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ANALISIS OF THE LOAD CAPACITY OF A SKELETAL SUPPORT CONSTRUCTION FOR USE IN DOG, OR BLIND-HEADING JUNCTIONS

ANALIZA NOŚNOŚCI SZKIELETOWYCH KONSTRUKCJI OBUDOWY ODGAŁĘZIEŃ WYROBISK KORYTARZOWYCH

The paper presents a new type of steel rib construction for the support of dog, or blind-heading junctions. The design, developed by the author, is currently being manufactured in the "Łabędy" Steel Plant.

Calculations of the strength of the support under assigned external loadings set up by rock mass were made using PRO-MES RAMA 3D program.

The values obtained for the moments of bending and stress, and the internal strength of the elements of the structure lie within permitted limits.

From observations and in-situ examinations it was demonstrated that the structure could find a useful application in the difficult geological and mining conditions in hard coal mines.

Key words: safety, mining, dog heading support, strength, parameter selection

W pracach badawczych i konstrukcyjnych obudowie połączeń wyrobisk korytarzowych poświęca się znacznie mniej uwagi niż obudowie samych wyrobisk (Chudek 1982). Dotychczasowa praktyka projektowania i wykonywania obudowy odgałęzień wskazuje, że dla danego odgałęzienia jest wymagane indywidualne obliczanie wymiarów kolejnych odrzwi oraz obliczenie każdorazowo długości i promieni gięcia elementów łukowych dla poszczególnych odrzwi (Niechciał 1970; Kowalski et al. 1989). Do wad tego typu konstrukcji należy konieczność stosowania nietypowych łuków o różnych krzywiznach, wykonywanie dużych wyłomów, stosowanie specjalnej technologii wykonania, niejednokrotnie dodatkowego wzmacniania obudowy za pomocą kotwi bądź obetonowania. Próby zastosowania trwalszych konstrukcji obudowy odgałęzień prowadzone były m.in. w Zakładzie Badawczo-Rozwojowym Budownictwa Górniczego BUDOKOP (Wojtusiak 1975) oraz w kopalni „Bogdanka" (Głuch, Limburski 1987).

Na podstawie dokumentacji oryginalnych konstrukcji, opracowanych w Głównym Instytucie Górnictwa (Stałęga 2001), uruchomiono produkcję na skalę przemysłową w Hucie „Łabędy" w Gliwicach typoszereg obudów odgałęzień od wielkości odrzwi W8-WI5.

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Komplet obudowy odgałęzienia (rys. 1) składa się z cztero- lub pięcioelementowych łukowych odrzwi przejściowych 1 zabezpieczających odcinek wyrobiska wejściowego przed portalem, portalu 2, wspornika 3 oraz trzyelementowych odrzwi uzupełniających 4. Konstrukcyjne elementy wspornika wyposażone są w łączniki 5 umożliwiające dwuprzegubowe, sworzniowe połączenie łukowych elementów odrzwi obudowy. Zarówno portal jak i wspornik w swoich dolnych częściach mają segmenty upodatniające 6, które zaczynają pracować po przekroczeniu określonych obciążeń portalu i wspornika, powodujących ścięcie sworzni usztywniających konstrukcję i zgniatanie stosu z impregnowanych płyt dębowych o łącznej wysokości 500 mm.

Szkieletowe konstrukcje obudowy modelowano elementami, które przenoszą siły osiowe, siły poprzeczne, momenty skręcające i momenty zginające.

Wyniki obliczeń przedstawiono w formie rysunków ugięć, rozkładu sił wewnętrznych, reakcji i naprężeń.

Obliczenia wytrzymałościowe obudowy odgałęzień typu „Łabędy" przeprowadzono przy wykorzystaniu metody elementów skończonych (Zienkiewicz 1973). Posłużono się programem komputerowym PRO-MES 5.1.RAMA 3D. Po wprowadzeniu wymaganych danych oraz założeń i warunków brzegowych wykonano obliczenia, w wyniku których uzyskano wartości sił wewnętrznych, naprężeń oraz przemieszczeń elementów obudowy powstałych na skutek działania założonego obciążenia zewnętrznego. Na tej podstawie, przy znanych parametrach wytrzymałościowych przekrojów poprzecznych elementów obudowy, obliczono wartość maksymalnego obciążenia zewnętrznego, jaką może przenieść ta obudowa.

Za podstawę do utworzenia dyskretnego modelu obliczeniowego (rys. 2) posłużyła dokumentacja konstrukcyjna obudowy odgałęzienia nr BK-946.00.00 (archiwum GIG-BG) , wykonana z wykorzystaniem wspomagania komputerowego (Pacześniowski, Skrzyński 1995). Na rysunku 3 przedstawiono usytuowanie kształtownika konstrukcji nośnej w modelu.

Dla obliczanej konstrukcji przyjęto obciążenie równomiernie rozłożone, działające na odcinku równym 0,60—0,65 długości elementu stropnicowego odrzwi (PN-92/G-15000/05).

Wartość obciążenia całkowitego odrzwi przyjęto w taki sposób, aby odpowiadała obciążeniu rozłożonemu liniowo q_o = 0,1 MN/m (rys. 4).

Model podparcia układu (warunki brzegowe) przedstawia rysunek 5.

Wyniki obliczeń przedstawiono w postaci graficznej oraz liczbowej. Obejmują one następujące wielkości charakteryzujące zachowanie się modelowanej konstrukcji, przy założonych warunkach brzegowych, pod wpływem działania przyłożonego obciążenia zewnętrznego:

- przemieszczenia węzłów konstrukcji wzdłuż głównych osi układu współrzędnych;
- reakcje w wybranych punktach podparcia konstrukcji;
- siły wewnętrzne występujące w elementach konstrukcji:
  - siły poprzeczne T_z, T_y względem głównych osi kształtownika,
  - siły podłużne N,
  - momenty zginające M_z, M_y względem głównych osi kształtownika,
  - momenty skręcające M_s względem osi przechodzącej przez środek ścinania przekroju poprzecznego kształtownika;
- naprężenia ściskające i rozciągające w przekroju poprzecznym kształtownika.

Wyniki obliczeń. dla obudowy, której konstrukcja nośna wykonana jest z kształtownika HEB 450, zaś odrzwia z kształtownika V29 przedstawiono na rysunkach 6 i 7.

Na podstawie uzyskanych wyników obliczeń wyznaczono maksymalną wartość występującego w odrzwiach naprężenia σ_z przy mimośrodowym ściskaniu, wyrażonego wzorem (l).

Wartość naprężenia σ_n, powodującego powstanie przegubu plastycznego określono według wzoru (2).

Na podstawie powyższych zależności określono warunek wytrzymałościowy wyrażony wzorem (3).

Z przytoczonych zależności wynika, że dla elementów konstrukcji, obciążonych zgodnie z założonym schematem, wartość obciążenia rozłożonego liniowo $q_{\max}$, powodująca powstanie przegubu plastycznego na skutek osiągnięcia przez stal granicy wytrzymałości na rozrywanie określa wzór (4).

Wartości całkowitego maksymalnego obciążenia dla rozpatrywanych przypadków zostały przedstawione w tablicy 1.

Badania dołowe w kopalni B (rys. 8) polegały na prowadzeniu cyklicznych (comiesięcznych) liniowych pomiarów szerokości i wysokości portalu i wspornika w charakterystycznych punktach, wielkości osiadań w węzłach upodatniających obu ram oraz zsuwów w złączach odrzwi obudowy. Mierzone wartości przemieszczeń elementów obudów przedstawia rysunek 1. Pozostałe konstrukcje różniły się między sobą gabarytami portali i wsporników oraz liczbą odrzwi przejściowych i uzupełniających.

Wyniki sześciomiesięcznej akcji pomiarowej dowiodły, że po tym okresie pracy obudowy wokół wyrobiska w górotworze wytworzyła się i ustabilizowała obszarowo strefa odprężona.

Analizę statystyczną wyników pomiarów dołowych przeprowadzono za pomocą programu komputerowego STATISTICA. W obliczeniach zastosowany został moduł „estymacja nieliniowa", pozwalający między innymi na zdefiniowanie przez użytkownika własnej funkcji regresji. Pod uwagę brano szereg funkcji, z których najwierniej zmiany szerokości portalu i rozpiętości wspornika oraz zmiany wysokości upodatnienia odzwierciedlały funkcje logarytmiczne. W tablicy 2 zestawiono funkcje regresji, natomiast na rysunkach 9–11 wybrane przebiegi funkcji regresji z zaznaczonymi punktami z pomiarów.

Opracowany typoszereg obudów odgałęzień wyrobisk obejmuje praktycznie wszystkie przypadki zabudowy ich połączeń w różnych warunkach geologiczno-górniczych, poprzez zmianę wielkości profilu belek nośnych, kształtownika odrzwi (V29, V32 lub V36) oraz rozstawu odrzwi w zależności od prognozowanego obciążenia ze strony górotworu.

Analizowane konstrukcje skutecznie zabezpieczają wyrobiska przed deformacjami. Nośne ramy obudowy (portal i wspornik) pracowały w zakresie odkształceń sprężystych, a w łukowych odrzwiach obudowy, poza niewielkimi zsuwami w złączach, nie stwierdzono nadmiernych odkształceń plastycznych. Świadczy to o prawidłowym doborze elementów obudowy odgałęzień, tj. wytrzymałości belek nośnych ram obudowy i wielkości kształtowników łukowych odrzwi, do istniejących warunków geologiczno-górniczych i występujących obciążeń ze strony górotworu.

Narastanie obciążeń ze strony górotworu zachodziło w okresie 3–6 miesięcy, licząc od momentu zastosowania obudowy, po czym odkształcenia jej konstrukcji wykazywały charakter malejący i ustalał się stan równowagi układu górotwór-obudowa.

Z uwagi na różnorodność warunków geologiczno-górniczych występujących w Górnośląskim Zagłębiu Węglowym, decydujących o współpracy obudowy z górotworem karbońskim, uważa się za niezbędne, aby każdy indywidualny przypadek zastosowania konstrukcji odpowiedniej wielkości z typoszeregu był poprzedzony rozeznaniem warunków geotechnicznych w miejscu jej zabudowy, w celu określenia wartości obciążeń konstrukcji.

Słowa kluczowe: bezpieczeństwo, górnictwo, obudowa chodnikowa, wytrzymałość, dobór parametrów

1. Introduction

In the Polish coal-mining industry, numerous junctions of horizontal access headings are made annually, often under difficult geological and mining conditions. In the works devoted to research into and construction of headings, remarkably little attention is paid to the support of dog heading junctions in comparison with that devoted to the support of the heading excavations themselves (Chudek 1982). The simplest and most commonly-applied assumptions made in the evaluation of the working conditions a dog heading support system tend to dismiss the junction as being only one part of the whole section.

Such an assumption allows the analysis of structural loadings and interaction with the surrounding rock mass in a flat, or 2-D system allowing the values of loads or internal forces in the support per unit of excavation length, to be obtained. Such a simplification, when constructing junctions, is inadequate, since the character of loads imposed and the forces exerted by junction support system are 3-dimensional, or spatial.

The most frequent heading-junction resembles a letter **y** in plan view. Its support structure consists of multi-element, flexible, arched frames, the width and height of which increases as the junction itself is approached. It is at this point that the heading reaches its largest dimensions, both in terms of height and width, thereby exposing the support structure to the largest rock-mass loading. Yet, considering the dimensions of the support-frame members, it is here that the lowest levels of load bearing capacity occur. Besides, in roof of the corner a space, allowing for hazardous gathering of methane and causing an extra air flow resistance, occurs. Also the increased roof height at the convergence of the heading creates an elevated pocket with reduced air movement and the risk of localised methane build-up.

The practise followed hitherto when designing a support system for a particular junction demands that frames be mechanically analysed and designed on an individual and sequential basis, involving the length and radii of curvature of various elements. (Niechciał 1970; Kowalski et al. 1989).

The faults of such a construction method lie in need use non-standard arches of various curvatures, large scale excavations, the application of a special techniques and often extra reinforcement using bolts, or concrete.

A unique design for a specialised support system for use at junctions has been developed at BUDOKOP, the Research and Development Plant of Mining Building (Wojtusiak 1975). The fundamental load-carrying element of the support constitutes three arched steel ribs joined radially in plan. The additional provision of a separating rib, in a plane of symmetry with the side-ribs allows the influence of uneven settlement of the rib-foundations to be absorbed by load-bearing structure. To enable the support-structure and the surrounding rock-mass to act co-operatively rather than in opposition the central joining-point of the arch-ribs is rock-bolted into position.

In the Lublin Coal Basin, in view of very difficult geological and mining conditions, in the 1980s, many changes concerning excavation and the construction of heading support structures , as well as the technology of its implementation at junction points, were introduced (Głuch, Limburski 1987).The essential element of the junction support — structure consists of, as applied in the "Bogdanka" Mine, a bowl shaped junction headwall, which allows load-bearing capacity to be increased and, also, to redistribute stress-patterns more evenly (prevent the nodal accumulation of stress concentrations). In order to risk collapse of the divergence pier (the divergence pier is the pillar of material where the two headings diverge) into the excavation , the headwall was dug out in such a way that the corner of the connection was shifted as far as possible towards the body of coal so that its width became 1.0–1.5 m. The headwall was formed in a spherical shape connecting the roof and wall part of the excavation with ceiling part with the aid of vertical pillars and the side parts with the aid of horizontal arches. The support was made from V type channel-bar. Across the width of the corner, four vertical arches made from double V-29 section were fixed, which rested with their ends on the largest support frames whilst the vertical arched resting on the frames of the entry headings and the largest frames of a branching heading. The vertical arches were joined to the horizontal ones by hook bolts and formed a steel grating.

In addition these solutions, it is possible to roof-bolt the rock mass and/or spray concrete which, does not however, always result in a considerable increase in load--bearing capacity of the support-structure. The necessity seek new solutions designed to ensure the full stability of support-structures for dog heading junctions, remains.

2. Characteristic of the skeletal construction of a junction support

On the basis of a unique design, evolved and analysed at the Central Mining Institute (Stałęga 2001), the production of a standardised series of junction-types has been started at the "Łabędy" Steel Plant on an industrial scale. The frame size of the supports ranges between W8–W15.

A set of supports (Fig. 1) consists of four-or five-element arched passage frames, a protecting section for the entrance excavation before a portal, a portal 2, a bracket 3,

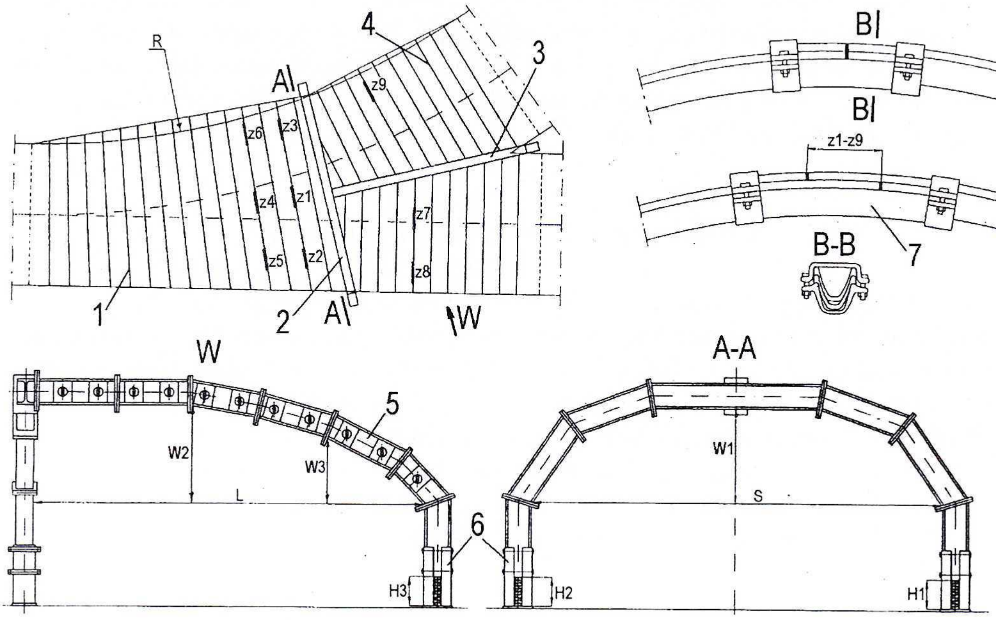

Fig. 1. Skeleton support construction of junctions of "Łabędy" type

Rys. 1. Szkieletowa konstrukcja obudowy odgałęzień typu "Łabędy" and a triple complementary frame 4, thereby supporting the entry excavation and branching connections of the excavations. The dimensions and shapes of arc elements are dependent on the junction geometry. The portal and bracket are made from construction elements in the form of box welded plate girders or a double-tee bar, possessing front plates set perpendicularly or at an appropriate angle to the longitudinal axis of the elements allowing them to be connected with the aid of bolts.

The constructional elements of the bracket include plates 5 allowing the arched elements of the support frame to be connected using twin bolt. This solution is characterised by its simple structure which does not require any labour-consuming processing, ensures joining of the connection of frame arches with the load-bearing support-structure and the reduction of both horizontal force component, acting on the frames and bending moments in the connectors, and also in the structural beam formed by the girders. Hence it eliminates many faults occurring in solutions of other firms (Heintzmann, FAZOS) where the shape of link-mechanisms sets up unfavourable stress concentration in girder-seatings and eyes connecting with roof-bar ends.

Both the portal and bracket at their bottom parts possess likening segments 6, which come into effect after pre-set loads of the portal and the bracket are exceeded causing the bolts stiffening the construction to shear off and compressing a 500 mm pile of impregnated oak plates. This ensures the flexibility of fundamental load-carrying elements of the junction support-structure. Flexibility of passage-supplementing frames is gained by arch-connecting elements of these frames 7. The other elements of the support such as sprags, cladding linings, are installed in accordance with appropriate regulations for installing arch-supports made from V-section shapes, while for crosswise frame stabilisation of the passage frames, sprags of controllable length are essential, whereas supplementing frame stabilization can be achieved by the use of sprags of adjustable or fixed length.

3. Numerical testing of support structure

The skeletal support-structure has been modelled with elements which transfer axial forces, transverse forces, twisting moments and bending moments. The elements have been connected by stiff nodes, which only after joints were locked could result in neutralization of an sufficient inter-nodal forces. According to assumptions made in PRO-MES system nomenclature they are treated as "beams".

For the assumed scheme the following loads have been calculated:

- linear and angular dislocations,
- internal forces of components required for the model type,
- reactions,
- stresses.

Extreme values (envelopes) can also be obtained for each result, calculated according to attributes. These extreme values could be assigned for each component independently or according to indicated value in association with other factors.

The system values allows to be read at any point, not only at model's nodes.

The results are presented graphically in the form of bending diagrams, internal force-distributions, reactions and stresses.

3.1. Method of calculation

The calculation of the strength of the "Łabędy" type junction support were carried out using the method of finished element (Zienkiewicz 1973). PRO-MES 5.1.RAMA 3D computer program was employed for this purpose. After introducing the necessary data, assumptions and boundary conditions, the calculations were carried out. These resulted in a series of values of internal strengths, stresses and dislocation of support elements which occurred as forces produced by the assumed external loading took effect. On the basis of these results and taking into consideration recognised strength parameters, cross-section of support elements, and the values of maximum external loads which could be transferred by the support were calculated.

In making the calculation the requirements of standards PN-93/G-15000/02, PN-92/G-15000/05, PN-93/G-15000/03, PN-H-93441-3;1994 and PN-90/B-03200 were taken into consideration.

As a basis for the elaboration of a discrete calculation model (Fig. 2) the constructional documentation of junction support Nr BK-946.00.00 (archive GIG-BG) was used, with computer assistance (Paczešniowski, Skrzyński 1995)

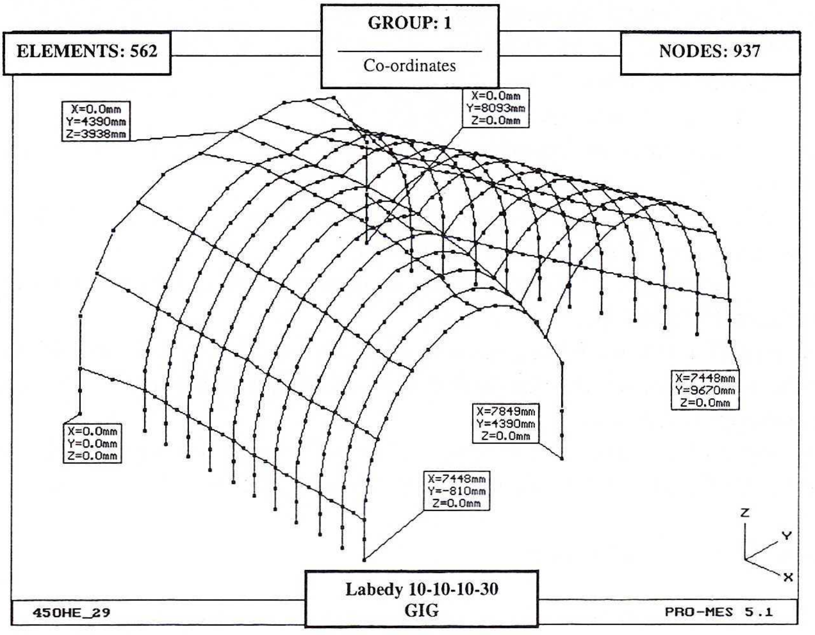

Fig. 2. Discrete calculation model (system geometry) of junction support

Rys. 2. Dyskretny model obliczeniowy (geometria układu) obudowy odgałęzienia

In order to design a discrete calculation model for PRO-MES program in analysed frames:

- the structure was divided into elements and appropriate node geometrical parameters were assumed,
- geometrical and material features of particular elements were defined,
- places and directions of load actions were defined,
- elastic constraints were replaced by equivalent static supports of adequate stiffness,
- the defined values were written down in a format according to the requirements of PRO-MES program.

3.2. Assumptions

The smallest values of material parameters were assumed for the calculations, using materials in the library of typical materials (among other things steel, wood) of PRO-MES program, namely:

- steel of the following parameters:
  - young's modulus 205000 MPa,
  - poisson ratio 0.3,
  - specific gravity 0.077 MN/m^3;
- portal and bracket tractable material;
- wood of the following parameters:
  - young's modulus 9000 MPa,
  - poisson ratio 0.065,
  - specific gravity 0.006 MN/m^3;
- bearings, on which the support is founded: of $1.0 \cdot 10^6$ MN/m (stiff bearings along fibres);
- bearings representing passive resistance according to PN-92/G-15000/05 (model of walls reaction) of stiffness 2.7 MN/m;
- bearing representing the action of support spatially-stabilising (strutting): of stiffness 15.0 MN/m;
- the area of active roof-load reaction according to PN-92/G-15000/05, that means in the area of the width equal to 0.6–0,65 of the free length of roof elements of the arch-support frames.

The strength characteristics of non-standardised shapes for support elements were calculated with the aid of MOMBEZ 3.2 modulus of PRO-MES 5.1 program.

3.3. Data for calculation

The construction documentation of header-junction support-structure Nr BK-946.00.00 (Archive GIG-BG), analysed with computer assistance, formed the basis for the formulation of a discrete calculation model (Fig. 2) (Pacześniowski, Skrzyński 1995). For the description of cross-sections of particular support elements has been assumed:

- Closed welded profile of 450 mm high manufactured by the "Łabędy" Steel Plant of the following parameters:
  - cross-sectional area $A = 0.022400$ m^2,
  - moment of inertia of cross-sectional area $I_x = 0.00076228$ m^4,
  - index of elastic bending $W_x = 0.0003388$ m^3.
- Double-tee bar HEB 450 of the following parameters:
  - cross-sectional area $A = 0.021800$ m^2,
  - moment of inertia of crosswise section $I_x = 0.00079890$ m^4,
  - index of elastic bending $W_x = 0.0003551$ m^3.
- Shape V29 of the following parameters:
  - cross-sectional $A = 0.003700$ m^2,
  - moment of inertia of crosswise section $I_x = 0.0000016$ m^4,
  - index of elastic bending $W_x = 0.0000937$ m^3.
- Shape V36 of the following parameters:
  - cross-sectional area $A = 0.004570$ m^2 ,
  - moment of inertia of crosswise section $I_x = 0.00000972$ m^4,
  - index of elastic bending $W_x = 0.0001365$ m^3.
- The limit of material plasticity $R_m = 340$ MPa.
- Immediate tensile strength of the material $R_m = 540$ MPa.

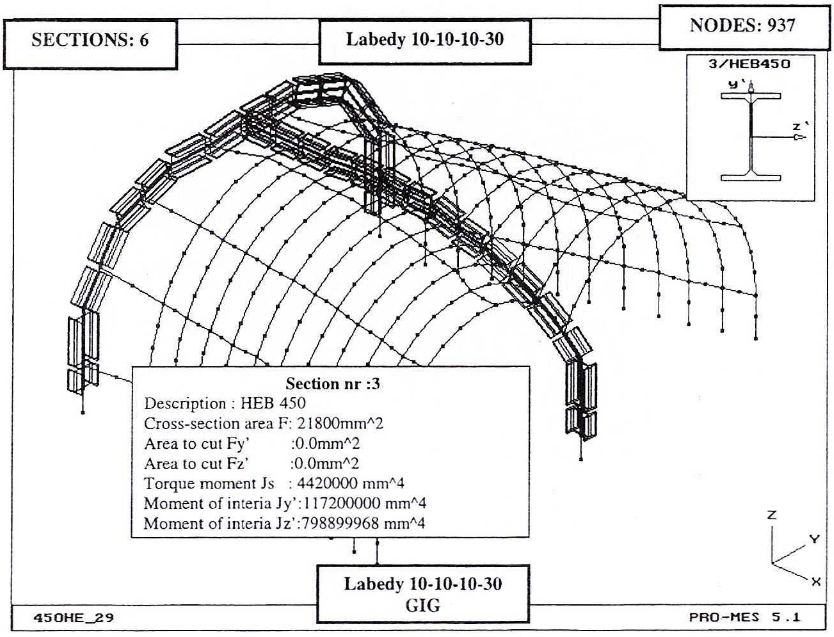

Fig. 3. Location of HEB450 I-section in the computational model

Rys. 3. Usytuowanie dwuteownika HEB450 w modelu obliczeniowym

In Fig. 3 the localization of the shape of load-carrying construction in the model is presented. As a shape of the sprags, a bar of circular section and strength characteristics equivalent to parameters of adjustable sprags used for supports, was assumed.

In order to calculate the construction, a uniformly distributed load , acting within a length equal to 60–65% of the length of frame roof element (PN-92/G-15000/05), was assumed. The detailed calculation procedure to obtain a value for the structural load is specified, among other things, in Instructions (1988).

The value of total load of the frame was assumed in such a way so that it is equivalent to linearly distributed load q_o = 0.1 MN/m (Fig. 4).

The model of the support-system (boundary conditions) is presented in Fig. 5.

Making use of the above discussed data and assumptions, calculations of the following specification of shapes of "Łabędy" type ,load-bearing junction support frames of 10-10-10-R30-P size, were carried out:

Profile of junction load-carrying	Profile of frame elements
• structure	
• HEB 450	V29
• Welded 450 mm	V29
• Welded 450 mm	V36
• HEB 500	V29

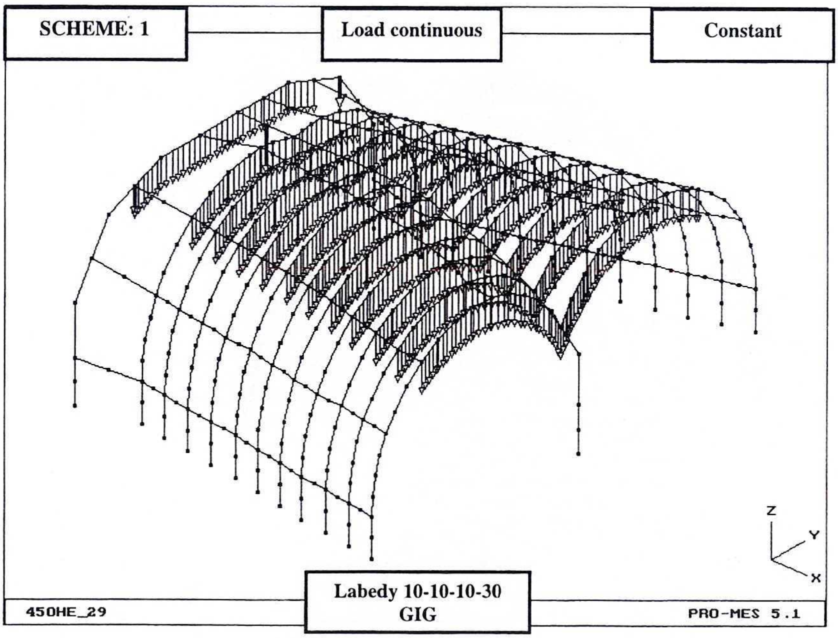

Fig. 4. Model of load having impact on junction support

Rys. 4. Model obciążenia działającego na obudowę odgałęzienia

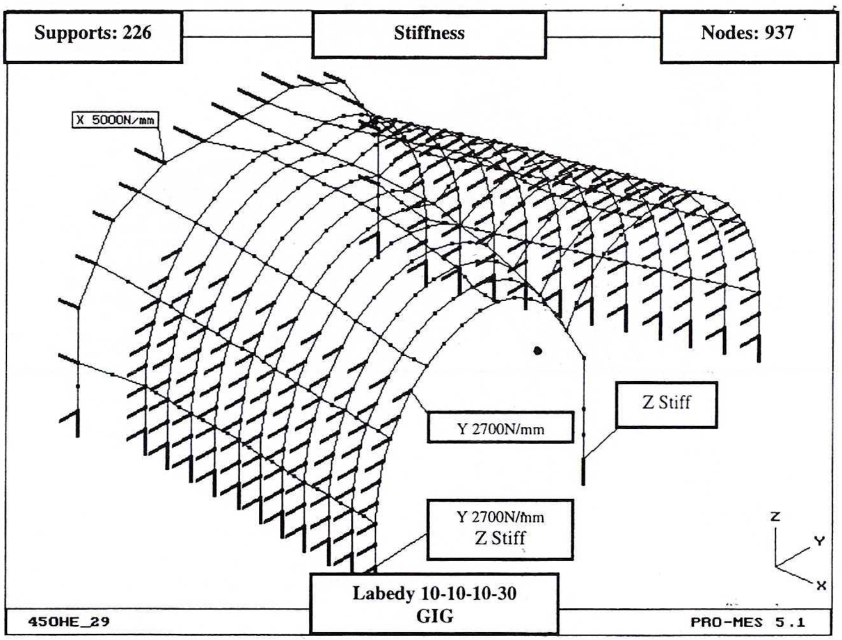

Fig. 5. Model of system bearing (boundary conditions)

Rys. 5. Model podparcia układu (warunki brzegowe)

• HEB 500	V36
• Welded 500 mm	V36
• HEB 550	V36

3.4. Results of calculation

The results are presented in graphical and numerical form. They consists of the following quantities characterising the response of modelled structure, at assumed boundary conditions, and under the influence of an imposed external load:

- dislocations of construction nodes along main axes of co-ordinate system;
- reactions at selected points of support-structure;
- internal forces occurring in structural elements:
  - transverse forces T_z, T in relation to main shape axes,
  - longitudinal forces N,
  - bending moments M_z, M, in relation to the main shape axes,
  - twisting moments M, in relation to the axis going through the cutting-off centre of the cross-sectional shape;
- tensile and compressive stresses in cross-sectional shape.

Calculation to determine the strength of a load-bearing support structure made from HEB 450 sections, with frames from V29 section, are shown in Fig. 6 and 7.

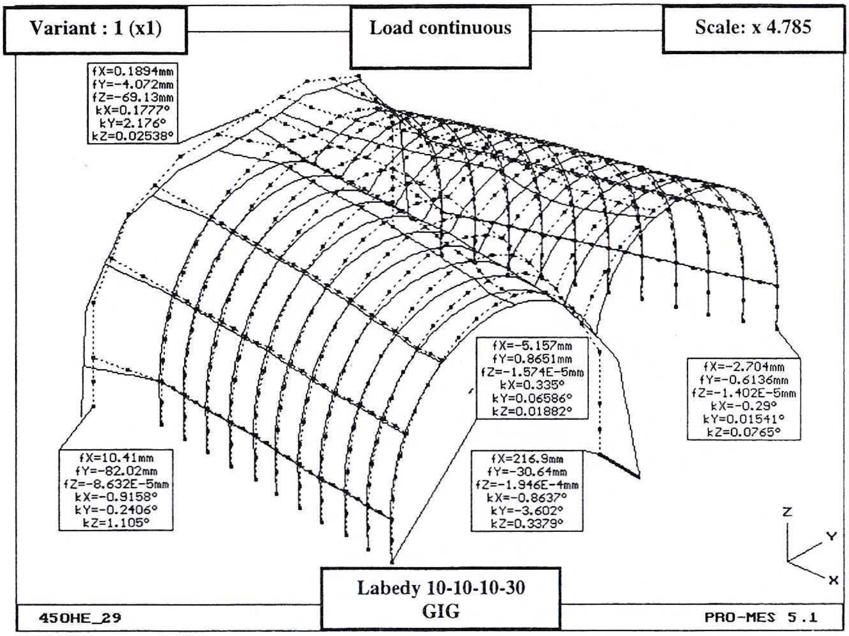

Fig. 6. Dislocations of construction nodes of junction support

Rys. 6. Przemieszczenia węzłów konstrukcji obudowy odgałęzienia

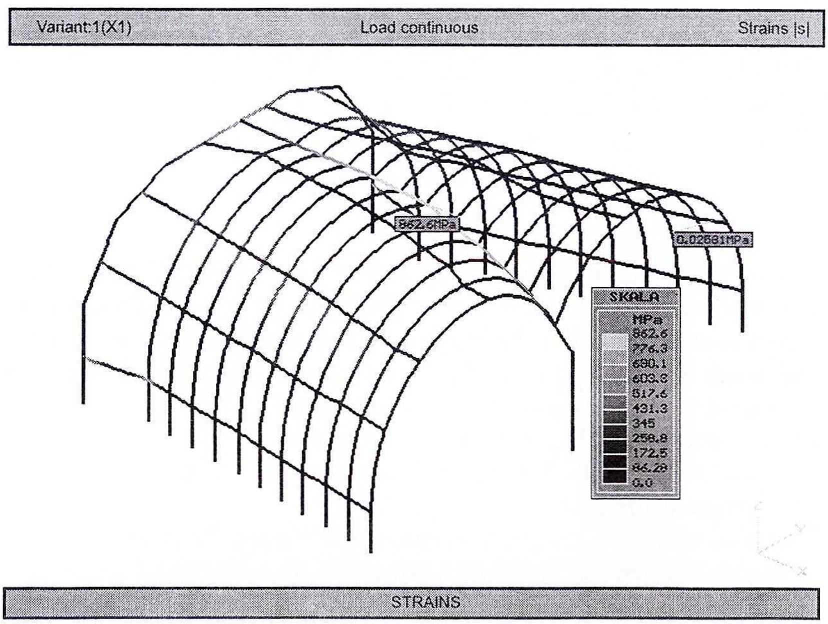

Fig. 7. Map of reduced stresses σ_z in support elements

Rys. 7. Mapa naprężeń zredukowanych σ_z w elementach obudowy

3.5. Analysis and interpretation of calculations

On the ground of the theoretical (modelled) results obtained the maximum value of stress δ_z, at off-centre compression, occurring in frames, is expressed by the following dependence:

$$\sigma_z = \left(\frac{N}{A} + \frac{M_g}{W_x}\right)_{\max} \quad [\text{MPa}] \tag{1}$$

where:

N — value of axis force [MNm],

M_z — value of bending moment [MNm].

The value of stress σ_n causing the occurrence of plastic joint has been defined as follows:

$$\sigma_n = \frac{R_e(m+n)}{\gamma_s} \, [\text{MPa}] \tag{2}$$

where:

n — reinforcement material index, expressed by the dependence

$n = \frac{R_m - R_e}{R_e}$ (dimensionless),

m — geometrical reinforcement index (dimensionless),

γ_s — material index according to PN-90/B-03200 equal to 1.15 (dimensionless).

On these dependences the following strength condition has been specified:

$$\sigma \leq \sigma_n \tag{3}$$

It may be inferred from the above dependences that for construction elements loaded according to assumed scheme, the value of a linearly distributed load q_{max} causing the occurrence of plastic joint as a result of steel reaching its uts (uts = ultimate tensile strength) amounts to

$$Q_{\max} = \frac{\sigma_n q_o}{\sigma_z} \, [\text{MN/m}] \tag{4}$$

After carrying out the calculations according to PN-90/B-03200 and taking into account the values of reaction sum occurring under the influence of external loads, the values of total loads which the support is able to bear, have been obtained.

The values of total maximum load for discussed cases are presented in Table 1.

A characteristic feature of the support-structure is that its load capacity is determined by the elements of load-bearing structural elements (portal and bracket), additionally keeping the frame in correct spatial alignment.

TABLE 1

Calculated values of total load of junction support-system

TABLICA 1

Obliczone wartości całkowitego obciążenia obudowy odgałęzień

Profile of elements of junction of load-carrying construction	Profile of frame elements	Total support load [MN]
HEB450	V29	3.2367
Welded 450 mm	V29	3.3795
Welded 450 mm	V36	3.3877
HEB500	V29	3.5010
HEB500	V36	3.5447
Welded 500 mm	V36	4.1028
HEB550	V36	5.2970

The frame's load capacity, connected with the type of section used (V29, V32 or V36) has a great influence on load uniformity, especially at values of those loads which produce boundary stresses in the frame structure.

Steel sprags applied to the construction of adjustable length and support stabilization index $w_n = 1.0$, fixed on the perimeter of the portal, as well as the passage supplementing frames set at intervals of 1.2 m, ensure additional protection from buckling and skewing.

4. Underground testing of junction support-structure

The junction was installed at Mine B within orzeskie layers in the vicinity of the protecting pillar for shafts I, II and III. The local geology consists of a mudstone and sandstone series of Carboniferous formation inter-bedded by uniform and irregular coal seams. There were no influences of coal seams at this stage. The site of the junction on the plan of development works, is drawn in Fig. 8.

The support structure was installed in an excavation secured by the frames of ŁP10 size and is adapted for transmit, primarily, loads from the over-lying rock strata in a vertical direction and, to a lesser degree, side pressures. Taking into account, however, the great depth of the location (1050 m), a more and versatile rock-mass influence on the support-structure might have been expected.

The construction was protected from excessive stress concentration leading to considerable element through the introduction of flexible connections in the support segments at the near-ceiling parts of the portal and bracket. This resulted in the

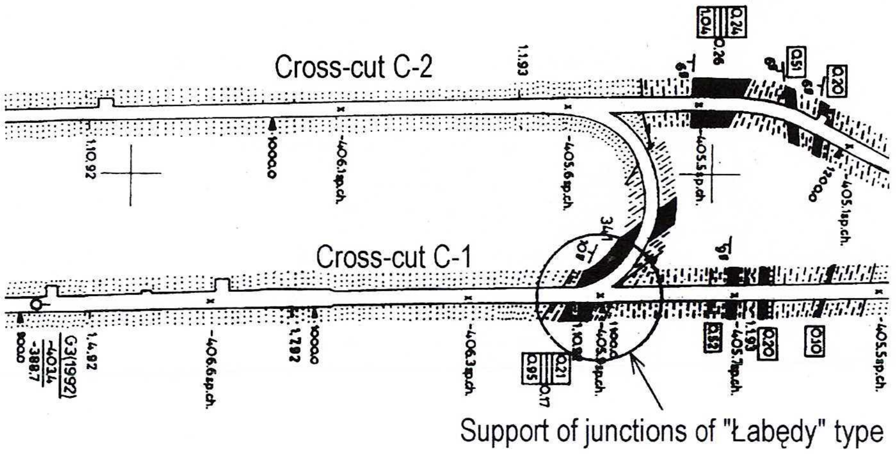

Fig. 8. Situation junction of main mine working in support of "Łabędy" type at B colliery

Rys. 8. Lokalizacja odgałęzienia przekopów w obudowie typu „Łabędy" w kopalni B

junction support movement being characterized by component element dislocation in the direction of the excavation.

The underground testing took place monthly and consisted of linear measurements of the width and height of the portal and bracket at characteristic points, the amount of subsidence in the load-distributing nodes of both frames and slides movements in the support frame connections. The measured of support-element dislocations of the mine B installation are enumerated in Fig. 1. Other structural components differed between one another; on the overall dimensions of the portal and brackets and on a number of passage and supplementing frames. The slide movements were small and ranged from 0 to 8 mm. No movements at the sites of connections of arches were noticed.

It seemed that due to horizontal component of force of rock mass pressure affecting the bracket, the portal might have tended to tilt from the vertical. This phenomenon was not observed , possibly because the friction forces between the bracket and roof rocks, in addition to that of the frames were bigger than this component. A greater flexibility of segments at the divergence pier than the near-wall segments should be considered advantageous as it influences, to a minor extent, a decrease in the cross-section of excavations.

The results of the six-month measurement programme demonstrated that after this period following the installation of the junction support-system a decompressed zone had been territorially stabilized. This was shown by the decrease in the numerical values of the measurements. It may also be inferred that the rate of load increase was greatest during the month immediately succeeding the placement of the support and that after 3–6 months the increase of deformations went asymptotically down to zero.

The statistical analysis of underground measurements was carried out with the aid of STATISTICA computer program. The modulus "non-linear estimation" was

TABLE 2

Selected regression function of tested junction parameters

TABLICA 2

Wybrane funkcje regresji badanych parametrów odgałęzień

Parameter	Regression equation	Index of correlation
Width of portal S	Y= –5.385065 ln(x) + 5438.156	0.988
Span of bracket L	Y= –7.766926 ln(x) + 7650.636	0.946
Height of flexibility H3	Y= –27.91027 ln(x) + 450.124	0.998

applied to the calculations which allows, among other things, the definition by the user of his own regression function. A series of functions were taken into consideration, of which changes of portal width, bracket span and changes of flexibility height were most truly reflected by logarithmic functions. The scope of these functions refers to parameter $x \geq 0.5$. In Table 2 regression function have been specified, while in Fig. 9–11 selected courses of a regression function with marked points from the measurements.

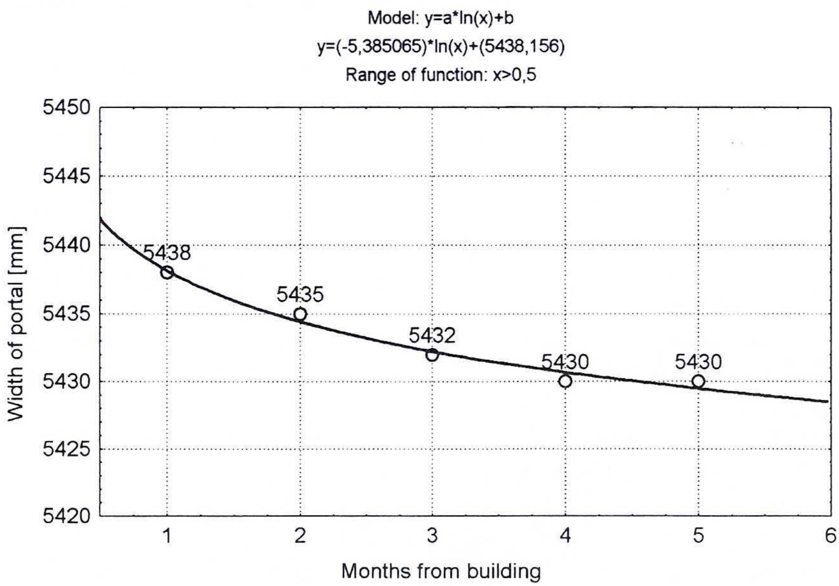

Fig. 9. Width of portal in the time function of support setting

Rys. 9. Szerokość portalu w funkcji czasu od zabudowy

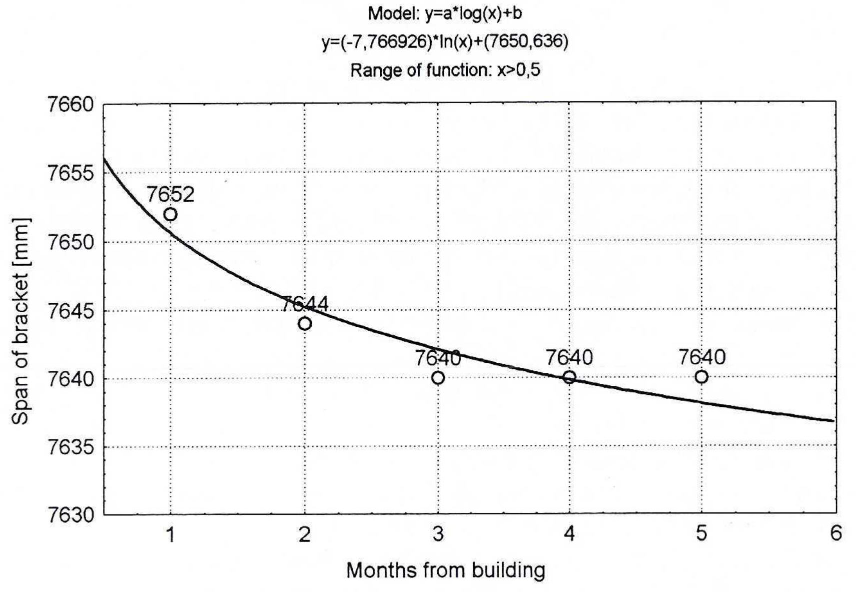

Fig. 10. Span of abutment in the time function of support setting

Rys. 10. Rozpiętość wspornika w funkcji czasu od zabudowy

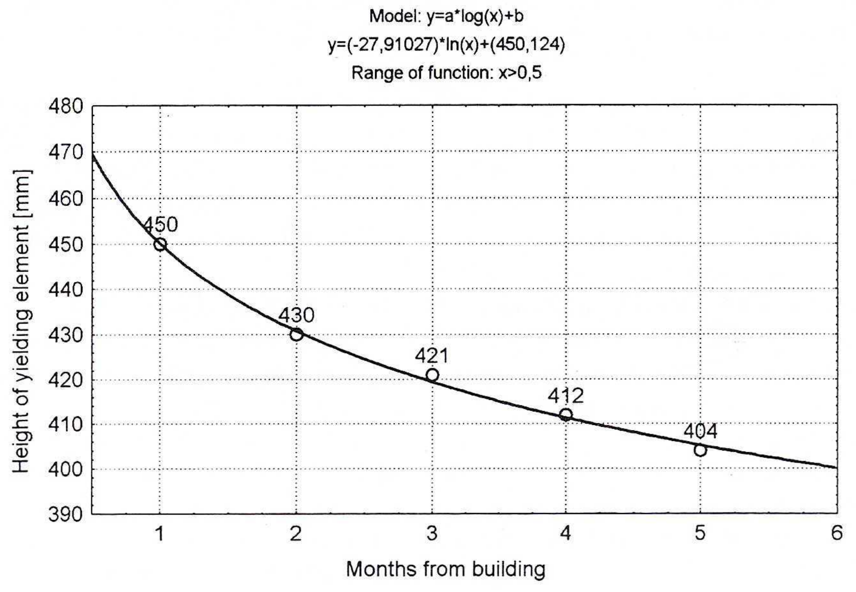

Fig. 11. Height of yielding element in the time function of support setting

Rys.11. Wysokość upodatnienia w funkcji czasu od zabudowy

5. Conclusions

1. The unique constructions of the "Łabędy" type support presented by the author effectively ensure the safety of the connection zones of dog heading even in difficult geological and mining conditions. These support-systems are characterised by their constant height along the whole length of excavation junction, and their fundamental load-carrying elements, in the form of portal and bracket, form compact skeletal structure with overall dimensions that ensure minimal intervals allowing means of transport to run through the excavations.

2. The series of support-systems for dog heading junctions variable through the change of the size of load-carrying beams, frame shape (V29, V32 or V36) and frame span makes the system a viable proposition in various geological and mining conditions, in relation to assumed rock mass load.

3. The analysis of calculations carried out for junction 10-10-10-R30-P, selected from the series of types and all sizes as well as element construction types of portal and bracket, allows the following statements to be made:

- The finished element method applied to strength calculations allowed optimal selection of the sizes of constructional component elements to be made, not allowing them to exceed their required strength for assumed external load configurations.
- The area of the highest stresses coinciding with the area of maximum bending moment in the load-bearing support construction occurs in the central part of the bracket. This applies both to a load-bearing support construction made from welded elements, or to one made from a deep-section, rolled, alloy-steel I beam.
- The positioning of the load-bearing construction (portal and bracket) in inter connecting excavations ensures a symmetrical bracket load and eliminates the possibility of a twisting moment being set up inside it. The characteristic feature of support-structure construction is the fact that its functionality is determined by portal and bracket load capacity, additionally ensuring the correct spatial alignment stabilization of the frames.
- Frame load capacity, associated with the section used connected (V29, V32 or V36), affects to a small degree, the load capacity of the whole construction but it greatly affects the uniformity of load distribution on the support, especially at loadings causing boundary stresses in the frame shape.

4. On the basis of the measurements and observations of the "Łabędy" type junction support-system work and the opinions of mining plants applying these supports, it may be stated that:

- The constructions analysed effectively protect excavations from deformations. Support load-bearing frames (portal and bracket) worked within the range of elastic deformations, and in the arched support frames, except for small slides in the connections, no excessive plastic deformations were found. It enables the right selection of junction support elements, that is the load capacity of the beams of the support frame and the sizes of arched frame shapes for existing geological and mining conditions and loads set up by the surrounding rock mass.

- As a result of rock mass influence, the reduction in the dimensions of both load-carrying frames (portal and bracket) in tested supports did not exceed 1.0%, whereas the extent of flexibility of the support structure was only 16%.
- An increase in external loads occurred in the 3 to 6 month period following the installation of the support structure. After this phase, structural deformations decreased and a state of equilibrium was reached between the rock-mass and the installation.
- Changes of geometry of junction load-bearing structure (portal and bracket) are most precisely depicted by logarithmic functions.
- In view of the widely varied range of geological and mining conditions in the Upper Silesia Coal Basin, which are crucially dependent on the interaction of support structures with the Carboniferous rock mass, it is considered necessary to examine each individual case prior to specifying the appropriate support structure. This examination must consider the geo-technical conditions present at proposed site in order to pre-determine the probable structural loadings.
- Underground testing and observations of installed heading-support systems in difficult geological and mining conditions proves that these structures are ideally suited for installation at heading junctions.

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