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ANALYSIS OF THE APPLICABILITY OF THE CONVERGENCE CONTROL METHOD FOR GATEROAD DESIGN BASED ON CONDUCTED UNDERGROUND INVESTIGATIONS

ANALIZA MOŻLIWOŚCI ZASTOSOWANIA METODY STEROWANIA KONWERGENCJĄ W PROJEKTOWANIU CHODNIKÓW PRZYŚCIANOWYCH W OPARCIU O PRZEPROWADZONE BADANIA DOŁOWE

The stability of gateroads is one of the key factors for the mining process of hard coal by a longwall system. Wrong designed and applied the gateroad support at the stage of drilling, may adversely affect the functionality of the gateroad and the safety of the crew throughout its existence.

The article presents the results of the underground tests and observations such as: convergence of the gateroad, stratification and the fractured zone range in the roof rocks, carried out in four longwall gateroads at the stage of their drilling.

The obtained test results were the basis for the assessment of the possibility of using a convergence control method in the design of the gateroad support. The method is based on three interdependent relationships, such as: Ground Reaction Curve (GRC), Longitudinal Displacement Profile (LDP), and a Support Characteristic Curve (SCC). All calculations were performed using numerical modeling in the Phase2 program, based on the finite element method (FEM).

Keywords: mining, gateroad, underground investigations, analysis, numerical modeling

Stateczność chodników przyścianowych jest jednym z czynników, które ma kluczowe znaczenie w procesie wydobywczym węgla kamiennego systemem ścianowym. Źle zaprojektowana i zastosowana na etapie drążenia obudowa chodnikowa może wpływać negatywnie na funkcjonalność wyrobiska i bezpieczeństwo załogi w całym okresie jego istnienia.

W artykule przedstawiono wyniki badań dołowych w zakresie: konwergencji wyrobiska, rozwarstwienia oraz zasięgu strefy spękań skał stropowych, przeprowadzonych w czterech chodnikach przyścianowych na etapie ich drążenia.

Uzyskane wyniki badań stanowiły podstawę do oceny możliwości zastosowania przy projektowaniu obudowy zabezpieczającej wyrobisko korytarzowe metodę sterowania konwergencją. Metoda ta bazuje na trzech powiązanych ze sobą zależności, takich jak: krzywa reakcji masywu skalnego (GRC), profil przesunięcia wzdłużnego (LDP) oraz krzywa charakteryzująca obudowę (SCC). Wszystkie obliczenia

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przeprowadzono z wykorzystaniem modelowania numerycznego w programie Phase2, opartego na metodzie elementów skończonych (MES).

Słowa kluczowe: górnictwo, chodnik przyścianowy, badania dołowe, analiza, modelowanie numeryczne

1. Introduction

The stability of gateroads is an important factor affecting the safety of the mining crew and the efficiency of ongoing mining operations. Protection of the stability of excavations requires proper design of the support, taking into account the existing load from the surrounding rock mass. The value of this load depends, among other things, on the fracture zone in the rock mass formed around the excavation. In the case of gateroads, the range of fractures varies over the entire period of its existence, i.e. from the moment of drivage, until its liquidation after particular longwall mining is completed, which in turn leads to significant changes in the load value to which support is subjected and its deformation.

Therefore, it is important that as early as at the stage of designing gateroads located in given geological and mining conditions, it was possible to determine the range of the fracture zone in order to select the appropriate support protecting these excavations. It is more and more frequent that bolt support is additionally used as a reinforcement element for the rock mass or else arch support in gateroads. Unfortunately, there are cases where the use of additional bolting does not fulfil its task due to support parameters inadequately matched to geological and mining conditions. Another possible reason is too late, with respect to resulting stratification, bolting time in the excavation.

At the Central Mining Institute, for several years, underground research has been carried regarding the assessment of the load acting on the support and convergence of gateroads as well as the range of the fracture zone in the rock mass around these excavations throughout their existence. This paper presents examples of underground research results that were carried out in four gateroads at the stage of their drivage. The obtained test results formed the basis for the assessment of the possibility of using gateroad convergence control method based on the Ground Reaction Curve (GRC) in the design process of the support protecting the excavation (Pacher, 1964; Panet, 1995; Carranza-Torres & Fairhurst, 2000; Brady & Brown, 2006; Esterhuizen & Barczak, 2006; Hoek et al., 2008). All calculations were performed using numerical modelling in Phase2 programme based on the finite element method (FEM).

2. Research on the range of the rock fracture zone and the deformation of driven gateroads

2.1. Characteristics of geological and mining conditions in the area of conducted underground investigations.

Underground measurements of deformation and the range of the rock fracture zone around the gateroads during their drivage were carried out in four gateroads, located in four mines within the Upper Silesian Coal Basin (USCB). For the purpose of the paper, these gateroads have been marked as: G-1, G-2, G-3 and G-4.

All gateroads were surrounded by rocks typical of the USCB, such as: clay shales, sandy shales and sandstones. Fig. 1 presents geological profiles showing the arrangement of individual rock layers in the vicinity of each of the examined gateroads. In addition, the gateroads in question were surrounded by the body of coal on both sides.

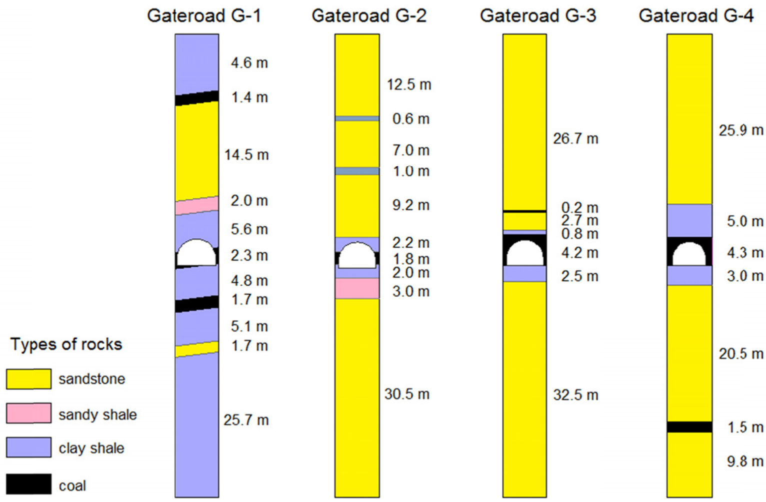


Fig. 1. Geological rock profiles around the gateroads where the investigations were performer

The gateroads were driven at depths from 730 to 1080 m in coal seams with a thickness from 1.8 to 4.3 m and an average inclination from 4 to 14°. The support of gateroads consisted of yielding steel arch frames ŁP9/V32 (size 9: width × height = 5.0 × 3.5 m) and ŁP10/V32 (size 10: width × height = 5.5 × 3.8 m), manufactured from section of V32 type (which means the weight was 32 kg/m). The spacing between steel arch frames was from 0.75 to 1.0 m. In none of the excavations was an additional reinforcement of the ŁP support by means of bolts or props used. The average daily face advance was from 5.0 to 9.0 m.

An important parameter, decisive for the deformation of the rock mass and the range of the fracture zone around the gateroads, is the uniaxial compression of rocks located in the direct vicinity of a given excavation. In the analysed cases, the average values of R_c strength, obtained on the basis of penetrometric tests, were from 4.8 to 21.08 MPa for coal, from 27.3 to 52.3 MPa for roof rocks, and from 22.0 MPa up to 30.0 MPa for floor rocks, respectively. Selected values describing geological and mining conditions in the area of the conducted research are presented in Table 1.

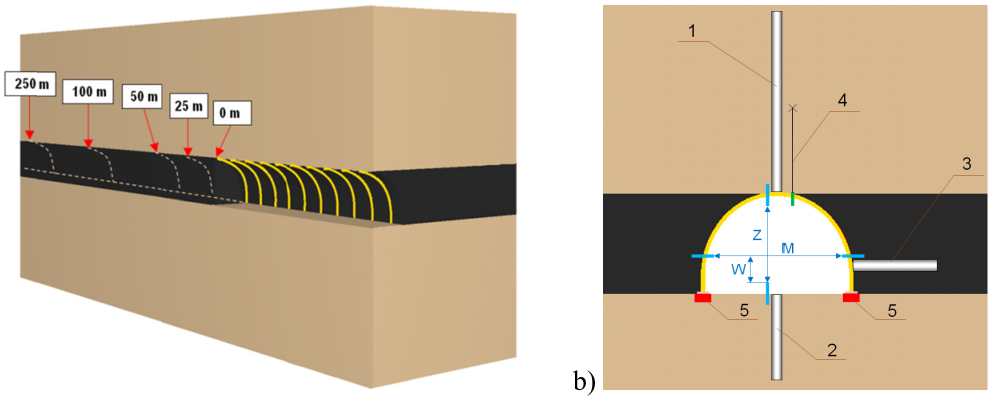
2.2. Methodology of conducted underground investigations

The measurements of rock mass deformation and the range of the fracture zone around the gateroads being in driven were carried out in measurement stations specially prepared for this purpose. These stations were located directly in the face of the excavation between the first and second arch support (counting from excavation face) according to the diagram shown in Fig. 2.

TABLE 1

The basic geological and mining-technical data characteristic of the areas where the referred to underground investigations were carried out

No	Gate-road	Depth, m	Seam inclination, °	Seam thickness, m	Compressive strength R_c, MPa			Support Width/Height/ Spacing	Average face advance m/day
					roof	coal	foor		
1	G-1	1035	12	2.3	27.3	10.7	26.0	ŁP10/V32/4 5.5/3.8/0.80 m	6.0
2	G-2	950	6	1.8	52.3	4.8	26.5	ŁP10/V32/3 5.5/3.8/0.75 m	9.0
3	G-3	1080	14	4.2	50.0	10.3	22.0	ŁP10/V32/4 5.5/3.8/0.75 m	5.0
4	G-4	730	4	4.3	35.7	21.1	30.0	ŁP9/V32/4 5.0/3.5/1.00 m	8.0



a) b)

Fig. 2. Diagram and localization of a measurement stadion in gateroad: 1, 2, 3) 10.0 m, 5.0 m and 3.0 m long boreholes for penetrometer and borehole camera; 4) low delamination-meter; 5) hydraulic dynamometers; Z) measure of vertical convergence; M) measure of horizontal convergence; W) measure of floor heave

In these stations, penetrometric and endoscopic investigations were carried out and vertical and horizontal convergence of the gateroad was measured. Moreover, the delamination value of roof rocks deposited in the gateroad within sections from 0.0 to 3.0 m (low delamination-meter) was determined. These measurements allowed to determine the values of direct roof displacement over the gateroad support. Additionally, in one of the G-2 gateroads, the ŁP support load was measured using hydraulic dynamometers, which were placed under the rib arch of the two neighbouring supports (Walentek et al., 2009).

In the first place, three boreholes were drilled in each of the gateroads in the roof, floor and side walls of the excavation with a diameter of Ø95 mm and lengths: 10.0 m, 5.0 m and 3.0 m (Fig. 2) in order to perform penetrometric tests. On the basis of the obtained test results, the compressive and tensile strength of rocks present in the vicinity of the gateroads in question was determined. Having completed the penetrometric tests, an endoscopic camera was introduced into these boreholes to assess the fractures in the rock mass surrounding the gateroads (Prusek, 2008; Walentek et al., 2009). It was assumed that rock fracture tests would be carried out in all boreholes; however, it was not always possible because of technical reasons. This was due, among other things, to the fact that some of the floor boreholes were flooded with water.

The first measurements were carried out at a distance of about 1.0 m from the face of the excavation, the subsequent ones depending on the position of the face with respect to the measuring station at distances of 25 m, 50 m, 100 m and 250 m.

The measurement of deformation of the excavation (convergence) took place in accordance with the research methodology (Prusek, 2008) adopted in the Central Mining Institute (GIG), consisting in installing bench-marks (bolts with a length of 0.5 m) between the ŁP supports in the roof, floor and coal side walls. During the measurements, these bench-marks were permanent reference points, which allowed monitoring the dimensions of the excavation constantly in the same plane.

2.3. The results of underground measurements

Due to the size and number of data obtained as a result of underground research carried out in four drilled gateroads, this paper presents the most important and selected results in the form of Tables in which values (Table 2) are summarized: vertical convergence, horizontal convergence, floor heave, roof subsidence and low delamination. Table 3 presents the results of endoscopic investigations on the basis of which the scope and the number of fractures were identified as well as spacing between fractures of rocks underlying the seam. In addition, the results of endoscopic investigations were also presented graphically in Fig. 3-6.

TABLE 2

The results of measurements gateroads convergence and roof delamination

No	Gate-road	Distance from the face, m	Vertical convergence, mm	Horizontal convergence, mm	Floor heave, mm	Roof subsidence, mm	Roof delamination within 0-3,0 m, mm
1	G-1	1.2	0	0	0	0	0
		39.0	380	191	295	85	53
		118.0	480	226	360	120	80
		222.0	500	226	375	125	80
2	G-2	1.0	0	0	0	0	0
		22.0	48	15	28	20	20
		60.0	253	100	178	75	30
		103.0	312	116	222	90	35
		215.0	339	138	248	91	35
3	G-3	1.2	0	0	0	0	0
		33.0	59	150	20	39	20
		76.0	126	262	55	71	40
		184.0	167	397	76	91	45
		279.0	193	441	95	98	45
4	G-4	1.0	0	0	0	0	0
		38.0	15	12	8	7	5
		106.0	86	37	56	30	18
		230.0	105	47	65	40	20

TABLE 3

The results of endoscopic measurements referring to the assessment of the range of the fracture zone in the roof

No	Gate-road	Distance from the face	Range of fracture zone in the roof, m	Number of fractures in the roof			Spacing between fractures in the roof, m		
				0-3.0 m	3.0-10.0 m	0-10.0 m	0-3.0 m	3.0-10.0 m	0-10.0 m
1	G-1	1.2	2.4	7	0	7	0.43	—	1.43
		39.0	6.4	40	30	70	0.08	0.23	0.14
		118.0	6.4	60	50	110	0.05	0.14	0.09
		222.0	6.4	60	55	115	0.05	0.13	0.09
2	G-2	1.0	3.9	4	1	5	0.75	7.00	2.00
		22.0	3.9	4	1	5	0.75	7.00	2.00
		60.0	3.9	7	1	8	0.43	7.00	1.25
		103.0	3.9	8	1	9	0.38	7.00	1.11
		215.0	3.9	8	1	9	0.38	7.00	1.11
3	G-3	1.2	6.4	19	4	23	0.16	1.75	0.43
		33.0	6.4	19	4	23	0.16	1.75	0.43
		76.0	6.4	20	4	24	0.15	1.75	0.42
		184.0	6.4	20	4	24	0.15	1.75	0.42
		279.0	6.4	20	4	24	0.15	1.75	0.42
4	G-4	1.0	0.6	6	0	6	0.50	—	1.67
		38.0	1.4	8	0	8	0.38	—	1.25
		106.0	1.4	8	0	8	0.38	—	1.25
		230.0	4.6	12	1	13	0.25	7.0	0.77

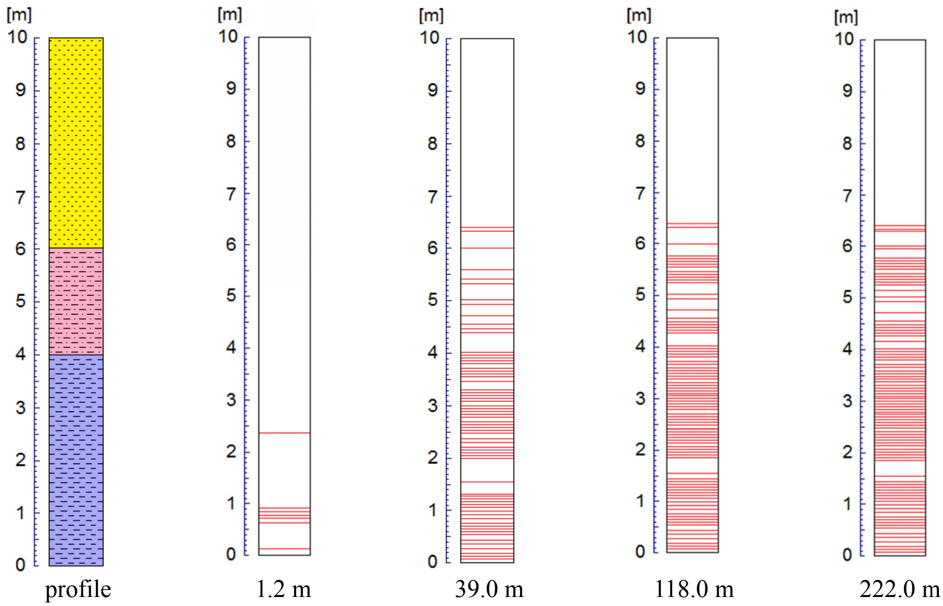


Fig. 3. Results of measurements of the fracture zone range around the G-1 gateroad depending on the position of the excavation face

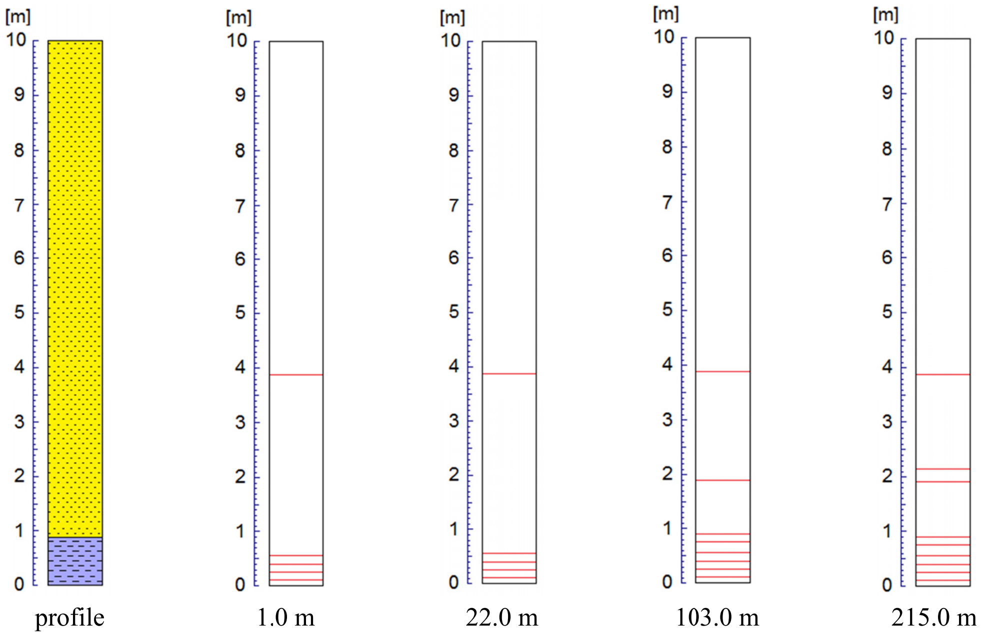


Fig. 4. Results of measurements of the fracture zone range around the G-2 gateroad depending on the position of the excavation face

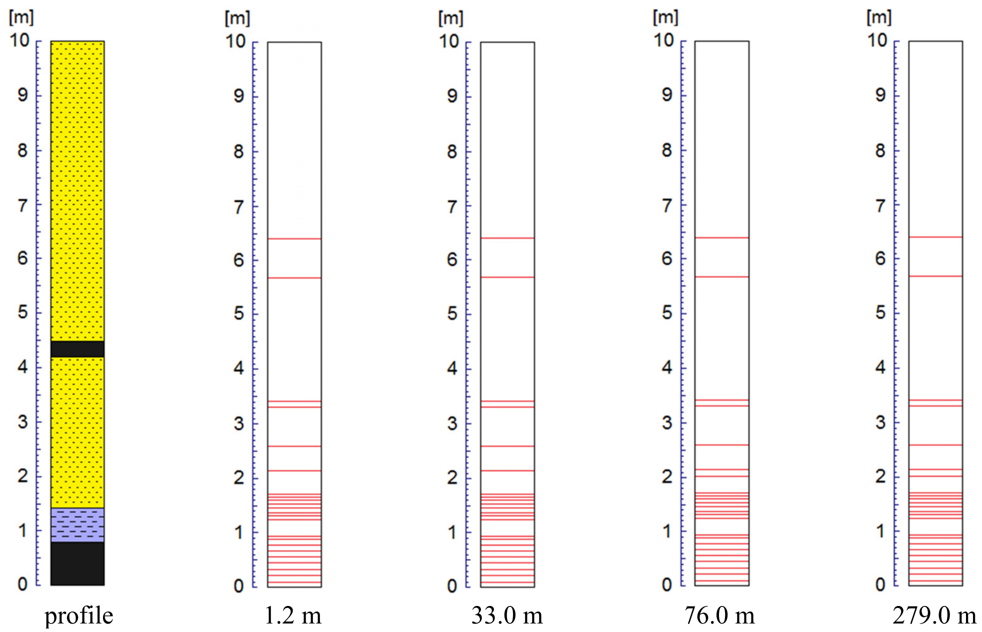


Fig. 5. Results of measurements of the fracture zone range around the G-3 gateroad depending on the position of the excavation face

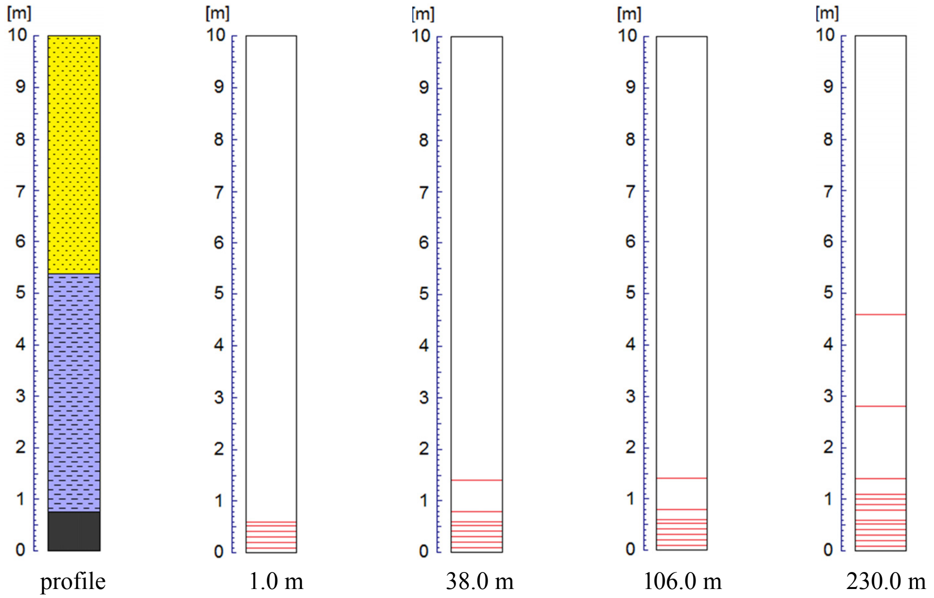


Fig. 6. Results of measurements of the fracture zone range around the G-4 gateroad depending on the position of the excavation face

2.4. Analysis of underground research indications

The results of underground investigations presented in this paper (Tables 2 and 3) show differentiation in the size of the rock mass deformation and the range of the fracture zone around the four gateroads being driven. This is influenced by, among others: the type and arrangement of rock layers surrounding excavations, strength parameters of rocks, and cross-sectional size of gateroads.

The first differences are visible in the results of vertical and horizontal convergence measurements in individual pavements (Table 2). The impact of the front moving away from the measurement station to the distance of about 250 m caused that the vertical convergence value in the G-1 gateroad was 500 mm, in the G-2 gateroad: 339 mm, in the G-3 gateroad: 193 mm, while in the G-4 gateroad 4: 105 mm. In the case of horizontal convergence, these values were 226 mm, 138 mm, 441 mm and 47 mm, respectively. It can be noticed that in the G-3 gateroad, the horizontal convergence value is twice as high as vertical convergence. In the remaining excavations, horizontal convergence constituted about 50% of vertical convergence.

The value of subsidence of roof layers is important for the design of gateroad support using the convergence control method. Underground investigations have shown that subsidence of the roof constituted from 20 to 45% of the total value of vertical convergence. Therefore upheave of roof rocks had a decisive impact on gateroad convergence. This fact is confirmed, among others, by the recorded indications of low delamination-meters where the maximum delamination values in a three-meter group of roof strata ranged from 20 to 80 mm.

In order to unify the results of roof subsidence measurements, all subsidence values in individual gateroads (Table 2) were related to their maximum values obtained in the last measurement (Fig. 7). Similarly, the results of the load measurements of the ŁP arch support in the G-2 gateroad which were mentioned in chapter 2.2, have been referred to. The obtained measurement results for averaged load values from the two neighbouring arch supports are also shown in Fig. 7.

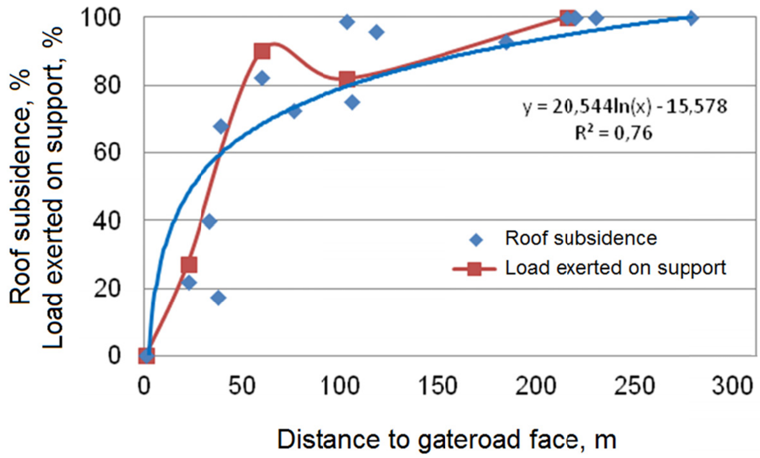


Fig. 7. The course of roof subsidence and load exerted on support in the tested gateroads during their drivage

From the results presented in Fig. 7, it can be concluded that the largest increase in subsidence of the floor during the gateroads drivage, in which the only support used to secure the excavation was arch support ŁP, occurs approximately 50 m behind the face of the excavation. The value of roof subsidence on this section of the gateroad constitutes about 60% of the total displacement of the roof during the drivage phase. From 50 m, the subsidence increase is smaller and stabilizes at a distance of about 250 m behind the gateroad front. Roof movement causes direct impact on the gateroad support, which is confirmed by the recorded results of load to the G-2 gateroad. What is more, at a distance of about 50 m behind the gateroad front, the increase in load was the highest.

The logarithmic function (Fig. 7) was adjusted to the obtained roof subsidence results in the driven gateroads, which allows us to determine the percentage increase in roof subsidence depending on the position of the gateroad face. The dependence for which the correlation coefficient R^2 =0.76 is as follows:

$$u = 20{,}544 \cdot \ln(x) - 15{,}578;\ \% \tag{1}$$

where: x – distance from the gateroad face

The above dependence may be useful when designing a gateroad support, especially in the case of the need to use additional elements reinforcing the support during the drilling of the excavation in order to limit its convergence.

The convergence of the excavation is a consequence of the fracture zone created in the rock mass, associated with the relaxation of the rocks around the excavation. This is confirmed by the results of endoscopic investigation carried out in the roof of the gateroads presented in Table 3 and Fig. 3-6.

The results presented in Table 3 indicate that both the range and number of fractures and the distance between fractures in the floor rocks over particular gateroads are varied and change with the distance from the face of the excavation.

The largest fracture range and the number of fractures was found in the G-1 gateroad, where in the distance of 39.0 it reached the height of 6.4 m and it was maintained until the last measurement conducted at the distance of 222.0 m. The maximum number of fractures in this case was 115, and the average fracture interval in the whole borehole was 0.09 m. On the other hand, within the three-meter group of rocks, the average fracture spacing was 0.05 m, which was the lowest of all gateroads tested. The reason for this was the low strength parameters of the rocks surrounding the excavation (Table 1) and the type and arrangement of rock layers (Fig. 1).

The same range of fractures was observed in the G-3 gateroad; however, the number of fractures in relation to the G-1 gateroads was more than four times smaller and amounted to 24. In the case of G-2 and G-4 gateroads, the number of fractures amounted to 9 and 13 while the maximum range of fractures was 3.9 m and 4.6 m, respectively.

The endoscopic examination showed that a zone of intense fracturing can be formed in the gateroad roof, with a large number of discontinuities (the number of fractures), which can lead to delamination of rocks and their movement towards the excavation space. Above this zone there are single discontinuities that do not have a significant impact on the stability of the excavation, e.g. of geological origin (G-2 gateroad and G-3 gateroad). In the case of G-2 and G-3 gateroads, the range of intense fracturing was 2.1 m and 3.4 m, respectively. Individual crevices appeared at the height of 3.9 m (Fig. 4) and at 5.7 and 6.4 m (Fig. 5).

Fractures in the rock mass are of such a significance in mining engineering practice that in many methods of stability assessment, the description of fractures fulfils the leading role. The most well-known engineering methods considering fracturing for the assessment of the usefulness of a rock mass for mining purposes are the so-called rock mass quality indicators such as: RQD (Deere & Deere 1988), RMR (Bieniawski, 1987), and Q (Barton et al., 1974). The endoscopic investigation of the extent of the fracture zone around the excavations, carried out in recent years, allowed the development of a new rock mass index, called Endoscopic Rock Mass Factor (ERMF) (Małkowski et al., 2008; Majcherczyk et al., 2005). Based on this index, it was determined that the excavations in question were driven in the rock mass classified into:

- class II – block type of rock mass; very good quality of rock massif – G-4 gateroad,
- class III – weakly fractured rock mass; good quality of rock massif – G-3 and G-2 gateroads,
- class VI – a type of rock mass completely destroyed; very poor quality of rock massif – the G-4 gateroad.

It follows from the above that the excavations in question were driven in extremely different conditions, affecting the size of their deformation and the destruction of the rock mass.

3. Numerical model of driven gateroad based on the convergence control method in tunnels

Having obtained the results of underground measurements in the area of driven gateroads, and especially with specific mechanical parameters of rocks and the measurements of the fracture zone range as well as the course of convergence, numerical calculations were performed for the excavations in question. The main purpose of numerical analyses was to verify the convergence control method in tunnels to map deformations and the range of the fracture zone of the rock mass measured *in situ*.

Numerical calculations for the gateroads were carried out using the program Phase2 based on the finite element method, assuming that the rock mass is an elastic-plastic and isotropic medium. The boundary state conditions were calculated according to the Hoek-Brown criterion (Hoek, 2007):

$$\sigma'_1 = \sigma'_3 + \sigma_{ci}\left(m_b \frac{\sigma'_3}{\sigma_{ci}} + s\right)^a, \text{ MPa} \qquad +(2)$$

where: σ'_1 and σ'_3 are the axial (major) and confining (minor) effective principal stresses respectively, MPa; m_b is a reduced value (for the rock mass) of the material constant mi (for the intact rock); s and a are constants which depend upon the characteristics of the rock mass; σ_{ci} is the uniaxial compressive strength (UCS) of the intact rock material, MPa.

For calculation objectives four models in the form of a shield with dimensions 70×70 m were carried out (Fig. 8), where the geological profiles used corresponded to the those presented in Fig. 1.

The mechanical properties of rock strata, including those describing the Hoek-Brown criterion (Table 4), were adopted based on results of rock strength tests conducted in the area of gateroads using RocLab (Hoek, 2007). It was also assumed that each rock stratum modeled behaves as an elastic-plastic isotropic medium.

TABLE 4

Basic parameters of rock layers adopted for numerical calculations

Model	Types of rocks	Young's modulus, MPa	Poisson's ratio υ	Compression strength, MPa	Parameter m_b	Parameter s
G-1	coal	1356	0.30	10.7	0.611	0.0008
	clay shale	2114	0.24	27.0	1.092	0.0030
	sandy shale	2857	0.21	26.2	1.248	0.0035
	sandstone	4203	0.20	48.0	2.145	0.0060
G-2	coal	1389	0.30	4.8	0.648	0.0008
	clay shale	2056	0.24	23.0	1.156	0.0020
	sandy shale	2148	0.22	30.0	1.175	0.0017
	sandstone	4099	0.20	40.0	2.206	0.0057
G-3	coal	1434	0.30	10.3	0.731	0.0006
	clay shale	2195	0.26	22.0	1.260	0.0015
	sandstone	4899	0.21	55.0	2.806	0.0067
G-4	coal	1470	0.30	21.1	0.073	0.0010
	clay shale	2056	0.26	31.0	1.205	0.0027
	sandstone	5967	0.21	40.0	2.237	0.0080

In addition the following assumptions were made in the models:
– boundary nodes along horizontal egde of the model are allowed to move only in horizontal direction,

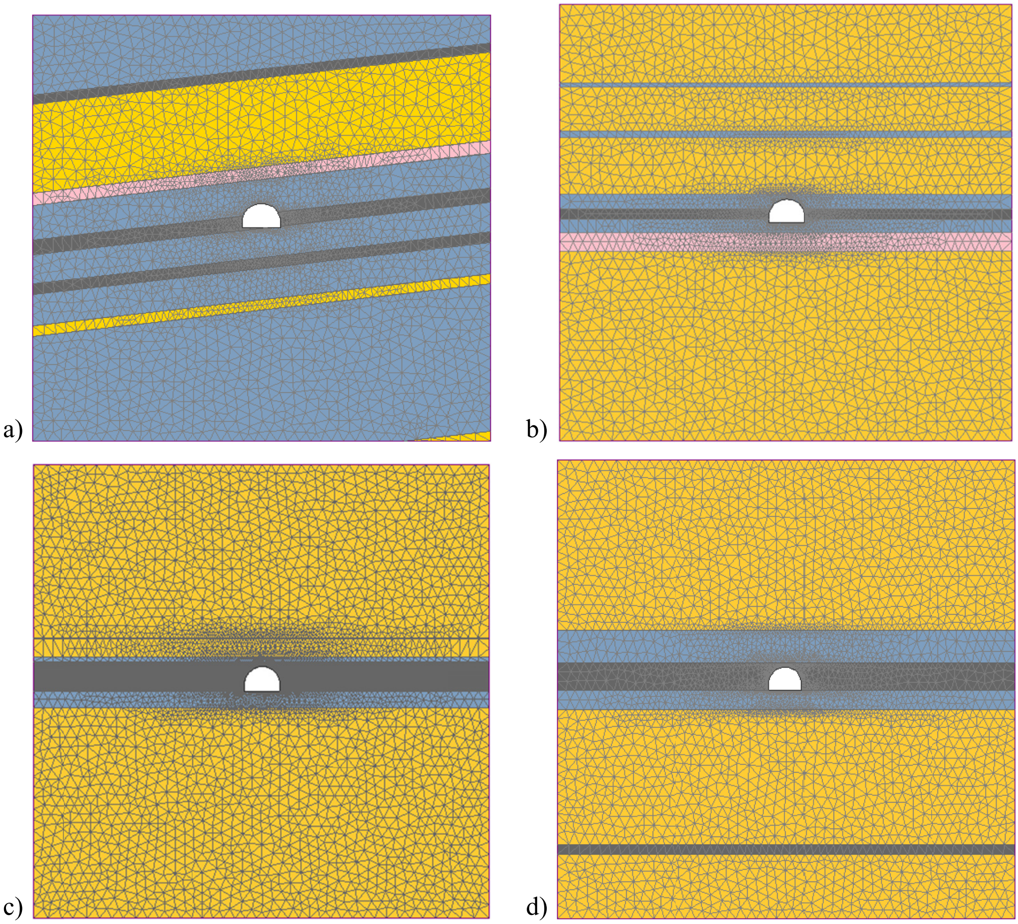


Fig. 8. Numerical models of the rock mass around the investigated gateroads: a) gateroad G-1, b) gateroad G-2, c) gateroad G-3, d) gateroad G-4

- boundary nodes along lateral edges of the model are allowed to move only in vertical direction,
- other nodes are allowed to move in XY directions,
- the initial stresses corresponded with the primary state of stresses, which results from the depth of workings' position and average weight by volume of overburden rocks,
- hydrostatic stress condition (vertical stresses equal to horizontal stresses).

In order to determine ground response curve (GRC) using the Phase2 program, the normal stresses (p_i) with values that will counteract the movement of the rock mass (u_i) should be applied to the contour of the excavation model. In subsequent calculation steps, the values of normal stress decrease asymptotically to zero, which allows obtaining the maximum displacement of the excavation contour (u_{im}). At the same time, a rock mass fracture zone (r_p), is formed around the excavation, which in the final stage of the calculation becomes the largest (r_{pm}) (Fig. 9).

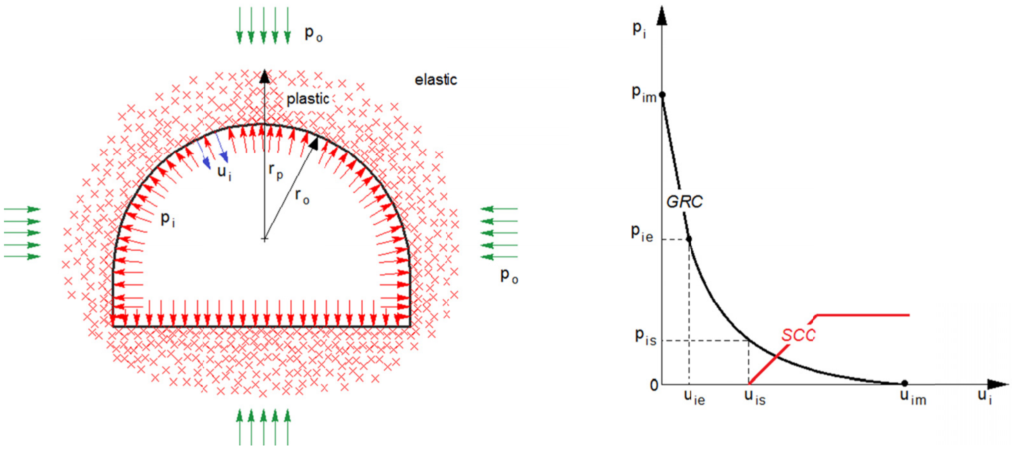


Fig. 9. Plastic zone surrounding a gateroad (left), ground response curve (right)

In the next stage of calculations, a support protecting the excavation, characterized by adequate stiffness, is introduced to the numerical model. For this purpose, a Support Characteristic Curve (SCC) is used for the calculation, which presents a graphical relationship between radial displacements (u_i) of the support with the external radial stress applied to the support (p_o).

In the excavations under study, the flexible arch support of the ŁP9 and ŁP10 type were used. Due to the inability to reproduce flexible support in the Phase2 program, the rigid support is used in the calculation models. Support Characteristic Curve for rigid support was developed based on the results of laboratory tests conducted at GIG (Rotkegel, 2013). An example of the results of laboratory tests carried out for the ŁP10 support in a rigid and flexible version is shown in Fig. 10.

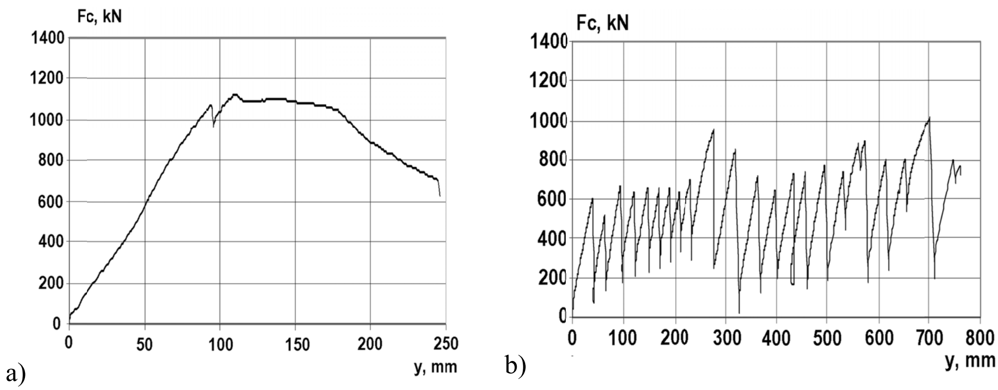


Fig. 10. Characteristics of ŁP support operation based on GIG laboratory tests:
a) rigid version; b) flexible version (Rotkegel, 2013)

During modelling of the excavation with the use of a gateroad support, it is assumed that it is installed immediately after gateroad is completed. This is not the appropriate procedure and may lead to overvaluation of the support load and cause a significant reduction of displacements around the excavation and reduction of the fracture zone range. In fact, every support is installed in the excavation after a certain period of time during which certain displacements of the rock massif occur.

Due to the fact that the numerical calculations of ground response curves (GRC) for the analysed excavations were carried out on models in a plane strain condition (shield perpendicular to the gateroad axis), it is difficult to present the deformation of the rock mass along the axis of the gateroad and install the arch support in real distance behind the gateroad face. To this end, in order to indicate the optimal moment of support installation in the numerical model, a computational method was used for circular tunnels (radius r_o) using the Longitudinal Displacement Profile (LDP) (Vlachopoulos & Diederichs 2009). This is the contour displacement curve of the tunnel contour along the section passing through its main axis. In the case of a circular tunnel, the graph shows the relationship between radial displacements and the distance from the face (Lato & Diederichs 2014) (Fig. 11).

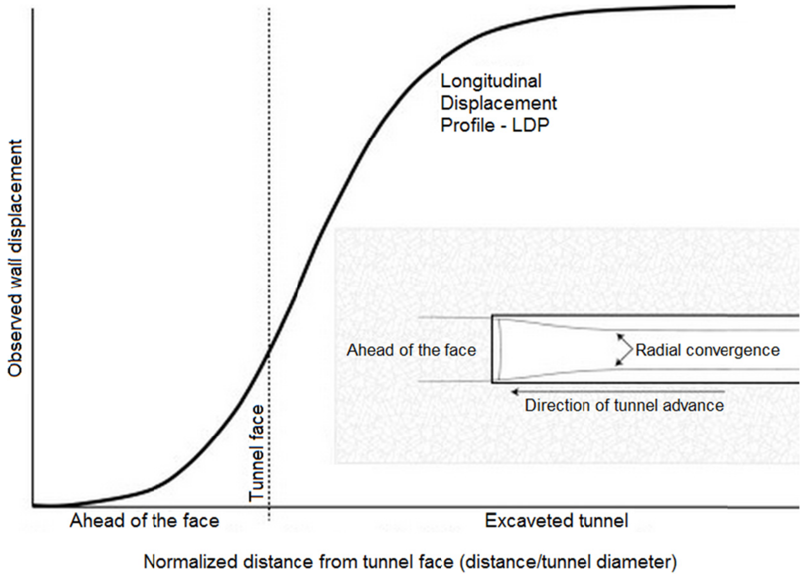


Fig. 11. Use of the longitudinal displacement profile, (LDP), to relate tunnel wall displacement as a function of distance from face (Lato & Diederichs 2014)

The LDP calculation takes into account the significant influence of ultimate (maximum) plastic radius. Failure to use the appropriate LDP can result in significant errors in the specification of appropriate installation distance (from the face) for roadway support systems.

The Longitudinal Displacement Profile, shown in Fig. 11, is required in order to establish the relative position of the tunnel face and the sections under consideration. It is necessary to carry out a three-dimensional analysis to determine this profile. The graphical tool presented in Fig. 12 (Vlachopoulos & Diederichs 2009) is for short term displacements occurring as a function of tunnel advance only.

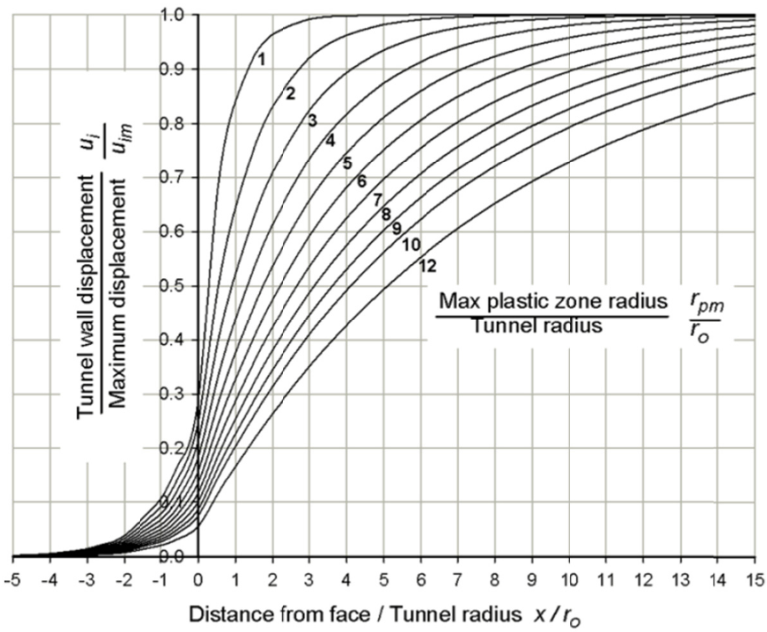


Fig. 12. Longitudinal Displacement Profile (Vlachopoulos & Diederichs, 2009)

Based on the waveforms of LDP curves shown in Fig. 12, displacement values of the excavation roof (u_{is}) were determined in relation to the actual distance of arch support installed in the coal face (x). It enabled indication of the moment when the gateroad support had to be installed in numerical models. The results for the individual components of the LDP curves are summarized in Table 5.

TABLE 5

Results of calculations of indicators

No	Roadway	Distance from the face (x), m	Tunnel radius (r_o), m	Maximum plastic zone radius (r_{pm}), m	Maximum tunnel roof displaces (u_{im}), m	Tunnel roof displaces before the support is installed (u_{is}), m
1	G-1	1.10	2.75	9.35	0.221	0.077
2	G-2	1.00	2.75	6.25	0.130	0.043
3	G-3	1.00	2.75	7.15	0.206	0.057
4	G-4	1.30	2.50	6.20	0.112	0.036

In accordance with the assumptions presented above, numerical models were used to perform complete simulations for the numerical models in the scope of mapping the results of underground tests of driven coal faces using the convergence control method in tunnels. The results of numerical calculations are shown in Fig. 13.

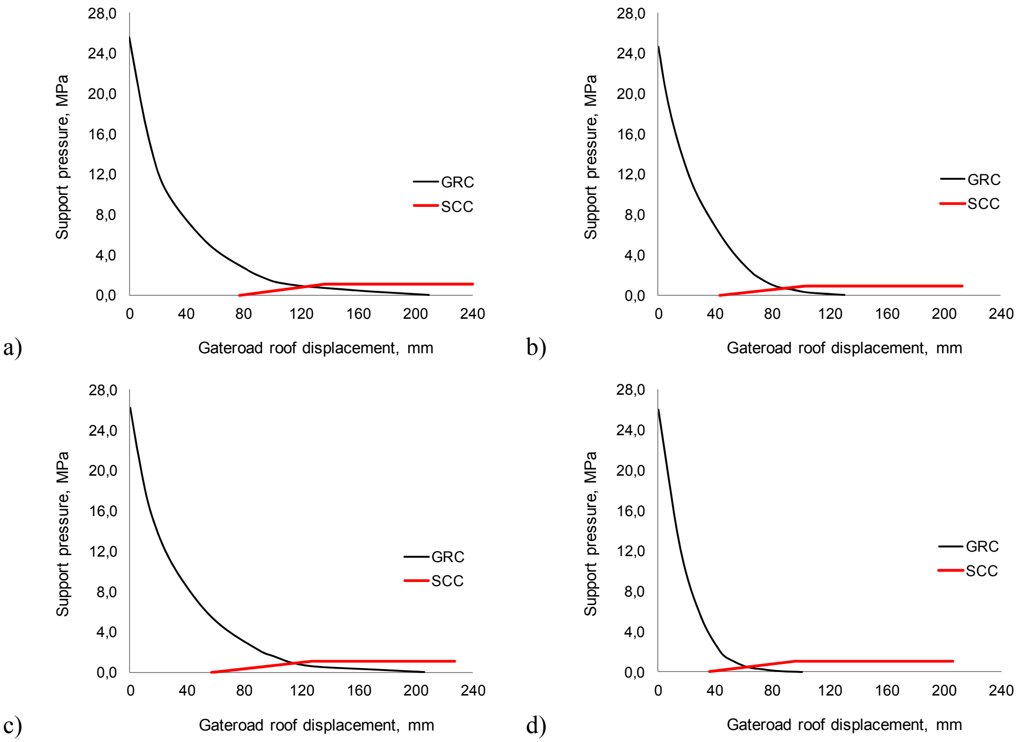


Fig 13. Results of GRC and SCC calculations for investigated gateroads: a) gateroad G-1, b) gateroad G-2, c) gateroad G-3, d) gateroad G-4

The numerical calculations carried out using the Phase2 showed that for the adopted method of ground response curve (GRC), the maximum values of displacements of the roofs unprotected by the support after stabilization of the rock mass were 211 mm, 130 mm, 209 mm and 91 mm, respectively. In comparison with the results of underground research, these values are higher by about 43% to 127%. It should be noted that the calculations for plotting the GRC curve did not take into account the ŁP arch support.

After installing, in numerical models, the ŁP arch support operating in accordance with the adopted characteristics (SCC), the maximum displacement values of the roof (at the intersection of two curves) for individual excavations are: 129 mm, 95 mm,107 mm and 44 mm. This time the differences between the results of underground tests and numerical calculations do not exceed 10%.

According to the laws of geomechanics, the displacement value is a consequence of the fracture zone created in the rock mass. Fig. 14 presents the results of numerical calculations showing the extent of the fracture zone around the analysed excavation at the moment of stabilization of the rock mass in conjunction with the ŁP arch support.

Referring the obtained values of the range of the fracture zone of roof rocks to the results of endoscopic examination, it can be concluded that the numerical calculations coincide exactly with the area of intensive fractures in *in situ* conditions. This confirms the theory that the zone of

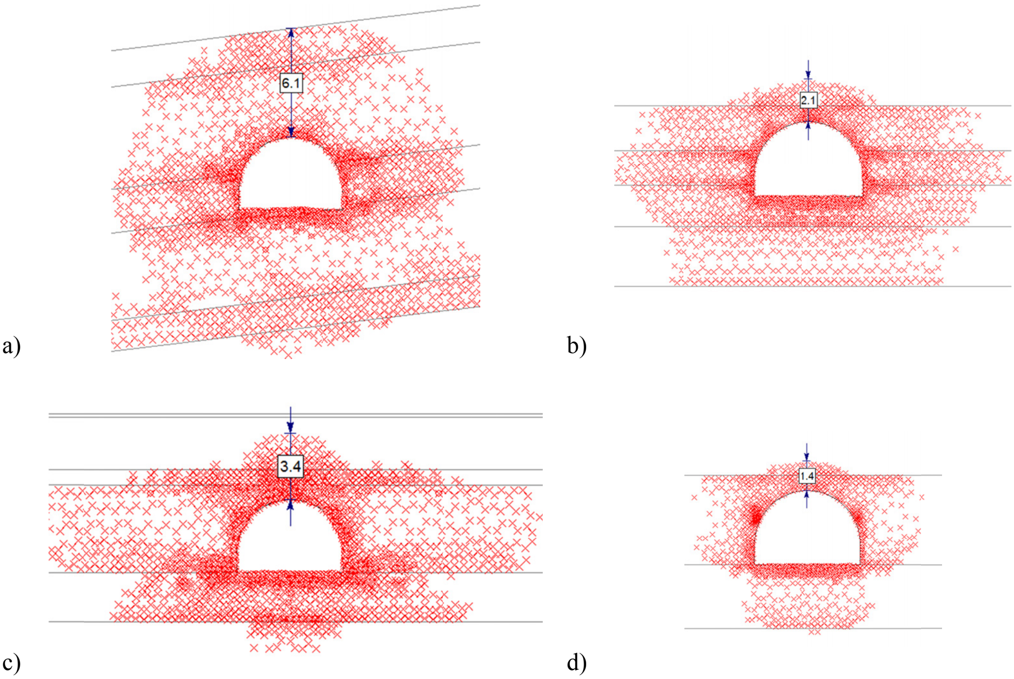


Fig. 14. Maps of fracture zone around the gateroads: a) gateroad G-1, b) gateroad G-2, c) gateroad G-3, d) gateroad G-4

these fractures is important in the convergence mechanism of the gateroad. Due to the fact that numerical models were made in a continuous medium, it is difficult to obtain single fractures outside the area of intense fracturing.

In order to compare the results of underground research with the results of numerical calculations in Table 6, the maximum values of: subsidence of the roof, low delamination and the range of the zone of intense fractures in the roof of the excavation are summarized. The obtained large convergence of results justifies the selection of the convergence control method in the roadways for the design of gateroads.

TABLE 6

Comparison of underground research results and numerical modelling of driven gateroads

No	Gateroad	Roof subsidence, m		Roof delamination 0-3.0 m, m		Range of intense fracturing zone, m	
		Measurement	Model	Measurement	Model	Measurement	Model
1	G-1	125	129	80	82	6.4	6.1
2	G-2	91	95	35	35	2.2	2.1
3	G-3	98	107	45	46	3.4	3.4
4	G-4	40	44	20	25	1.4	1.4

4. Conclusions

Conducting mining works, including gateroads drivage, is significant in the proper functioning of mines in the context of effective longwall management (Prusek et al., 2016). The quality of drilling work, as well as design and measurement *in situ* is of a decisive importance whether the excavation will optimally fulfil its purpose throughout the entire lifetime. Errors that will be committed at the outset will result in difficulties in maintaining the stability of the gateroads during longwall mining and may pose a threat to the working crew.

This paper presents the results of measurements of deformations in the excavations and the development of the fracture zone in the roof of the seam together with the progressive gateroad advance. The results of these tests, formed the basis for gateroad support design process using the convergence control method in tunnels, based on ground response curve (GRC). The obtained results of numerical calculations showed the possibility of using this method in the conditions of mines located in the USCB.

This method provides a lot of information about the cooperation of the support with the rock mass due to the use of Support Characteristic Curve (SCC) and Longitudinal Displacement Profile (LDP), which allow us to determine the time of installation of the support in the rock mass model.

The obtained results of underground research also provide a lot of valuable information regarding the mechanism of fractures formation, deformation and the load of the support when driving gateroads. This is important in the correct design of the ŁP arch support as well as the optimal selection of the bolt support. Most often in gateroads, bolting is performed at a distance from the front of the longwall, or behind the front of driven gateroads, often at distances above 100 m. Considering the fact that one of the main tasks of the bolt support is to prevent the delamination of rocks in the vicinity of the excavation, it is commonly considered that the delayed installation of the bolts is not favourable.

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