SMRI Spring Meeting, Houston,* p.1-18.

Feuga B. (2002) Old salt mine at Dieuze (France) revisited 150 years after being abandoned. *Proc. SMRI Fall Meeting, Chester*, p. 114-128.

Jeanneau V. (2005) The sinkhole of the cavity LR 50/51 in La Rape Area, a case history. RHODIA Company. *Proc. SMRI Fall Meeting, Nancy,* p.9-24.

Minkley W. and Menzel W. (1996) Local Instability and System Instability of Room and Pillar Fields in Potash Mining. *Proc. 3rd Conference Mechanical Behaviour of salt. Clausthal-Zellerfeld, Germany: Trans Tech Publishers,* p.497-510.

Rothenburg L., Carvalo Jr A.L.P., Dusseault M.B. (2007) Performance of a mining panel over tachyhydrite in Taqari-Vassouras potash mine. *Proc. 6th Conference Mechanical Behaviour of Salt. Taylor & Francis, London UK,* p.305-314.

Salençon J. and Matar M. (1982) Bearing capacity of axially symmetrical shallow foundations. (Capacité portante des fondations circulaires). *Journal de Mécanique Théorique et Appliquée, Vol. I, n°2, p.237-267 [*in French].

Van Sambeek L. and Thoms R. (2000) Pre- and Post-Flooding Surface Subsidence Rates at the Retsof, Belle Isle, Jefferson Island Salt Mines. *Proc. Technical Class and Technical Session, SMRI Fall Meeting*, *San Antonio,* p. 75-85.

Vouille G. (1986) Varangéville Mine, Mechanical Behaviour of Floor Marls when put in contact with Brine. (Mine de Varangéville, Comportement mécanique des marnes du mur en présence de saumure). *Rapport R 86/3 de l'Ecole des Mines de Paris pour la CSMSE, unpublished* [in French].

Vouille G. and Humbert B. (2001) Dieuze Mine. Mechanical Stability of the old mining works. *Rapport R2001/11/ROC/MR.* [in French].