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DESIGN OF THE SHAFT LINING AND SHAFT STATIONS FOR DEEP POLYMETALLIC ORE DEPOSITS: VICTORIA MINE CASE STUDY

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Abstract: In order to properly design a shaft it is necessary to acquire full information about the rock mass in the exploration area. It is especially crucial in the case of the deposit of an unusual vertical intrusion shape, occurring at a great depth. Such a situation implies that the shaft lining design must take into consideration not only the geomechanical properties of the rock mass but also the virgin stresses (often having significant values). In this paper, the methodology of the shaft lining and shaft station lining design for a deep shaft is presented based on the Victoria Mine located in Canada. Taking into consideration the geological structure as well as the results of the laboratory tests, the properties of the rock mass were derived. Next, the numerical calculation was performed based on the elasto-plastic model of the rock mass. The numerical analysis consisted of simulation of the multistage technology of the shaft excavation and lining execution. This allowed to estimate the forces in rock bolts of the temporary ground support as well as stresses in the final concrete lining of the shaft.

Keywords: *shaft lining, numerical methods, concrete, Sudbury Basin*

INTRODUCTION

Thorough geological exploration and rock mass identification is a necessary condition for a proper shaft lining design. Detailed information regarding the geological structure of the region around the deposit is crucial for, e.g. planning of the scope of

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laboratory investigation for the intact rock samples. Precise exploration gives the assurance that the construction will be stable and safe. With the increasing depth of the deposit, the virgin stresses, associated with the weight as well as the seismic activity of the rock mass, are becoming larger (Butra *et al.*). Therefore, careful geomechanical analysis is essential in order to determine the stresses in the designed construction.

In this paper, the methodology of the shaft lining and shaft station lining design for a deep shaft is presented, based on the Victoria Mine case study. Victoria Mine is located approximately 32 km southwest of Sudbury, Ontario, Canada. Research completed in this area shows large deposits of nickel, copper and precious metals. Copper and nickel sulphide mineralization was discovered on the Victoria property in 1886. Exploration is a challenge due to its depth – it starts at 1,000 metres from the surface and remains open at 2,000 metres. The depth of the planned production shaft is assumed to reach 1,860 m. The shaft includes ten shaft stations. This paper presents the results of the 2D numerical analysis of the shaft and shaft stations lining in Victoria Mine. Taking into consideration the geological structure as well as the results of the laboratory tests the properties of the rock mass are derived. Next, the numerical calculation is performed based on the elasto-plastic model of the rock mass. The numerical analysis consists of simulation of the multistage technology of the shaft excavation and lining execution. As a result, the forces in rock bolts of the ground support as well as the stresses in the final concrete lining of the shaft are estimated.

The article is constructed as follows. In the next section, the methods for shaft and shaft station lining selection are presented. Polish regulations and guidelines are characterized and compared to the design methodology used in Canada, USA etc. Some common empirical formulas are provided for quick estimation of the rock bolt length. The following chapter deals with the Victoria Mine case study, namely the application of the numerical analysis for the design of the shaft and shaft station concrete lining. Geological information about the Sudbury region as well as the results from the shaft pilot hole survey are presented. The methodology and input parameters for the numerical calculation, performed with the use of FEM Phase^2 are introduced. Finally, the results of the analysis are presented including the stresses in the shaft lining as well as the forces in the rock bolts. Final conclusions end the paper.

DESIGN METHODS FOR SHAFT CONCRETE LINING AND SHAFT STATIONS

In many countries there are certain regulations that standardize the mine design. In Poland, the Regulation of Ministry for Economy is applied (2002), which defines the requirements that the shaft anchored lining has to fulfil in the specified conditions. There are additional standards that are in use, i.e. PN-G-05016 (1997) and PN-G-05015 (1997), which determine the methods for load calculation as well as give the guidelines for concrete lining design. The thickness of the necessary concrete lining, according to PN-G-05015, can be estimated based on the following formula:

$$d_b = R_w \left(\sqrt{\frac{f_{cd}}{f_{cd} - m \cdot p \cdot \sqrt{3}}} - 1 \right) \tag{1}$$

where R_w represents the internal radius of the shaft lining, f_{cd} indicates the design compressive strength of concrete, m is the correction factor depending on the type of rock and p symbolizes the horizontal design load on the shaft lining calculated according to PN-G-05016, based on e.g. the weight as well as the physical properties of the rock mass.

Unfortunately, the standard methods of concrete lining design in Poland are somewhat crude due to the fact that the formulas provided take into account only the virgin stresses resulting from the weight of the rock mass while completely ignoring the magnitude and directions of other types of stresses, e.g. tectonic ones. It is obvious that in the situation of deep shafts, exceeding 1000 m, neglecting this kind of stresses might have catastrophic consequences for the structure. Of particular interest are not only the values, but also the directions and asymmetry of the stress tensor components. Thus, in complicated cases, the complex geomechanical analysis is applied, commonly with the use of numerical methods, allowing modelling of the rock mass and the structure under investigation, taking into consideration all the significant factors.

Nevertheless, it is beyond any doubt, that polish regulations and standards are adjusted to the geological conditions that are present in Poland for the shafts not deeper than 1200 m. In other countries, e.g. Canada, Republic of South Africa, Australia, USA, the design methodology for shaft lining is based on the mining traditions and experience of those regions.

Design methods for the shaft lining and shaft stations can be divided into three main groups, namely: analytical, numerical and empirical ones. Analytical methods are the deterministic solutions of closed form. They involve application of generalized models which do not take into account particular, sometimes unusual, rock mass conditions. The main advantage of analytical methods is that they give a possibility to understand the mechanism and physical effects of the occurring phenomena. On the other hand, empirical methods are the formulas determined, in general, based on the experience of different mines with respect to the laboratory and *in-situ* test results. In literature, there is a number of empirical formulas, developed over the years, which give the possibility to estimate, e.g. the rock bolt length based on the span of the opening. One of them is Stillborg equation (Stillborg, 1994):

$$L = 1.4 + 0.184a \tag{2}$$

where L indicates the necessary rock bolt length and a is the span of the opening.

Barton *et al.* (1980) suggested Eq. (3) for rock bolt length estimation as follows:

$$L = 2 + \frac{0.15a}{ESR} \tag{3}$$

where ESR is the excavation support ratio, ranging from 1.6 for permanent mine openings up to 3.0 to 5.0 for temporary mine openings. For vertical shafts with circular section, it is equal approximately 2.0. For the design of roof rock bolts, the span or width of the opening are considered in Eq. (3), while for the wall supports, the height of the opening should be taken into account.

Alexander & Hosking (1971) gave a different formula for the rock bolt length:

$$L = 1.82 + 0.0004B^2 \tag{4}$$

where B is the span of the opening. It is noticeable that each of the formulas determines a different minimum rock bolt length, i.e. 1.4 m, 2.0 m and 1.82 m, respectively.

One of the commonly applied empirical approaches in mining engineering is the use of rock mass classifications. It is a very effective and practical design method for engineers. The classifications take into consideration the so-called "scale effect" of the rock mass. The laboratory results of rocks mechanical properties are always investigated on the, relatively small, intact rock samples. As a result, the influence of the discontinuities, which has a significant impact on the rock mass strength, is neglected. Therefore, it is crucial to decrease the laboratory test values in order to obtain a realistic strength of the rock mass. This procedure is incorporated in the methodology of the rock mass classifications.

First rock mass classification was formulated by Terzaghi in 1946 with the use of rock loads. In 1970 Deere et al. correlated Terzaghi's rock loads (Terzaghi, 1946) with RQD (Rock Quality Designation) values and approximate fracture spacing (Deere, 1970). This resulted in the development of a new classification of the rock mass, RQD, giving the possibility to describe the character of the rock mass. Furthermore, Deere published ground support recommendation based on RQD. RQD is a basis of other classifications, i.e. Geomechanics Classification RMR by Bieniawski, published in 1979 (Bieniawski, 1979). It uses several parameters to assess the rock mass quality, namely: uniaxial compressive strength of rock, RQD, spacing, condition and orientation of the discontinuities and groundwater condition. The most detailed rock mass classification system is the NGI Q-System developed by Barton, et al. in 1974 (Barton, 1974). The value of the rock quality index Q can be estimated as follows:

$$Q = \frac{RQD}{J_n} \frac{J_r}{J_a} \frac{J_w}{SRF} \tag{5}$$

where J_n is the joint set number, J_r indicates the joint roughness number, J_a is the joint alteration number, J_w is the joint water reduction factor, whereas *SRF* is the stress reduction factor.

In 1980, Hoek and Brown (1980a, 1980b) proposed a new method for obtaining estimates of the strength of jointed rock masses, based upon an assessment of the interlocking of rock blocks and the condition of the surfaces between these blocks. This method was modified over the years and lead to the development of a new classification called the Geological Strength Index (GSI). GSI coupled with Hoek and Brown non-linear failure criterion, presented later in this paper, are valuable tools in geomechanical analysis. In addition, the GSI system was found useful as the reliable input data into numerical analysis and closed form solutions for designing tunnels, slopes and foundations in rocks (Marinos & Hoek, 2005), which gives it a significant advantage over other classifications.

Even though the rock mass classifications are very useful from practical point of view, in more complex geological conditions, they might not be sufficient. Numerical methods give the biggest possibilities in terms of concrete lining design. They incorporate nonlinear analysis, anisotropy and discontinuities of the rock mass, complicated geometry of the problem and troublesome geological profile, if necessary. Nevertheless, they are often time-consuming and, as a consequence – expensive. Furthermore, the code has to be verified by comparison of the obtained results to the similar cases, existing mines etc. In spite of some drawbacks, numerical methods are developing very fast and they have a lot of potential. This paper presents an application of the numerical calculation for the ground support design. In addition, GSI classification is applied as an input parameter for the geomechanical analysis.

GROUND SUPPORT NUMERICAL ANALYSIS: VICTORIA MINE CASE STUDY

Victoria Mine site is located around 30 km from Sudbury in Ontario, Canada. Copper and nickel sulphide mineralization was discovered on the Victoria Property in 1886 (Farrow *et al.*, 2001). The Victoria property is situated at the junction of the Sudbury Igneous Complex (SIC) and the Worthington quartz diorite Offset Dyke. Both intrude sheared and metamorphosed mafic volcanic and sedimentary rocks. Zones of Sudbury Breccia occur throughout the property as discontinuous lenses. Late quartz diabase and olivine diabase dykes cross-cut all lithologies. The Cu-Ni sulphide mineralization at the Victoria property is characterized by a group of irregular lenses of chalcopyrite, pentlandite and pyrrhotite (Farrow *et al.*, 2001). The ore production and shaft sinking started in 1900. In 1918, a three-compartment production shaft was sunk to a depth of 918 m. In the beginning of the mine operation, the ore mineralization reached 2.99% Cu and 2.12% Ni. In 1923 the mine was flooded after its closing. It was not active for the next decades, until 1973, when the production resumed after dewatering in 1969. A total of 649,000 tons of ore, averaging 1.26% Cu, 0.83% Ni, was produced until 1978 when the mine was, again, closed and flooded. The site of the old shaft has been fenced off and grassed over (Farrow *et al.*, 2001).

Since 2011, there have been plans to build a new Victoria mine in order to access the orebody. For this purpose, a new shaft has been designed, i.e. 7.6 meter diameter, circular, concrete lined shaft, which will be sunk to a depth of 1,861 meters with shaft stations excavated every 300 m. In this section of the paper, a detailed 2D numerical analysis for the Victoria Shaft concrete lining as well as one of the shaft stations is presented.

GEOLOGICAL CONDITIONS

The origin of the Victoria Mine deposit is a controversial topic, still widely analysed by the researchers. The most recent theory states that the geological structure of Sudbury Basin in Ontario, Canada is the result of a meteor impact, which occurred approximately 1.95 billion years ago. The Sudbury Crater has a diameter of 130 km and it is a third, confirmed, crater in the world in terms of size. In addition, it is one of the most frequently analysed terrestrial impact craters (e.g. Dietz, 1964; French, 1967; Grieve *et al.*, 1991; Pope *et al.*, 2004; Przybylski & Badura, 2004; Petrus *et al.*, 2014). There is an ongoing discussion regarding the initial size of the crater, due to the impact erosion and tectonic deformations, which modified its shape. Post-impact deformations and the erosion of the Sudbury impact zone resulted in creating an elliptic basin, with impact melt rocks in the form of elliptic rings, visible on the surface, with the dimensions of 60×30 km, being a part of the Sudbury Igneous Complex (Fig. 1).

The geological conditions for the shaft design were evaluated based on the information obtained from the Shaft Pilot Hole located in the centre of the planned structure. Nine types of rocks were identified in the investigated Shaft Pilot Hole, namely: Quartzite, Diabase, Metasediment, Rhyolite, Metabreccia, Metacrystal Gabbro, Metabasalt, Metagabbro, Quartz-Diabase (Report, 2012).

The depth of the particular rock layers and their thickness are presented in Fig. 1. The quality of the rock mass was evaluated based on the standard, commonly used, indexes characterizing the structural features of the rock (e.g. discontinuities, joints, weathering etc.). In general, the rock mass was evaluated as a good quality one. In addition to the rock mass structural assessment, the intact rock strength was tested. Twenty-one samples representative of the various rock units were selected. The tests were conducted at Queen's University in Kingston, Ontario. The following parameters were examined: Uniaxial Compressive Strength (UCS), Young's Modulus E, Poisson's ratio ν, tensile strength, density of the rock. Most of the rock samples showed the *UCS* above 100 MPa. The Young Modulus values were between 17 GPa and 62 GPa. The lowest results were obtained for the Quartz-Diabase sample from the depth interval of 1,654.0-1,654.2 m (*UCS* = 46 MPa, *E* = 17,6 MPa) and for Metagabbro sample from the depth interval of 1,432.0-1,432.3 m (*UCS* = 48.2 MPa, *E* = 29). The weakest layers were chosen for the numerical analysis.

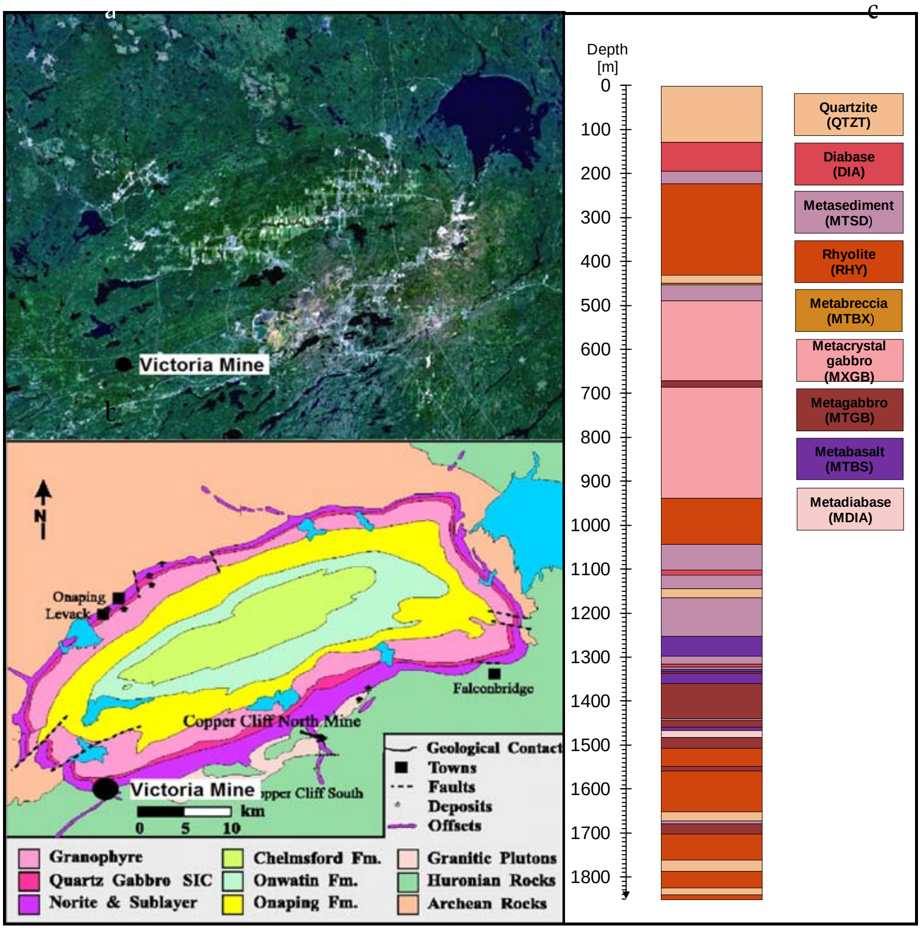


Fig. 1. a) Satellite image of the Sudbury crater (NASA), b) Simplified geology of the Sudbury basin and the surrounding area (emg.geoscienceworld.org), c) geological profile based on data obtained from the Victoria shaft pilot hole

Testing campaign consisted of estimation of the *in-situ* stresses magnitude and orientation. In many Canadian hard rock mines, ground stresses are one of the major concerns due to potential ground failures and rock bursts (Herget & Arjang, 1997). In order to avoid unnecessary risk, the mini-frac *in-situ* stress testing was performed. A total of twenty-four borehole mini-frac hydraulic fracturing *in-situ* stress tests were attempted in borehole at depths between 371 m and 745 m (Report, 2013). However, only eight tests gave reliable results in the range of 388 m and 689 m depth. The stress regime in the vicinity of the borehole was estimated as follows: vertical stresses σ_V = 0.0275 MPa/m, minor horizontal stresses σ_{hmin} = 0.0467 MPa/m and major horizontal stresses σ_{hmax} = 0.0824 to 0.110 MPa/m. It is surprising that the difference between horizontal stresses is extremely high – the minor stress value is around 2-3 times lower than the major one. Due to the large scatter and significantly low number of reliable data, the results of the tests cannot be considered trustworthy. Furthermore, the tests cover only part of the shaft depth range, therefore, they are not taken into account in the numerical analysis. However, a significant phenomenon was indicated by the tests results, namely: the asymmetry and relatively high values of the horizontal stresses which may have a substantial impact on the behaviour of the rock mass.

METHODOLOGY AND INPUT PARAMETERS

The principal method of analysis chosen to assist in predicting the behaviour of the shaft lining was a Finite Element Method (FEM). There are a number of shaft lining calculation examples in literature (e.g. Jia *et al.*, 2013; Chen *et al.*, 2014), however this article presents a method which is less detailed but more engineering-friendly. The software used for the FE analysis was Phase^2, a two dimensional commercially available computer package, which has been specifically developed for the analysis of underground facilities (RocScience, 2015).

The full methodology of the numerical analysis is presented in Figure 2. It consists of formulating the assumptions and gathering the input data, followed by the calculations and results. The initial phase of the analysis consists of calculation of the rock mass parameters using the laboratory results for the intact rock. For this purpose, the well-known Hoek-Brown criterion has been applied formulated as follows (Hoek *et al.*, 2002):

$$\sigma_1' = \sigma_3' + \sigma_{ci}\left(m\frac{\sigma_3'}{\sigma_{ci}} + s\right)^a \tag{6}$$

where σ_1' and σ_3' denote the major and minor effective principal stress at failure, respectively, σ_{ci} is the uniaxial compressive strength of the intact rock sample, whereas m_b, a and s denote the constants which can be derived according to the formulas below:

$$m_b = m_i \exp\left(\frac{GSI-100}{28-14D}\right) \tag{7}$$

$$s = \exp\left(\frac{GSI - 100}{9 - 3D}\right) \tag{8}$$

$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-\frac{GSI}{15}} - e^{-\frac{20}{3}}\right) \tag{9}$$

where m_i is the constant for intact rock, D is the disturbance factor depending on the degree of disturbance to which the rock mass has been subjected by blast damage and stress relaxation. D varies from 0 for undisturbed rock masses to 1 for very disturbed rock masses. *GSI* denotes Geological Strength Index introduced by Hoek, Wood and Shah (1992) and Hoek, Kaiser and Bawden (1995).

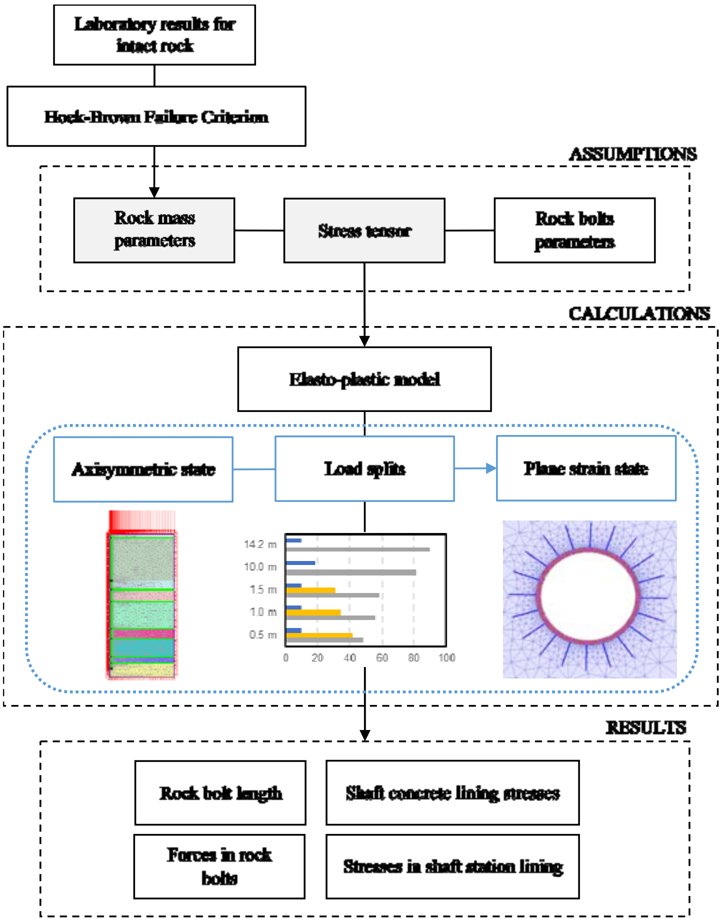


Fig. 2. Numerical analysis methodology

It can be expressed as:

$$GSI = 9\ln Q^{'} + 44 \tag{10}$$

where *Q'* is the Modified Rock Tunnelling Quality Index that can be calculated according to the following formula which is the modification of Eq. (5):

$$Q^{'} = \frac{RQD}{J_n}\frac{J_r}{J_a} \tag{11}$$

As it was mentioned earlier, the calculations for both shaft and shaft station are performed for the weakest layers of rock which have been selected based on the laboratory tests results of intact rock properties. The final design parameters calculated with the use of Hoek-Brown criterion are listed in Table 1 with assigned type of rock and depth of the layer bottom.

Tab. 1 Design parameters of the rock mass

Rock type	Depth [m]	*UCS* [MPa]	*E* [GPa]	Cohesion [MPa]	φ [°]
Metagabbro	1,440	35.5	35.8	10.46	48.1
Metagabbro	1,557	29.1	24.9	8.60	48.1
Rhyolite	1,653	18.2	22.0	12.40	41.1
Quartz-diabase	1,671	19.9	20.0	10.20	39.0
Metasediment	1,675.5	12.8	19.4	7.40	39.4
Metagabbro	1,701	29.1	24.9	8.60	48.1
Rhyolite	1,761	18.2	22.0	12.40	41.1
Quartz-diabase	1,785	33.3	30.3	17.20	39.1
Rhyolite	1,824	15.9	23.8	10.80	41.1
Quartz-diabase	1,839	33.4	30.3	17.20	39.1
Rhyolite	1,851	15.9	23.8	10.80	41.1

In addition, the state of stress needed to be assumed. The pre-mining far-field stress regime used in the Sudbury Basin is derived from the 1980s measurements due to the absence or inconclusive recent results. In general, the major principal stress is horizontal and trending E-W whereas the minor principal stress is vertical (Trifu & Suorineni, 2009). The assumed relations are presented below:

$$\sigma_1 = 10.9 + 0.0407Z \tag{12}$$

$$\sigma_2 = 8.7 + 0.0326Z \tag{13}$$

$$\sigma_3 = 0.029Z \tag{14}$$

where σ_1 the maximal horizontal stress, σ_2 is the minimal horizontal stress, σ_3 is the vertical stress and Z depicts the depth (in meters). The stress tensor components versus depth are shown in Figure 3. Red line represents the depth below which the calculation was performed.

The analysis, both for the shaft and shaft station, consists of two main stages, i.e. the axisymmetric state calculation and plane strain modelling. In both models, the multiphase technology of shaft excavation and lining installation is simulated. The axisymmetric model gives the possibility to estimate an average plastic zone range for each of the rock mass layers. In addition, the values of the horizontal displacements that have been completed until the temporary and permanent ground support is installed as well as the total displacements are calculated, in this state. This results in the determination of the load split coefficients for the plane strain model. Those models are necessary in order to take into consideration the asymmetry of the horizontal stresses.

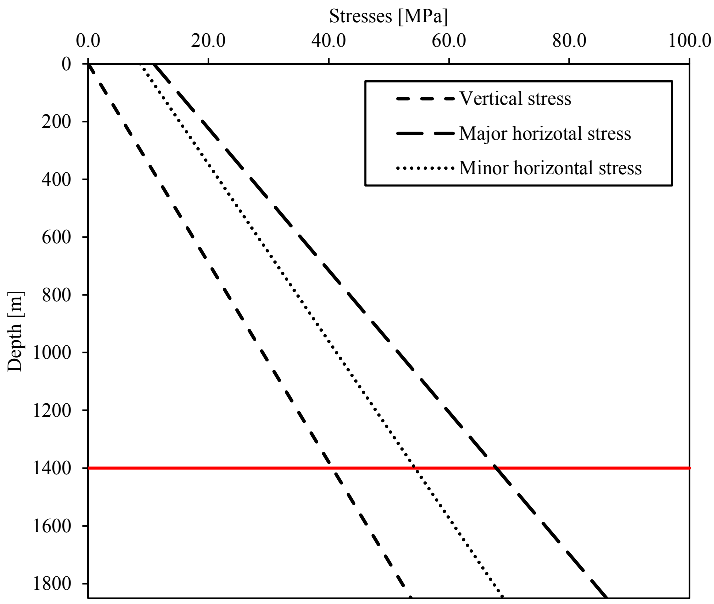


Fig. 3. Stress tensor components vs. depth (red line represents the depth below which the calculation was performed)

RESULTS AND DISCUSSION

SHAFT LINING

A three-phase axisymmetric computational model was applied for the calculation of the shaft lining in order to estimate the horizontal displacements which are being realized before installation of the ground support.

In Phase 1, the rock mass is loaded with the pressures resulting from the rock mass weight, calculated for each of the layers. The model contains ten layers of rock from the bottom of the shaft. It is assumed that the first layer from the top has elastic properties and the rest are elasto-plastic ones. This ensures that the boundary problem of elastic-plasticity belongs to the cases of confined plastic flow and numerical computation process converges.

In Phase 2, the excavation for the shaft is made until the bottom of the Quartz-Diabase layer, to depth of 1,671 m. In Phase 3, the excavation reaches the bottom of the shaft.

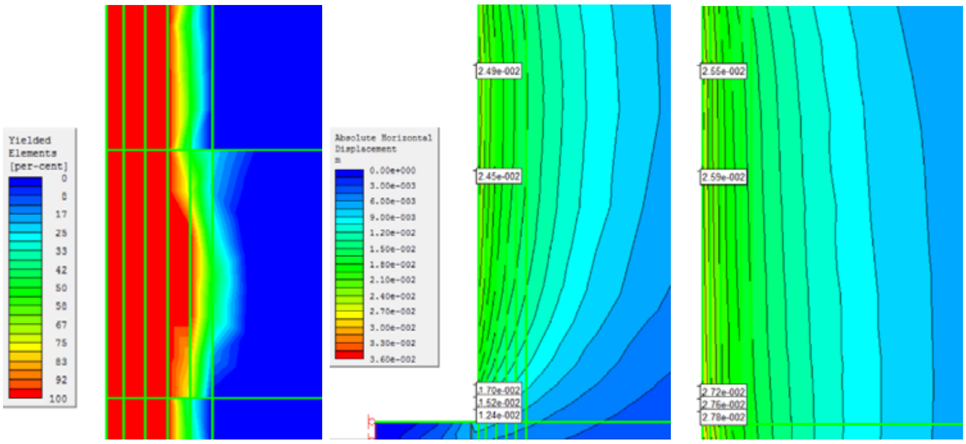


Fig. 4. Phase 1 plastic zone of the shaft wall in the Quartz-Diabase and Metasediment layer (1671m and 1675.5m, respectively) and displacement of the rock mass in Phase 2 and Phase 2

The right panel of Figure 4 shows the plastic zone range for the Quartz-Diabase and Metasediment layers. It is visible that the maximum range of the plastic zone is approximately 1.5 m. In order to have the necessary data for the rock bolt length estimation, the plastic zone length was estimated in all the rock layer under investigation. The results are listed in Table 2.

In addition, the displacements for each of the calculation phases were established. Based on those values, the percentage of the rock mass displacement before the ground support installation was determined and the load split coefficients were computed. In other words, the loading, that the shaft concrete lining and the rock bolts are subjected to after execution of the excavation, was calculated. The aforementioned loading becomes then the input parameter in the plane strain analysis.

Tab. 2 Range of the plastic zone at particular depths

Rock type	Depth [m]	Thickness [m]	Plastic zone 100% [m]	Plastic zone 5% [m]
Rhyolite	1,653	96	0.7	1.4
Quartz-diabase	1,671	18	1.2	1.8
Metasediment	1,675.5	4.5	1.5	2.4
Metagabbro	1,701	25.5	0.8	1.5
Rhyolite	1,761	60	0.7	1.4
Quartz-diabase	1,785	24	0.4	1.0
Rhyolite	1,824	39	1.1	1.8
Quartz-diabase	1,839	15	0.7	1.4
Rhyolite	1,851	12	0.9	1.6

The plane strain calculation phases are demonstrated in Figure 5. In Phase 1, the rock mass is subjected to the horizontal virgin stresses of 78.9 MPa and 63.2 MPa vertical ones resulting from its weight – 48.5 MPa (perpendicular to the plane of the drawing). The load split coefficient was assumed to be equal to 0.625.

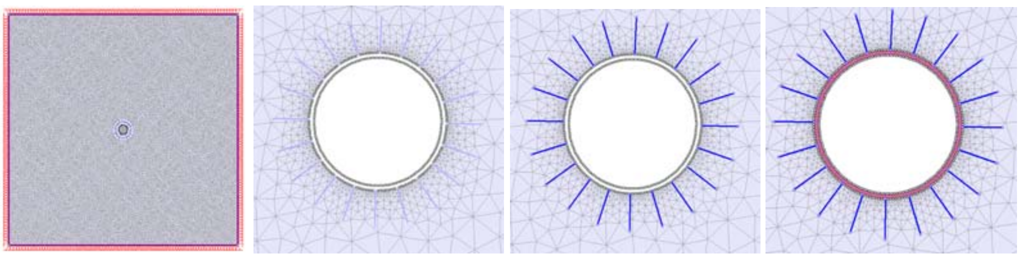
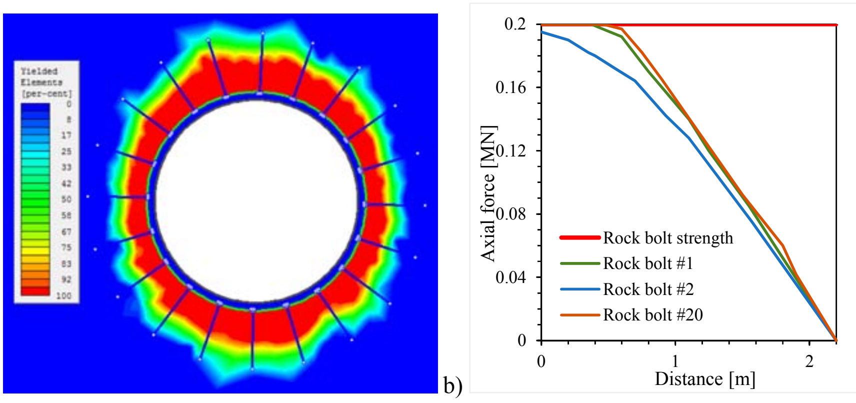


Fig. 5. Phases of the plain strain state assessment: Phase 1, Phase 2, Phase 3, Phase 4

In Phase 2, the excavation for the shaft is executed. The displacements equal to 0.625 of the virgin stresses are completed. In Phase 3, the rock bolts are installed and the virgin stresses are increased to the 0.321 of their final value. In the final phase of the plane strain model, a permanent concrete lining is assembled and the virgin stresses are equal 0.054 of their final value.

Figure 6 presents the results of the rock bolt analysis. Firstly, the end-anchored rock bolts were applied, however, the stresses were too high and the failure occurred. For this reason, the yieldable 0.2 MN rock bolts were suggested, e.g. Swellex, Cone Bolts. Swellex rock bolts are made of a welded tube folded on itself and sealed at one extremity. It is expanded using a high pressure water flow provided by a special pump.

As the anchorage principle of Swellex is a tight interlock with the borehole, corrosion can develop if it is installed in an aggressive environment (Minova, 2009). For this purpose, rock bolts with different coatings are available. The main advantage of the Swellex rock bolts is that the expansion of the bolt, inside the borehole, creates a friction and interlocking anchor, which provides support on the whole length of the borehole. The working principle of the Cone Bolts is similar to Swellex ones. They absorb the seismic energy through the action of ploughing through the resin and steel stretching. They are a better solution for the permanent ground support due to the fact that they are less prone to corrosion than Swellex rock bolts.

a)  b)

Rys. 6. a) 2.2 m yieldable rock bolts in the yield zone,
b) Axial force distribution in 2.2 m 0.2 MN yieldable rock bolts no. 1, 2 and 20

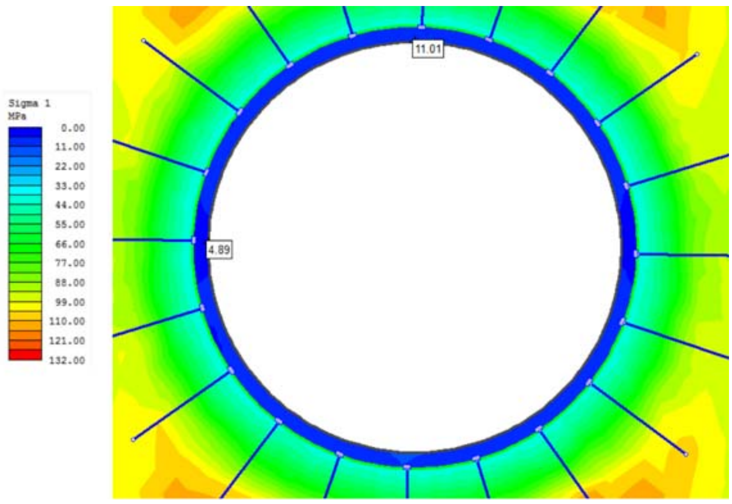


Fig. 7. Minimal and maximal hoop stresses in shaft concrete lining

In Figure 7, stresses in shaft concrete lining are presented in the final stage of the calculation. It is noticeable that the maximum stress in concrete is equal to approximately 11.0 MPa. It indicates, that failure of the shaft lining made of C30/37 concrete, is not going to occur.

SHAFT STATION GROUND SUPPORT

For the calculation of the shaft station, a 2D elasto-plastic model of the shaft station was created and the calculation has been divided into 7 modelling phases. First one contains only the calculation of the rock mass without any excavation. In the second phase, the excavation is executed to the back of the shaft station. In the third phase, the excavation is in the middle of the station height and the shaft lining is constructed. In the fourth phase, the excavation does not change but another segment of lining is added. Phase 5 assumes increase of the excavation depth below the station bottom. In the Phase 6 the shaft station tunnel is excavated. Phase 7 is the final phase, where the shaft excavation reaches shaft bottom level. All the phases are presented in Figure 8.

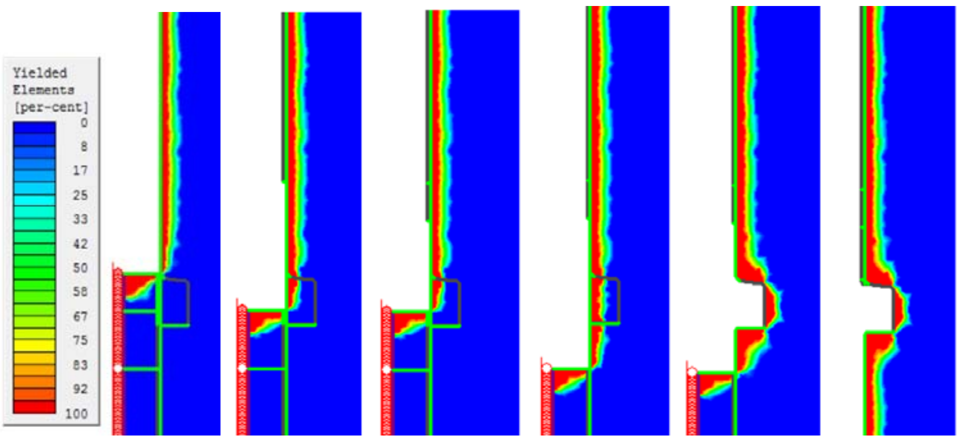


Fig. 8. Phases of the axisymmetric numerical calculation for the shaft station, starting from the left: Phase 2, Phase 3, Phase 4, Phase 5, Phase 6 and Phase 7

The displacements in different phases of the shaft station excavation are presented in Figure 9. As mentioned earlier, they were used to estimate the split load coefficients applied in the plane strain model.

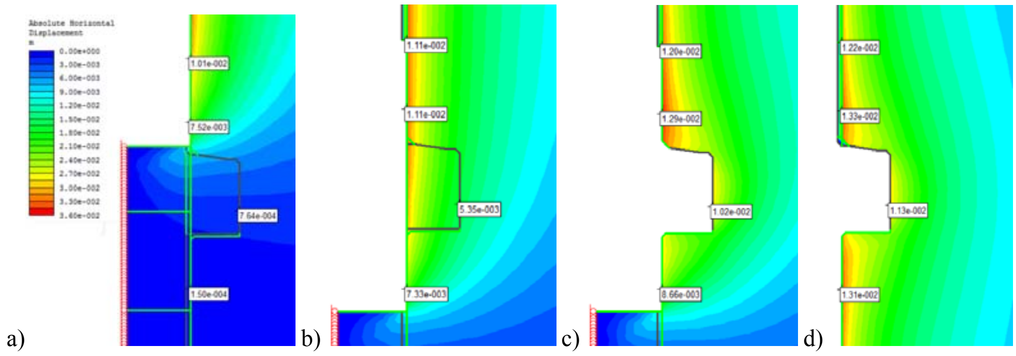


Fig. 9. Shaft station rock mass displacements in: a) Phase 1, b) Phase 4, c) Phase 5, d) Phase 6

The results of the plane strain model calculation for the rock bolts above the station are shown in Figure 10. Again, the yieldable rock bolts were assumed (5.0 m long). The failure does not occur.

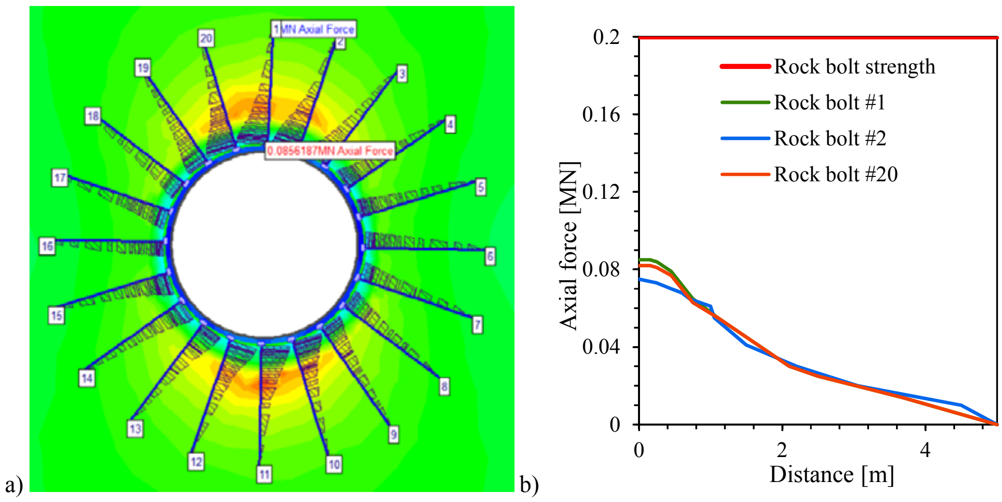


Fig. 10. a) Axial force distribution in 5 m MN yieldable rock bolts above the shaft station, b) Diagram of the axial force in rock bolts number 1, 2, and 20

In addition, the rock bolts in the shaft station were evaluated (Fig. 11). The end-anchored bolts were assumed and they proved to be sufficient in this case. It is noteworthy, that in the corners of the shaft station excavation, the concentration of the plastic zone is visible. For this purpose, the additional rock bolts were planned in those zones.

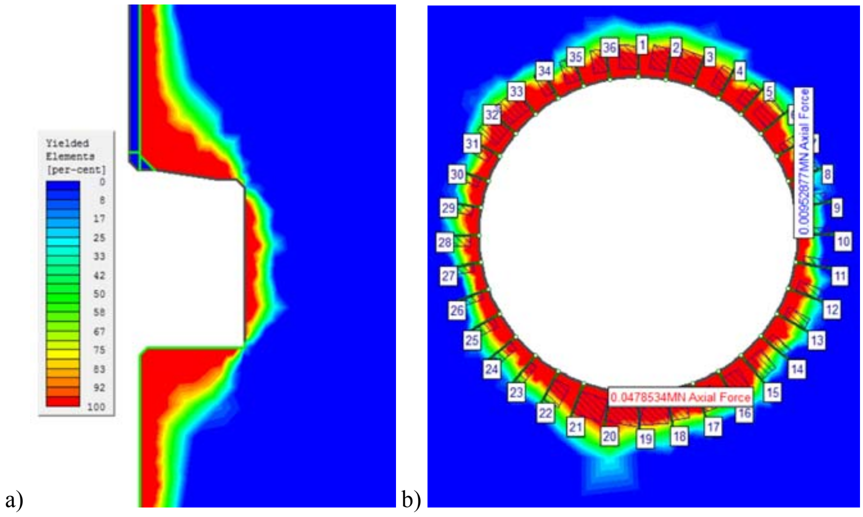


Fig. 11. a) Plastic zone in around the shaft station, b) Axial force distribution in 1,8 m end-anchored rock bolts in the shaft station walls

As for the shaft concrete lining, for the shaft station, the analysis of the stresses in concrete was performed. It can be noticed in Figure 12 that the maximum stresses are equal around 6.0 MPa, which is even less than for the shaft lining.

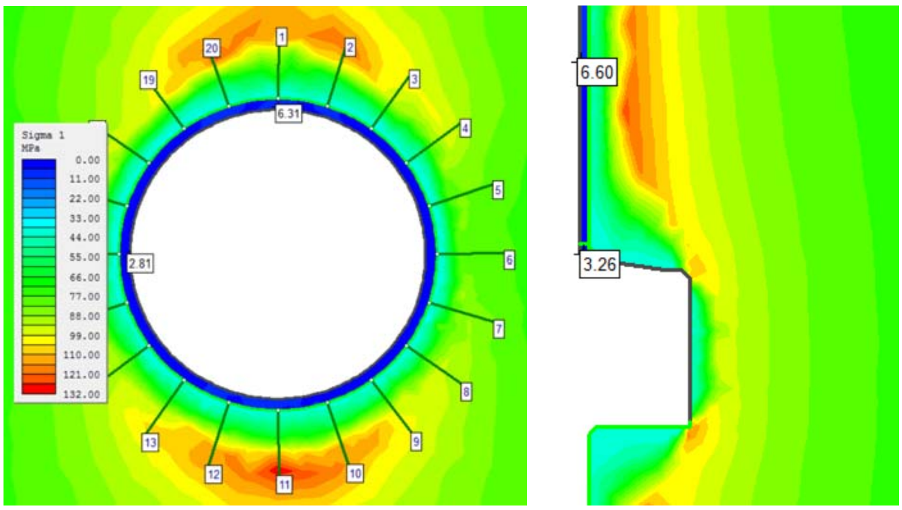


Fig. 12. Maximal and minimal hoop stresses in the shaft concrete lining in close proximity of the shaft station

SUMMARY

The article deals with the ground support selection for the shafts in deep polymetallic ore deposits. In the paper, the importance of the proper shaft lining design was underlined. Different traditional methods for shaft lining design were introduced, taking into account Polish as well as foreign conditions. In addition, a 2D numerical analysis of the shaft and shaft station stability was performed for Victoria Mine located in Canada. Phase^2 finite element method software was utilized. This software allows to perform analysis with multi-stage models. Due to this fact it is possible to simulate real progress during shaft sinking with all stages established. Input material properties were assumed based on the laboratory and *in-situ* tests performed for the Victoria Project. The elasto-plastic model of the rock mass was applied. The scope of study included calculation of the forces in the rock bolts as well as stresses in the shaft concrete lining. The plastic zone ranges were estimated in order to evaluate the rock bolts length.

The study demonstrated the advantages of numerical methods application for deep shaft design. There are no doubts that for unusual geomechanical conditions, a far more complex analysis is needed than the one based on standards. In the Victoria Mine case, the asymmetry of the horizontal stresses plays a crucial role in the design of the shaft, therefore it was necessary to evaluate the selected ground support with the use of numerical modelling. The approach proposed in the paper proved to be useful for solving engineering problems.

Furthermore, the *in-situ* stress testing did not give trustworthy results due to the fact that the number of tests was too low and the scatter was high. The empirical stress formulas give more reasonable results, however future testing programme is needed in order to confirm their compliance with the actual stress tensor occurring in the Victoria Shaft vicinity.

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DOBÓR OBUDOWY SZYBU ORAZ WLOTÓW DO SZYBU W WARUNKACH GŁĘBOKO ZALEGAJĄCYCH ZŁÓŻ RUD POLIMETALICZNYCH NA PRZYKŁADZIE SZYBU KOPALNI VICTORIA

Do procesu projektowania obudów wyrobisk udostępniających złoże wymagane jest posiadanie pełnych informacji o górotworze rejonu złożowego. Jest to szczególnie ważne w przypadku złoża zalegającego na dużej głębokości, o nietypowym kształcie pionowej inkluzji, gdzie dobór obudowy musi uwzględniać nie tylko własności geomechaniczne masywu skalnego, ale również naprężenia tektoniczne (pierwotne), niekiedy o znacznych wartościach. W niniejszym artykule zaprezentowano metodykę doboru obudowy szybu oraz obudowy wlotów dla głębokiego szybu na przykładzie Kopalni Victoria w Kanadzie. W oparciu o informacje o budowie geologicznej oraz bazując na wynikach badań prowadzonych dla potrzeb rozpoznania złoża określono własności masywu skalnego. Na ich podstawie opracowano numeryczny, sprężysto-plastyczny model górotworu. W analizie numerycznej, wykorzystującej możliwości posiadanego oprogramowania zasymulowano zgodnie z założoną technologią wieloetapowe wykonywanie wyrobiska szybowego wraz z jego obudową, co pozwoliło na oszacowanie wielkości sił w kotwach obudowy wstępnej szybu oraz wlotów podszybi, jak również naprężeń w betonie obudowy ostatecznej szybu.