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Stability assessment of a shallow abandoned chalk mine of Malogne (Belgium)

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ABSTRACT

The Malogne Phosphatic chalk quarry was developed by the rooms and pillars mining method within an area of 67 ha. The site is partially flooded and located in close proximity to important infrastructure as railway, highway, and residential houses. During and after its exploitation several significant ground collapses were registered. The last one, with an area of 1.2 ha and 3 m amplitude occurred in 2015 nearby the railway line. To assess the behavior of the underground openings a combination of in situ structural measurements and laboratory mechanical characterisation in dry and saturated state of the main lithology type in the quarry – white and phosphatic chalk, hard ground and calcarenites, are performed. The UCS of the chalk ranges from 4 to 10 MPa while its tensile strength is between 0.75 and 2.1 MPa. For the hardground, these values are over ten times higher. The results reveal significant influence of the water on the chalk properties by reduction of almost twice its strength. The data obtained was implemented in a conceptual geomechanical model, using the 2D FEM. Numerical results show a failure pattern that was confirmed from the in situ observations.

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Chalk; shallow depth; room-and-pillar; stability analyses; FEM

Introduction

Very often, underground cavities that were developed at a shallow depth and in relatively soft rocks can induce stability problems on the ground surface. A typical example of that is underground quarries for chalk extraction where collapses occurred. Most often, the stability problems led to failures that, together with significant landscape changes, could affect near situated surface infrastructure and inhabited area. In this way, they could have an important influence on the local communities. This type of problem has been the subject of numerous studies that attempt to propose solutions by analytical and numerical methods (Al Heib, 2016; Al Heib et al., 2015; Bekendam, 2004; Castellanza et al., 2013; Gombert et al., 2019; Laouafa & Tritsch, 2006; Lewis & Donovan, 2018).

In this paper, a case study on the stability assessment of an underground chalk quarry developed by room-and-pillar mining method (Leclercq & Bouko, 1985), mainly on one level, is presented. The underground quarry is situated in the central part of the Mons sedimentary Basin (western Belgium), where the underground and open-pit exploitation of the Upper Cretaceous Ciply-Malogne Phosphatic Chalk Formation has been started in 1873. Initially, it began with the exploitation of several separate underground quarries that were later joined into one large with a total area of 67 ha, now known as the Malogne Phosphatic chalk quarry (Figure 1, red contour). The mining operations in the site have reached the maximum during 1904 and then again after the First World War. They were closed completely in the 1950s. The quarry was developed at a shallow depth with a hangingwall generally comprised between

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Figure 1. (a) Localisation of the Malogne underground quarry (red contour) and the surface infrastructure that could be influenced by the mine instabilities and (b) Position and aerial photograph of the collapse from 22 April 2015 (CACEff, 2015).

20 m and 25 m below the ground surface in the area of the Cuesmes village. Today it is located very close to regional important infrastructures, such as the railway Brussels-Paris and the highway R5. The total length of the galleries in it was estimated to about 175 km. The extraction ratio of the quarry varies approximately between 60% and 70%.

During the active mining operation, for a certain period, some of the excavations have been partially backfilled with slag. However, later it has been almost entirely removed and reused for different purposes (i.e. agricultural) due to the residual phosphates content. Currently, only a very small area of the quarry still remain backfilled (Leclercq & Bouko, 1985). Today about two-thirds of the quarry, at some places partly flooded, is available for observations. Due to the general layers dipping the remaining one-third of the quarry, mainly its northern part, is permanently below the groundwater level.

The precise location of the contour of the Malogne quarry was superimposed on the surface and shows that some parts of it are situated just below infrastructure and residential houses (Figure 1(a)). It became even critical as in 2015 a big collapse with an area of 1.2 ha and maximum amplitude of 3 m in the southeastern part of the quarry nearby the railway happened, affecting mainly agricultural land (Figure 1(b)). This indicates the high potential geotechnical risk above the underground excavations. Therefore, for the proper management of the site, the estimation of the surface instability risk at different places of the quarry becomes necessary. This event was a trigger for a new interest among public authorities and the scientific community. However, the current work does not aim to represent the collapse of 2015 and the conditions that induced this event.

The main objective of the study presented herein is to investigate the current state of the rock mass and its potential evolution of the underground openings and the overall stability of the pillars in the quarry. To understand the behavior of the abandoned underground phosphatic chalk quarry and to address the problem with the stability of the underground openings, geomechanical modeling needs to be performed. The stability analyses of soft rock masses as chalk usually are based on either a continuous or a discontinuous approach by different numerical methods. Descamps et al. (2019) investigated the influence of a shallow underground cavity developed in carbonate rocks by room-and-pillar mining on the stability of an open-pit quarry based on a 2D FEM parametric study. Perrotti et al. (2019) discussed the application of FEM-based stability charts for underground cavities in soft carbonate rocks and their validation by 2D and 3D models.

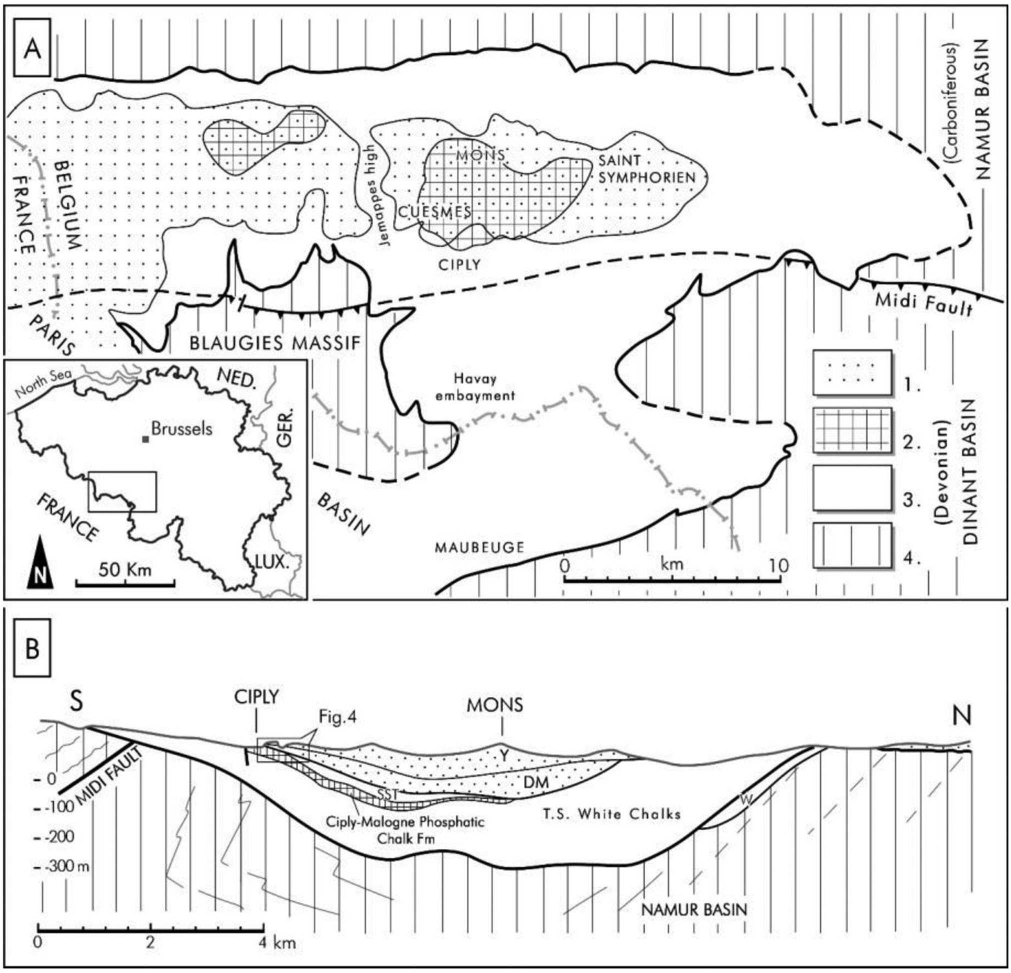

Figure 2. Position and schematic geological map (a) and schematic geological cross-section (b) of the Mons basin with the location of the Malogne quarry. Position of Figure 4 is indicated. 1 – Tertiary calcarenites, sands and clays; 2 – Maastrichtian Ciply-Malogne Phosphatic Chalk Formation; 3 – Upper Cretaceous marls and white chalks; and 4 – Paleozoic basement. T. S., Turonian-Senonian; SST, Saint Symphorien Formation; DM, Danian and Montian; Y, Ypresian (after Robaszynski & Martin, 1988).

From regional point of view, during the last decades petrophysical and mechanical properties of the chalk have been studied in several regions of W Europe as the Mons Basin, Belgium (Megawati et al., 2015; Schroeder, 2002), in Boulonnais region, France (Amédro & Robaszynski, 2001; Robaszynski & Amédro, 1986), in the Danish central North Sea (Fabricius, 2007; Fabricius et al., 2008) as well as in Flamborough Head and Sussex, England (Menpes & Hillis, 1996; Whitham, 1993). Summarising some of the data Descamps et al. (2017) analysed the relationships between the petrophysical and geomechanical properties from different chalk lithotypes in NW Europe and the factors that affect them. They established a linear relationship between the porosity and the logarithm of the permeability as well as between the porosity and the UCS.

Until the moment, no particular data for the Ciply-Malogne Phosphatic Chalk Formation was reported. Therefore, systematic sampling and laboratory mechanical characterisation were conducted in this study. The results have been used later on for developing a geomechanical model of the quarry and studying the behavior of the rock mass in dry and water-saturated conditions.

Geological and hydrogeological settings

The Mons sedimentary Basinis build up by Cretaceous, Paleogene and Quaternary formations that unconformably overlie the Paleozoic basement (Figure 2), including formerly exploited coal-bearing sediments

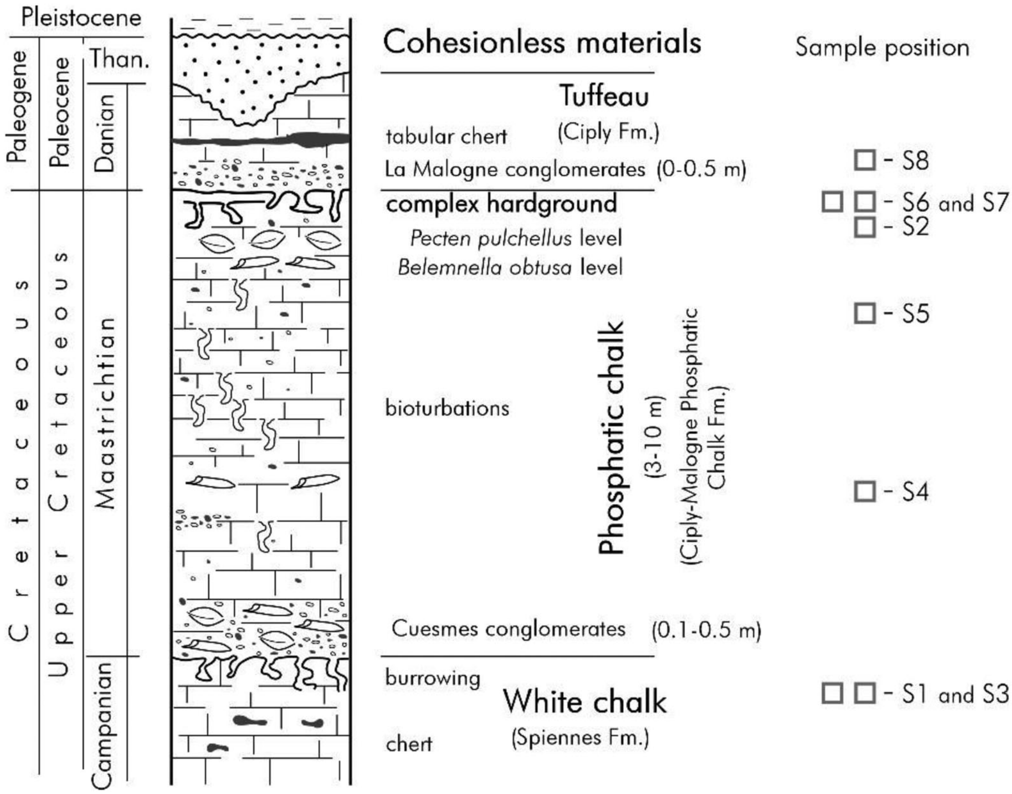

Figure 3. Lithostratigraphy of the Malogne quarry (after Vandycke, 1987; Robaszynski et al., 2002) and position of the samples for laboratory analysis.

of Upper Carboniferous. To the west, it is connected with the Paris Basin (Schnyder et al., 2009; Vandycke, 1992). From a regional point of view, the Mons Basin is a pull-apart structure located between the Brabant Massif to the north and the Ardennes Massif to the south where several stages of subsidence were distinguished (Dupuis & Vandycke, 1989; Vandycke et al., 1991). In the Malogne underground quarry, two tectonic events were identified: (1) strike-slip faulting (Vandycke, 1987) and (2) synsedimentary normal faulting (Dupuis & Vandycke, 1989).

From the sedimentological point of view the chalk, represented by several lithological varieties is the most common lithotype in the Upper Cretaceous series of the Mons Basin (Robaszynski et al., 1987, 2002). During Late Maastrichtian, because of sea-level fluctuations condensed beds and hardgrounds or even hiatus have been developed (Pirson et al., 2008).

The Maastrichtian phosphatic chalk (Ciply-Malogne Phosphatic Chalk Formation) that has been the object of intensive mining extraction is represented by fine to medium-grained limestone (Figure 3). The structure is massive to thick-bedded and contains rounded phosphate chalk conglomerates that form distinct levels (Vandycke, 1992). The base of the formation, that is, its contact with the underlying Campanian white chalk (Spiennes Formation), is an intensively burrowed surface covered by a bed of phosphaticpebbles and marks the footwall of the quarry. The uppermost part of the Ciply-Malogne Phosphatic Chalk Formation is characterised as a hardground level, which has an average thickness of about 60 cm (Vandycke, 1987). The top of the hardground marks the end of sedimentation of the Ciply-Malogne Phosphatic Chalk Formation and represents the hangingwall of the Malogne underground quarry. The Ciply-Malogne Phosphatic Chalk Formation is covered by the Danianthe Ciply Formation (known as tuffeau) which consists of a white-yellowish, marine calcarenite (Robaszynski et al., 2002). The overlying Thanetian to Pleistocene as age beds are considered as cohesionless materials.

According to Leclercq and Bouko (1985) an aquifer with a capacity of about 800,000 m^3 could be identified in the NE parts of the Malogne underground quarry. Currently, the underground water is permanently pumped. However, the water level varies with the seasons, so that a certain part of the pillars

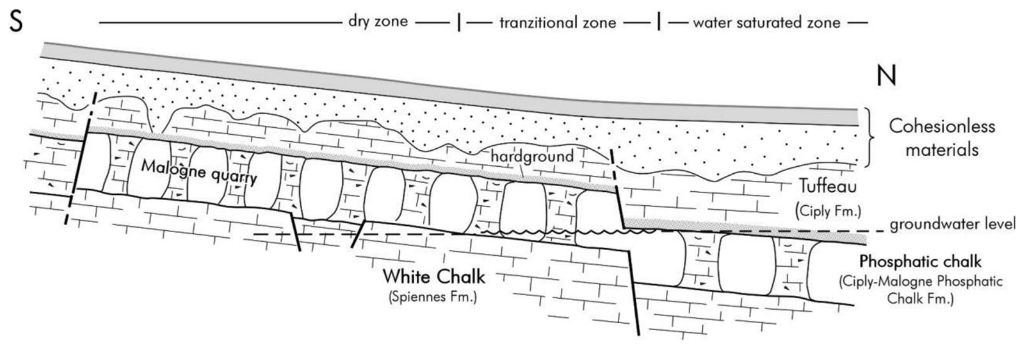

Figure 4. Schematic cross-section (not scaled) of the Malogne quarry (after Pacyna, 1992; with modifications) with the area of the dry, transitional and water-saturated zones.

are exposed to cycles of water-saturating and drying throughout the year. Three different areas in terms of the groundwater level can be identified in the quarry: (1) dry, (2) transitional and (3) water-saturated zones (Figure 4). According to Pacyna (1992) and Funcken and Welter(1996), the variation of the water level would be related directly to the stability of the quarry. Several phenomena, connected with the water weakening effect on chalk, such as short- and long-term debonding and grain dissolution process, have been identified and described during the last decades (Ciantia et al., 2015; Lafrance et al., 2014; Risnes & Flaageng, 1999).

Material and methods

To characterise the rock mass properties and behavior of the Malogne quarry, more than 30-field observations and measurements were realised. Sampling and laboratory tests of all main lithological types outcropped in the quarry, or forming the beds above and below it, have been carried out (Figure 5). The sampling consists of selecting representative samples from different levels of the stratigraphic succession taken from the pillars (S4 and S5), the hangingwall (S2, S6, S7 and S8) and the footwall (S1 and S3) of the quarry. Their orientation has been recorded. The collected rock samples are relatively equally distributed in the area of 25 ha in the central part of the quarry (Figure 5). In total, 197 cylindrical core samples with a diameter of about 40 mm and height of 40 mm (for Brazilian tests) and 80 mm (for UCS test) were drilled from the phosphatic chalk, the tuffeau and the white chalk. For true triaxial tests, 16 cubic samples from the hardground with dimensions of $31 \times 30 \times 30$ mm were prepared.

As the water-saturation of the chalk can have a strong effect on its strength (Descamps et al., 2018; Gombert et al., 2013, 2019; Gombert & Al Heib, 2013), in this study, the rock samples have been tested in dry and water-saturated conditions.

The porosity was estimated according to API (1998) standard procedures prior to the mechanical tests as its values could have a significant influence on the mechanical properties of the rock mass (Descamps et al., 2012; Palchik & Hatzor, 2004; Ramos da Silva et al., 2010).

The UCS measurements were done through a manual loading rate applied on a stiff frame with a maximum capacity of 380 kN, except for the hardground samples where an automatic press was used. The load was performed continuously and pressure transducers (350 bars) were used to measure the axial stress. A gage with axial and lateral displacement transducers was used for recording the strains for each sample. The cubic samples of the hardground were used for performing the true triaxial tests under the confinement of 5 and 10 MPa.

The experimental procedure applied for the Brazilian test complies with the ISRM suggested methods (ISRM, 2014). The loading on the samples was applied with the same manual-controlled testing machine, as for the UCS.

Mechanical characterisation of the rocks

Prior to the mechanical tests, the porosity of all rock specimens was estimated. The obtained data shows that the phosphatic chalk has an average value of about 39% (varying between 35 and 41%) which is

Figure 5. Map of the Malogne quarry (provided by SPW) with locations of the rock samples for laboratory tests: S1 and S3 – white chalk; S4 and S5 – phosphatic chalk; S2, S6, and S7 – hardground; S8 – tuffeau.

very similar to those in the white chalk, while in the hardground samples these values are significantly lower, ranging from 9% to 12% (Table 1). The highest values were obtained for the tuffeau that has a porosity between 48% and 51%.

The samples were used to conduct rock mechanical tests in dry and saturated conditions. The experimental results show that in dry conditions the compressive strength of the phosphatic chalk is varying from 6.3 to 8.9 MPa (on average 7.8 MPa) and can be characterised as a low (Table 1). In a water-saturated state, it decreases approximately by half and is defined as a very low ranging from 3.2 MPa to 4.6 MPa and an average value of 4 MPa. For the dry samples of white chalk, the compressive strength shows slightly higher average values of 10 MPa varying from 9.5 MPa to 10.9 MPa. In water-saturated conditions, its strength decreases approximately twice with an average value equal to 4.9 MPa. Because the tuffeau interval is usually above the groundwater level in the quarry, these samples were tested only in a dry state. Its strength ranging from 1.3 MPa to 2.0 MPa with an average value of 1.7 MPa. In contrast, in dry conditions the hardground samples exhibit UCS values in the interval from 88.5 MPa to 128.4 MPa (on average 101 MPa) that is significantly higher than the other lithologies. After water-saturation, the UCS tends to decrease to 59 MPa.

The experimental results show that compressive strength and elastic modulus of the hardground are notably higher than those of the other tested materials (the phosphatic chalk, the white chalk, and the tuffeau). This indicates that the hardground is more rigid, while the chalks and the tuffeau are more plastic. Based on the stress-strain curves obtained from the UCS tests, it can be concluded that in an atmospheric condition the tested materials exhibit elasto-plastic behavior with brittle failure (Figure 6). In addition, the triaxial tests of the hardground show that under confinement its behavior changes to perfect-plastic and hardening. Except for the hardground, where even with a low porosity the uniaxial strength in dry conditions is approximately 42% higher than for the water-saturated one, results from the

Table 1. Rock mass properties obtained from the laboratory tests in dry conditions.

Property	Unit weight (MN/m³)	Unit weight (increased) (MN/m³)	Porosity (%)	UCS (MPa)	Tensile strength (MPa)	Friction angle (°)	Cohesion (MPa)	Young's modulus (MPa)	Poisson's ratio
Cohesionless materials[a]	0.013	0.030	—	—	0	30.0	0	20	0.33
Tuffeau	0.013	0.030	50	1.7	0.5	40.0	0.5	300	0.20
Hardground	0.024	0.055	12	100.7	15.2	46.5	19.5	32,000	0.11
Hardground (GSI = 60)[b]	—	0.055	—	—	0.7	30.4	5.4	16,600	0.11
Phosphatic chalk	0.017	—	39	7.8	1.5	46.0	1.5	2200	0.23
White chalk	0.015	—	45	10.0	2.1	39.5	2.3	2700	0.25

[a]Data retrieved from the literature.
[b]Reduced by GSI mechanical properties used for the numerical model.

Table 2. Rock mass properties obtained from the laboratory tests in water-saturated conditions.

Property	Unit weight (MN/m³)	Unit weight (increased) (MN/m³)	UCS (MPa)	Tensile strength (MPa)	Friction angle (°)	Cohesion (MPa)	Young's modulus (MPa)	Poisson's ratio
Hardground	0.025	0.058	58.7	12.4	39.5	13.4	22,900	0.17
Hardground (GSI = 60)[a]	—	0.058	—	0.4	30.36	3.2	11,900	0.11
Phosphatic chalk	0.021	—	4.0	0.7	45.0	0.9	1150	0.23
White chalk	0.019	—	4.9	1.1	37.0	1.3	1410	0.25

[a]Reduced by GSI mechanical properties used for the numerical model.

tests indicate the link between the high porosity of the tested rocks and their sensitivity of water (Table 2).

The tensile strength has been measured by the Brazilian tests in dry and saturated conditions. The results of the dry phosphatic chalk samples vary between 1.3 and 2.3 MPa with an average value of 1.5 MPa. Slightly higher values, between 1.8 and 2.4 MPa, were obtained for the dry white chalk samples (2.1 MPa on average). In the water-saturated state, the tensile strength of the phosphatic chalk is in the range from 0.6 to 0.85 MPa (on average 0.7 MPa), while for the white chalk it is between 0.9 MPa and 1.3 MPa, showing the pronounced effect of the water on the chalk strength. Such effect was not detected in the hardground samples, where the tensile strength in dry conditions varies from 10.9 to 21.3 MPa (in value of 15.2 MPa) and water-saturated samples show results in the range from 11.1 MPa to 12.9 MPa. The tuffeau was tested in dry conditions only. The results obtained range between 0.48 and 0.58 MPa with an average tensile strength of 0.5 MPa.

The data above was used to determine the constitutive law of the main lithological types included in the numerical modeling. The values of the mechanical properties of the rocks used as input data in the numerical model are presented in Tables 1 and 2. Only the uppermost lithotype has been assigned as a cohesionless homogeneous material (Figure 4) with properties based on published sources. The input values of the mechanical properties in the numerical model are presented in Tables 1 and 2.

2D Finite element model

Using RS2 software (Rocscience, 2015) a set of 2D finite element models (FEM) were accomplished for assessment of the behavior of the underground openings of the Malogne quarry. For that purpose, first a representative geometrical model was developed which later one has been used for conducting geomechanical analyses. They were used for estimating the behavior of the underground openings in dry and saturated conditions. This was made based on the estimation of different parameters such as the vertical

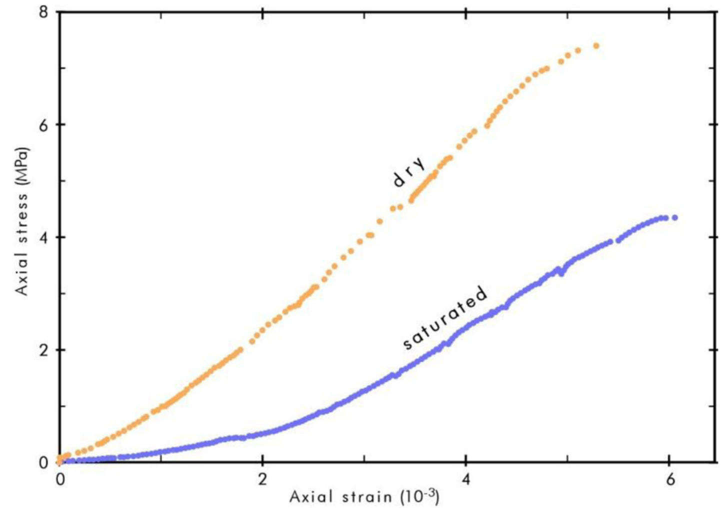

Figure 6. Typical stress-train curves for the phosphatic chalk in dry (blue) and saturated (orange) state.

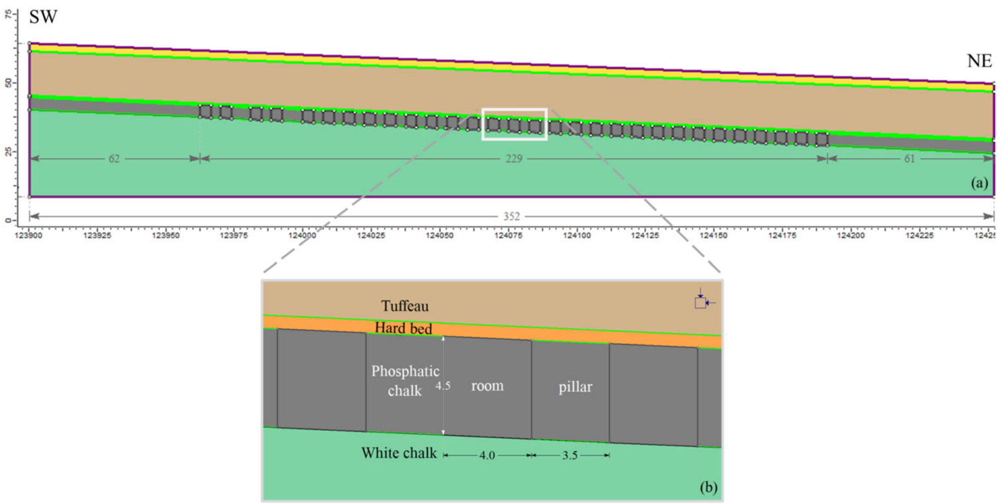

Figure 7. SW-NE cross-section of the numerical model: (a) general view showing the geometry and the different lithotypes and (b) close view on the rooms and pillars dimensions.

stress distribution and displacements. In addition as an indicator of the rock mass stability, the strength factor was estimated by dividing the rock strength (based on the M-C criteria) by the induced stress at every point in the mesh. In RS2, the strength factor can be considered as 3D since all three principal stresses have an influence on it (Rocscience, 2015). Later on, the obtained results were discussed.

Model geometry

First, using the GEMS software, a 3D geometrical model of the footwall and the hangingwall of the quarry and the boundaries between the different geological formations has been developed. For that purpose, all available mining maps developed after the end of the active mining operations that contain the dimensions of the rooms and pillars in the mined-out area were used. From that 3D model, a representative 352 m wide and 56 m high NE-SW cross-section of the quarry was created (Figure 7(a)). To minimise the boundary effects, the limits of the cross-section are much larger than the underground excavations. The thickness of the formations overlying the mined out phosphatic chalk is considered to be 19 m in total, and their bounding surfaces are dipping to the North. The geometrical model of the openings consists of 29 pillars with 3.5 m width (w/h ratio approximately equal to 0.8) and 30 rooms which width and height are 4.0 m and 4.5 m, respectively (Figure 7(b)).

Input parameters

The prevention of possible scale effect, due to the relatively small size sample used for mechanical characterisation of the studied lithotypes, requests the laboratory results to be adjusted using Geological Strength Index (GSI) (Hoek et al., 1998; Marinos & Hoek, 2000). On the other hand, according to Marinos et al. (2007), the GSI is not applicable for a very soft rock mass with no or few discontinuities. In such conditions, the rock mass can be considered as an intact with mechanical properties given by the laboratory test (Descamps et al., 2019; Perrotti et al., 2018, 2019). As the strength of both types of chalk studied was assessed as very low to low (ISRM, 2014) and the in situ structural survey indicated that they are rarely jointed, their laboratory mechanical properties were applied in the modeling without any corrections. In contrast, the field surveys and the values for the compressive strength of the hardground samples show that the upscaling approach is applicable for it and thus the GSI equal to 60 was assigned.

Since the stability analyses in a 2D plane strain surface would underestimate the load on the pillars at the transverse excavations, the effect of the extra ground load was simulated by increasing of the unit weight of the overlying layers (Deiveris & Bernardos, 2017; Descamps et al., 2019; Mohanty & Vandergrift, 2012). Thus, the pillar stress was computed in a 2D plane strain model (Equation (1)) and in a 3D case with square pillars (Equation (2)) and the ratio of 2.3 was obtained. It was applied as a coefficient for increasing the weight of the lithotypes overlying the pillars (Tables 1 and 2). The rocks are defined as isotropic plastic materials. No triaxial tests were conducted at this stage of the study, but chalk is known to exhibit plastic behavior under confinement (Schroeder, 2002). Therefore, an elastic perfectly plastic constitutive model characterised by the Mohr-Coulomb criterion was used in the simulations. Restrictions of the vertical and horizontal displacement were specified along the bottom of the model, while the lateral sides were restricted of the horizontal displacement. Free displacements were allowed along the ground surface.

$$\sigma_{v,2D} = \frac{F_{2D}}{L_p} = \frac{\gamma(L_c + L_P)\ H}{L_p} \tag{1}$$

$$\sigma_{v,3D} = \frac{F_{3D}}{L_p^2} = \frac{\gamma(L_c + L_P)^2\ H}{L_p^2} \tag{2}$$

where: L_c and L_p, width of the room and pillar respectively (m); H, thickness of the overlying layers (m).

Results

Initially, the model was run for the dry rock mass conditions, in a regime of increased unit weight, intact rock mass for the chalks and GSI = 60 for the hardground (Table 1). The results reveal that after the excavation the underground openings are overall stable. Only in part of the hangingwall, those formed by hardground, limited plastic zones in a scale of 25–30 cm are developed (Figure 8(a)). These zones are result of shearing and tensile yielded stresses. According to this model, the tensile stress at the roof of the rooms ranges in the relatively narrow interval from 0.2 to 0.3 MPa reaching its maximum in the middle of the rooms. At these places, the strength factor is less than 1 that could initiate a local roof failure (Figure 8(b)). At the same time in the pillar corners the maximum vertical stress reaches 6.74 MPa, which could lead to local failure of these overstressed zones.

In terms of vertical displacement, this model shows that its values at the hangingwall of the openings reach 1.2 cm while at the footwall it is 0.5 cm. At the ground surface, this displacement is 3.3 cm that, probably, could be a result of a subsidence process.

The second model was run for water-saturated rock mass conditions (Table 2). In this case, the cavities are still stable. In comparison with the dry conditions model, the height of the plastic zone here increases

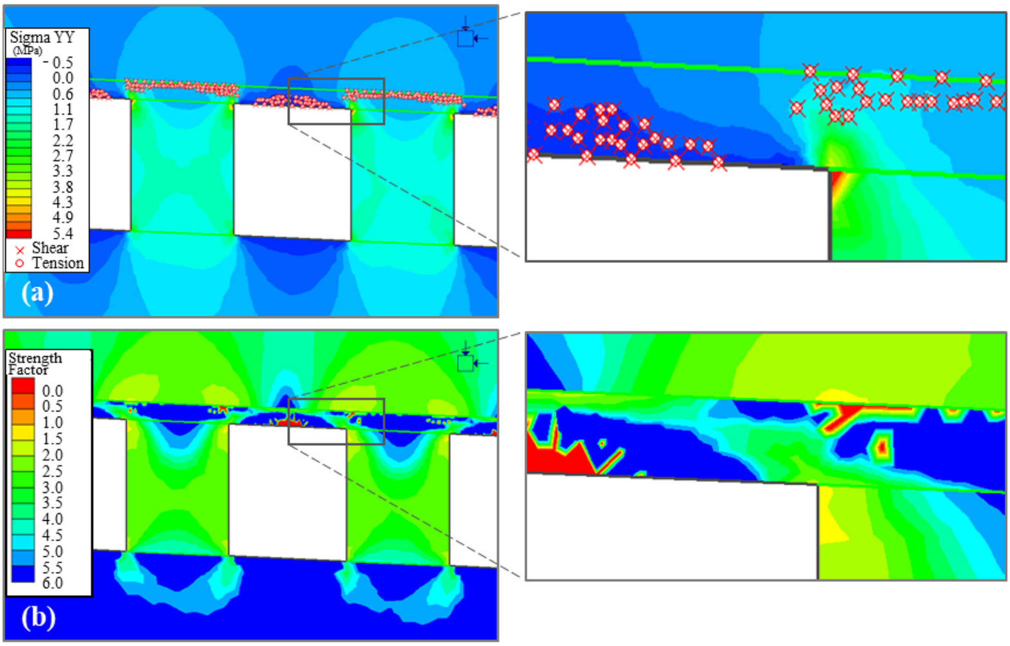

Figure 8. Results of the numerical model in dry conditions with (a) vertical stress distribution with yielded elements and (b) strength factor.

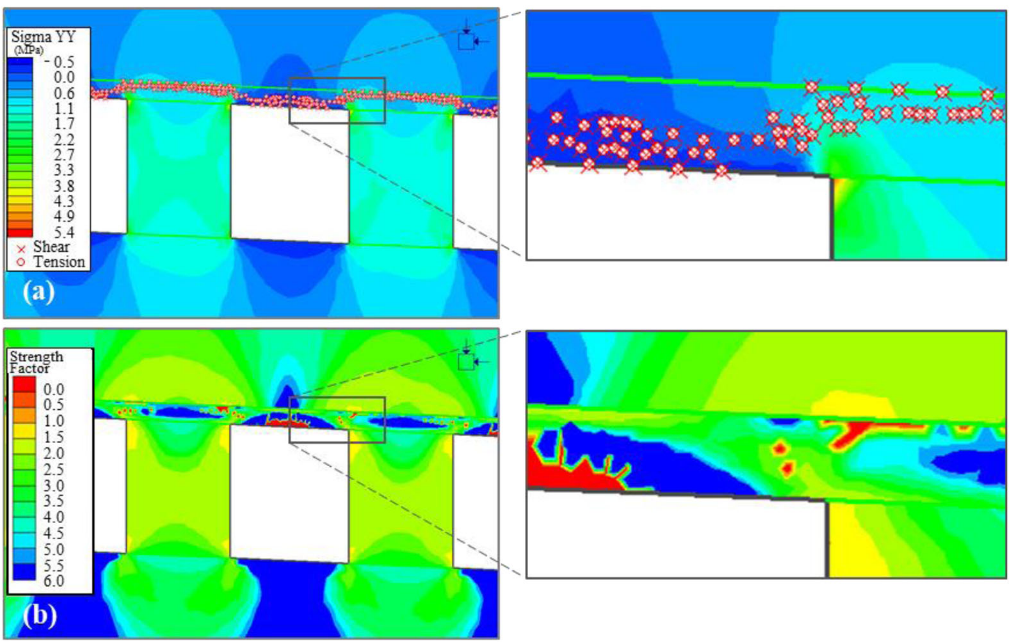

Figure 9. Results of the numerical model in saturated conditions with (a) vertical stress distribution with yielded elements and (b) strength factor.

to 35 cm. The distribution of the vertical stress demonstrates the same pattern as in the first model in the same magnitude (Figure 9(a)). At the corners of the pillars, the values of the vertical stress reached 5.4 MPa. All these values indicate instability in those parts of the hangingwall of the rooms that could lead to a partial failure (Figure 9(b)). This could be connected with the mechanical degradation effect of

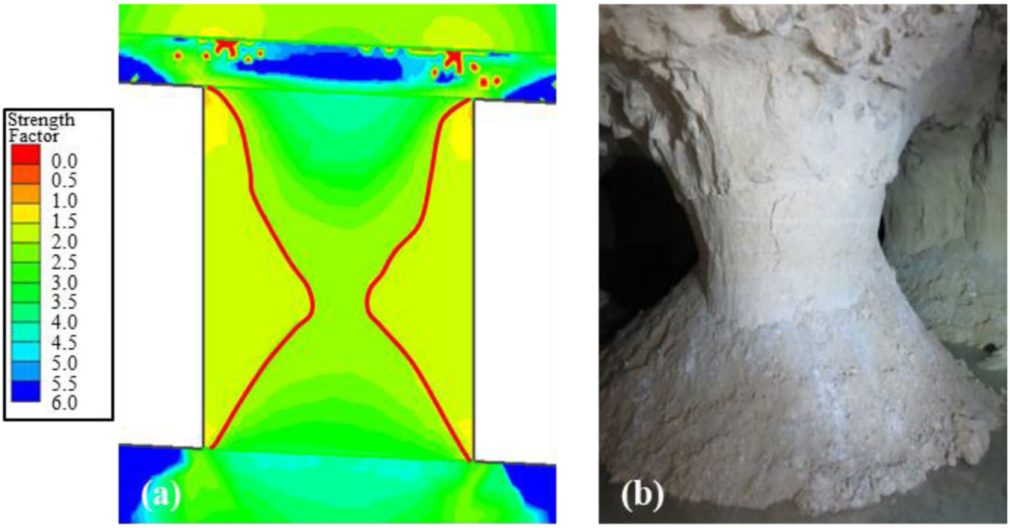

Figure 10. Comparison between the numerical model results and the in situ observations on a single pillar: (a) strength factor plot for a single pillar in water-saturated conditions and (b) spalling pillar in the transitional zone between the dry and flooded parts of the quarry.

the water (Ciantia et al., 2015) which results in decreasing in the rock strength values. Indications of such processes were found during the in situ observations when in the transitional zone of the quarry collapsed from the hangingwall blocks were identified.

In terms of calculated displacements, in this model they are greater than in the dry rock mass model with amplitude ranging from 1.5 cm around the hangingwall to 1.1 cm at the footwall. Here, it has to be noted that in both models the strength factor below and along the boundary between the hardground and the tuffeau materials is less than 1 that probably is caused by the significant difference in the stiffness of these rocks.

Discussion

The verification of the conceptual model was done by comparison of its results with the current state of the rock mass in the accessible part of the quarry. For instance, the range of the strength factor in the hangingwall in the center of the rooms (0–0.5) corresponds well to the observed roof fall. These failures could be connected with the induced tensile stress that leads to development of extension cracks in the hardground level. Later, due to shearing, this could lead to restricted as area failure.

Another process predicted by the modeling is the possible spalling of the pillars documented in some parts of the studied area during the in situ observations (Figure 10(b)). Its development probably is connected with the decrease of the strength factor in the outer part of the pillars (Figure 10(a), red contour). Although its values here are still too high (between 1.5 and 2); this could be interpreted as a sign of potential instability. It should be noted that currently the parameters of the model do not take into account factors like weathering, drying-saturation cycles, or even freeze-thaw cycles. All they could affect the mechanical behavior of the rock and therefore to lead to a decrease in the strength factor value.

Another verification step of the obtained numerical results was done by applying of the tributary area method in calculation of the load on the pillars. The estimated by this method value of 1.13 MPa is in agreement with the computations of the vertical stress distribution (from 1 to 1.5 MPa) in the model.

The presented models provide new data on the role of factors such as extraction ratios and position of specific lithology levels in the mining profile on the overall stability of the rooms and pillars and the scale of the failures. Analyzing large areas of mining operations with undersized pillars and high extraction ratios, Franck et al. (2019) consider that such a factor could be the presence of a rigid layer in the overburden of the quarry. Mottahed and Szeki (1981) suggested that the cause of sudden collapse of room and pillar works is the uncontrolled energy release by the strong strata above the pillars in the roof. Later, Fairhurst et al. (2003) showed that the pillars and the immediate roof continue to deform and the load is transferred to the resistant bed up to certain limits, at which a sudden shear collapse occurs.

In the Malogne quarry, these factors are represented by the appearance of the hardground level in the hangingwall and the relatively high extraction ratio ranging approximately between 60% and 70%. However, in the proposed model, a total failure was not predicted. Only local instabilities with a semi-arch shape at the roof of the rooms were obtained in the model.

Regardless of the currently achieved results, the simulations do not explain all the observed failure types. Although these models suggest the observed spalling and failure of the roof in the quarry the effect of some additional factors has not been considered. Based on different case studies, Hudson (1993) considered the presence of hydrostatic pressure as an important factor of roof collapse. According to Bekendam (1998) and Renaud et al. (2019), the time dependency after the end of the mining works, the presence of discontinuities and heavy rains resulting in changes of the water saturation degree are recognised as essential sudden failure mechanisms factors. They were not included in the proposed model at the current state of the study, which, probably could explain the obtained result for the overall stability of the underground openings, but certainly they will be the subject of future research.

Conclusions

To investigate the rock mass response of the underground openings in the shallow situated Malogne chalk quarry, a 2D Finite Element Model was created. For determining the input parameters of the numerical simulations laboratory tests in dry and water-saturated conditions were conducted on the main lithotypes outcropped in the quarry. The model is based on several hypotheses, including a generalisation of the rooms and pillars geometry, increasing of the unit weight of the overlying beds, intact rock mass properties of the chalk and reduced by GSI $= 60$ for the hardground.

The numerical simulations assume the overall stability of the underground opening and only local zones of failure were identified. These failures are caused by tensile and shear stresses induced after the rooms opening. The recognised failure mechanisms are connected with block collapse from the roofs and spalling pillar development, both confirmed by in situ observations. The intensity of such processes slightly increasing towards the zones with water-saturated materials.

The proposed models for the behavior of the underground openings in the Malogne quarry corresponds relatively well to the current state of the cavities.

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