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Content

Foreword

Dear Colleagues,

You are just opening the proceedings of the extended abstracts of the papers of the Fourth Traditional International Colloquium on Geomechanics and Geophysics. The richness and volume of these contributions, as well as the wide extent of participation in this meeting, confirm once again the vitality and necessity of the meeting of specialists; not just from our own region; in the fields of geomechanics and geophysics.

This year marks the 30^{th} anniversary of the foundation of the Institute of Geonics of the Academy of Sciences of the Czech Republic. This year's meeting is once again organised by this Institute in collaboration with Green Gas DPB.

Distinguished foreign specialists, who have been involved in the field of rock engineering over many years, have been invited to take part in this meeting. These specialists are Professors Jan Drzewiecki and Gregorz Mutke (GIG Katowice, Poland); Professor Yuzo Obara (Kumamoto University, Japan) and Professor Ove Stephansson (Geo Forschungs Zentrum, Germany) all of whom will present of keynote lectures. International participation is also enhanced by other foreign experts dealing not only with various aspects of geomechanics and geophysics, but also with geotechnics. Experts from Australia, Germany, Hungary, India, Japan and Poland will also present many valuable scientific results and impart their practical knowledge.

At both an international and a domestic level, the exchange of information and knowledge, from the viewpoint of different experiences, professional expertise and of different generations, is both valuable and necessary for our joint work. Within the parameters of the Colloquium, we think that we are successfully maintaining the tradition of professional discourse and, more recently, the steadily increasing necessity of collaboration between science, research and practice.

Organising this meeting would not have been possible without the important support of our partners. We wish to thank primarily our general partner, OKD and our acknowledges belongs to companies Dräger Safety, GEOtest, Hilti ČR, Huddy Diamonds, Minova Bohemia, NOVUM Servis, Sandvik Mining and Construction too as well as colloquium scientific partners which are Central Mining Institute in Katowice; Faculty of Earth Science and Engineering, University of Miskolcs; The Institute of Rock Structure and Mechanics, Academy of Sciences of the Czech republic and Faculty of Mining, Ecology, Process Control and Geotechnology, Technical University of Košice. Last but not least we thank the LANDEK Ostrava Foundation, financial supports the publishing of the proceedings of the extended abstracts.

We would also like to take this opportunity to express our acknowledgment to all our reviewers of the presented contributions, who undertook the difficult task of professional guarantors of the Colloquium. We thank Associated Professor Jiří Fries, Associated Professor Radomír Grygar, Dr. Josef Holečko, Dr. Karel Holub, Associated Professor Eva Hrubešová, Professor Petr Klablena, Dr. Petr Koníček, Dr. Jan Kozák, Dr. Alena Kožušníková, Professor Petr Martinec, Associated Professor Rostislav Melichar, Professor Karel Müller, Associated Professor Blažej Pandula, Dr. Josef Pek, Dr. Jiří Ptáček, Dr. Kamil Souček, Dr. Lubomír Staš, and Associated Professor Richard Šňupárek. Not least, we thank our colleagues Lucie Georgiovská and Vendula Stašová for their patient and final formatting of the proceedings.

We trust that our meeting in the new surroundings of the Beltine Hotel in Ostravice and the countryside of the Moravian-Silesian Beskids will be as pleasant as in the Sepetná Hotel, where our meetings were held in the past.

Together with all the team responsible for the smooth running of the Colloquium, we express the hope that all participants will enjoy informative and interesting discussions at both a formal and informal level, as well as professional success and personal happiness.

Wishing you "Good Luck!" *Petr Koníček & Kamil Souček*

DIRECTED ROCK MASS FRACTURING AHEAD OF LONG WALL FACE AS A CONSEQUENCE OF THE INTENSITY OF EXPLOITATION

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KEYWORDS: Rock burst, rock mass, fracturing, stress, longwall face

Carboniferous rock mass in the Upper Silesian Coal Basin (USCB) is inhomogeneous. It is result of complex geological processes connected with formation of hard coal deposit and long term and intensive mining exploitation in this area (Jaroszewski, 1972). Localization of discontinues and their density determine inhomogeneous of the rock mass. Primary divisibility (textural) and secondary divisibility (mechanical) have a strong influence on value of seismic energy which is accumulated in the rock mass and in consequence for a possibility of occurrence of dynamic phenomena destroying the work environment in underground hard coal mine.

It is difficult for reliable quantification of all factors determining the safety at work in coal mine. For a given area of the mine is possible predict the level of seismic hazard due to the specific situation of exploitation coal seam. Unfortunately, as experience shows, despite the seemingly good knowledge of the rock mass and the use of a number of preventive treatments, there are events which effects disturb the production process or prevent its continuation. The existing natural discontinuities in the rock mass favor the generation of dynamic phenomena accompanying the mining operation. Proper assessment of the destruction of the rock in the excavation process help to characterize the environment in the vicinity of rock excavation and the impact of mining on its destruction.

Measurements of displacements the elastic layers in the floor revealed are subject to displacement of the large distances from the longwall face (J. Drzewiecki, 1995). This indicates a high capacity for transverse preserving of continuity of the rock mass layers, despite of their strong deformation in the area of dynamic edge, which is the longwall face.

In real conditions, the longwall face is moved at variable speed, resulting from the current technical conditions, geological and existing natural hazards. In the rock mass, which is able to accumulate in a layer an elastic energy, face advance determines which layers and which parts are involved in the process of accumulating the energy. Their understanding allows to predict the accumulated energy and its change depending on the intensity of exploitation.

From the safety point of view of mining crews, particularly it is appropriate to explain the genesis of mining tremors, which sources are located in the face of the longwall. Knowledge of the destruction of rocks in this area is essential to develop effective rockburst prevention or reduce high-risk seismic events.

Research and measurement realized in the Central Mining Institute resulted in developing a series of analytical solutions, which allow calculating and illustrating the set of values characterizing the rock mass disturbed by longwall operation. Particularly suitable for the definition of deformation and destruction processes, which are the result of the mining are programs based on the analytical methods of calculation:

- the level of pressures on the horizon, and exploited seams and tremor-prone rocks layers including faults, (Kabiesz J. and team, 1994: J. Drzewiecki, 2011),
- the size and range of deformation of selected tremor-prone layers in the roof of the operated seam (Szpetkowski, S., 1988, 1995, J. Drzewiecki, 2004),
- principal stresses in the rock mass, (Makówka J., Drzewiecki, J., 2011),
- mechanism of sources of mining tremors (Stec K. Drzewiecki J., 2012).

The use of the above programs to determine the causes, mechanism and prognosis of strong dynamic phenomena allows for analysis of this type of phenomena in the context of destructive processes induced by mining activities.

For current exploitation major tremors are localized on the longwall face. In this area are the movement of the crew, haulage and transport of spoil material. Practice shows that in areas with geological disturbances faults are a source of dynamic phenomena.

It should be emphasized that there are technical methods for initiating this type of dynamic phenomena by making the fractures in well-defined volumes of rock mass by directed fracturing.

These types of fractures have reference orientation and their dynamic phenomena accompanying the propagation of seismic events. They may be a direct source of high seismic tremors or the initiator of other mechanism of the source.

Using the empirical method of binding anomalous seismic wave velocity and stress (Dubiński J., 1994), analytical and empirical methods for forecasting stress fields in the rock mass (Kabiesz J., 1994) and supplement of parameters to estimate the impact of fault on the range its impact (J. Drzewiecki, 2011) allows to calculate and present in graphic form the distribution of isolines of stresses in the seam and in higher horizon.

The results of calculations and their graphical images suggest potential dynamic phenomena in the seam and the strong elastic layers above the seam. At the same time, they are the basis for predicting energy of dynamic phenomena with which should be expected during an exploitation in the areas of the largest impact of stress anomalies.

Execution of presented analyzes and measurement of works to optimize rockburst prevention both passive and active, both for planned and realized mining operations. Discontinuities formed in rock mass layer or cracks weaken and reduce the ability to accumulate energy. The amount of energy accumulated depends on the thickness of the layer. It is appropriate to divide thick layer against infringement exploitation works. Possibility of using several methods of calculation, measurement and implementation of programs for predictions of future changes that will take place in case of their taking strong elastic layers, creates the conditions for the execution of a preventive treatments in the band discontinuities that weaken the rock mass.

Knowing and estimation of determining parameters such as increased rockburst hazard decide and determine the safety of mining crews. Implementation of research aimed at understanding the causes of this threat, by direct measurement of the main stress, changes of the fragment of the rock mass and its potential displacement are important to define and describe the processes of mining in the specific deposit conditions.

The proposed unit of measurement and analysis contributes to the development of effective seismic and rockburst prevention, both passive and active. Passive, to prescribe parts of the rock mass particularly exposed to the influence of strong seismic events. Active, involving not only the active execution of a series of discontinuities that weaken the rock mass, but also the development of individual metrics of a longwall face advance to minimize energy of tremors.

It should be emphasized that the creation of discontinuities in rock mass of a pre-selected volumes, allows to "control" the locations of tremors sources, which is crucial for their alienation from the active excavations.

REGISTRATIONS OF SEISMIC FOCI UNDER MINING COAL SEAM - THE EXPERIENCE OF POLISH MINES

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KEYWORDS: depth of mining seismic events, location and mechanism of mining tremors

1. INTRODUCTION

Taken in the article subject is important in terms of understanding the mechanisms of mine tremors foci and in terms of the recognition of real impact of mining tremors on the underground excavations. Currently, it is fairly widely accepted that the operational mining tremors are associated with dynamic cracking of undermined rigid roof rock or an explosion of coal seam resulted from accumulation of elastic energy in the presence of stress concentration zones. Models of mining tremors, associated with cracking of strong, direct roof rocks, were intensive research in Polish coal mines in the last three decades [Drzewiecki, J., 2001]. Also in seam seismic events, associated with the local high static load of the coal seam or dynamic stress resulting from close mining tremors, were the subject of numerous studies [Szuścik and Zastawny, 1980].

It is true that most mining tremors are the result of inelastic deformation of the direct strong roof layers. This point of view was confirmed by a lot of seismological observations in the mines. But now a number of seismic registrations are observed under extracted coal seam. In recent years, observations of deep tremors have been made possible by installing in a few mines the modern seismological network, allowing the reliable interpretation wertical position of the mining tremors foci (Fig.1). For safety and economics of coal extraction the information on depth of mining tremors is very important. For example, sources of mining seismic events localized shallow under coal seam, require a different prevention than the mining tremors localized in direct rock roof. When the seismic events are located much deeper under mining coal seam and are far from excavations, the amplitude of ground motion are significantly damped and dynamic loads to mine workings are relatively low and don't cause danger to the stability of mine workings and miners [Dubiński and Mutke, 1996].

The spatial location of seismic sources and analysis of their position in relation to mine workings is therefore essential for rockburst hazard assessment and thus for safety in operation. This is a very practical aspect of the study foci of deeper mining tremors. At present only occasionally we can meet research related to deep mining seismic sources and their mechanisms. Undoubtedly one of the reasons for this is in principle poor quality of seismic network in Polish mines in relation to small number of seismic stations and their spatial deploy. In general, there is so little opportunity to document the depth of recorded mining tremors. Analysis of mechanisms of mining tremors conducted in the Upper Silesia have shown, that shear mechanisms (dip-slip, strike-slip) are most popular for foci located in rock roof and explosive mechanism for seismic sources located in coal seam [Stec 2007]. Also studied the mechanism of mining seismic events resulting from adding mining stress to existing residual tectonic stress were undertaken, but depth of mining seismic foci only occasionally were studied [Mutke and Stec 1997].

In the last few years we have been recording more and more mining tremors localized deeper than level of mined coal seam. The reason for this change was to install in a few Polish mines the modern seismological network consisting of 32 or 64 channels and well-distributed spatially seismic stations.

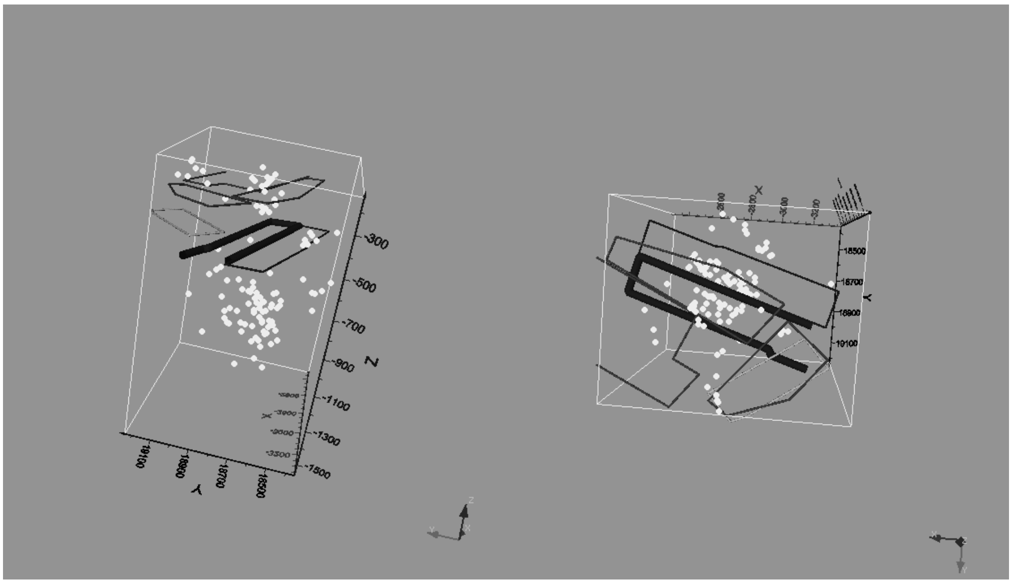

Fig. 1 Example of 3D location of seismic events in “Wujek-Slask” mine recorded by Seismic Observation System (SOS). On the left picture - side view, and on the right picture – top view (gray dots - hypocenter of seismic events, thick line - contour of longwall 2JD, thin lines - edges).

The most deep mining seismic events in Upper Silesia were related to regional geological structures focusing stress (e.g. syncline) or tectonic structures (younger faults), where mining stresses were only factor triggering tectonic seismic events. The deep tremors were characterized by a shear mechanism in the source (double-couple force solution). Tremors located directly under the coal seam floor very often were characterized by essential contribution of compensated linear vector dipol mechanism CLVD (uniaxial force solution). It could be means that we have relatively large horizontal uniaxial compression comparing to vertical one, $\sigma 3$. During mining are created new gobs. In such case the rock slabs are pushed up to empty space in the new gobs (vertical extension).

The spatial location of seismic sources and analysis of their position in relation to mine workings is essential for rockburst hazard assessment for choice of prevention methods and thus for safety in operation. This is a very practical aspect of the study foci of deeper mining tremors.

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CROSS-SECTIONAL BOREHOLE DEFORMATION METHOD (CBDM) FOR MEASREMENT OF ROCK STRESS CHANGE AND ITS APPLICATION

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KEYWORDS: cross-sectional borehole deformation method, rock stress change, monitoring, rock cavern

1. INTRODUCTION

The Cross-sectional Borehole Deformation Method (CBDM) developed by Obara et al. (2004, 2010, 2011a, b, 2012) is a method which two dimensional state of stress change within rock mass in a plane perpendicular to a borehole axis can be measured. In this paper, using this method, the stress change of immediate rock mass of a rock cavern is measured. Firstly, the theory of the CBDM is described, as well as the prototype instrument with the laser displacement sensor. Secondly the measurement site and results are described. Then the stress changed due to excavation of the cavern is discussed with time and space. Finally, it makes clear that stress change of immediate rock mass can be estimated by the CBDM and that the CBDM is available for measuring stress change.

2. MEASUREMENT RESULTS AND DISCUSSION

2.1. SITE DESCRIPTION

The plan view of measurement site in Kamioka Mine is shown in Fig.1 (Obara et al. 2011b, 2012). A cavern was excavated at a depth of 900m within gneiss. The Young's modulus and Poisson's ratio are 30GPa and 0.2 respectively. The dimension of the cavern is 15m by 21m and 15m in height. The borehole with a length of 5m for measurement of stress change was drilled from the gallery to the cavern before the start of its excavation. The width of the rock between the gallery and the cavern is about 7m. The borehole for measurement was drilled in the wall. The measuring points are located at depth of 1.0- 4.5m. The measuring points are determined from condition of the recovered core.

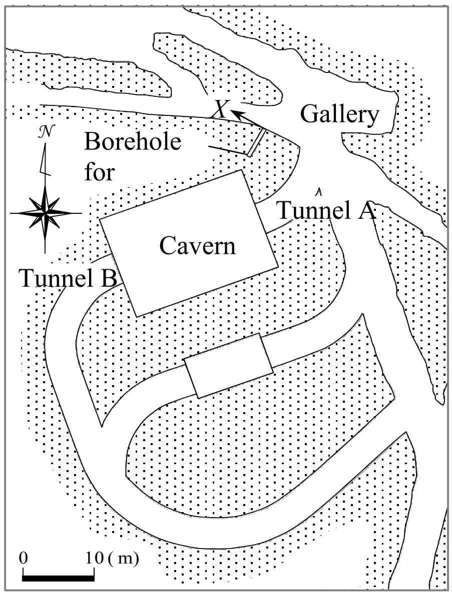

Fig. 1 Location of borehole for measurement and cavern in the plan view of measurement site in Kamioka Mine.

2.2. STRESS CHANGE

The distribution of stress change along the borehole axis at each stage shown in Fig.2. The positive stress change means increase of compressive stress in this paper. The vertical cross section along the borehole axis is in Fig.2(d). In shear stress change $\Delta\tau_{XY}$ of Fig.2(c), the stress change is relatively small along the borehole axis during excavation. This means that there is not very much change in principal direction with time and space.

The vertical stress change $\Delta\sigma_Y$ in Fig.2(b) is comparatively large. The stress change at depth of 1.8m, 4.0m and 4.5m is remarkable.

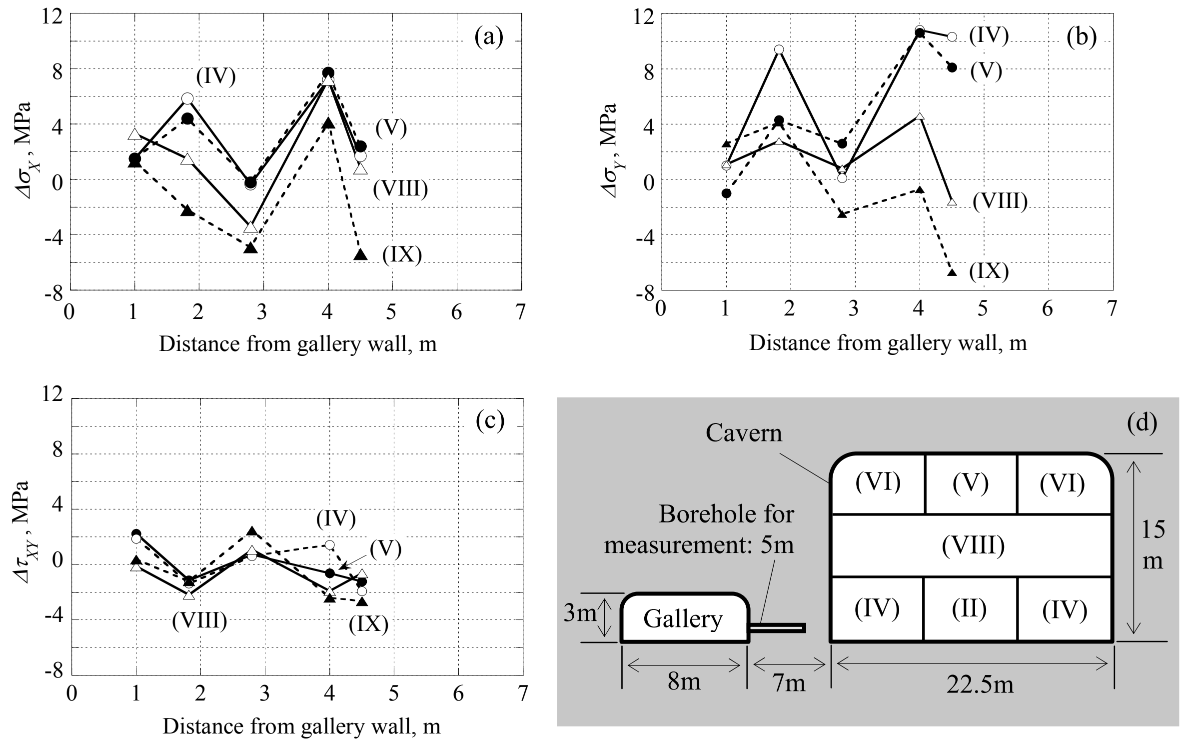

Fig. 2 Distribution of stress change along borehole axis: (a) $\Delta\sigma_X$, (b) $\Delta\sigma_Y$, (c) $\Delta\tau_{XY}$, (d) vertical cross section.

At a depth of 1.8m, the stress represents the maximum value at a stage of IV, then it decreases to a half of the maximum value at a stage V. This value is maintained until a stage of IX, which is completion of excavation. This means that the rock mass near a depth of 1.8m was not damaged. On the other hand, the stresses at depth of 4.0m and 4.5m also represent the maximum value at a stage of IV, then they decreases gradually with advance of excavation. At the final stage IX, the stresses decrease to the stress level lower than that before excavation. It is considered that the rock mass near depth of 4.0 - 4.5m was damaged due to excavation. These trends can be seen in horizontal stress change $\Delta\sigma_X$ shown in Fig.2(a). However state of damaged zone is not clear. Therefore that state should be confirmed by other methods such as numerical method.

The stress change at a depth of 1.0m is small. As this point is near the gallery wall, the rock mass in this area is considered to be damaged. On the other hand, the stress of $\Delta\sigma_X$ and $\Delta\sigma_Y$ at a depth of 2.8m do not increase at a stage of IV, nevertheless the stress increase at a depth in front and behind. It is considered that fracture around the borehole may occur and that discontinuities may move due to excavation.

3. CONCLUSIONS

The Cross-sectional Borehole Deformation Method (CBDM) is successfully applied to measurement of stress change under the excavation of cavern for six months. Based on the estimation of stress changes, it was made clear that the stress components increased in former period of excavation and decreased in latter period gradually and continuously and that there were two types of location along the borehole axis, namely one is the point that stress change was relatively small, the other is the point that stress increased, represented maximum value then decreased. From the results, it is concluded that the CBDM is available for measuring stress change.

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WORLD STRESS DATABASES AND ROCK STRESS MODELS AS RESOURCES FOR ROCK MECHANICS AND ROCK ENGINEERING

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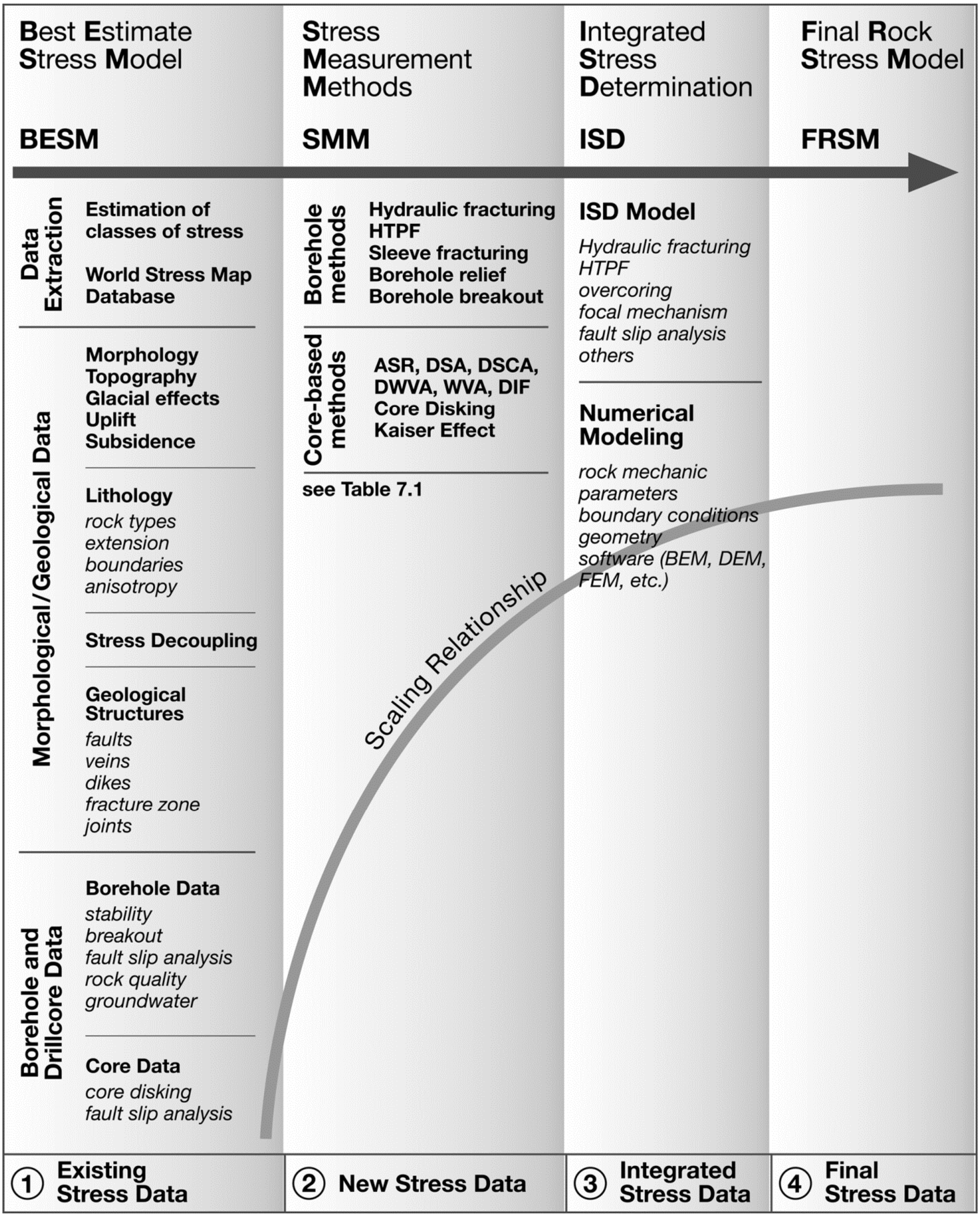
KEYWORDS: *Stress database, stress model, rock mechanics*

Knowledge of the in-situ stress state is of key importance for rock engineering. We present the World Stress Map (WSM) database and its application to rock mechanics and rock engineering purpose, and in particular the orientation of maximum horizontal stress. We discuss the World Stress Map and the quality ranking system of stress orientation data and show one example of discrete-measured and computed-smoothed stress orientations from central and northern Europe with respect to relative plate velocity trajectories. We present one example of stress decoupling where stress orientation differs below and above soft strata in a geological sequence. We give first insights into ongoing development of a second, more Quantitative World Stress Map (Q-WSM) database which compiles globally rock-type specific stress magnitudes versus depth. We discuss the vertical stress component, and the lateral stress coefficient versus depth for different rock types. We display stress magnitudes in 2D and 3D stress space, and investigate stress ratios in relation to depth, lithology and tectonic faulting regime.

In the second part of the presentation we describe the methodology to determine the Best Estimate Stress Model (BESM) and the Final Rock Stress Model (FRSM) as a part of a site investigation or an investigation of an area. We suggest that the BESM is generated from collecting existing stress data in archives, analyzing morphology, topography and geology in the field and stress information from boreholes and drill cores. We believe the established model is the result of the integrated study and the outcome is used in selecting the most appropriate stress measurement technique. We strongly emphasise that available stress data from BESM and measured new data from stress measurements are integrated and analyzed be means of least-square criterion, Monte Carlo simulation or generic algorithms and that the use of numerical modelling shall help in obtaining an overall understanding of the state of stress at the site or for a region. In our opinion the modelling results shall also contribute to the estimation of the variability and uncertainty in presenting the final rock stress model FRSM.

In Zang and Stephansson (2010) we present a brief description of the combination of available stress data from the best estimate stress model BESM, new stress data from stress measurement methods on site, integrated stress determination using previous data plus numerical modelling in order to generate the Final Rock Stress Model FRSM at a site or an area, see Figure 1.



1. **Fig.1** Establishment of the Final Rock Stress Model (FRSM) from combination of available stress data from the Best Estimate Stress Model, new stress data from stress measurement methods on site (SMM), integrated stress determination (ISD) using previous data and numerical modelling. After Zang and Stephansson, 2010.

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THE CONSEQUENCES OF THE ROCK BURSTS HAZARD IN THE SILESIAN COMPANIES IN POLAND

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KEY WORDS: Rock burst, deformation, stress, rock burst analysis, roadways, longwall face

The seismic and rock burst hazard still appears to be important in most of hard coal mines in Poland. In comparison with the past, there was a significant increase of seismic activity in rock mass in the Upper Silesian Coal Basin (USCB) in 2011. 33 rock bursts occured there in the period 2001-2010.. The causes of rock bursts occurrence are presented based on the analysis of the rock bursts that occurred in the Polish hard coal mines. The scale of the rock burst hazard has been characterized with the respect to the mining and geological conditions of the contemporary exploitation.

The existing methods of hard coal mining in Poland have been used since the sixties and seventies of the 20th century. When the methods and technology for rock burst protection were launched, the number of rock bursts in hard coal mines decreased drastically from 39 in 1972 to 2-5 in the last years. The decrease of rock bursts in the last years has been achieved due to evaluation of rock burst hazard source and condition. It enabled a proper design of planning of coal seams exploitation located in the rock burst hazard areas, as well as development of rock burst preventing method, especially the method of directional hydraulic fracturing of rocks (UHS) directed blasting fracturing (USS) (Dubiński, Konopko 2000). The reduction of production volume in USCB is also not without a meaning.

Seismic character of USCB is monitored several years by seismic data collecting by net of seismic stations. GIG archives has shown that there are two types of seismic activity: mining activity and mining-tectonic activity (Stec 2007). The first type of seismic character is related directly to the involvement of mining operations, therefore this event will occur mainly in the areas of active mining regions. The second type of seismic character is a result of interaction between the mining and tectonic factors. Most of these quakes are located in the areas of tectonic faults. They feature significantly higher seismic energy.

The rock burst occurrence inseparably generates the mine workings deformations and damages as well as casualties. This aspect of rock burst hazard appears to be mostly undesirable from the practical point of view. The measures undertaken to prevent the hazard of mine tremors and rock burst occurrence are designed to reduce these dangerous symptoms of the rock burst and seismic events.

In the last ten years (in the period 2001-2010) there occured 33 rock bursts at the depth of 550-1150 m (average depth 839,45 m). In the seams of stratigraphic group 500 occured 22 rock bursts. In the seams of stratigraphic group 400 in the same time period were noted only 6 rock bursts and in the seams of stratigraphic group 600 – only 1 rock burst, in the seams of stratigraphic group 700 – 4 rock bursts.

The data shows that both the depth of exploitation and rock bursts hazard growth constantly. The depth of coal seam stratigraphic group 500, at which as far as 22 rock bursts have occurred (in the period 2001-2010), extends from 550 m to 970 m. It is hard to admit that this depth could be the most dangerous. The number of rock bursts should rather be referred, for example, to the number of coalfaces or to the output from the depth of the intervals. A depth of 777 m could be suspected to be associated with the highest output and hence with the highest number of rock bursts. It is important to note that, nowadays, the average depth of mining in the USCB mines is 702 m.

Essential part of the rock bursts prevention is the analysis of the extent of damaged and/or destroyed headings, particularly in the field service environment. Proper assessment of the rock burst risk in the area of exploited seam is very important too, as well as the selection of the lenght of the expected rock burst impact zone affects, the accuracy of it and decision on the prevention of rock burst hazard. Central Mining Institute own a source material (the rock bursts Data Bank on USCB (Patyńska 1987-2010) relating to existing rock bursts, where details of geological, mechanical and technical properties are stored. These data were reviewed and there were selected lenghts in a way that rock burst analysis of the prevalence of various ranges of effects and their distance from the longwall face. They dealt with the incidence of various ranges of damage and/or destruction headings in the vicinity of longwall face. The purpose of these studies was to determine the lenght of the rock burst safe zone for different degrees of records.

88 specific cases were selected from the rock bursts occurred in the years 1987-2010 in the USCB for the analysis. The effects in the longwall in the workspace and/or its surroundings, with a range of excavations varied between 0 m to 300 m. Comparision of different rock burst effect rangs, depending on the effects of energy shock, which caused the rock burst, illustrate the scale of the analyzed event. The Data Bank of Rock bursts shows that the scale of effects refers to seismic shocks caused by rock bursts of energy 104 - 108 J. The rock bursts were noted for the depth of exploitation in the range of 410-1150 m (average 768 m) surrounded by front of the longwall with a lenghth of 45 m to 400 m (average length of the front was about 163 m). In order to determine changes in the prevalence of various ranges of rock bursts effects, we calculated the incidence of absolute, relative and cumulative frequency, grouped in class intervals in increments of 10 m, 25 m and 50 m. Distributions of different ranges of effects and the decrease of frequency clearly indicates of their variations, depending on class frequency and length of the interval. Logarithmic curves and equations trend of these changes indicates differences in the slope distributions.

Despite a smaller number of rock bursts and their consequences recorded during past 10 years, we can observe a certain upward trend in the state of the rock burst hazard. Especially as far as the seismicity of the rock mass. It concerned probably with the amount of output and work concentration at individual longwall panels as well as with the depth of the excavation.

The synthesis of the conditions of rock burst occurrence in hard coal mines in the years 1989-2010 allows summarizing the following statements and conclusions:

The hard coal output declined from 177,6 mln tons in 1989 to 76,1 mln tons in 2010. The average mining depth increased from about 524 m to 702 m during that period.

In the period 1989-2010 were noted 132 rock bursts in the Silesian Companies in Poland. Only in the seams of stratigraphic group 500 occured 117 rock bursts. In the stratigraphic seams of group 400 in the same time period only 8 rock bursts were noted and in the seams of group 600 – 3 rock bursts, in the stratigraphic seams of group 700 – 4 rock bursts. The depth of deposition of the mined all groups seams was from 400 m to 1150 m (average depth 760 m).

In accordance with the statistics of rock bursts in the period 1989-2010, the rock bursts were accompanied by rock mass quakes of energy 103 ÷108 J. The mining operations were inducing mine tremors of energy up to 109 J. The energy of the tremors constantly were increasing in average from 1,67·107 J (in the period 1989-2010) to 2,94·107 J (in the period 2001-2010).

132 rock burst were analyzed in the years 1987 - 2010, of which 95 cases have had their impact solely in the surroundings of longwalls. The study summarizes the number of rock bursts with the range of effects in the area of the longwalls; main gates and bottom gates and working spaces - in the form of damage to the walls and/or destruction of the excavations.

Analyzed set of ranges of rock bursts effects limited to a ranges of lengths from 0 m to 265 m, which, after rejecting the outliers and extreme ranges, refers to only 88 cases.

In order to determine changes in the prevalence of various ranges of effects, we calculated the absolute, relative and cumulative frequency, grouped in class intervals in increments of 10 m, 25 m and 50 m.

The frequency ranges of effects clearly show their differences, depending on the grade and length of the interval frequency ranges.

The study indicates the diversity of different distributions of confidence of the long range effects, also the most reliable set is the distribution range in increments of 50 m, in the form of the confidence curve.

On this basis, the areas (roadways) of the safe zones were selected as the safe to rock burst hazards in roadways conducted in terms of assessment for different degrees of the risk, as follows:

Roadways carried out in I, II and III degree of rock bursts hazard should be covered by rock bursts prevention to at least 150 m before longwall front;

Another zone is the roadway from 150 m to 220 m before longwall front, which can be described as a dangerous zone for the second and third degree of rock bursts hazard;

Other sections above 220 m before longwall faces can be classified as those in the first and second degree of the rock bursts hazard scale and they are the least dangerous section of roadway by the rock bursts.

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IN SITU PHYSICAL MODEL OF A CLAY BARRIER

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KEYWORDS: bentonite, in-situ experiment, physical model, geotechnical monitoring

1. INTRODUCTION

In order to guarantee the safe disposal of high level nuclear waste, a number of conditions must be fulfilled. The fundamental requirement is the isolation of hazardous radionuclides for an extremely long period of time. Research on engineered barriers, the role of which will be to isolate such radionuclides, has been in progress for a number of decades and has confirmed the usefulness of physical modelling which allows the in-situ monitoring of on-going processes, including the verification of the constancy of the rheological properties of the materials employed. An in-situ physical model of the vertical disposal of a container with radioactive waste (spent nuclear fuel) is currently under construction at the Josef Underground Research Centre (URC) operated by the Centre of Experimental Geotechnics (CEG) of the Faculty of Civil Engineering, CTU in Prague. The model will allow the monitoring of the behaviour of a barrier consisting of pressed bentonite loaded with temperature and simultaneously saturated with granitic ground water. The model, as well as its immediate surroundings, will be fully instrumented. The article is intended to acquaint the reader with the current phase of preparation of the physical model project.

The physical model will build upon earlier research on engineered barriers for the safe disposal of high level radioactive waste in the Czech Republic. The main task of underground repository multi-barrier systems is to reduce to the absolute minimum the migration of radionuclides into the biosphere. One type of engineered barrier will consist of clay material (bentonite). The clay barrier will be required to restrict to the maximum the movement of transmission media between the encasing unit containing high level nuclear waste and the natural barrier (the rock massif).

The model will be the first physical in-situ model of its type to be assembled in the Czech Republic. The natural barrier used in the experiment will consist of the granitic rock environment of the Josef Gallery (Fig. 1).

Following the completion of the design phase of the project, site work on the assembly of the model commenced in 2011 which involved, in particular, the laboratory testing of the clay material, the preparation of the selected niche and the boring of a large-scale repository well. An important part of the experimental model concerns the design of the instrumentation installed.

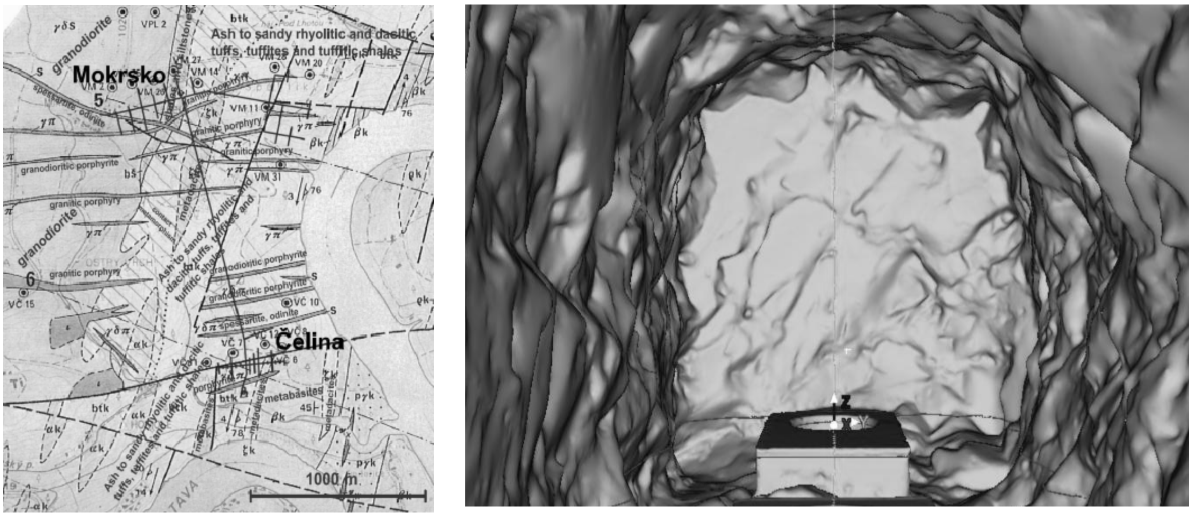

Fig. 1 Geological map of the Josef Underground facility (left); the oval indicates the granite section of the underground area. On the right is a simulation of the selected experimental gallery with the disposal hole prepared for the model

2. MODEL CONSTRUCTION

This physical model is a follow-up to a previous experiment which was performed at the CEG's laboratory from 2000 to 2008 (Svoboda J., 2010). The barrier of highly compacted bentonite bricks will be designed in the form of a super container which (including all the instrumentation) will be assembled at the above-ground laboratories of the Josef URC Regional Underground Research Centre and subsequently transported to the experimental site in the underground complex of the Centre. The material selected for the construction of the barrier is Czech Ca-Mg bentonite (Obrnice treatment plant). The input geotechnical parameters of the selected bentonite (referred to as B75) were defined by CEG laboratory staff prior to super container assembly.

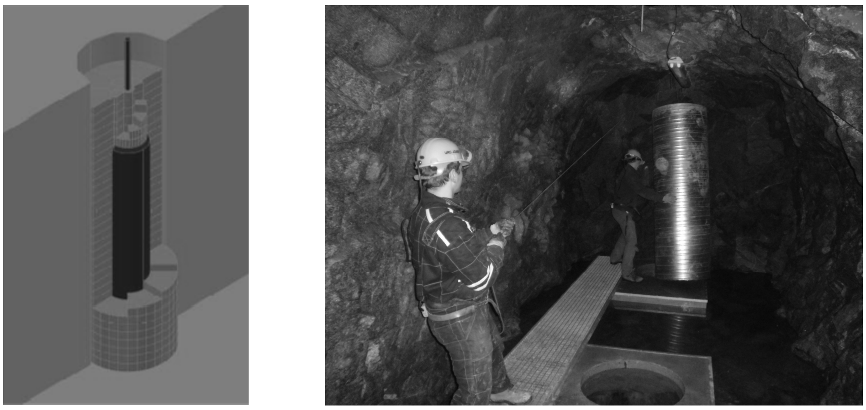

Fig. 2 Design of the model in the disposal hole showing the position of the heater and the surrounding bentonite blocks (left) and physical verification of the dimensions of the model with respect to the disposal hole prior to commencement of the experiment proper (right).

3. MONITORING OF THE PROJECT

The barrier, loaded with temperature and simultaneously saturated with granitic groundwater, will model real processes which will occur in bentonite barriers. The barrier will be fully instrumented. The sensors for the monitoring of the development of swelling pressure, temperature and moisture will be mounted along five measurement profiles, two of which will be located under the heater, one at heater level and a further two above the heater. Each measuring profile will house between 14 and 17 sensors which will provide continuous data reading. The surrounding rock will be monitored for changes in stress state and temperature. Surface deformations of the stope will be monitored by means of convergence measurement.

4. CONCLUSION

The placement of an in-situ physical model in a granitic rock medium replicates the presumed environment of a future deep underground repository in the Czech Republic. The loading of the bentonite barrier with temperature corresponds to the behaviour of the container with spent nuclear fuel and the gradual saturation of the bentonite barrier with granitic groundwater simulates the expected real conditions of deep underground repositories. It is hoped that the findings obtained from long-term monitoring may contribute towards the safe design of repository nests. The resulting data will be further employed for the numerical modelling of the long-term behaviour of multi-barrier systems. The experiment is planned to be launched in December 2012 and will run for a minimum of 3 years. Tests performed during the preparation stage of the project showed clearly that B75 bentonite fulfils all the requirements prescribed for use in the clay barrier of deep underground repositories and is a suitable material for use in the new in-situ experiment.

ACKNOWLEDGEMENT

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COMPUTERIZED MODEL SUPPORTING OPTIMIZATION OF DRIVE FOR THE BELT CONVEYOR SYSTEM IN OPENCAST MINING

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KEYWORDS: belt conveyor, energy saving, computerized model, force distribution

1. INTRODUCTION

Electric energy utilized in driving conveyor tracks operating in the opencast mining sites and belt conveyors operating at different special mining machinery is a significant cost item for the operating companies. For this reason, every saving achieved in this area results in a major and remarkable reduction of fixed costs for the company.

Sizing of the drive system should provide for a power necessary to the material flow demand and input the required hauling power even under extreme conditions. At reduced transport demands or under improved conditions of motion, installation of a moving power able of operation in extreme conditions is superfluous. By partial switch-off energy saving can be achieved in the drive system and the excess energy can be switched back in due time, whenever the circumstances make it necessary. The decision of switch-off and return should, however, be preceded by a preliminary assessment. With considering the results of this assessment incorrect decisions may be avoided. The assessment is effective only if its time requirement is able to follow changes in the operational conditions.

The computerized modeling system to be demonstrated in the following paragraphs runs in an EXCEL environment and - with its computing capacity - is able to determine, as well as visually represent the conveyor forces adapted to the modified conditions in a short time frame. In addition, as a result of the completed calculations, the model provides supplemental information for the user that helps operate the belt conveyor track with a greater safety and better efficiency.

Beyond the above items, the model can also be well utilized in training. It can help improve efficiencies of both the demonstrating and the inspection stage of training, thus improving degree of utilization of the available training time.

2. DESCRIPTION OF THE OPERATOR ENVIRONMENT

The modeling program communicates with the operator by the use of eight EXCEL sheets. It is not practical, however, to input data of the conveyor track to be modeled by directly entering cell data on the input sheet. A special Windows panel serves for this purpose, to be called by running an EXCEL macro. After starting the macro, the *Input basic data* window appears. (Shown in Figure 1.) Leaving a particular field one can obtain an error message. This is because the program checks, whether value of the data to be entered is within the range acceptable for a belt conveyor. After leaving the field a pop-up window will notify in case the user is about to validate an out-of-the-range figure, at the same time giving the valid range of the particular parameter. Exactly this is the main function of data entry via the *Input basic data* window.

The model manages the drive arrangement solutions usually applied in belt conveyor systems driven at the track end. Selectable layouts: 0;1;2;3 drive units in front and 0;1 drive unit at the back.

On the other hand, it is able to handle dynamic effect of the acceleration phenomenon occurring at starting the belt and it considers not only the static forces but also the power requirement of the system acceleration. To do this, time period of the starting process can be entered in the *Input basic data* window.

3. DISPLAYING THE RESULTS

The remaining seven sheets of the workbook feature displaying the calculated results. These sheets show trends of the hauling force distribution along the track. Each sheet contains diagrams of the possible hauling force distributions belonging to the possible drive layouts. Field at the bottom right corner contains the most important quantities of the track and results of the calculations. These are the following: power demand of the drive by drive units, drive reliability factor valid for each drive unit, tensioning force to be applied at the tension site, the transported mass flow, track length, average force calculated from the forces along the track, elevation of the track.

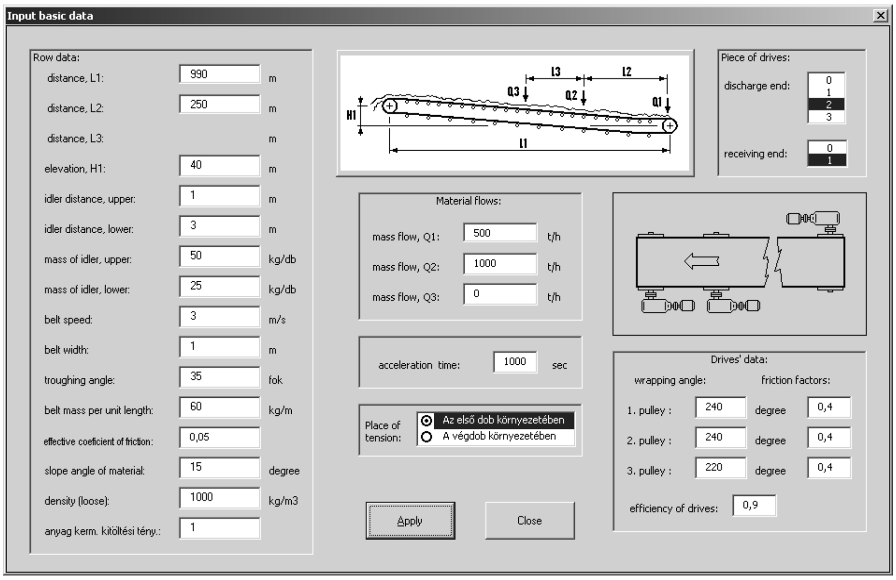

Fig. 1 The window of input data

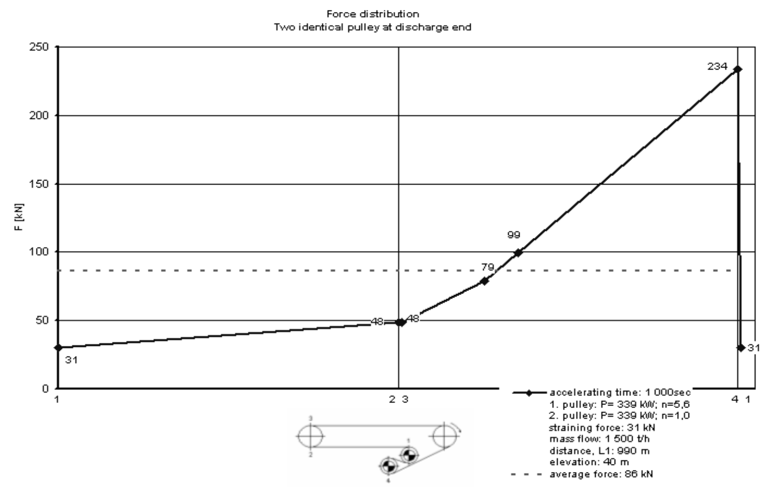

Fig. 2 Daigram of draging force

Figure 2 shows one of the seven result sheets belonging to the basic data obtained from Figure 1. For documentation purposes the integrated print function of EXCEL can be used, with all options offered thereby. All other EXCEL functions are, of course, available for the user, except for those protected by a password. Thus, any stage of the modeling can be saved and then reloaded.

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SOME POSSIBILITIES OF TECTONIC AND STRESS ANALYSIS APPLICATION IN MINING IN THE UPPER SILESIAN COAL BASIN

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KEYWORDS: Stress, structural analysis, stability, mine working.

Distribution of stress fields affects significantly the behaviour of rock mass, thus influencing considerably mine workings stability in rock mass. Except for irregular stress distribution in rock mass is a decisive factor for the occurrence of rockbursts, the good knowledge of rock stress is important for excavation in general. Mine workings are affected both by natural stress, including gravitational, tectonic, hydraulic and residual stresses and by the stress induced by mining. While the stress induced by mining operations can be influenced, to a certain extent (e.g. by the appropriate exploitation of mine workings in time and space), primary stress fields are defined by the geological structure and by rock properties. The influence of the perpendicular component of stress tensor is commonly regarded and it is used for designing of mine workings support. The influence of horizontal stress components on mine working are used to a lesser. The deformation of mine working is different by different orientation of horizontal stress components. Thus, good knowledge of stress distribution in rock mass is necessary for the rock burst risk prediction, as well as for the mine technique. Since 1994, a few dozens measurements of recent horizontal stress at a depth of 600 to 800 metres under the surface have been carried out. In addition to the measurement using the hydro-fracturing method, a few measurements were carried out by the modified overcoring method. It was shown recently that tectonic structure of rock mass can significantly affect stress fields even in a wider area of individual structures. Rezidual tectonic stress is possible to interpret by analysis of geological structures in rock mass. The use of recent stress tensor data, its comparison with the results of structural analysis and their generalization for the Karviná subbasin can be significant contribution for optimal spatial and time arrangement of mine workings.

The experience gained from excavations of mine workings suggests a mutual relationship between the direction of drivage of mine workings and the orientation of the maximum horizontal stress (σH). Thus the inappropriate direction of excavation mine workings against σH can affect negatively the stability and convergence of supports in the working mined, the floor stability, the character of failure of adjacent rocks and particularly, not last the stress release in the roof of mine workings – seismic events. The experience gained in the working in the ČSM Mine shows that the most suitable direction of a mine working is parallel with the maximum horizontal stress (σH) and the worst conditions occur when a mine working is mined perpendicularly on the maximum horizontal stress, i.e. parallel with the minimum horizontal stress (σh).

In many cases, residual tectonic stress in the vicinity of fault structures can be considered for an important element affecting the resulting stress fields in rock mass. Stress fields in the Czech part of the Upper Silesian Basin (in the Ostrava-Karviná Coalfield - OKR) have been primarily affected by the Varisan orogenic processes, as early as the period of the basin formation. Generally, the west-east orientation of the principal component of horizontal stress $\sigma 1$ is assumed. Based on the structural-tectonic analyses, it was concluded that in the Upper Silesian Coal Basin its intensity decreases to the north at the same time. From this a dextral rotation of the force results. Strike slip movements occur along the west-east fractures consequently.

Their crossing with the north-south system of faults causes the complicated stress conditions. The so-called corner structures come into existence there with a potential tendency to gravitational compensation of unbalanced stress condition. The exploitation in the vicinity of such structures disturbs temporarily a stress balance in wider regions. The re-distribution of stress in one tectonic segment causes the increase of the stress in the adjacent segments. An example of such behaviour can be shown on the excavation in the neighbourhood of the crossing between the Hlubinská fault and the Eleonora fault. It is shown there, that it is necessary to coordinate the plans for mine workings both from the aspect of space and time.

Stress fields are unambiguously one of the most important factors affecting the underground mining in connection with the exploitation of mine workings. The experience with stress release at exploitation of mine workings in the east part of the Karviná Coalfield, and based also on the effect of residual tectonic stresses in the space of the crossing of faults, shows a mutual relationship between the time and space aspect of the excavation of a mine working and the orientation of the components of stress tensor. In order to generalize these interpretations, it is necessary to supplement the measurements of recent stresses in the OKR, because until now the network of measurements has been rare and unequal. Thanks to the good knowledge of stress conditions at the excavation it is possible to optimize their projection and to contribute to the improved safety and mine productivity.

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STRUCTURAL ANALYSIS IN THE ROCK MASS BETWEEN ROŽNÁ AND OLŠÍ URANIUM DEPOSITS (STRÁŽ MOLDANUBICUM) FOR STRESS FIELDS INTERPRETATION IN THE REGION OF UNDERGROUND GAS STORAGE

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KEYWORDS: Structural analysis, stress field, underground gas storage

The construction of underground gas storage is planned in the area of the declined uranium deposit. The underground storage are designed in the underlying rocks of the deposit in the space not affected by extraction of uranium ore; on the location where favourable geological and geomechanical conditions for the construction are assumed. The uranium deposit is formed with a complex of rocks of the Strážecké Moldanubikum consisting primarily of fine grain and medium grain biotic gneiss, with cordierite here and there, migmatizated gneiss, migmatite and granitized gneiss. In the upper part of the complex a band of amphibolites and amphibolitic gneiss is localized. The structure of the whole complex was analyzed within the geological and geo-technical research in the area of the planned construction of gas storage. Moreover, the structural analysis was one of the supporting material for the determination of geometry and the method of designing the mine workings and potential exploration mine workings for the gas storage.

Both uranium deposits, among which the gas storage tanks are designed, the Rožná deposit and Olší deposit, are situated in the east part of the Strážecké Moldanubikum in the north-east edge of the core of the Bohemian Massif. Isoclinal folds of various vergency are typical for peripheral parts of the Strážecké Moldanubikum (Melka et al. 1992). The Rožná-Olší ore fields are situated in the east periphery of vast anticlinorium. The region between the Rožná and Olší deposits consists of anticline identified as the Principal anticline (also called the Roženska-rozsošská anticline).

The fault systems can be identified, according to geometric relationship to foliation, as longitudinal, transverse and diagonal systems. They play an important role in the whole structure of this part of the Bohemian Massif.

The structural analysis was focused primarily on geotechnical aspects, but the wider structural-geological relationships in the area investigated were taken into consideration.

The compass measurement of foliation (ductile elements) and discontinuities (ruptures) were carried out at the levels XVIII and XX in the vicinity of the shaft R3 and in the exploration crosscut V1at the levels XXI. A pole diagram of fractures construed from the values measured at the level XVIII shows the paired systems, one with a steep inclination to the NNW direction and the second system, which is not so apparent, with the inclination almost to the SSW direction, which passes, here and there, into the direction of foliation. These systems are accompanied with the sub-horizontal system of fractures and with the system of fractures copying the orientation of foliation. The similar systems are shown in the chart of fractures construed on the basis of measurements at the level XX.

Until now, the compass measurement of the foliation and the discontinuities in the crosscut V1-XX1 have been carried out in the stationing between 210 and 495 m and in the exploration gate GR1-XXI. The compass measurement carried out within our research was supplemented with the measurement carried out by the geological service of the mine in the stationing of 150 m to 209 m in the area of occurrence of amphibolites. These measurements have been statistically processed in the same manner as the relevant compass measurements. The results were used for the definite estimation and quantification of fracture systems for the determination of geotechnical coefficients RQD, RMR a Q.

In addition the structural data gained from the mine geological maps 1 : 2000 using also the detailed geological documentation of roadways in scale of 1:200 have been statistically prepared and analyzed. In order to interpret vertically the area as large as possible, the data from the selected crosscuts in the underlaying rocks of zone 1 at the level XVIII, level XXI and level XXIV have been evaluated. Important fractures and faults have been analyzed from the geological maps. These fractures and faults have been statistically evaluated and documented in the contour diagrams. The distribution of fractures in the diagrams differs in different parts of a working field and the orientation of both statistic maximums and the frequencies of individual paired fracture systems as well as.

The results of interpretation from the particular measurements of ductile elements (foliations) and fractures (ruptures) from the mine maps can be summarized as follows:

Foliation follows generally the orientation of vein structures, i.e. approximately in the N-S direction to the NNE- SSW direction. In the north part of the planned gas storage of Rožná rotates locally in the NE – SW direction.

Two paired systems of diagonal fracture systems were recorded as the most frequent systems NW - SE direction and NE – SW direction, which are most important for the stability of mine workings in the location for the planned gas storage. The mutual proportion of the frequency of both diagonal systems changes.

Diagonal fractures show a low persistency only. While in the crosscuts, their thickness reaches up to the 10-1m orders, they are not caught in paralel gates, even if the distance is several tens of metres only.

The fractures or fault zones are filled, in the majority of cases, with carbonates and, to a lesser extent, with the crushed rock, often mylonitized and kaolinitized, mostly with the presence of chlorite on contact surfaces. Water inflow is sporadic only with negligible capacity. All these fractures do not affect negatively the stability of mine workings.

It is logical that the frequency of fractures will be highest in a close vicinity of ore zones where the rock mass is most affected from tectonic aspects. The reduction of the frequency of fractures can be legitimately assumed in the east direction into the underlying strata of ore zone. This is also suggested by the measurement in mine workings oriented to the E-W direction in the underlying strata of zone 1.

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EFFECT OF HIGH TEMPERATURES ON CEMENT COMPOSITE MATERIALS IN CONCRETE STRUCTURES

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KEYWORDS: high temperature, resistance, concrete, tunnel

1. INTRODUCTION

Concrete is flexible construction material, which can be used by various technological applications for underground structures and reinforcement of mine works (adits and tunnels). In such applications, concrete has many functions – static function, water-tightness is often required as well as gas-tightness and resistance to action of aggressive waters or durability. In railroad and road tunnels construction there is one important question– influence of high temperatures on concrete lining during wildfire. The paper focuses on analysis of behavior of cement composite materials (concrete) exposed to high temperatures. Several possibilities of increasing resistance to cement composites to high temperatures are proposed as well as recommendation for reduction of damage on structures exposed to high temperature.

2. USE OF CONCRETE IN UNDERGROUND STRUCTURES

Concrete as construction material can have various specific properties (self-compacting, high-strength, direct-finish, fast-setting, sprayed) and has a wide range of application in underground construction industry. Most frequently, concrete is used for construction of railroad and road tunnels, where concrete is used at two stages of the construction. First, as primary lining, in the form of sprayed concrete, which is applied directly on stope in rock. Such concrete has to have specific properties: fine grained concrete, mostly with addition of dispersed reinforcement and setting accelerators. At the second stage, concrete is used for concreting of secondary (final) lining of the tunnel. Since concrete at this stage is placed by pumping into formwork and thickness of layer is variable, rheological properties are the most important ones. Moreover, after removing formwork, surface of the secondary lining is the face of inner walls of the tunnel, which increases requirements for appropriate composition of the concrete mix. Apart from construction of railroad, road and subway tunnels, concrete in underground construction industry can be applied for civil and industrial constructions for construction of basement structures (foundations, base plates, piles) and for direct construction of underground structures (underground garages, basements). Construction of underground collectors is another case, where the only material used is concrete. However, concrete for such structures is mostly in the form of prefabricated elements.

3. EFFECT OF HIGH TEMPERATURE ON CONCRETE

Road traffic keeps growing, which brings higher number of road accidents in road tunnels. This increases requirements of high temperature resistance of tunnel lining. Final surfaces of most of road tunnels are direct finish concrete walls. Therefore, concrete walls are directly exposed to action of high temperatures during fire. Concrete structures with higher resistance to effect of high temperatures imply higher demands on individual components of concrete. The most important part is of course cement. However, other components are also important: aggregate, types of admixtures and additives, method of reinforcement, addition of polypropylene fibers and steel wires.

Temperature during fire in road tunnel can rise as high as to 1200°C, which completely destroys concrete structure. Experiments proved that even much lower temperatures (around 200°C) can cause explosive flaking of concrete in some cases. The main questions of influence of high temperature on concrete involve complex identification of changes in cement matrix and study of transport phenomena. Analysis is complicated by the fact, that cement based concrete is composite material consisting of two very different components: mastic cement and aggregate. Moreover, various types of aggregate have different mineralogical composition. Minerals exposed to high temperature show metamorphic changes, which are typical and different for each mineral. In the end, many changes go on in heated concrete, which result in changes of physical, thermal and mechanical properties (Kodur V., 2008).

4. METHODS OF EXPERIMENTAL WORK

Experimental work focused on designing and verification of mix-design with higher resistance to action of high temperature. In the first stage, input material was selected. Then, mix-designs were designed and tested, in particular as regards appropriateness of composition from the point of view of rheological properties. For further testing, 5 mix-designs were selected, 4 of them with additions improving properties of concrete exposed to high temperatures. Manufactured concrete specimens were exposed to thermal load at the temperatures 200°C, 400°C, 600°C and 900°C with isothermal dwell 60 minutes in ceramic stove. After exposition to high temperature, basic physico-mechanical properties were tested (volume weight, compressive strength) and compared to reference specimens, which were not exposed to high temperature. Surface of testing specimens was analyzed and damage of surface was defined (Xing Zhi, 2011).

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ANALYSIS OF TRANSFER CONSTANTS BASED ON SEISMIC MEASUREMENTS IN TUNNELS ON THE RAILWAY CORRIDOR IV

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KEYWORDS: tunnel, blasting, seismic measurements, Langefor's relationship.

1. INTRODUCTION

Load structures are most often assessed by the maximum amplitude of particle velocity Vmax and theprevailing frequency oscillations. There is a general effort to build a relationship that will predict the speed depending on the size of the total charge (or size bombs detonated per delay) Q and the distance L. For setting maximum velocity in the remote zone using an empirical relationship called Langefor's (Pandula and Kondela, 2010, Dojčár, Horký, Kořínek 1996). This relationship is often provided in the form:

$$V_{max} = K \,.\, Q^m \,.\, l^n \tag{1}$$

kde:

V_{max} - maximum vibration velocity (mm/s),
Q - charge weight (kg),
l - distance from blasting (m),
K, m and n are empirical parameters.

In many publications are given empirical relations similar to Langefor's relationship that optimally suited to the location and conditions for implementing the blasting. All are based on a knowledge of empirically derived constants that are characteristic for the habitat and can be obtained only by parametric measurements. This relationship can be very well defined, the complex geological conditions, however, tends to have very low correlation (*Holub, 2006, Pandula, Bocan, Kondela, 2007*). As a result of the above geological and technological influences is that the size dependence of prediction sought maximum velocities, partial load weight and distance can be determined only approximately, using statistical methods. The actual maximum value of the induced velocity should be determined (checked) by monitoring, and in most instances the measured values show considerable scatter. CSN 73 0040 informative value of the transfer constant K in the subsoil of rock and weak rock and other minerals out of rocks in the saturated environment, depending on the distance from the blast. These values are used to estimate the maximum velocity amplitude of vibration using Langefor's relationship (CSN 73 0040 considering the empirical parameter m = 0.5 and n = 1):

$$V_{\max} = K\frac{\sqrt{Q}}{l} \tag{2}$$

These values are especially suitable for large-scale blasting in surface mines and for distances of hundreds of meters from the blast site. Further examples will be shown from measurements, as can be a constant variable transmission within a distance of hundred meters of blasting in which the total load does not exceed 100 kilograms.

2. EXPERIMENTAL MEASUREMENT

Seismic experimental measurements were carried out in tunnels IV. railway track section on Votice - Benešov (*Gramblička, Mára and Mařík, 2008, Šponar and Kossler, 2010*), namely the tunnel Olbramovice, Tomice Tomice I and II. All measurements were carried out seismic sensor apparatus Gaia2T with sensor ViGeo2. To process the entire data set was chosen methodology, which are deducted at the maximum amplitude value of vibration velocity on the individual components (vertical, horizontal, radial and transverse horizontal) and then the spatial component is calculated as:

$$v = (v_x^2+v_y^2+v_z^2)^{0,5} \tag{3}$$

From calculated maximum amplitudes of the spatial components of velocity constants were then calculated transfer constants of transmission environment K to third relationship. The data processing was the frequency analysis.

3. ENVIRONMENTAL ANALYSIS TRANSFER COEFFICIENT OBTAINED UNDER SEISMIC MEASUREMENT

Based on the standard CSN 73 0040, based on equation (2) were plotted on a graph the coefficients K (interpolated on a straight line) depending on the distance from the source of the dynamic stress (red curve). These coefficients were compared according to standards with the calculated transmission coefficients specific to the environment observed for all three tunnels.

4. CONCLUSION

In the paper was presented analysis of the transmission coefficient K. Real environment coefficients K were obtained by experimental measurements in the near zone of the three tunneling projects in the Czech Republic. The basis for interpretation of measurements was chosen Langefor's analysis of the relationship according to CSN 73 0040 - Actions on structures technical seismicity. Transfer coefficient to the environment (among others depending on the shooting conditions, the transmission properties of the environment, type of explosive,) becomes the reference set of data values from a very wide interval (X0 - kg-1/2.m2.s-1 x00). For each case were introduced two values thus calculated transfer constants K, for the smallest (K1) and largest (K2) reduced distance (at the maximum amplitude of vibration velocity in the reduced distance) (Table 1).

Tab. 1 Overview of the extreme values of the constants K for the observed transfer tunnels

	Olbramovice		Tomice I		Tomice II	
	<u>K1</u>	<u>K2</u>	<u>K1</u>	<u>K2</u>	<u>K1</u>	<u>K2</u>
	109,92	216,06	137,69	157,03	137,91	80,06
v_{max} [mm.s^{-1}]	42,31	11,34	23,09	7,66	35,55	3,51
L [m]	4,5	33	16	55	8,5	50
Q [kg]	3		7,2		4,8	

Figure 1 documents the finding that for distances below 50 m (ie close to the zone) are transmission coefficients environment to a large variance, and this dependence is not well defined. The coefficients in the graph are confronted with the theoretical transmission coefficients environment according to CSN 73 0040 (red curve). Theoretical values do not correspond to real values.

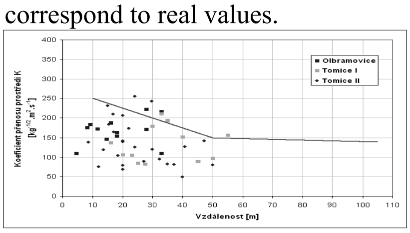

Figure 1 Empirical transfer coefficient depending on the distance to the environment of the monitored data from three tunnels - confrontation with CSN 73 0040

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THE MINING SEISMICITY INFLUENCE ON SURFACE IN OSTRAVA-KARVINÁ COALFIELD - FIRST RESULTS OF MONITORING WITH NEW SEISMIC NETWORK

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KEYWORDS: Seismic stations, surface tremor, induced seismicity, coal mines, prediction.

1. INTRODUCTION

The new seismic network was built on the basis of management decisions OKD, a.s. in 2009. The seismic stations on the surface (PS) were installed in mining areas in Karviná part of Ostrava-Karviná Coalfield (OKR). In the article the first results from this seismic monitoring are presented.

2. INDUCED SEISMICITY

Seismic stations on the surface (PS) were installed at a total of 15 localities during the first half of 2009 (Holečko 2010). Six stations have been moved to the areas of increasing occurrence of strong seismic events and surface vibrations in years 2010-2011. Measuring apparatus is AMAX-GSI - made in GIG Katowice. Sensitivity registration and detection capability are limited by level of seismic noise (≈0,1 mm.s^{-}1, occasionally even 1 mm.s^{-1}) and by maximum value 200 mm.s^{-1}, frequency range is 2-50 Hz. Records of seismic events (triggers) are sent via the Internet to the evaluation center in Paskov.

The basic information about the current measurement and about the maximum values of horizontal peak ground velocity VH_{max} are shown in Tables 1 and 2.

Table 1 The number of seismic events in OKR in the period 10. 9. 2009 - 31. 12. 2011

Energy [J]	Seismic Events OKR / year			Seismic Events PS / year			Seismic records PS / year		
	2009	2010	2011	2009	2010	2011	2009	2010	2011
Total / year	278	250	344	139	186	204	314	500	530
Total 2009-2011	872			529			1344		

Table 2 The number of seismic records at PS in the period 10. 9. 2009 - 31. 12. 2011

15 seismic stations	Number of seismic records / VH_{max} (mm/s)					Total records	VH_{max}	Dep min	Dep max
	< 3.0	3.0 - 4.99	5.0 - 9.99	10.0 - 14.99	≥ 15.0		(mm/s)	(km)	(km)
21 localities	1 209	66	51	13	5	1 344	33.4	0.12	11.69

Table 1 shows the number of seismic events (energy from 1,0x104 J) registered in OKR in the period 10.06.2009 - 31.12.2011 - total 872 seismic events. Only 529 seismic events were recorded by fifteen seismic stations on the surface and total of 1000 records were obtained from these seismic events. The largest energy was 1,4x107 J, the largest value of magnitude was 2.61. The number of records varied from one station to another signicantly. Values of horizontal peak ground velocity VHmax were mostly smaller than 3.0 mm.s-1, but strong seismic events reached values over 10 mm.s-1, the largest value was 33.4 mm.s-1 - see Table 2.

Tables 1 and 2 show that the current number of records especially of the strong seismic events is too small, and therefore, is still insufficient for the determination of reliable relationships between VHmax and seismological parameters (energy and hypocentre distance of seismic events).

3. PREDICTION OF HARD ROCK GROUND VELOCITY MOTION AT THE SURFACE

3.1. GENERAL PREDICTION

The regional formula for general forecasting of ground velocity in conditions of OKR was tested first. For the Upper-Silesian Coal Basin's area in Poland (GZW) a formula for amplitudes of ground velocities at bedrock has been developed (Mutke 1991, Mutke and Dworak 1992). The empirical relationship used for determining vibrations of bedrock takes the following form for OKR:

$$V_{MD} = [1.48 \cdot 10^{-3} (\log E_p)^{1,23} - 0,011][1.55 R^{0.135} \exp(-0.77R) + 0.04] \;,\; R = \sqrt{D^2 + h^2} \;,$$

where: $E_p = 386 \cdot E^{0.7895}$, E – seismic energy [J] , D – epicentre distance [km], h – focus depth [km] .

The general prediction must take into account the influence of local near-surface strata structure on ground vibrations at the surface - coefficient of amplification Wf .While calculating Wf it is necessary to take into account thickness and lithological structure of overburden strata and/or velocity of transverse wave propagation Vs in the overburden strata up to a thickness of 30 m. The final formula for the maximum value of horizontal peak ground velocity VHmax predicted at the ground surface shall have the following form:

$VH_{max} = V_{MD} \;.\; W_f$.

This value of VHmax may be a measure of instrumental intensity of the vibration. The formula is applied for assessing results of mining-induced seismic events with the help of the GSIGZW scale in Poland (Dubiński at al. 2009) - it is necessary to adopt the calculated value of ground velocity VHmax and vibration duration above 3 s.

3.2. LOCAL PREDICTION

The detailed local prediction requires an appropriate number of tremors recorded at the surface. We decided to process the first local test formulas based on the record of surface seismic stations. The local formula for conditions of OKR should be more accurate than the regional formula for GZW. The formula for relationship between VH_{max} and energy and hypocentre distance of seismic events is based on a model Joyner-Boore (Joyner and Boore 1981) - the level of vibration increases with energy and decreases with epicentral distance. A linear relationship was calculated in the commercial program STATISTICA by regression method.

An example of regression testing formula, determined for the station 13 (in Doubrava) for seismic events with a duration 1.5 - 3s, epicentre distance 0.15 - 4.00 km, seismic energy $1x10^4$ J - $1x10^7$ J, is given:

$\log(VH_{max}) = 0.58\log(E) + 0.37\log(D) - 6.39$, where: E – seismic energy [J] , D – epicentre distance [m]

Values of VH_{max} for record of the seismic events 29. 6. 2011, energy $4.4x10^6$ J, D = 0.24 km are: regional prediction (10.0 $mm.s^{-1}$, vibration duration > 3 s), local prediction (22.0 $mm.s^{-1}$, vibration duration 1.5 - 3.0 s), measured (31.6 $mm.s^{-1}$, vibration duration 2.0 s). Local prediction is better than general prediction, but both predicted values are very different from the measured value at the seismic station.

4. CONCLUSION

The current data set of seismic records is too small to develop credible local empirical formulas for predicting maximum ground velocity amplitudes and vibration duration. Formulas may be developed for a larger data set of strong seismic events. Neither is there enough data yet to verify the scale GSI in conditions of OKR.

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NUMERICAL MODELLING OF STRESS INFLUENCE TO FLOW AND TRANSPORT IN FRACTURED ROCK

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KEYWORDS: fracture dilation, numerical simulation, Bandis-Barton model, coupled problem .

1. INTRODUCTION

The study is motivated to analyze a change of isolation properties of rock due to excavation and heat load, in particular for safety assessment of spent nuclear fuel disposal. Both experimental and numerical tests show nonuniform change of fracture aperture for stress change, which can cause higher speed of water flow and radionuclide migration.

Standard solution with distinct element method (e.g. implemented in well known software UDEC by ITASCA CG Ltd) combine simulation of stress on discontinuities (fractures) and inside the deformable matrix blocks. We show how also a simple model of fractures stress and deformation, with projection of external stress to the normal and tangential direction of each fracture and use of non-linear constitutive law for the fracture, can give comparable results of stress influence to the flow.

2. MAIN MODEL FEATURES

The solved benchmark problem is a 2D fracture network – system of 7797 lines in the square model domain 20m x 20m, stochastically generated with parameters from field analyses, with set of prescribed mechanical pressures in two directions and two directions of hydraulic gradient (Fig.1). The problem has been defined as a benchmark within the project DECOVALEX, aimed to comparison of coupled models of thermo-hydro-mechanical phenomena in rock and follows the former studies of stress influence to hydraulic properties (Min et al 2004, Baghbanan a Jing 2008).

The prescribed gradient for hydraulic problem is 10^4Pa/m and the variants of stress load are given by ratio of horizontal and vertical components, k=1,2,3,5 (vertical is always 5MPa, except unloaded state denoted as k=0). The constitutive relation stress-deformation for fractures comes from the Bandis-Barton model and is used according to Baghbanan a Jing (2008) a Hudson et al (2008). In the normal direction, the relation is nonlinear hyperbolic type and represents rise of stiffness during fracture closure (full closure for infinite stress). In the shear direction, the elasto-plastic model with Mohr-Coulomb slip criterion is considered, with normal dilation in the slip part (opening of fracture caused by shift of uneven fracture surface). If we apply the constitutive relation for each fracture individually (i.e. we project full external stress to only the particular fracture and decompose to normal and shear, without influence of other fractures), it is not possible to determine displacement from given stress in the perfect plasticity model (in the slip regime). Thus we derived a simple model allowing to determine equivalent (nonzero) stiffness of fracture in the slip regime, which is in fact given by stiffness of rock matrix.

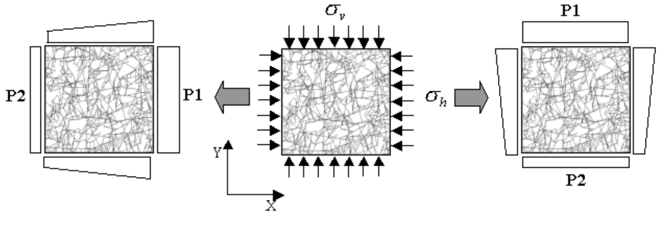

Fig. 1 Definition of boundary conditions for the stress problem (in the middle) and for the flow problem (two variants – horizontal gradient left and vertical right) (from Hudson et al. 2008).

3. RESULTS

The evaluation comprises a lot of particular results of models – stress/deformation, water flow, and solute transport – which are connected between each other. The results of mechanical model is the set of new apertures, which is the input for the flow model (hydraulic). The results of the flow model is the field of velocity and pressure in the fractures, while the total flowrate representing equivalent hydraulic conductivity is used for evaluation (Fig.2.). The velocity field is input for the solute transport model, whose result is the breakthrough curve, i.e. outflow concentration or aggregated mass as function of time. Besides that, mean residence time and variance of residence time are calculated.

We observe influence of changing horizontal/vertical stress ratio to evaluated quantities of flow and transport. With respect to unloaded state, the fractures naturally close with larger stress and thus the total flowrate decreases and mean residence time of particles rises. For the largest stress ratio (25MPa horizontally and MPa vertically), the slip criterion is exceeded for many fractures and they open due to dilation, but only for certain range of fracture orientation. Thus the flowrate moderately rises, but only for horizontal pressure gradient.

The reaction of residence time (and the migration velocity as well) is different then the flowrate – the relative change is smaller (Fig.2 right) and thus simulations confirmed the hypothesis of opening small number of most conductive fractures and thus the solute migration is quicker than could be expected from the hydraulic conductivity change (channeling effect). Results of other teams within the DECOVALEX project, including the comparison with those here presented, were compiled to the submitted paper Zhao et al 2012.

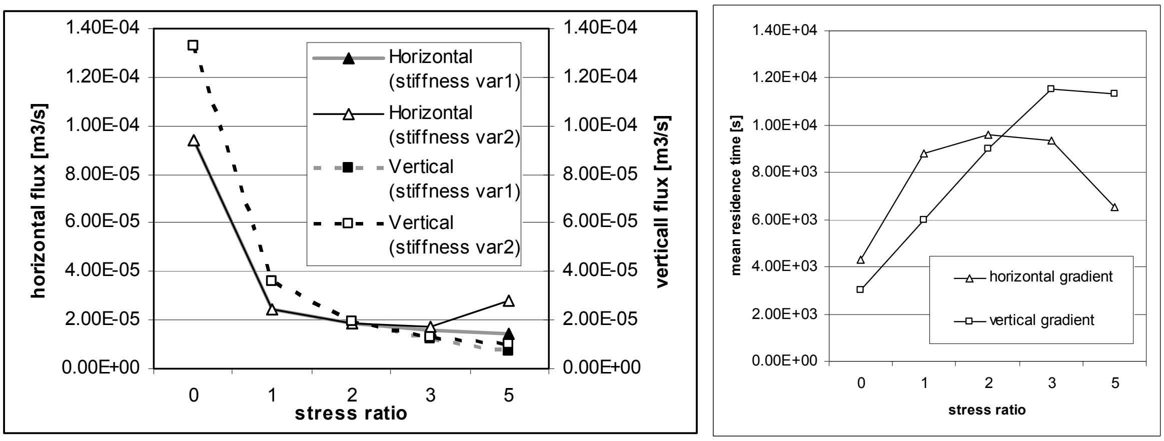

Fig. 2 Influence of stress ratio on flowrate (left) and on the mean residence time (right), for horizontal and vertical pressure gradient.

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GEOPYSICAL AND GEOMECHANICAL MONITORING OF THERMAL EXPERIMENT AT THE JOSEF GALLERY IN MOKRSKO

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KEYWORDS: energy storage, granit, temperature, heating, thermal features

1. INTRODUCTION

There are several possibilities of storing redundant energy produced by renewable sources such as wind power plants or photovoltaic power plants. Along two the most famous approaches – water heating and salt melting – a rock heating is starting to be similarly promising. This procedure can be realized at a real rock massif or at a concrete block with heating exchangers. The in situ rock heating expects a specific rock features as thermal conductivity, thermal load stability etc.

2. ABSTRACT

Within the reasearch project no. MPO TIP-FR-TI 3/325, investigated by Czech Geological Survey in cooperation with Arcadis Geotechnika a.s., ISATech s.r.o., Technical University in Liberec and Institute of Rock Structure and Mechanics ASCR v.v.i., were done preliminary works and technological installations for performing in situ rock heating experiment in the Josef gallery in Mokrsko.

The initial steady state of rock massif have to be described. Its basic geomechanical and geophisical features must be determined to be able to objectively explain changes made during the rock heating phase. Because of these reasons geological and hydrogeological studies were carried out along the geoelectric and shallow seismic surveys. Results of these works are being described here and expected changes of the rock features are discussed. It is going to be verified during the heating experiment as a complete hypothesis.

A mathematical model of environment is a basic part of thermal features, thermal conducting, thermal storage and thermal regaining study. There are assumed many geomechanical, hydrogeological, geochemical and geophysical methods for real state of rock continuous monitoring. This monitoring will be performed using the know-how a technology developed by Institute of Rock Structure and Mechanics. Current schedule suppose pressure monitoring in the frontal exploring drill hole, temperature of nearsurface rock measuring, air temperature measuring, air moisture monitoring and seismic backgorund monitoring. Furter there will be installed up to fifty thermometers into drilled holes, seismo-acustic resivers for micro-crack monitoring, fault zones convergency monitrors, drill-hole convergency measuring and continuous tension monitoring. Along the in situ preparation phase there are being performed laboratory tests on rock samples from the area of interest. These samples are subjects of experiments at normal and expected final in situ conditions.

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INDUCED SEISMIC ACTIVITY IN THE SOUTHERN PART OF THE OSTRAVA-KARVINÁ COAL MINES

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KEYWORDS: Ostrava-Karviná coal mines, seismic network, induced seismicity, location plot

1. INTRODUCTION

Apart from the eastern part of the Ostrava-Karviná coal mines also its southern part is of considerable interest. The two mentioned parts are incommensurable, particular with regard to the thickness of their coal seams and level of induced seismic activity. Whereas the thickness of the coal seams in the Karviná mines reaches almost 6-7 m, in the southern part the thickness is roughly up to 1.5 m. It is indeed the extent of winning in the past and present, and the complicated geological structure of both parts, in which mining causes instabilities within the rock massif and their gradual, but sometimes even sudden, compensation or redistribution of residual stresses, generates increased seismic activity. As opposed to the Karviná Basin, in which rockbursts and shock phenomena in general represent considerable danger for mine workers, sometimes causing partial or complete devastation of mine workings, in the southern part of the district the mine workings are endangered by phenomena such as outbursts of coal and gas. These outbursts have occurred from time to time in the past and they also occur in the present, their occurrence frequency being lower, but the consequences in mine workings as compared to rockbursts in the Karviná area are substantially less devastating. The seismic phenomena are generated mostly as a result of either spontaneous caving falls, or blasting in the roof.

The problem of induced seismic activity in the Ostrava-Karviná coal mines, with particular regard to its southern part, has been the subject of papers published earlier, e.g., Holub et al. (2002), Holub and Rušajová (2004), Holub et al. (2004). This paper essential ties in with the referenced papers and in particular summarizes new results of the research into induced seismic activity in the southern part of the coal deposit in the interval from 2008 to February 2012.

2. MONITORING NETWORKS

For the purposes of detection and localization of induced seismic events in the Ostrava-Karviná coal mines, a seismic polygon of 15 stations (Klíma et al., 1984) was originally planned in the middle of the 1980's and, in the course of being established, was reduced to 10 stations, equipped with digital instruments in the Karviná part of the coal deposit which is run by Green Gas DPB, a.s. To satisfy the original intention of comprehensive monitoring of induced seismic activity also in the southern part of the deposit, it was decided that the Mining Institute of the AS CR (now the Institute of Geonics of the AS CR) in Ostrava-Poruba was to be entrusted with the development and manufacture of the equipment for the 5-station network in the Frenštát area. The new system was designed with parameters similar to that of the system of the 10-station system with data radio transmission using the retranslation path via Lysá hora to the Geophysical Laboratory of the Mining Institute AS CR, i.e. functioning as an independent seismic network. A brief description of the new equipment, including the SW, is given in the papers of Toth (1991) and Knejzík and Zamazal (1992). However, as it turned out in the course of operating the Frenštát Seismic Polygon (SPF) the stations of this network recorded, e.g., natural microearthquakes, quarry blasts carried out in neighbouring quarries, but most frequently seismic phenomena, including rockbursts, which occurred in the mines of the Ostrava-Karviná coal mines, but also in the coal and ore mines in neighbouring Poland.

The Frenštát seismic network mentioned has contributed significantly to the study of induced seismic activity in the Paskov and Staříč mine fields having recorded a number of seismic events in the course of its operation, 50 of which were successfully localized. The increased seismic activity in this area caused the equipment of the surface station in the area of the ČSM mine in the Karviná area to be moved to the Brušperk locality, which is substantially closer to the focal regions in the Paskov and Staříč mine fields, in order to improve detection and thus also the accuracy of localization. The SPF network was operated without failure from December 1998 to January 2002, and the assessment of its contribution to the monitoring of induced events in the Frenštát area has been described in the summarizing paper by Holub et al., (2004). After the operation of the SPF was terminated, the quality of the information on the activity in both the mine fields deteriorated.

3. PRESENT CONDITION OF MONITORING INDUCED EVENTS IN THE PASKOV-STAŘÍČ AREA

After the operation of the local SPF seismic network was terminated, only station Brušperk alone remained in the originally well monitored area, because most of the SP stations of Green Gas DPB Paskov, a.s., whose data can be used for localization, are located in the eastern part of the Ostrava-Karviná coal mines. In order to record at least the stronger events from Staříč, station Brušperk now has a larger weight for triggering events. The stronger of these, which are localized, are then stored in the database of seismic events and annotated "Staříč". A number of weaker events are also detected, but because they cannot be localized, their records are labelled as noise, which is later erased.

This was later reflected in the course in the interpretation and localization of events recorded at the local stations run by the Institute of Geonics of the AS CR, the Institute of the Physics of the Earth of the Masaryk University in Brno, and the station of the nationwide network of stations of the Institute of Geophysics of the AS CR in Prague. After comparison with the data of the Green Gas DPB, a.s., it was found that not all seismic events or blasting operations from the mine area have been included in the database of events, because some of them were recorded and localized only by stations operated by various institutions mentioned before. The results of the localization of all seismic events can be seen well in Fig. 1, which also proves that induced seismic events from the region of interest are also being recorded to date.

Fig. 1 Calculated focus positions of seismic events within the periods 1992-2002 (○) and 2008-4/2012 (×).

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APPLICATION OF ROTATIONAL SENSOR FOR MONITORING OF MINING INDUCED SEISMIC EVENTS IN KARVINA REGION

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KEY WORDS: rotational sensor, mining induced seismicity, time-frequency analysis

1. EXTENDED ABSTRACT

Rotational seismology points to the fact that to the full description of the vibration is in addition to the three translational components and strain necessary to know also three rotational components (e.g. Båth, 1979, Teisseyre et al., 2006, Lee et al., 2009). Russian seismometer signed S-5-S was adapted by staff of Institute of Geonics ASCR in 2010 to the stationary sensor measuring rotational component of seismic vibrations in the frequency range 0.2 - 25 Hz (Knejzlík et al., 2011a). Industrial Property Office of the Czech Republic registers Utility model for this adaptation in the following year. This is the one-component sensor that can register the rotational motion around either vertical (Figure 1) or horizontal axis. It is designed to obtain either velocity or acceleration of rotational motion. The basic tests of adapted sensor, which is marked as S-5-SR, were realized using rotary test table in the Geophysical Institute ASCR, v.v.i., Prague (Knejzlík et al., 2011b).

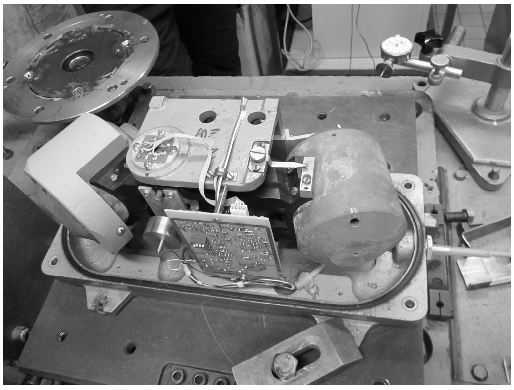

Fig. 1 The S-5-SR seismometer for the measurement of the rotational components of vibration around the vertical axis (without cover of sensor)

Karvina area, in which foci of mining induced seismic events are localized under seismic station, was elected to verify the functionality of the sensor S-5-SR in the field (Doležalová et al., 2008, Kaláb et al., 2011a). The latter mentioned condition is necessary because it is possible to record rotational components in epicentre area only, outside of which these components are quickly attenuated. First records of rotational components were obtained during experimental measurement in 2011/2012; it confirmed their existence also for the mining induced seismic events. Measured values of rotation motion around the vertical axis exceeded 1 mrad.s^{-1} during operating seismic stations in Doubrava and Orlova villages; it was for mining induced seismic event with energy 10^5 J (Kaláb and Knejzlík, 2012).

Record example of translational components of vibration velocity (signed as Z – vertical, N – horizontal N-S, E – horizontal E-W components) and the rotational component of vibration velocity around the vertical axis (signed as ROT) is presented on figure 2. Presented wave pattern is stronger mining induced seismic event with hypocentre distance about 1.5 km, according to S-P differential time. This record was obtained in the Orlova

station where the sensors are located in the basement floor of the large building. The sensors are connected to the seismological apparatus, in which the record was digitised using 100 Hz sampling frequency (which is sufficient in relation to the frequency range of the sensors). The maximum recorded value of vibration velocity for this event has reached nearly 8 mm.s-1; the maximum measured value of rotation velocity was almost 0.3 mrad.s-1. These rotational components of vibration may not present a danger for the building design, they may, however, cause torsional stresses on the structure in the epicentre area for the very intensive seismic event.

The results of time-frequency analysis of digital records are also presented in the article. Seismological software that was developed by Prof. Lyubushin from Moscow for special analysis of digital records (time-series) of different types was used. This software package, in addition to the ordinary operations with digital data as view, selection of the interval, re-sampling, ..., enables data processing using Fourier transform and wavelet one, it provides a spectrum of singularity, wavelet decomposition and further processing of the data (Kaláb et al., 2011b). Records of technical seismicity (e.g. the passage of trucks) do not obtained recognizable values on the rotational component. The records of distant events, such as the effect of mining induced seismicity from the Polish part of the Upper Silesian Basin, have no detectable rotational vibrations.

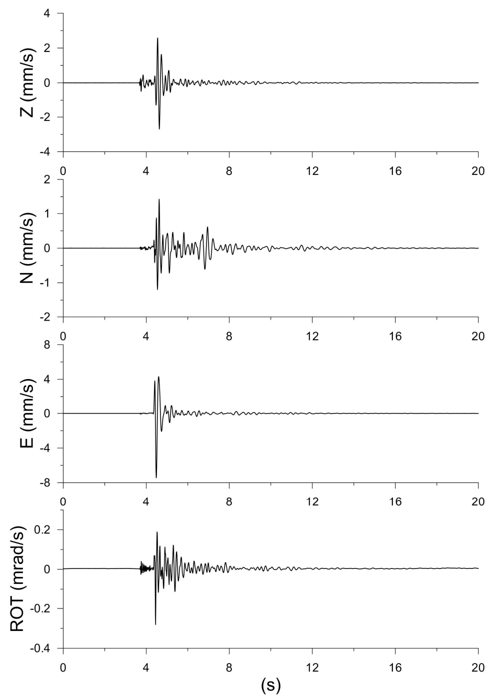

Fig. 2 Example of translation components of velocity vibration (signed as Z, N and E components) and the rotational component of velocity vibration around the vertical axis (signed as ROT); horizontal axis is time.

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CURRENT RESULTS AND STATUS OF TESTING "STATIONARY ROTATIONAL – SLIDING TUBES SYSTEM (DRR)" IN OKD COAL MINES TERMS

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KEYWORDS: prediction, stress in rock massif, drilling tests, stationary rotational – sliding tubes.

1. INTRODUCTION

In the Czech Republic is monitoring of stress changes in rock massif (Jiránková et al., 2012) during operating solved by using legal regulations in form continuous prediction. Continuous prediction contains the method of individual gateroads watching, drilled tests method (small diameter drilling testing or high diameter indication distressing boreholes) provided in coal seam and the seismological monitoring method of rock massif.

In the Federal Republic Germany has been find and developed, for purpose of monitoring stress changes in area closed intermediatelly to gateroads, method named - „stationary rotational – sliding tubes system (DRR)". This method is used in borholes after drilling tests after conditions noticed in directive of regional mining authority (valid for North Rhine - Westphalia area) for purpose of protect against hazards caused by mining tremors (rockbursts). This Directive sets out criteria, which can not be drilled test bores equiped with "DRR" system. Possibility of application this metod in OKR terms is actually verified.

2. METHODOLOGY OF USE „DRR" SYSTEM IN OKR TERMS

For application of the DRR in OKR terms were taken establishing criteria under which conditions can be drilling test borehole after use. These criteria are enshrined in the German legislation aimed at mining (Directive, 2008).

The criteria under which it is not possible after done drilling test equipped borehole with "DRR" system:
a) drilling crushed yield is greater than 6 l / m (Baltz, 2002)
b) occurrence of one or more cracks in the rock,
c) clamping of drill rods caused by stress in rock,
d) pulling the drill rods into the borehole.

The parameters of boreholes used for the DRR method:
a) diameter 42 mm
b) length max. 16 m (Dvorsky, Kubica, 2010)
c) spacing of 20 m to 30 m
d) recommended an advance ahead working face front – "L".

The procedure for measuring and evaluating of "DRR" system:
a) each operating day during preparation shift will be done manually physical inspection of rotating all installed swivel tubes (rotating tube shall extend from a borehole 0.3 m to 0.5 m for manual rotation option)
b) if it is not possible rotate with outer rotating tube, will be done control of the movement the inner sliding tube (placed inside the rotating tube)
c) result of the control rotation of swivel tube or sliding motion tube "yes" or "no", will be recorded in the primary documentation
d) if it will be not possible sliding movement of the inner tube, this state will be evaluated as adverse, and at a distance of 2 to 3 m on either side of this adverse control drilling well will be made control drill tests of length equal to the length of adverse borehole

e) in case of adverse results of drilling tests will proceed according to the Annex to the technological procedure "Special measures against rockbursts".

3. RESULTS OF VERIFICATION SYSTEM "DRR" IN TERMS OF OKR

In the OKR terms is using of the "DRR" method checked at the chosen sites of Karvina Mine (plant CSA) and Darkov Mine (plant Darkov 2) in seams No. 34 (558), No. 37 (530) and No. 40 (504). At the Karvina Mine (plant CSA) there are panels No. 22 3452, No. 22 3750 and No. 22 3752 in the area of 22nd mining block, i.e. abolished safety pillar of Doubrava shafts. It was further verification of "DRR" method carried out in panel No. 1 4068, in the 1th mining block of Doubrava mining field and at the Darkov Mine in panel No. 340 206, in the 2nd mining block on plant 2 - Gabriela aera.

The results so far obtained by physical control of "DRR" system, maps and partial evaluation of the application of a "DRR" method in panels No. 1 4068 and No. 22 3452, it is possible to draw following findings. Method "DRR" can be considered as a Express authentication prediction method of stress state around the working face of gateroads. Like when by using of the drilling test method are verified stress conditions in this particular place, i.e. spot information is obtained only (Ptacek, 2011).

Noteworthy is the fact, when the face of panel No. 340 206 after approaching to backfilled roadway No. 340 224 at a distance of about 100 m and less („L“ = 117 m), starts the loss of the DDR probe at a distance of about 70 m from the working face front. A unique case is the situation where in the same face the working face front approached to mined out overlying coal seam No. 37f (about 85 m above the seam No. 40) at a distance of about 15 m and there was a gradual loss of the probes at a distance of about 40 m to 85 m from the working face front. In other areas of panel No. 340 206, outside area of this mined out overlying coal seam No. 37f, the working face front approached to the probes at a distance of 5 meters. In these rare cases, can not draw definite conclusions, but it is necessary to obtain more detailed information to continue the verification"DRR" method mainly in the areas of panels with greater target length. All cases of non-functioning "DRR" system were verified by performing verify drilling tests, drilled from both sides of this adverse borehole, with the same length as was the length of the averse borehole bad. In addition to the performance of this „DRR“ method is in the gateroads face at a distance L from the panel coalface continuous prediction made by using drilling tests.

The disadvantage of the method "DRR" is a necessity, after the loss of system functionality, execution of a pair conventional drilling tests to verify the stress state in this particular location. Then there is the fact that in case of the loss of system functionality (retraction tubes) must be given state always considered an adverse, while in case by using drilling tests are crushed yield from tests between each meters set in a definitive liter range.

Due to the fact that the method "DRR" is not accepted by the Czech legislation as the official prediction method, this method can be consider only as an additional prediction method.

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PERMEABILITY AND VOLUMETRIC CHANGES IN COAL UNDER DIFFERENT TEST ENVIRONMENT

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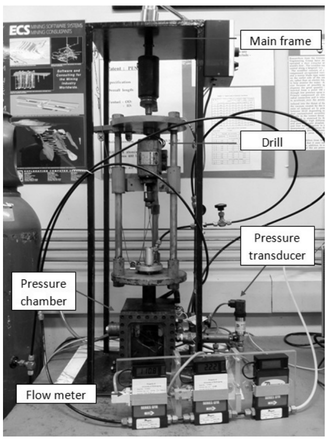
KEYWORDS: coal mine gas, coal permeability, volumetric changes

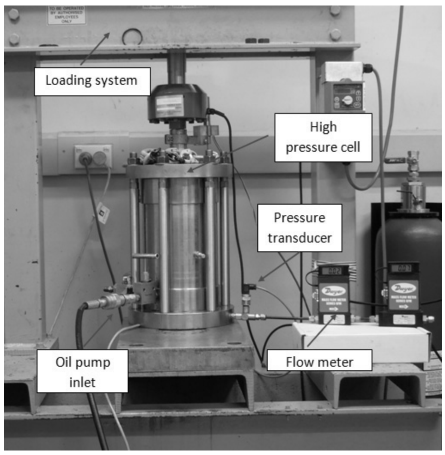
1. INTRODUCTION

Permeability refers to the ability of coal to transmit gas when a pressure or concentration gradient exists across it. Permeability is considered by many researchers to have a significant impact on a coal seam's ability to produce gas). Permeability is closely related to the coal fabric (i.e. cleat spacing and aperture width) and varies significantly as fluid pressure changes during coal seam gas production. Permeability has a strong effect on the gas production profile and gas well performance and is dependent upon various factors, which include effective stress, gas pressure, water content, disturbance associated with drilling and matrix swelling/shrinkage due to adsorption/desorption.

A laboratory permeability testing programme was initiated by the gas research group of the University of Wollongong., to investigate and evaluate the permeability and drainability of coal collected from the samples from the Bulli seam, Sydney Basin, NSW, Australia. The programme consisted of duplicate testing of coal using two different permeability testing apparatus. Both tests were carried out under triaxial test conditions. The first permeability testing method was carried out using a Multi Function Outburst Research Rig (MFORR) , is illustrated shown in Figure 1(Aziz, and Li-Ming, 1999). In this test, the coal sample was enclosed in a triaxial gas chamber, and then subjected directly to gas as the confining pressure. The pressured gas was made to filter through the coal sample while it is being loaded axially. A centrally drilled hole in the coal sample allowed the gas to flow out of the chamber in a controlled manner. The second permeability testing apparatus used in this study is a high pressure triaxial cell (Figure 2), initially built for determining the relative permeability of coal measure rocks under two-phase flow conditions.



1. **Obr. 1** Multi Function Outburst Research Rig



2. **Obr. 2** Triaxial High Press cell

The permeability of the coal sample tested in MFORR was calculated using the following Darcy's equation:

$$K = \frac{\mu Q \ln\left(\frac{r_0}{r_1}\right)}{\pi L\left(P_1{}^2 - P_2{}^2\right)} \qquad (1)$$

Where K is the permeability of coal, μ is the viscosity of gas, Q is the flow rate of gas, L is the height of the sample, ro and ri are the external radius and internal radius of sample, P1 and P2 are absolute gas pressure inside and outside of chamber, respectively. And the permeability of the coal tested in high pressure cell was calculated by using the modified Darcy equation, in which A is the cross section of specimen.

$$K = \frac{2Q\mu L P_2}{A\left(P_1^2 - P_2^2\right)} \qquad (2)$$

Figure 3 shows the permeability test result with MFORR apparatus, which is being pressurised by N2 gas, at different applied vertical stress levels. For each of the vertical stress level, the coal sample permeability decreases with increasing gas pressure and at higher gas pressure, coal permeability stays stable and changes very little, under different vertical stresses. The test results show that the permeability values stay below 2 mD when the applied confining gas pressures became greater than 0.5 MPa.

Figure 4 shows the triaxial permeability test results with N2 at different gas pressures carried out at 6 MPa confining vertical stress. The samples behaviour laterally was monitored with two strain gauges. The coal sample permeability decreased with the increase in gas pressure. At higher gas pressures, coal permeability remained constant, a similar trend as with the permeability test results with MFORR. At each vertical stress, coal permeability test decreases with increase in horizontal stress.

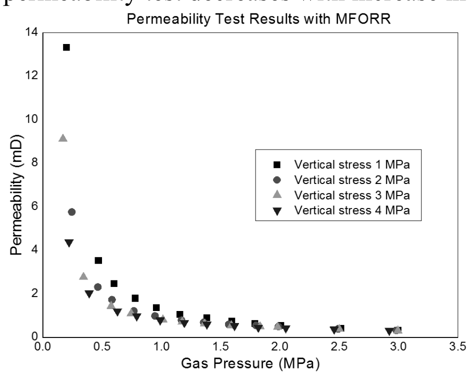


Fig. 3 Coal permeability test results with MFORR

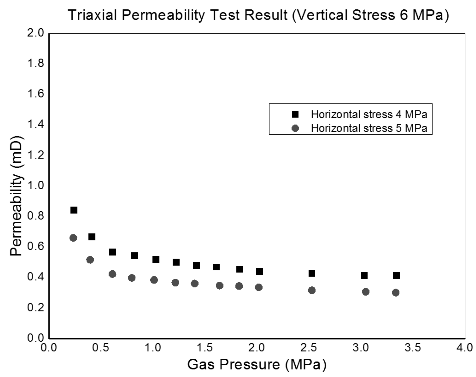


Fig. 4 Coal permeability test at 6 MPa vertical stress using high pressure triaxial cell

Figure 5 shows a comparison of the permeability results between the MFORR and triaxial tests at various vertical stresses. Although the results show some significant difference in permeability values at lower confining gas pressure because of the relatively low confining pressure of MFORR test, the permeability converges to a steady level below 2 mD under high triaxial stress conditions, portraying the near in situ conditions of the Bulli seam of Sydney Basin. No significant mathematical difference between the two different types of testing apparatus and calculation method were evident.

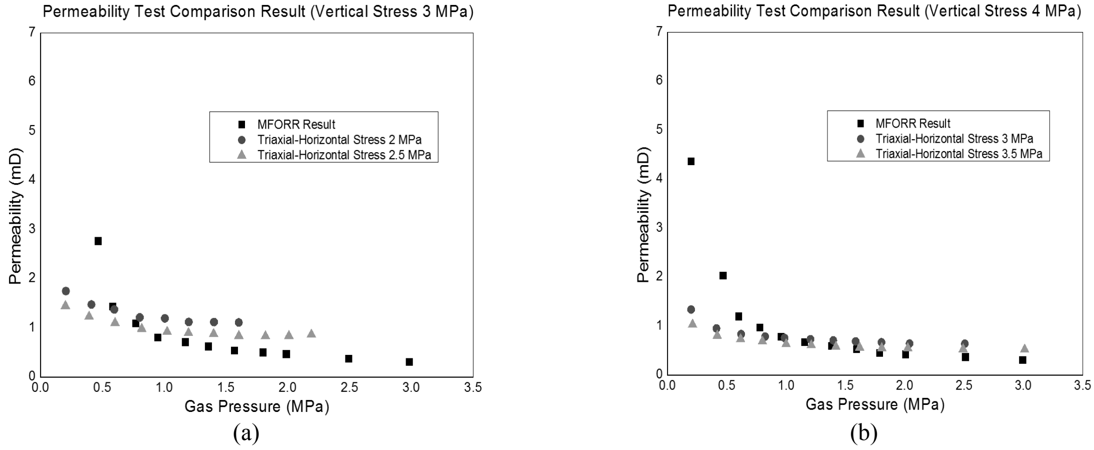


Fig. 6 MFORR permeability and triaxial permeability test results comparison

Similar results were confirmed with the other studies; Hayes (1982) reported that the Bulli seam coal permeability was considerably less than 1 mD. Lingard et al. (1984) reported permeability of Australian coals from Appin, West Cliff and Leichhardt collieries that varied from less than 0.1 mD to 100 mD. Recently the Bulli seam coal permeability was measured using a combination of injection / falloff and step-rate testing methods and the results from 31 locations of the Bulli seam at West Cliff Colliery showrd the average in situ permeability of coal as 2.2 mD, with the range extending from a low of 0.005 mD to a high of 5.8 mD.

In conclusion the study has demonstrated that the permeability of coal decreases with increasing gas pressure irrespective of the type of the apparatus used. The level of the permeability decrease however, tapers to a constant level at increased vertical stress and confining pressures. There is no significant mathematical difference between the two types of testing apparatus and calculation methods used. Both permeability test results are comparable and tally's well with the Bulli coal seam test result calculations.

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STABILISING ROCK SURFACES WITH A GLASS REINFORCED POLYMER SKIN.

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KEYWORDS: Bolts and anchors, laboratory testing

1. INTRODUCTION

Mine roadways developed in highly stressed strata are subject to roof shear, which under severe conditions may manifest as the well known symptom of guttering, particularly at the roadway edge leading into the major horizontal stress as shown in Figure 1. This roof shear can progressively reduce the effectiveness of bolt confinement of the strata within the lower roof horizon affecting stability of the immediate roof. This paper presents the results of a study to investigate the effectiveness of polymers as skin reinforcement in highly stressed coal mine roadways, as they may provide better roadway skin support than the currently used steel mesh. In order for polymer liner products to be regarded as a suitable replacement for steel mesh, the advantage of polymer reinforcement must be clearly demonstrated.

2. COMPARING THE SKIN SUPPORT CAPABILITIES OF STEEL MESH AND POLYMER SKIN

To assess the reinforcing capabilities of steel mesh and glass reinforced polymer skin supporting damaged sedimentary strata a steel mesh and 5mm polymer layer was bonded to a jointed concrete block as shown in Figure 2

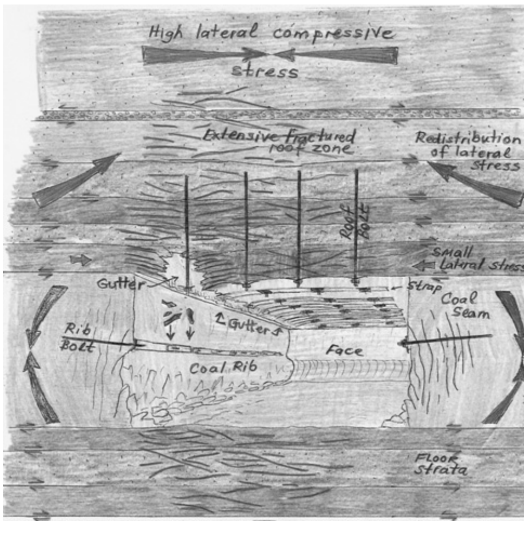

Fig. 1 Typical roof conditions in coal mine roadwayin a high lateral stress environment

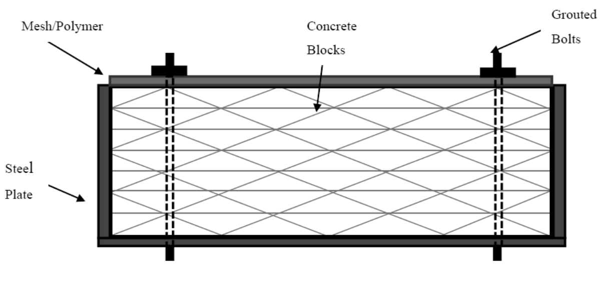

Fig. 2 Test specimen formed from concrete prisms to imitate fractured strata

Three large scale tests were conducted. In addition to a test with no skin reinforcement, which produced a predictable skin failure, a second test used the glass fibre reinforced polymer for skin support and another used steel mesh. The block dimensions were restricted to 800 x 400 x 400 mm in size due to the loading machine size. The confined concrete block was then mounted into the loading machine and loaded at a rate of 0.5 mm per minute while the load and displacement were monitored (Figure 3).

The test results presented in Figure 4 indicate that the glass reinforced polymer bonded to the jointed concrete block provide stiffer and higher reinforcement than the passive support of steel mesh. On the whole, both the steel mesh and the polymer skin did not reach their ultimate strength as each experiment was terminated due to excessive movement of the concrete blocks and the unsafe conditions that occurred at the later stage of each test. The initial stages of load versus displacement for both tests indicate similar behaviour, however, at the later stage accelerated loading of the sample reinforced with the polymer skin was experienced.

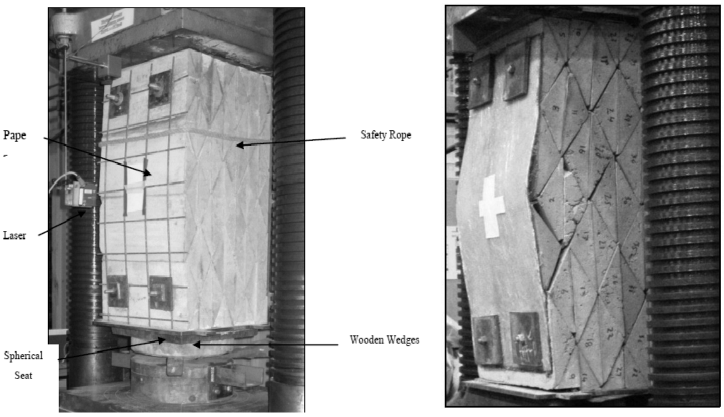

Fig 3 Loading of two concrete specimen supported with steel mesh and reinforced with polymer skin

The experimental tests were simulated using the Universal Distinct Element Code (UDEC). The properties of steel mesh are very complex, as they are highly dependent on the direction in which the mesh is loaded. For the steel mesh sample, the modelled properties were adjusted to represent the percentage of the steel mesh actually covering the total surface area. The numerical models examined the steel mesh and the polymer surface support and its ability to confine the strata. The modelled block was positioned in the same way as the tested physical model. Graphical representations of the loaded models are shown in Figure 5 (a) and (b) respectively.

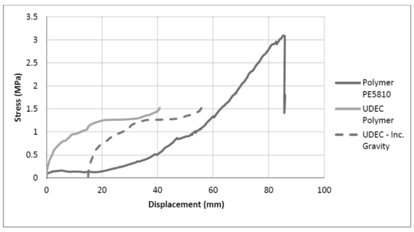

Fig 4 Load vs displacement resultsfor tested concrete specimen reinforcedwith the polymer skin and steel wire mesh

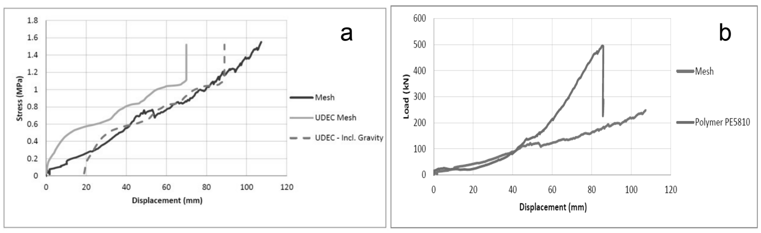

Fig 5 Loading and displacements of UDEC model supported with (a) steel wire mesh, (b) glass reinforced polymer skin, compared to the experimental data.

3. CONCLUSION

Steel mesh provides passive support to the substrate, while it is expected that a polymer liner would adhere to the substrate and act as composite with the rock. These tests demonstrated the concept of effective skin reinforcement. The modelled results clearly support the laboratory experimental results and demonstrate that the glass reinforced polymer skin offers a stiffer support system.

Polymer liners have the potential to replace steel mesh as the major form of surface support in underground coal mines. Results obtained from experimental investigations and computational models clearly establish that the fibre reinforced polymers tested are better than steel mesh in resisting skin displacements. The polymer has the ability to penetrate into fractures bonding adjacent fragments together and providing stiffer, more effective support. The polymer not only provides better resistance to rotation and deflection but can also sustain higher loads than the mesh. In addition to this, the polymer can bond satisfactorily to itself allowing fractures or damaged areas to be easily repaired.

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MONITORING OF STAND-AND-ROOF-BOLTING SUPPORT: DESIGN OPTIMIZATION

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KEYWORDS: stability of underground headings, monitoring.

1. INTRODUCTION

Preserving stability of underground headings in order to fulfill their technical functions without any°disturbance and provide a safe workplace for mining staff seems to be a fundamental issue in mining activity. Headings are usually protected with prop-type roof support, which can be additionally reinforced in difficult geological conditions or changing mining conditions. In recent years, bar rockbolts or cable bolts have been most frequently applied as means of reinforcement. Such a construction is often referred to as a stand-and-roof-bolting support.

Stand-and-roof-bolting support is constructed in many variants or versions as there are numerous types of frames and practically infinite number of possible schemes of rockbolt distribution. Regarding varied options of prop-type support with roof bolting, a proper selection of support scheme for particular geological and mining conditions appears to be highly problematic (Majcherczyk et. al. 2011). Although the effectiveness of such a solution seems to be well documented with numerous practical experiments, however such research is most often based only on visual evaluation of the state of headings and support schemes, which definitely fails to provide a sound basis for estimating the behavior of support and surrounding rock mass in a longer period of time or in changeable mining conditions. Therefore, the only way to optimize the support scheme for particular conditions is a proper monitoring of stand-and-roof-bolting support carried out in natural conditions (Layer 1996, Bawden & Tod 2002, Majcherczyk et al. 2006). Such a method allows to obtain work specification of particular elements of support, which can verify the appropriateness of the applied scheme.

The paper presents sample results of monitoring stand-and-roof-bolting support schemes selected out from numerous research projects carried out by the authors. The results discussed below are based on the measurements of strength parameters of rocks in laboratory and in-situ research, convergence of underground excavations, forces in rockbolts, separation of surrounding rock strata tested with extensometric probes, cable telltales and endoscopes, as well as frame loading tested with dynamometers. Complex measurements of stand-and-roof-bolting support alongside with specifications of geological, mining and geomechanical conditions allowed to formulate a proper evaluation of support behavior and its effectiveness for particular conditions.

2. MONITORING METHODS OF SUPPORT BEHAVIOR IN UNDERGROUND HEADINGS

The monitoring of underground workings is usually carried out in three areas specified below: measurements of changes in rock mass, measurements of loading in particular support elements, measurements of excavation geometry change.

In most cases, support monitoring makes it possible to determine optimal conditions, in which number of rockbolts, their length and frame size can be minimized, whereas frame spacing can be increased. As a result, such monitoring procedures offer numerous benefits, such as: an increase of safety potential offered by the applied solutions, facilitating an on-going search for more innovative support schemes, entailing cost minimization of exploitation and providing optimal protection of headings with an ideal adjustment of support scheme to dynamic requirements.

3. SELECTED APPLICATIONS OF SUPPORT REINFORCEMENT IN UNDERGROUND HEADINGS AND THEIR STABILITY EVALUATION

As it was pointed out in the introduction, there is a plentitude of possible schemes of stand-and-roof-bolting support. The following section presents results of monitoring carried out in three selected underground headings with varied schemes of support, located in different mining and geological conditions. Various types of measurement devices were used in order to evaluate the stability of prop-type support with anchored roof bars, reinforced with roof bolting between arches, reinforced with cable bolts between steel frames and reinforced with binding joists anchored with cable bolts.

4. CONCLUSIONS

Stability monitoring of selected headings with frames reinforced with bar or cable bolts allows to formulate the following practical conclusions:

- Additionally reinforced prop-type support proves its usefulness in various mining and geological conditions. In particular, its scheme should be adjusted to roof conditions, whereas the potential influence of exploitation on a working should be assumed in support design.
- In the case of high strength parameters of rock mass and lack of influence from serious mining factors (e.g. the influence of caved goafs or the influence of longwall), a standard frame spacing of the support (i.e. 0.75÷1.0 m) can be increased even to 1.5 m. Obviously, in such a case it is absolutely necessary to properly reinforce the rock mass with rockbolts. In the analyzed case, applying the frame spacing of 1.5 m entailed an effective use of rockbolts' work at the level of 70-80%.
- As the research shows, application of long cable bolts is also effective in seismic hazard conditions owing mainly to their bearing capacity exceeding 400 kN and large deformation resistance. The expertise gathered during the research study indicates that rockbolts with their potential to absorb elastic strain energy in certain ranges can successfully counteract roof separation. The measurements carried out as part of the present research did not reveal any changes in the analyzed working after the occurrence of rock burst in the immediate neighborhood.

It should be also pointed out that a long-term coal-mine research illustrates the evolution in the range of applied rockbolts. During the last two decades, there has been a spectacular change in types and lengths of implemented rockbolts. Initially, bar bolts with the length of approx. 2.0 m and bearing capacity ranging 120÷200 kN were used. Then, cable bolts appeared with the length of approx. 6.0 m. Currently, longer cable bolts are applied in order to reinforce the working in mines: their usual length ranges between 6 and 10 m but sometimes it reaches even 12÷15 m, with their bearing capacity of more than 420 kN. On the one hand, such a rate of progress was enforced by increasing dimensions of workings and the appearance of larger influences acting on rock mass. On the other hand, the development was made possible by the implementation of more advanced technologies and innovative materials in anchoring.

The instances of monitoring the stability of headings discussed in present paper suggest that coal-mine tests allow the researchers for a precise evaluation of the behavior of rock mass surrounding the working and particular elements of its support. The measurements of similar parameters with the use of varied methods (e.g. estimating roof strata separation with endoscope and extensometric probes) often produce completely different results. The values obtained in monitoring are strictly related to the properties of rocks and a type of applied support scheme.

What seems to be a crucial element of coal-mine measurements is the possibility to utilize their results in numerical modeling (Procházka & Trčková 2008, Małkowski et. al. 2008). The application of the so-called reverse analysis allows the researchers for a proper selection of rock mass models, their properties and rate of strata influence on support. A model calibrated in this way can serve as a basis for designing underground headings with the use of numerical methods.

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CLARIFYING GEOLOGICAL STRUCTURE FOR COAL AND MARSH GAS DEVELOPMENT USING MAGNETOTELLURIC METHOD

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KEYWORDS: Apparent resistivity, Cretaceous formation, Kushiro City, Osotsunai fault, Harutori fault

1. INTRODUCTION

Main generation ability rate in Japan are natural gas (29.4 %) and atomic energy (29.2 %). However, a review of use the atomic energy is carried out because of the Tohoku earthquake and demand for coal which is cheaper than oil and natural gas is raising now1). But usage of the coal higher than 99% depend on the import and quantity of coal is only mined approximately 1,300,000 tons a year in Hokkaido. To estimate detailed coal deposits, it is necessary to clarify the geological structure around the coal area in detail from the shallow to the depth. In this study, Magnetotelluric (MT) and Audio magnetotelluric (AMT) method are applied to clarify the geological structure for estimating of coal abundance nearby Harutori fault. Basic principles of MT and AMT method are almost same, but penetration depth and resolution is different because of difference of a bund of frequency. In this area, Osotsunai fault is existed which can be continued to Harutori fault. Studies of these faults are a few. Hence, these faults structures are unknown.

2. STUDY AREA AND MEASUREMENT SITES

Harutori fault is in southern area of Kushiro city, Japan. Recoverable coal reserves is estimated about 120,000,000 ton. However, investigations of Harutori and Osotsunai faults are not carried out. Measurement sites of MT and AMT method are set on both sides of these faults. A site of north side is called S1, the other is called S2. S3 is set on away about 1.5 km from S2. In addition, a reference site is set about 30 km away from measurement sites. Measurement and reference sites and estimated faults position2) is shown in Fig .1. MT and AMT machine are made by Phoenix Corporation, which are called MTU-5U and MTU-5A respectively. These machines can get the data of two of horizontal electric field, and two of horizontal and one of vertical magnetic field. Measurement was carried out from August 22th to September 2nd, 2011.

3. RESURTS AND DISCUSSION

Resistivity distributions of S1 and S2 are shown by Fig. 3. Land surface is wed because wed selected hard rocks are deposited on surface layer. Land surface is wed because wed selected hard rocks are deposited on surface layer. For this reason, it is estimated that low resistivity zone in shallow part of S1 and S2 is corresponded with groundwater. Low resistivity zone in the elevation from -100 to -120 m in S1 is estimated coal or clay layar of Yubetsu coal layer, but low resistivity zone is not appeared at the position of geological column in the elevation of -330 m which is correspond with coal layer. From this, it is estimated that Osotsunai fault exist in the elevation from -200 to -330 m and coal layer is displaced by the fault. Therefore, Osotsunai fault is exist between the bottom of Yubetsu layer and Tennei gravel, and it connected the Cretaceous layer. Under the elevation of -350 m is estimated the Cretaceous land block which are divided into Osotsunai and Harutori faults. In addition, low resistivity zone in the elevation of -10 m in S2 is corresponded with aquifer under Oboro gravel. It is thought that Harutori and Osotsunai faults are in the north side of S2, and Each dip (Harutori fault: north dip of 210 m, Osotsunai fault: east dip of 180 m) almost coincide with resistivity distribution dip of -400 m. Resistivity distribution of S3 and geological column away about 1,500 m from S3 are shown by Fig. 4. Low resistivity zone that Harutori and Osotsunai faults are in the north side of S2, and Each dip (Harutori fault: north dip of 210 m,

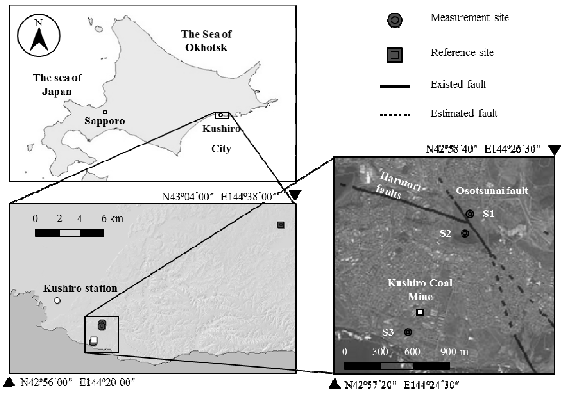

Fig. 1 Study are and measurement sites

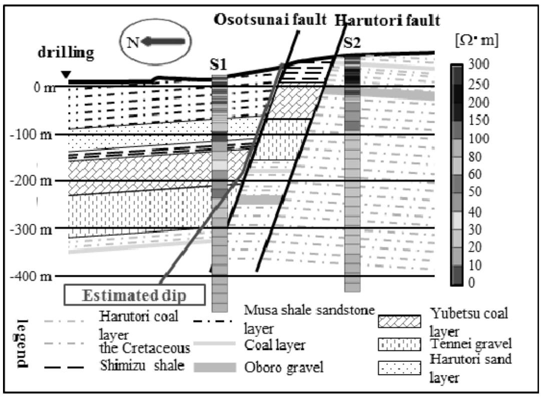

Fig. 2 Resistivity distribution of S1 and S2 and geology

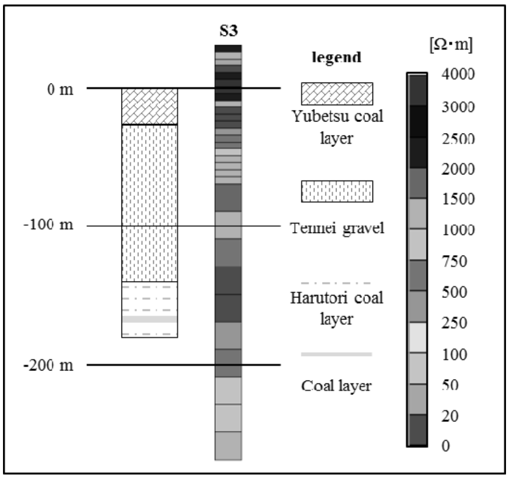

Fig. 3 Compared resistivity distribution of S3 and geological column

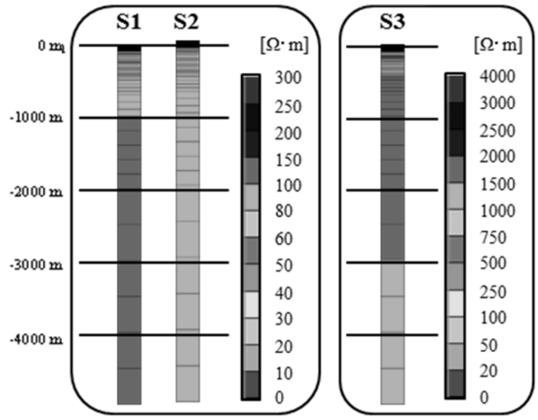

Fig. 4 resistivity distributions of all sites until the depth of -5,000 m

Osotsunai fault: east dip of 180 m) almost coincide with resistivity distribution dip of -400 m. Resistivity distribution of S3 and geological column away about 1,500 m from S3 are shown by Fig. 4. Low resistivity zone at the elevation of -30 m is caused by groundwater which is in the shallow part of S1 and S2. High resistivity zone at the elevation of -100 m is corresponded with Harutori coal layer because Harutori coal layer in S2 shows high resistivity value. Resisitivity distributions in all sites until the depth of -5000 m are shown by Fig. 5. The bottom of the Cretaceous layer does not appear in any sites. Therefore, layers under the depth of -500 m are regarded as the same, and Cretaceous layer thickness is estimated 4,000 m or more in this area. Because there is study of the Cretaceous layer thickness more than 3,000 m, this is a proper guess[3].

4. COCLUSION

In this study, geological structure is estimated using MT and AMT method around coal mining area. As a result, dip angle of faults and existence of aquifer which is about 200 m thick in the place deeper than -500 m of the Cretaceous layer were estimated. For these things, exploration drilling depth is needed more than 800m. In addition, thickness of the Cretaceous layer can be estimated 4,000 m or more. To compare that results and geological column, faults structure, aquifer, and coal layer can be estimated.

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MECHANISM OF RIGID OVERLAYING STRATA FAILURE IN FACE MINING IN CONDITIONS OF MULTISEAMS CARBONIFEROUS DEPOSITS

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KEYWORDS: subsidence trough, rock mass, overlying strata, longwall mining, tensometric measurement.

1. INTRODUCTION

The rigid overlying strata failure assessment method is applicable in the deep mining of thick coal seams (thickness of more than 1 m) by means of the method of longwall mining with controlled caving. The rigid overlying strata failure assessment allows to distinguish, whether the rigid overlying strata of rocks has been deformed or a strutting arch has been formed over the goaf and below it is the area of free from stress. Good knowledge of the mining, technical and geological conditions of a given site is a prerequisite for successful evaluation.

The advantage of utilising surface measurements is a possibility to interpret the effects of changes in rock mass, especially in areas of high overlying strata. The practical importance of the overlying strata failure assessment consists in determining the size of the mined-out area where the deformation of the rigid overlying strata occurred in dependence on the character of rock mass.

The paper will be put into the context of the expected width of the goaf during the deformation of rigid overlying strata with parameters that describe the mining and geological conditions of the locality. There will also be put into context the changes in the area of the goaf from results of tensometric measurements.

2. DESCRIPTION OF THE SITE

The article presents results from assessment of the failure of overlying strata by mining coal face 340 206 at 40 seam in 2th block Darkov mining area. Mining operations started in July 2011. The extraction is performed by advanced longwall mining with controlled caving with an average thickness extracted 5 m. Advance of longwall face and previous mining in area around the assessed faces is shown in Fig. 1.

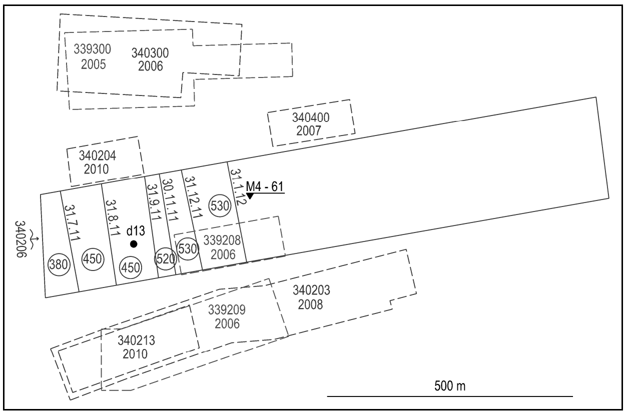

Fig. 1 Situation of borehole M4 – 61 and a surface point d13 towards face 340 206 and previous mining.

Information about the conditions of the overlying strata were obtained from borehole M4 - 61, in the line, there are 26 m thick interlayer between the coal seams 39 and 40. The interlayer is mostly composed of coarse-grained sandstones and conglomerates. The interlayer between the coal seams 37f and 39 are coarse-grained sandstone and fine grained sandstone layers with the thickness of approx. 44 m.

Tectonic evolution in the area is very complicated. In west direction of 340 206 face runs tectonic fault Gabriel, on east Elizabeth tectonic fault, on north tectonic fault Lezata. In order to measure movements on the surface, points were fixed in a surface network whose position and height is periodically surveyed by GNSS (Kajzar et al., 2012). For the presentation the current subsidence on surface and seismological activity in the mining 340 206 coalface as an identification point was determine d13, Fig. 2.

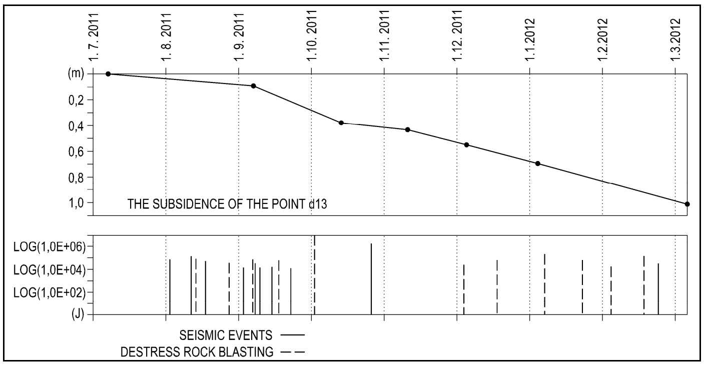

Fig. 2 The subsidence of the surface point d13 with graphic display of registered significant seismological events in the area.

3. CONCLUSION

It is necessary to regard the rigid overlaying strata as a stratified non homogeneous beam composed of layers varying in thickness. The method of the overlaying strata failure assessment of extracted seams is based upon the simultaneous assessment of surface subsidence and seismic activity considering spatio-temporal progress of mining depending on the rock mass character. It means that at the time of the rigid overlaying strata breakthrough only those layers with the small capability of deflection are deformed in a rigid manner. The deformation of flexible layers at the time of a complete failure is not fragile, a fragile failure of these layers occurs only subsequently with the mining progress.

The development of surface subsidence presented by subsidence curve of d13 point, record of registered significant seismological events and previous experience (Jiránková, 2010) show that during the extraction of 340 206 face there was a regular deformation of overlying strata. From Fig. 2, the apparent onset of surface subsidence is clear, accompanied by the occurrence of natural seismic events and seismic events from performed BTPVR (destress rock blasting). At this time (August-October 2011) a regular deformation of overlying strata occurred. After the second made BTPVR 10th 2011, the working face front stopped for about 3 weeks, which resulted in reduced growth of surface subsidence. The following mining brought new growth of surface subsidence and was accompanied by significant seismic events that originate only from BTPVR. It can be assumed that at this time elastic deformation of the overlying strata dominate over deformation in a rigid manner.

The paper reviews the findings of the overlaying strata failure assessment and it is confronted with the results of tensometric measurements in boreholes (Staš et al., 2011).

ACKNOWLEDGEMENT

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STRUCTURAL CHANGES OF ROCKS SUBJECTED TO HIGH TEMPERATURES AND THEIR IMPACT ON THERMAL PARAMETERS

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KEYWORDS: structural rocks parameters, high temperature, thermal rocks parameters, rocks' furnacing.

1. INTRODUCTION

This article presents the results of laboratory tests of rocks surrounding a geo-reactor designed in Poland: changes to bulk density, specific density, and porosity due to high temperature. The above results were compared with the results of tests of thermal conductivity of the rocks, their specific thermal capacity and capacity to conduct temperature. The study involved claystones, siltstones and sandstones. The division of rocks was done after mineralogical investigation. All rocks in the study were heated for this purpose to the temperature of 1000^0C or 1200^0C, depending on the distance of the stratum from the designed geo-reactor. It was proved that a temperature up to 600 ^{0}C can still exist at the distance of 2 meters from the burned coal seam and for crushed rock mass – even at a distance of 6 meters. The correlation between the analysed physical properties of the rocks was conducted, as well as the mineralogical investigation after kilning.

1. CHANGES TO STRUCTURAL PARAMETERS

The impact of the high temperature on the change of rock structure and texture was determined by observing rock behaviour during their kilning, and then by performing measurements of the parameters specified above on the rock samples after cooling.

First of all, a clearly different reaction to high temperature was observed for particular types of rocks (Fig. 1-3), while in its phases similar to the observations made by Mao (Mao et al. 2008). Claystones were burnt, completely changing colour to brown (Fig. 1), and also underwent stratification and cracked into fine pieces. This is due to their clay mineral content.

Siltstones generally did not change their appearance, but often cracked along the stratification surface (Fig. 2). Laminas of clay materials were overburnt and changed colour to browny-red. Sandstones completely preserved their shape (Fig. 3), while some minerals changed their colour to red or brown. The lack of reaction to the high temperature for siltstones and sandstones was caused by the small amount of matrix, below 30 %. Amount of over 50 % accelerates the decay.

The studies of structural physical properties, made on 26 samples, show that after kilning all analysed rocks increase their bulk density and decrease their specific density. The greatest changes are recorded for siltstones which may increase their bulk density ρ_o by over 20 % (series 8.7), with the average value amounting to 9.76 %. Among the rocks in this group, there were also the great changes to the structural parameters. Siltstones change their bulk density on average by 8.19 %, while sandstones – by 5.73 %. In one case in the series 4.5, sandstones practically did not change ρ_o.

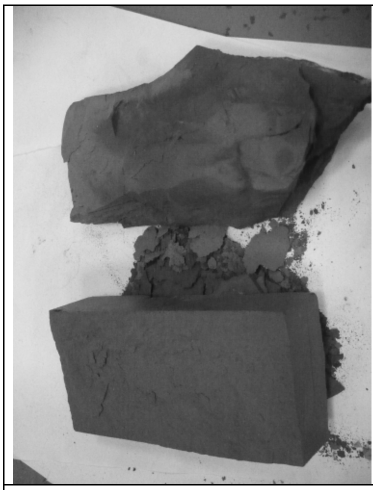

Fig. 1 Claystone after kilning

Fig. 2 Siltstones after kilning

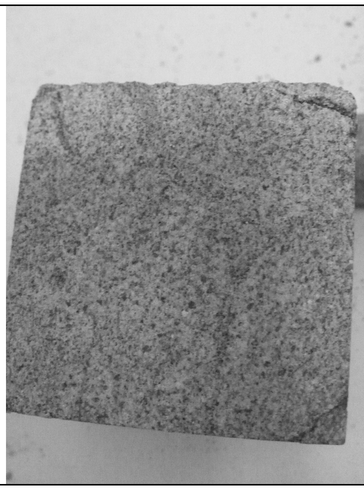

Fig. 3 Sandstone after kilning

2. CHANGES TO THERMAL PARAMETERS OF THE ROCKS

The analysis of limit values of the thermal conductivity of carboniferous rocks at the temperatures of 20^0C and 1000^0C shows that the values of factor λ after rock kilning have a reversely proportionate tendency when original value. For the thermal conductivity factor equal to approx. 1 W/m·K at room temperature, after rock heated to 1000^0C, its values rapidly increase even to 14 W/m·K (Fig. 4). Together with the greater initial thermal conductivity, rock kilning at high temperature begins to bring the reverse effect to its thermal conductivity. With the initial value λ equal to 3 W/m·K or more, after kilning, the factor usually does not exceed the value of 2 W/m·K. What is also noticeable is the clear logarithmic nature of the changes of rock capacity to conduct heat before and after kilning them at high temperatures. The coefficient of determination for the samples analysed is equal to 0.70, which due to instability of thermal processes in the rocks and their strict dependence on mineralogical composition must be considered unusually high.

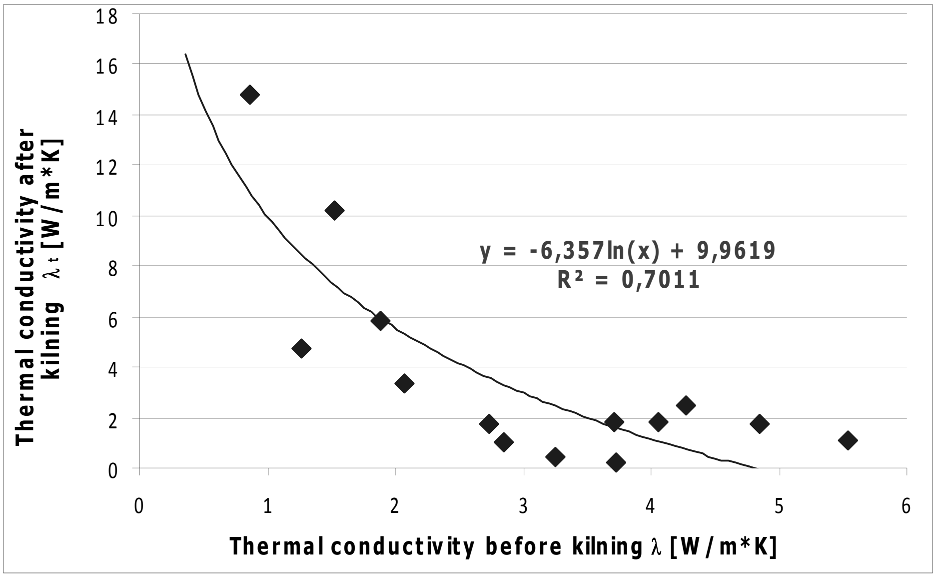

Fig. 4 Dependence between thermal conductivity factor γ before and after γt kilning of the same rock

3. CONCLUSION

The studies on the thermal properties of rocks are rather rare. They are most frequent in the areas of nuclear power plant constructions. Underground coal gasification is another reason for which thermal physical properties of rocks must be analysed in the surrounding of the designed geo-reactor.

As a result of rock kilning, there is clear increase to their total porosity up to approx. 4% in the case of sandstones, and up to 11-18% in the case of claystones and siltstones. The study performed, however, does not point to a change in porosity or the change of rock density but the meaning of amount of quartz and matrix in rock samples for its behavior in high temperature was proved..

The analysis of the thermal conductivity factor at the temperatures of 20^0C and 1000^0C shows that auto-correlation occurs, with the logarithmic function. For low values of thermal conductivity factor's equal to approx. 1 W/m·K at the temperature of 20^0C, after rock kilning to 1000^0C its values increase to 14 W/m·K, whereas with initial value equal to approx. 3.0-3.5 W/m·K, after heating λ amounts to approx. 1-2 W/m·K. It was also stated that due to rock heating, very high increase to their specific heat capacity c_p occurs. Its value for rocks analysed at a temperature of 20^0C amounts to approx. 0.5-1.5 kJ/kg·K, whereas after kilning – even 13 kJ/kg·K. In the case of this parameter, however, auto-correlation does not occur.

To conclude, it can be stated that subjection of the rocks to very high temperatures strongly distorts mutual relations between their physical properties. This also refers to the thermal properties of the rocks that are also strictly dependent on the temperature in which they are set. The change of rock structure does not, however, directly translate to its conductivity capacity and heat accumulation.

The study was performed under Research Task No. 3 entitled: "Development of coal gasification technology for highly efficient production of fuels and electricity" - Point 1.2.1. "Mining and environmental requirements with modelling of geo-gas-dynamic processes", financed by the National Research and Development Centre within the strategic programme of scientific research and development "Advanced technologies for energy acquisition". Contract No. AGH 23.23.660.8902/R34.

FRACTURE TOUGHNESS – A PARAMETRIC STUDY

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1. INTRODUCTION

Each rock is characterized by specific mechanical, temperature and chemical properties, obtained during its genesis before millions years ago. From mechanical point of view the failure study and failure criteria determination of rocks in connection with kind of their loading represent one of the basic rock engineering problems. Very often the phenomenological theories of rock fracture are used. These theories quantify spatial orientation of the failure plane in relation to the stress state in the rocks. It is for example a criterion of maximum shear stress, Coulomb fracture criterion and the generalized Mohr criterion (Stephansson 2010). On the other hand the mechanistic fracture theories assume existing incipient cracks in the rock, which by nature represent a concentrator of the local stress. These cracks control the rock material failure under the specific conditions. The fracture mechanics deals with fracture toughness study, which is a material attribute. Recently, the study of the rock fracture toughness is one of the current and frequent directions of the research in the area occupying with rock failure. Presented paper describes the parameters effect influencing the resultant rock materials properties (displacement rate, rock moisture) during the fracture toughness measurement at different kind of the rock.

KEYWORDS: fracture toughness, rock, density, displacement rate

2. FRACTURE TOUGHNESS

Ordinary rock material contains cracks (in fact, there are also pores, impurities, dislocations, etc.). There is a high stress concentration, which occurs on the tips of these cracks during their loading. Occurrence of small cracks greatly decreases of the rock material resistance to external loading (cracks can propagate uncontrollably).

The unstable crack propagation by Griffit´s theory occurs when the stress intensity factor (K) reaches a constant critical value. This value is called the fracture toughness of rocks (K_c). The parameter K, reflecting the stress intensity factor at the crack tip, and index are quoting, that distinguish three basic modes of loading for a crack (Figure 1).

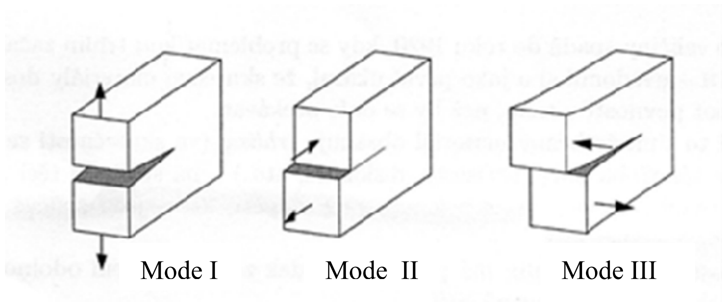

Fig. 1 The three basic modes of loading for a crack (opening mode, sliding mode, tearing mode)

3. MEASUREMENT METHODOLOGY

The measurement of rock fracture toughness was performed using three-point bending test on selected rocks using a specimen type CB (Chevron Bend), with the method of loading - Mode I (Figure 1). Figure 2 shows the test set with a cylindrical specimen and extensometer of clip on a cage type. Due to the extensometer it is possible to measure the crack face opening (COD – crack opening displacement).

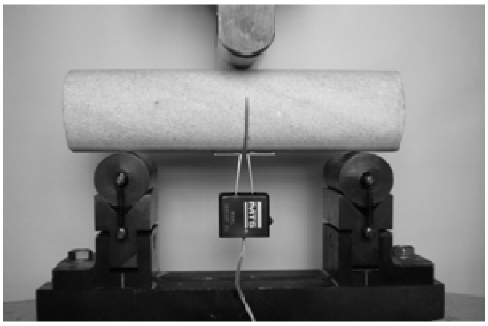

Fig. 2 Test equipment with the extensometer

Fracture toughness measurements were carried out on four rocks types. It is the medium grained carboniferous sandstone, medium grained sandstone from the Javorka site, fine-grained granite from the site of Černá voda and coarse-grained marble from the site of the Horní Lipová. The fracture toughness measurements were performed in both dry and saturated samples by water at the carboniferous sandstones. In addition, comparative measurements were performed for the other above mentioned types of rocks at the comparable displacement rate and depending on different displacement rates.

4. CONCLUSION - EXPERIMENTAL RESULTS

On the basis of the resultant fracture toughness values of the analysed rocks we can state:

- the minimal fracture toughness values were found out at the sandstone sample, which reach c. 17% - 30% of measured fracture toughness values in the analysed granite and marble samples (see Table 1 and Figure 3)

Table 1 Ratio of fracture toughness values of sandstone (Javorka) to granite and marble values

	Displacement rate [mm.min^{-1}]		
	1	**0.1**	**0.01**
Sandstone/Granite [%]	0,31	0,20	0,26
Sandstone/Marble [%]	0,28	0,20	0,17

- Generally the major fracture toughness values of analysed rocks were recorded at lower loading displacement rates, which probably are related to rheological properties of rocks and amount of elastic deformation energy realized on the test system per time unit.

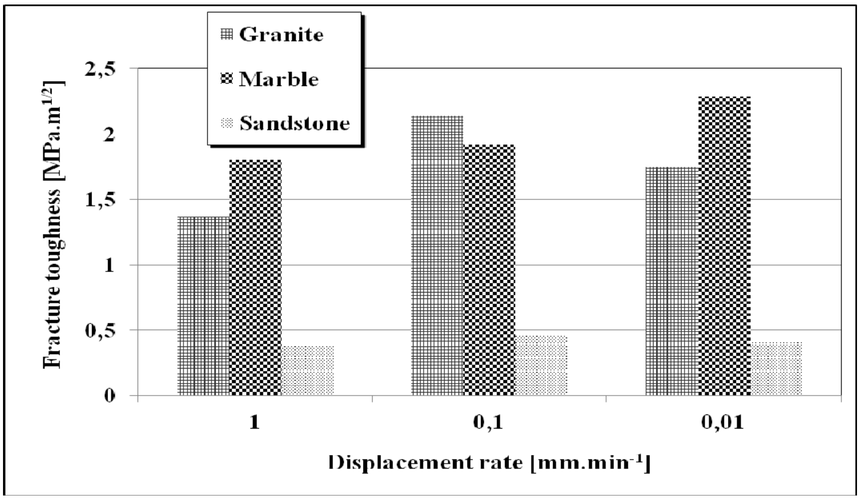

Fig. 3 Fracture toughness values of analysed rock

- As in case of the other rock mechanical properties (e.g. uniaxial compression strength), as well as in this case the higher sandstone (carboniferous) moisture influences the degreasing of fracture toughness values and increasing of their deformation ability. (see Figure 4).

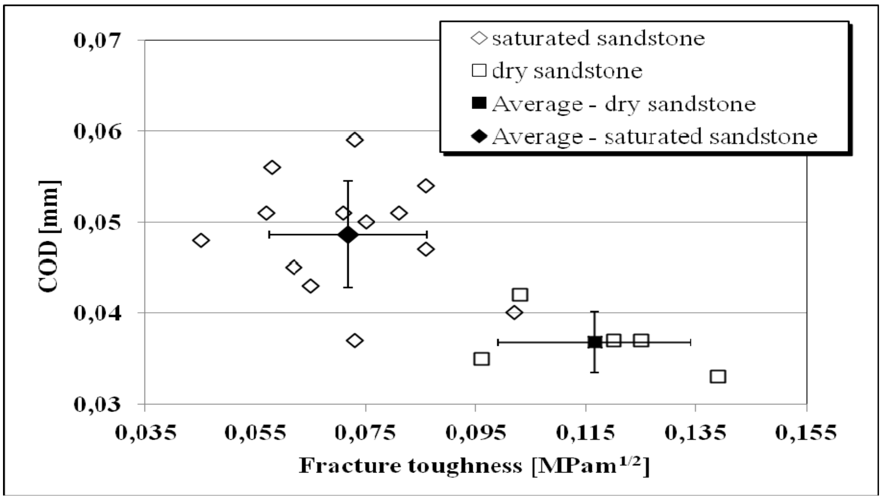

Fig. 4 Fracture toughness values of analysed dry and saturated sandstone

- From the graph in the Figure 5 it is clear that the COD values found out in granite and marble are much lower (they range in values at the most up to 0.04mm) than in the sandstone samples (Javorka sandstone, the values range is from 0.05 mm to 0.09 mm). These values reflects the more brittle behaviour of granite and marble samples.

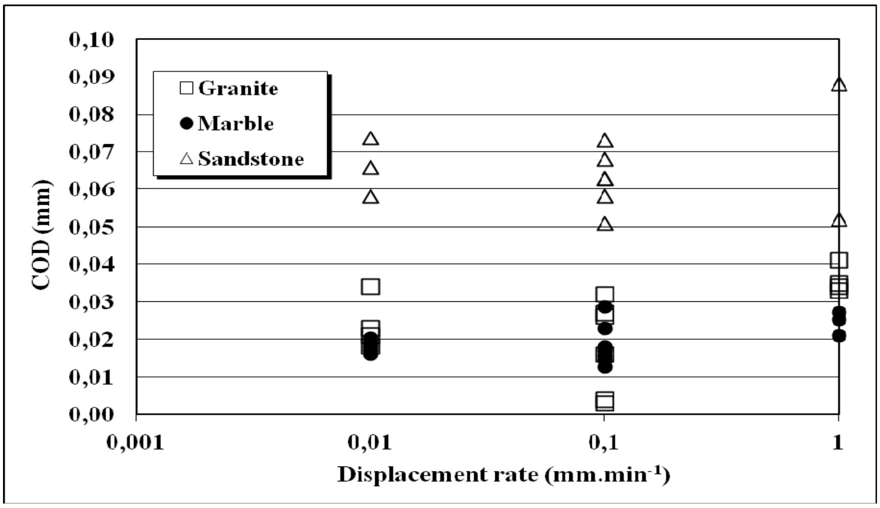

Fig. 5 COD (crack opening displacement) vs. displacement rate

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3D VISUALISATION OF GPR DATA IN VOXLER SOFTWARE

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KEYWORDS: Ground Penetrating Radar, GPR, Voxler, 3D Visualisation, image angular transformation

1. INTRODUCTION

Ground Penetrating Radar (GPR) survey might significantly contribute to the understanding of sub-surface processes at areas of interest in geoscience and related technical fields (Jol, 2009). It is usually necessary to cover the construction area with a detailed grid of measurements (within the first 100 survey lines) to identify the critical anomaly or damaged construction part. Evaluation and presentation of this huge collection of data by the author is difficult to explain to the client due to the complexity of the situation. Graphical visualisation of the georadar results in 3D combined with technical maps of the area could reduce the complexity of the output and be beneficial for all users. This graphical presentation can't be obtained using specialized GPR processing software alone.

Presented poster is focused on the 3D visualisation of GPR data in Voxler 2 software (Voxler, 2010). Source data was processed using Reflex-Win 6 software package (Sandmeier, 2012) and EKKO-Mapper 4 (EKKO-Mapper, 2009). GPR data processing involved advanced numerical corrections, including deconvolution, migration etc., and a conversion of radargrams captured in time to the depth-profiles based on 2D velocity model, which was created on site using CMP.

Visualisation itself is created for (i) the original source data (scatter plot), (ii) for their 3D interpolation (volume rendering, etc.), (iii) for the classic raster images (radargram, time slices) including the visualisation of georeferenced images in horizontal layout or their angular transformation up to the vertical profile layout. Real field data (Hubatka, 2012) is used to document the advantages of the of 3D graphical presentation for identification of sub-surface anomalies.

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DIMENSIONING PILLARS FOR MINING METHOD ROOM-PILLAR IN PROTECTION PILLARS OF SHAFT CSM NORTH MINE

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KEYWORDS: dimension, room and pillar, OKR

1. INTRODUCTION

Protective pillar NORTH CSM mine attaches considerable coal reserves, which are not allowed to mining by the classic long wall cave by which there would be dips of pits and surface deformation of objects. Therefore, it is planned to use the method of OKD - room and pillar with stable pillars. It is assumed that this mining method is friendly to the protection of the surface structures.

Dimensioning of pillars deserves attention from both the technical and operational- security and economic. The special importance of this issue is that by using the room and pillar, because an essential feature of this method is that a part of seam is mined and the other part remains as a temporary or permanent support of roof. It is an effort under certain natural and technical conditions of mining; secure the necessary degree of safety of workplaces with the smallest losses of coal reserves. Size of pillars left is determined on the experience and laboratory experiments.

2. THEORY OF PILLAR DIMENSION

In basic consideration for pillar dimension is important the geometrical shape of pillar:

a) square pillar
b) rectangle pillar

At both methods is facultative the width of room „B“, and the width of pillar „w“, and the other parameters are given.

Size "B", which is to be maximized, has it upper limit in dependence of strength of the overlying strata, according to the vault and beam theory. Size "w", which is to be minimized, is determined by the condition that the left pillar, has to balance at least the forces that act on it from surrounding strata. (Mikeska et al, 1970). The issue of dimensioning square pillars describes a condition where the left side of the equation (1) is expressed by the load and the pillars on the right side of his strength:

$$(w+B)^2 \cdot \sigma_z \leq w^2 \cdot \sigma_u \cdot K \tag{1}$$

S_p - load of pillar [MN]
σ_p - strength of pillar [MN]
σ_z - vertical stress [MPa]
σ_u - uniaxial compressive strength [MPa]
K - slenderness ratio

3. COMPUTE OF PILLAR DIMENSION

Questioned seam 30 in protection pillar of shaft NORTH CSM Mine is deposits at a depth of 700-900 m below the surface with incline 12° to the northwest. Seam thickness ranges from 1,9 to 3,3 m and strength of 30th coal seam in the uniaxial pressure is 15 MPa.

For the calculation the width of pillars were used the following values:

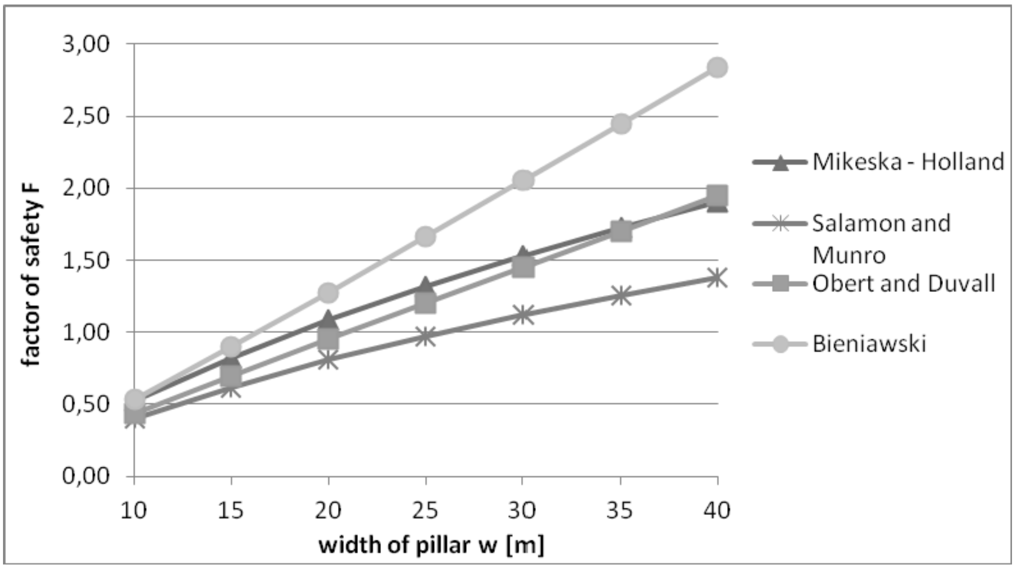
width of driving gate	B = 5,2 m (given by continuous miner)
thickness of mined seam	h = 3 m
specific weight of overlying strata	γ = 0,025 MN·m^{-3}
depth of imbed	H = 900 m
uniaxial compressive strength	σ_u = 15 MPa

Table 1 Compute of safety factor for different width of pillar.

size of pillar	recovery	Mikeska			Holland			Salamon and Munro		Obert and Duvall		Bieniawski	
		pillar load	pillar strength	safety factor	pillar load	pillar strength	safety factor	pillar strength	safety factor	pillar strength	safety factor	pillar strength	safety factor
w	e	S_p	σ_p	F	S_p	σ_p	F	σ_p	F	σ_p	F	σ_p	F
[m]	[%]	[MN]	[MN]	-	[MPa]	[MPa]	-	[MPa]	-	[MPa]	-	[MPa]	-
10	57	5198	2739	**0.53**	52.0	27.4	**0.53**	21.0	**0.40**	22.8	**0.44**	27.6	**0.53**
15	45	9181	7547	**0.82**	40.8	33.5	**0.82**	25.2	**0.62**	28.3	**0.69**	36.6	**0.90**
20	37	14288	15492	**1.08**	35.7	38.7	**1.08**	28.8	**0.81**	33.9	**0.95**	45.6	**1.28**
25	31	20521	27063	**1.32**	32.8	43.3	**1.32**	31.9	**0.97**	39.4	**1.20**	54.6	**1.66**
30	27	27878	42691	**1.53**	31.0	47.4	**1.53**	34.7	**1.12**	45.0	**1.45**	63.6	**2.05**
35	24	36361	62763	**1.73**	29.7	51.2	**1.73**	37.3	**1.26**	50.5	**1.70**	72.6	**2.45**
40	22	45968	87636	**1.91**	28.7	54.8	**1.91**	39.6	**1.38**	56.1	**1.95**	81.6	**2.84**

Compute after Mikeska and Holland are the same. Both authors used the same slenderness ratio. More information concerning the dimensioning of pillars see (Bieniawski, 1984) and (Mikeska et al, 1970).

Graph 1 Influence of width of pillar on it disturbance.



4. CONCLUSION

In the present article summarizes the theory and design of the pillars dimension and has been calculated for the design of mining seam 30th in the CSM Mine NORTH CHAMBER - protection pillar, according to four different theories. The results are presented in Table1 and Chart1. It is evident that by using the calculation according *Bieniawski* would suffice square pillars of width 20 m *(F=1,28)*. When using a calculating *Mikeska (Holland)* and *Obert and Duvall*, which is very close to each other, meets the safety factor *(F=0,53 to 1,5)*, 30 m wide pillars. Calculation according to *Salamon and Munro* is not suitable for conditions in OKR, as it was used for mining in South Africa to a depth of 300 m

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STEEL MINE SUPPORTS

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1. INTRODUCTION

Technology of steel mine support production was under development intensively in 80-th of last century [1]. A lot of metallurgical and size changes were prepared during this period regarding to innovation of German standard and changing of mining conditions [2]. The production was previously focused especially to smaller size groups started for instance by section K21, K24, TH 16,5 and TH21. Size groups were led gradually to higher weight sections and finally to unification of size types of sections. Development in mining technology gives a possibility to progress of new modification of steel mine supports and using of new progressive material in their production.

Steel mine profiles are one of the profiles which are produced on the Heavy Section Mill in ArcelorMittal Ostrava a.s. All of these steel mine profiles are rolled according to the national KN standards or German DIN standards [2]. The processing of steel arches consists of several important technological steps [3] hot rolling, cold straightening and cold bending to final steel mine supports. Some of the profiles can be heat treated before final cold bending as well. The heat treatment allows to achieve higher mechanical properties than the conventional processing technology. Final steel arches are composed of several shorter bended parts connected together by special joints in mines.

Mechanical properties, including the Charpy test after aging, are the main quality parameters of steel mine supports. Apart from these testing methods, two types of special tests can be performed on full section - special bending test and slip test. Yielding steel arch supports are used in mines the most frequently due to their good capability to slide inwards when their load-bearing capacity is exceeded. Good slip resistance before plastic deformation of steel arch is reached by defined overlapping of the profile and type of connection. The testing of all mine steel frame is very complicated and needs a special testing machine that is available only in several laboratories around the world.

Steel mine support designed for mine roadways is under high static and dynamic loading of surrounding rocks. Progressive technology and mechanization in mine technology leads to successive increasing of cross section of mine roadways. It leads also to modification of steel arch support shape, its construction and new material solution of steel profiles. ArcelorMittal Ostrava a.s. cooperates very tightly with one of the biggest consumer of steel mine supports OKD a.s. on several developing projects regarding of new SP shape of steel mine support and development of new grade of steel sections [4]. The company cooperates also with other research and science centers as Faculty of metallurgy and material engineering and Faculty of Civil engineering of VŠB-TU Ostrava or ITA s.r.o. company. Together with these partners and by support of grant agency TAČR ArcelorMittal Ostrava a.s. cooperated on development of new mine supports produced from microalloyed steel of steel with higher tensile properties after hot controlled rolling.

The poster deals with the current technology used for production of steel mine supports, their mechanical properties, cold bending in a bending shop and shortly describes the last development of new steel arches and new steel grade of mine support. According to setting of suitable combination of chemical composition, rolling and cooling conditions we can achieve demanded mechanical properties according to standards or increase their level. The works on new grade development continue while development of new SP profile has been finished.

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