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Original article

Traffic infrastructure in mining areas (selected problems)

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ABSTRACT

The article discusses the issues of safety of operation of road and rail transport infrastructure objects in mining areas. In particular, the following issues were discussed: the general characteristics of the mining deformations of the rock mass and the terrain surface, the impact of continuous and discontinuous deformations on the safety of the operation of land transport infrastructure facilities and reinforcements of land transport infrastructure structures in mining areas.

Examples of land transport structure reinforcements situated within the reach of mining influences are given.

KEYWORDS

land transport structures, mining areas,
soil compacting and building embankments

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Introduction

Underground mining of the deposits upsets the rock mass balance and causes the rocks to move from the surroundings of the selected deposit towards the void created after the mining operations have been terminated [1, 2]. These phenomena are known to commonly cause rock mass quakes and changes in water conditions. Depending on the geological and mining conditions and the deposit mining system, destruction zones tend to form above the mined deposit (characterized by caving, cracking and deflection), which most often occur in various combinations [2]. In the vicinity of communication objects such as highways and expressways, in such cases deformations in the form of land depression, horizontal deformations, slopes and curvatures should be taken into account. This type of deformation can cause mining damage, which can result in the road being out of service. In the case of other engineering structures, including bridges, one should take into account that mining damage may have an impact on the occurrence of significant damage, among others: changes in height and situational position of supports, the creation of additional compressive, tensile, torsional stresses, tilting of supports in the direction of the axis longitudinal object, changing the length of the spans, or tilting the supports in a direction perpendicular to the longitudinal axis causing the support structure to twist [3].

1. General information on mining deformations of the terrain surface

The deflection of rocks in the shape of a basin moves through the rock mass towards the surface of the ground, acting on elements of the traffic infrastructure. The elements which shape the influence of underground mining on the ground surface within the borders of the road strip are presented schematically in Figure 1 according to [1, 4]. The influence of mining is most often considered in the aspect of emerging continuous, discontinuous deformations and quakes [1, 2].

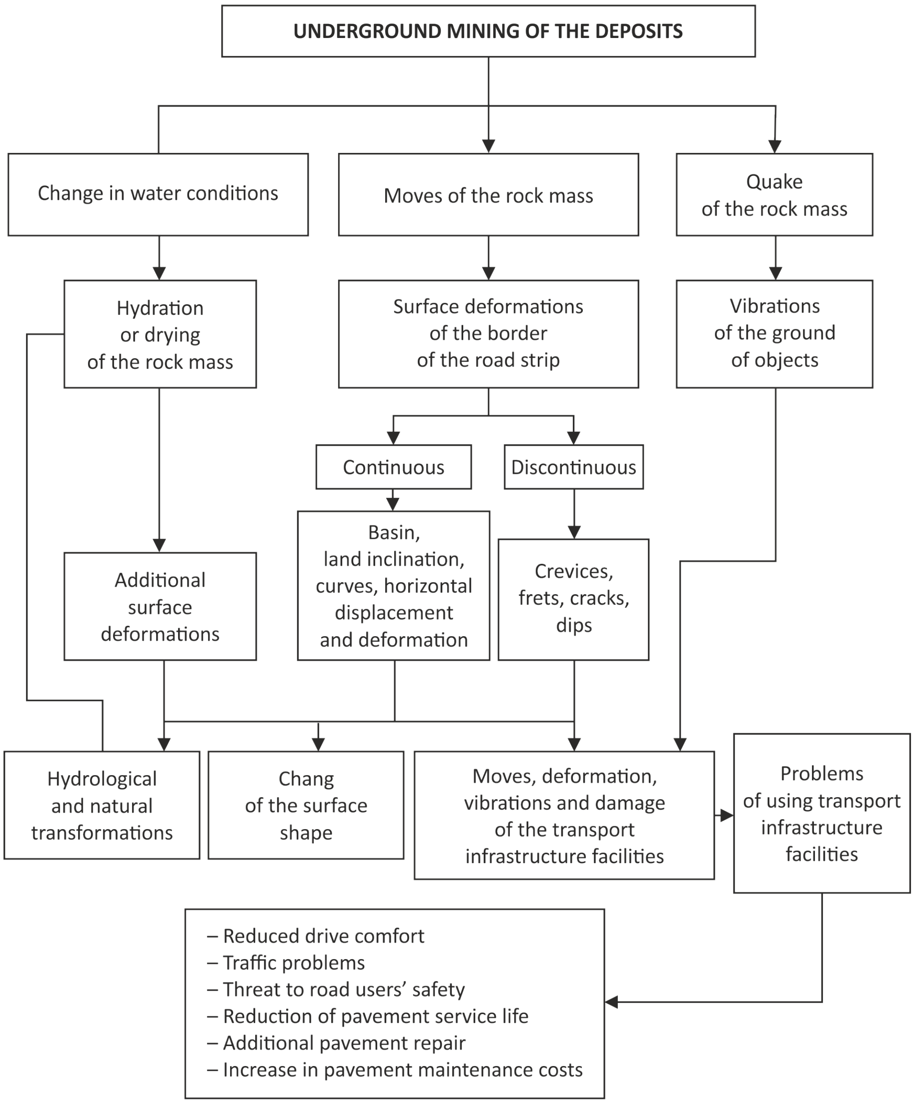

Fig. 1. Elements shaping the influence of underground mining on the ground surface within the limits of the road strip

Source: [1, 4].

Continuous deformations are characterised by a gentle lowering of the ground surface in the form of subsidence basins, the range of which goes far beyond the scope of operation. A continuous basin is described by five terrain deformation indicators [2] (Fig. 2): *w* – basin, *T* – land inclination, *ε* – horizontal deformation, *u* – horizontal displacement, *R* – curve radius. The following classification of mining areas was developed depending on the intensity of the impact of continuous deformation of the terrain, depending on the limit values of deformation indices (*T* – land inclination, *R* – curve radius, *ε* – horizontal deformation) [1, 2, 5-8]:

– category 0: $T \leq 0.5$ mm/m, $|R| \geq 40$ km, $|\varepsilon| \leq 0.3$ mm/m,

– category I: $0.5 < T \leq 2.5$ mm/m, $40 > |R| \geq 20$ km, $0.3 < |\varepsilon| \leq 1.5$ mm/m,

– category II: $2.5 < T \leq 5$ mm/m, $20 > |R| \geq 12$ km, $1.5 < |\varepsilon| \leq 3.0$ mm/m,

– category III: $5 < T \leq 10$ mm/m, $12 > |R| \geq 6$ km, $3 < |\varepsilon| \leq 6$ mm/m,

– category IV: $10 < T \leq 15$ mm/m, $6 > |R| \geq 4$ km, $6 < |\varepsilon| \leq 9$ mm/m,

– category V: $T > 15$ mm/m, $|R| < 4$ km, $|\varepsilon| > 9$ mm/m.

Category zero stands for minimum impact, whereas category V stands for maximum impact.

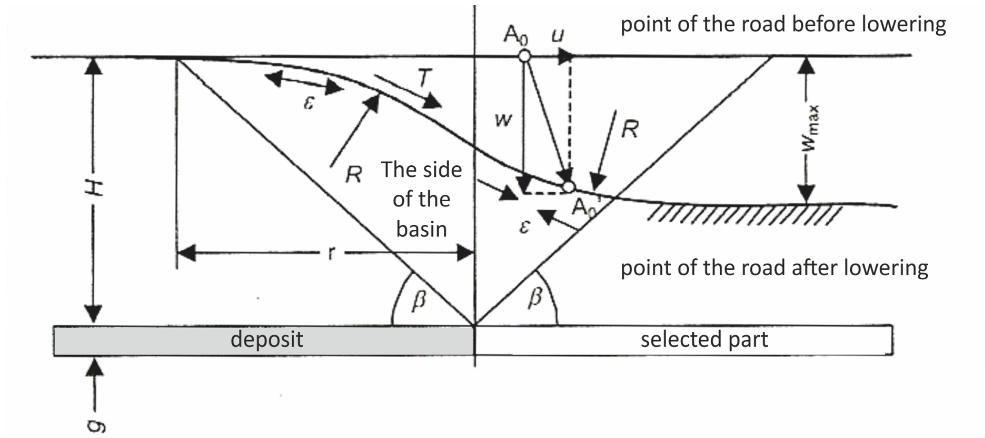

Fig. 2. A diagram of the determined basin (profile of vertical displacements concerning continuous deformations): *w* – lowering of the terrain, *T* – inclination of the terrain, *ε* – horizontal deformation, *u* – horizontal displacement, *R* – curve radius, *H* – depth of the mined deposit, *g* – thickness of the mined deposit, *β* – angle of the range of mining influences, *r* – radius of the range of mining influences
Source: [2, 6].

The appearance of a basin is manifested, among others, by deformations of the road gradient (i.e. changes in its longitudinal inclination), which in turn may imply a reduction in the efficiency of surface drainage. Marginal reservoirs forming in roadside ditches are an example of this negative phenomenon [1] (Fig. 3), where, after the formation of the depression basin, the culvert is not situated at the lowest point of the ditch bottom.

Mining deformations of the terrain may lead to a change in the lateral inclination of the roadway. In a particular case, there may be a reversal of transverse drops in the horizontal curve area, posing a threat to the safety of road users [1] (Fig. 4).

Fig. 3. Marginal reservoirs forming in roadside ditches in maximum land depression areas. The case where, after the formation of the depression basin, the culvert is not situated at the lowest point of the ditch bottom grade
Source: [1].

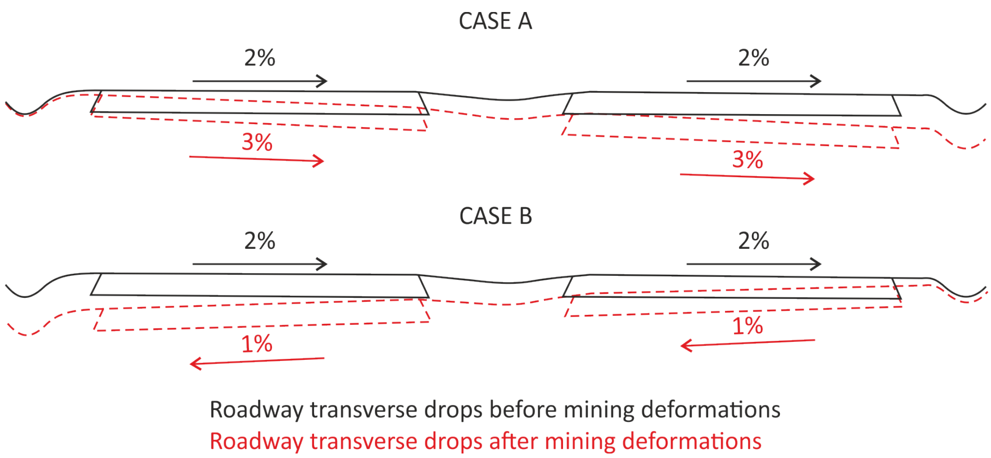

Fig. 4. Cross section through a dual carriageway on a horizontal curve: case *A* – increase of roadway transverse drops due to mining deformations, case *B* – reversal of transverse drops due to mining deformations
Source: [1].

Discontinuous deformations [1] (Fig. 5) interrupt the continuity of the terrain surface, including e.g. road surface [1] (Fig. 6). They take the form of crevices, fault ledges, cracks, cave-ins, sinkholes and occur mainly with shallow mining (up to the depth of ca. 80 m according to [1]) and at significant depths of longwall mining conducted at high speed or in multiple layers [2]. The probability of occurrence of discontinuous deformations depends mainly on the thickness of the overburden [9] and, according to E. Skrzynski [9], the highest probability of occurrence of cave-ins is expected when the overburden thickness is lower than 25 m. As the thickness of the overburden increases to a limit value of approx. 70 m (according to [9]), the probability of cave-ins decreases, but the funnels formed have bigger and bigger diameters, however, usually not exceeding 10 m. Figure 7 presents a discontinuous deformation in a railway road, resulting from the occurrence of mining damage [9]. This is characterized as a cave-in type of settlement.

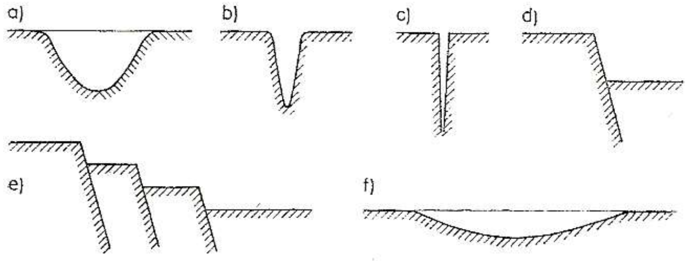

Fig. 5. Forms of discontinuous deformations:
a – funnel, b – trench, c – crevasse, d – fault, e – ledges, f – local basin
Source: [1, 7].

Fig. 6. View of damage to the road surface in the area
of discontinuous linear terrain deformations
Source: [10].

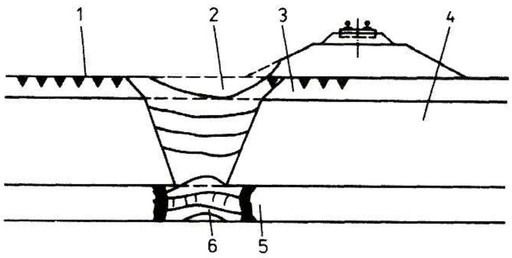

Fig. 7. Discontinuous deformation of the terrain around a railway line above mine workings.
Land settlement forming a cave-in, resulting from the occurrence of mining damage:
1 – surface area, 2 – cave-in, 3 – near-surface layer, 4 – indigenous rock, 5 – deposit,
6 – part of the rock mass filled with a caved cover
Source: [9].

The authors of articles [1, 10] treat discontinuous deformations as a special case of mining impact on the surface and introduce the concept of linear discontinuous surface deformations. The origins of these deformations is related to the impact of multiple, deep mining. This type of damage has a very negative impact on the evenness of road or rail surface and on the safety of the users. Therefore, they require immediate intervention, connected with limiting the speed of driving and the possibly fast shaping of the surface or, in extreme cases, it is necessary to close a section of road or railway line until the repair works are completed.

2. Selected problems of cooperation of structures with the mining base

The most important factors which determine the cooperation of the structure with the mining base are [2]: uneven vertical depressions creating the terrain curvature and uneven horizontal displacements of soil particles, which are the cause of its horizontal deformations. Horizontal deformations of the soil may have different characteristics, depending on the location of the mining front or the mining edge in relation to the surface point. These deformations are generators of soil relaxation (crawling) or compaction (creep).

Designing foundations of structures according to limit states requires the application of partial safety factors, addressed to loads and geotechnical parameters. In the case of mining areas, the procedure is similar – the values of characteristic deformation indices (the "*k*" index) and computational indices (the "*d*" index) are taken into account. Two limit states are analyzed: load-bearing capacity and use. At the ultimate limit state, the design values are expressed by the following formulas [2, 6]:

$$\varepsilon_d = k_{wp}\,\gamma_{f\varepsilon}\,\varepsilon_k, \tag{1}$$

$$T_d = \gamma_{fT}\,T_k, \tag{2}$$

$$K_d = (R_d)^{-1} = k_{wp}\,\gamma_{fK}\,K_k, \tag{3}$$

where:

R_d – terrain curvature [m^{-1}],

ε_d, T_d, K_d – design values of deformation indices,

ε_k, T_k, K_k – characteristic values of deformation indices,

$\gamma_{f\varepsilon}$, γ_{fT}, γ_{fK} – partial safety factors,

k_{wp} – the coefficient of working conditions.

The values of partial safety factors are adopted as follows [2, 6]: for horizontal terrain deformations $\varepsilon \rightarrow \gamma_{f\varepsilon} = 1.10$; for an inclination of terrain $T \rightarrow \gamma_{fT} = 1.50$ in high structures with a base dimension of < 15.0 m and $\gamma_{fT} = 1.20$ in other structures; for curvature of terrain $K \rightarrow \gamma_{fK} = 1.70$.

The values of the k_{wp} coefficient should be adopted depending on the ratio of the length of the object *L* to the radius of the range of mining influences *r*. This coefficient takes into account the change of predicted values of horizontal deformations and terrain curvature along the length of the structure [2, 6]. If the mining deformation forecast does not allow to determine changes in the values of parameters ε and r (in practice with $L/r \leq 0.3$), it is possible to assume $k_{wp} = 1.0$ according to [2, 6].

At the use limit state, the characteristic values are the predicted deformation values [2, 11].

The history of research carried out in natural (field) and laboratory conditions on the influence of mining on the change of physical and mechanical properties of soil dates back to the 1960s [2]. The interpretation of the results of many cohesive and non-cohesive soil tests became the basis for the development of empirical relationships determining the reduction of the values of strength parameters of the soil subjected to the process of loosening. Studies have pointed to a reduction, mainly in soil cohesion [2, 12-14]. According to [13], the relationship defining the change (decrease) of soil cohesion occurring in the process of its loosening can be formulated as follows:

$$c_R = c_0\,[1 - \varepsilon_R\,(\alpha_R + \varepsilon_R)^{-1}] + c_{kr}\,\varepsilon_R\,(\alpha_R + \varepsilon_R)^{-1} \tag{4}$$

where:

- ε_R – horizontal relaxing deformation,
- c_R – reduced soil cohesion (with $\varepsilon_R > 0$),
- c_0 – standard soil cohesion (with $\varepsilon_R = 0$),
- c_{kr} – critical cohesion at stabilization of soil strength drop ($\varepsilon_R \geq \varepsilon_{kr}$)
- α_R – coefficient describing the reduction of cohesion; $\alpha_R = 0.5 \div 4.0$; $\alpha_R < 1.5$ is assumed for the spreading of weak soils.

E. Stilger-Szydlo [2] also gives other proposals for the description of the reduction of cohesion Δc and the angle of internal friction $\Delta\tan\varphi$:

$$\Delta c = 11\,\varepsilon_R^2 - 50\,\varepsilon_R + 19 \tag{5}$$

$$\Delta c = 0{,}4\,\varepsilon_z^2 + 7\,\varepsilon_z + 4 \tag{6}$$

$$\Delta\tan\varphi = 0.6 - 0.7\,\Delta c - 0.005\,\Delta c^2 \tag{7}$$

where:

- c – ground cohesion,
- φ – internal ground friction angle,
- ε_R – horizontal relaxation ($0 < \varepsilon_R \leq 3$ mm/m),
- ε_z – horizontal compaction ($0 < \varepsilon_z \leq 9$ mm/m),

The following formulas describe the shear strength of cohesive soil as a function of the predicted amount of relaxation according to [2, 15]:

$$\tau_R = \sigma\,\tan\varphi + c_R \tag{8}$$

$$\tau_{R,min} = \sigma\,\tan\varphi + c_w \tag{9}$$

$$c_R = A + B\,e^{\lambda} \tag{10}$$

where:

τ_R – soil shear strength, corresponding to the relaxation expressed by an inequality $\varepsilon_R \leq \varepsilon_{R(kr)}$, which implies that the horizontal loosening strain ε_R can reach a critical value of $\varepsilon_{R(kr)}$, as illustrated in Figure 8 [2, 16],

σ – normal stress,

c_R – reduced soil cohesion corresponding to relaxation $\varepsilon_R \leq \varepsilon_{R(kr)}$,

$\tau_{R,min}$ – minimum value of shear strength of cohesive soil,

c_w – hydrocolloidal component of soil cohesion,

$\lambda = (-N\,\varepsilon_R)$ – experimental parameter,

A, B, N – oefficients determined experimentally, depending on soil humidity.

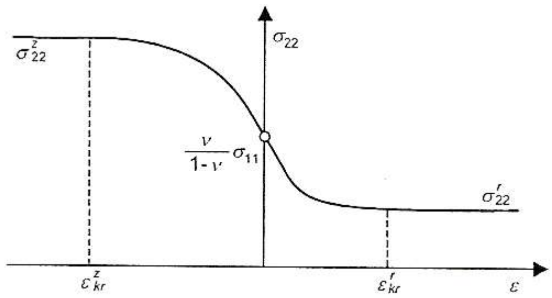

Fig. 8. Dependence of horizontal stress σ_{22} on the horizontal deformation of the substrate ε
Source: [2, 16].

The following can be stated in a conclusion to the study [2]:

– significant influence of horizontal deformations on the change of strength parameters of cohesive soils,
– in cohesive soils, there is a large decrease in soil cohesion, where the soil is loosened by mining operations; e.g. in silty clays, in the case of horizontal substratum deformations of ε_R = 3÷10 mm/m, a decrease in cohesion of 20÷45% is observed,
– shear strength of loosened non-cohesive soils $\varepsilon_R \leq \varepsilon_{R(kr)}$ remains practcally unchanged.

On the basis of the results of the experimental studies mentioned above, the relationships showing the distribution of horizontal stress in the near-surface layer of soil subjected to horizontal tensile and compressive deformations, assuming a flat state of deformation, have been developed (Fig. 8). At rest, the stress distribution in the ground is described by the following relation [2]:

$$\sigma^0_{22} = K_0\,\gamma\,h = \nu\,(1-\nu)^{-1}\,\gamma\,h \qquad (11)$$

where:

K_0 – resting pressure coefficient,

ν – Poisson's coefficient,

γ – volumetric weight of the soil,

h – thickness of the overburden.

In the case of spreading ε^r, to which the ground layer is subjected (with the angle of internal friction φ, cohesion c and volumetric weight γ), variability of the pressure coefficient K_r (when the soil is relaxed) is expressed by the following relation [2]:

$$K_r = K_a + (K_0 - K_a)\,[1 - \varepsilon^r\,(\varepsilon^r_{kr})^{-1}]^n \tag{12}$$

where:

$K_a = \tan^2(0.25\,\pi - 0.5\,\varphi)$ – active pressure coefficient,

$\varepsilon^r_{kr} = -(1 - \nu)\,(2\,G)^{-1}\,(\sigma^r_{22} - \sigma^0_{22})$ – critical deformation when the soil is relaxed, dependent on vertical load expressed by vertical stress σ_{11} (ε^r_{kr} [mm/m] = 2.4 σ_{11}),

n – experimental factor; $n = 3.8$ [17] should be adopted for non-cohesive soils,

G – transverse modulus of elasticity,

σ^r_{22} – critical horizontal stress when relaxing the soil (in active state).

The stress value σ^r_{22} is calculated according to the following formula:

$$\sigma^r_{22} = \gamma\,h\,K_a - 2\,c\,(K_a)^{0.5} \tag{13}$$

In the soil layer subjected to compressing deformations ε^z (compaction), the variability of the lateral pressure coefficient K_z (when the soil is compacted) is calculated using the following formula [2]:

$$K_z = K_p + (K_p - K_0)\,[1 - \varepsilon^z\,(\varepsilon^z_{kr})^{-1}]^m \tag{14}$$

where:

$K_p = \tan^2(0.25\,\pi + 0.5\,\varphi)$ – passive pressure coefficient,

m – experimental factor, ranging from 2.9 (for fine sand) to 3.4 (for coarse sand),

$\varepsilon^z_{kr} = -(1 - \nu)\,(2\,G)^{-1}\,(\sigma^z_{22} - \sigma^0_{22})$ – critical deformation at soil compaction (in non-cohesive soils ε^z_{kr} = 31 [mm/m]),

σ^z_{22} – critical horizontal stress during soil compaction (passive).

The σ^z_{22} stress value is expressed by the following formula:

$$\sigma^z_{22} = \gamma\,h\,K_p + 2\,c\,(K_p)^{0.5} \tag{15}$$

Horizontal stress σ_{22} is related to the vertical stress σ_{11} by means of the lateral strut ratio K as follows [18]:

$$\sigma_{22} = K\,\sigma_{11} \tag{16}$$

3. Assessment of the stability of traffic embankments in the mining impact zone

The loss of embankment stability is one of the forms of exceeding the ultimate limit state. Load-bearing capacity, on the other hand, is the value of the limit load determined on the basis of the design characteristics of the material and the parameters of the substrate. In the process of numerous studies and observations, it was shown [2] that the stability of embankments is primarily determined by the influence of horizontal deformations of the mining substrate ε_R, connected with the convex part of the mining basin's bank. These deformations lead to the loosening of the soil structure, the change in the stress state and, consequently, to an unfavourable change in the strength characteristics of the soil medium. The above mentioned loosening of the soil is the cause of uneven subsidence of embankment, and the formation of landslides. Analyzing the cooperation of the earth structure with the mining ground base, the limit value of horizontal relaxing strain ε^{r}_{kr} is important, since, when exceeded, the state of limit equilibrium occurs [2, 16, 19] (Fig. 8).

The stability of a ground structure situated on a mining ground can be assessed by comparing the values of the limit deformation of the embankment soil (ε_n) with the limit values of ground deformation (ε_p), with the projected mining deformation ε_R. Block procedures are among the most prominent methods of assessing the stability of earth structures. The representative A. Kezdi's method assumes the displacement of the escarpment block along the ceiling of the weaker layer [2, 20]. The calculation diagram of the damage to the embankment slope concerning this method is shown in Figure 9 [2]. This diagram can be used for limit deformations defined by the following inequality: $\varepsilon_n > \varepsilon_p$. The embankment stability factor F is calculated as the quotient of the holding force S_u and sliding force S_z [2, 20]:

$$F = S_u\,(S_z)^{-1} = (T + E_{p2})\,(E_{a1} + E_{a2})^{-1} \tag{17}$$

where:

- T – shear resistance at the level of the bottom of the weak layer (a component of the holding force),
- E_{p2} – weak layer resistance (a component of the holding force),
- E_{a1} – active pressure in the embankment (a sliding force component),
- E_{a2} – active pressure in the weak layer (a slip force component).

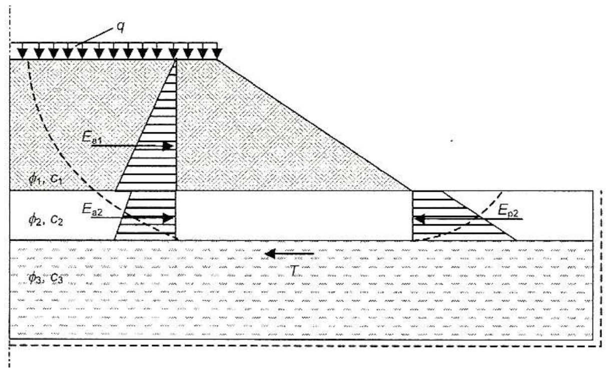

Fig. 9. Calculation diagram for the assessment of embankment stability based on weak bearing soil according to A. Kezdi's method: E_{a1} – active pressure in the embankment, E_{a2} – active pressure in the weak layer, E_{p2} – weak layer resistance, T – shear resistance at the level of the weak layer floor, φ_1, c_1 – strength features of the embankment; φ_2, c_2 – strength features of the weak substrate layer
Source: [2].

4. Maintenance of traffic facilities located in mining damage areas

4.1. General remarks

In areas affected by underground mining, where deformations, loss of load-bearing capacity and stability of the subsoil of traffic structures occur, engineering practice offers at least two effective solutions:

– shifting the route of an endangered section of a road or railway line beyond the cave-in area,
– crossing the section with cave-ins with a viaduct or a flyover.

However, these solutions cannot always be implemented for such reasons as expropriation restrictions (in the case of a route shift) or anticipated threats to the stability of engineering structures' supports (when crossing a mining damage area to the viaduct or flyover is planned). In this situation, compacting of the mining base may be undertaken. Some examples of such solutions are presented below.

4.2. A shock absorbing layer integrated between the substrate and the track surface

The solution shown in Figure 10 concerns the railway track located in a mining area, where deformations takes the form of surface discontinuities (sinkhole funnels) [21]. In this case, the maintenance work was reduced to [21]:

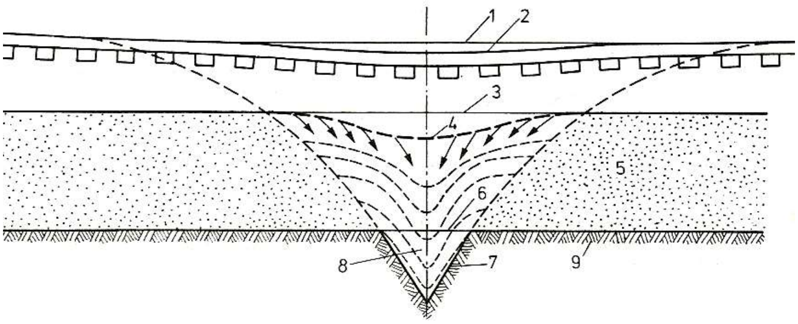

Fig. 10. A fragment of the longitudinal section of a railway track with a shock absorbing layer, installed between the track ballast and the subsoil: 1 – rail head grade before deformation of the substrate, 2 – position of the rail head after deformation of the substrate, 3 – position of the ceiling cushioning before substrate deformation, 4 – position of the ceiling of the cushioning layer, after deformation of the substrate, 5 – shock absorbing layer, 6 – position of the cushioning layer 's bottom before substrate deformation, 7 – position of the base of the cushioning layer after deformation of the substrate, 8 – cave-in, 9 – ground surface exposed to mining damage
Source: [21].

– installation of a shock absorbing layer between the ground and the track surface, which changes the discontinuous form of deformation occurring in the part of the bottom layer in contact with the ground, into a continuous form of a local basin,
– reinforcement of the track surface with a load-bearing structure made of rail bundles or I-beams of equal strength characteristics along the entire length of

the protected track section (Fig. 11 according to [21]). The number of rails forming the bundle or the dimensions of the I-beam shall be sufficient to achieve the required cross-sectional strength index of the track grate (the grate is formed by rails attached to the sleepers). Reinforcing the surface with a load-bearing structure does not ensure the removal of the mining origin of deformations in the ground surface of the track, but only limits the possible effects of their occurrence during train passage.

It should be stressed that the basic measure to ensure traffic safety in areas with random discontinuous deformations not signalled by visible signs is to limit the speed of trains. The prevalent rule of the Polish Railways is to limit the speed to 15 km/h on railway lines led through cave-in areas [21].

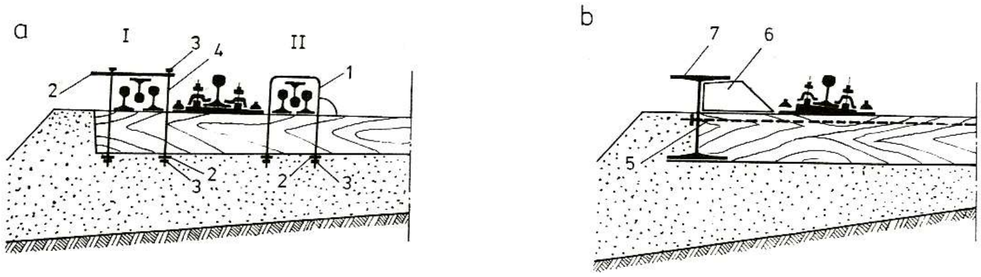

Fig. 11. A fragment of cross section through the railway track protected against discontinuous deformations by means of a load-bearing structure: a – load-bearing structure made of from two bundles of rails on both sides covering the rail course of the track, b – relieving structure made of an I-beam consisting of an I-beam which encloses a track gauge on one side and the front of the sleeper. Markings: 1 – collar, 2 – joint bar protruding from rail connectors, 3 – nut, 4 – steel rod, 5 – steel bracing, 6 – stiffening wedge, 7 – I-beam
Source: [21].

4.3. A railway embankment reinforced at base level with a geosynthetic "mattress"

According to [22], an effective way of limiting destructive changes at the base of a railway track located in an area affected by underground mining is presented. It consists in an earlier strengthening of the ground substrate using a geosynthetic "mattress". The effectiveness of this solution is conditioned by the proper selection of polymers, with particular emphasis on their rheological properties.

Applying the Z._Soil [20] program basing on the finite element method, a numerical model of an earth structure located in a mining area was developed. A double-track railway line was located on top of an embankment [22] (Fig. 12). The research was comparative: the model model consisted in an embankment without a geosynthetic mattress. In turn, the test models consisted in embankments with a single mattress in the substrate, a double mattress (at the base of the embankment and at the tip of the embankment, right under the surface) and a triple mattress. Geotechnical and material parameters were adopted for individual layers of the model:

– mining substrate: Coulomb-Mohr material model, deformation modulus E = 30 MPa, cohesion c = 33 kPa, internal friction angle φ = 18°,

– railway embankment: Coulomb-Mohr material model, E = 60 MPa, c = 20 kPa, φ = 30°,
– protective layer under the track ballast: Coulomb-Mohr material model, E = 200 MPa, c = 3 kPa, φ = 42°,
– track aggregate ballast: elastic material model, E = 2800 MPa, c = 0 kPa, φ = 45°,
– mattress coating geogrid: material model as anisotropic membrane, tensile strength R_r = 40 kN/m for the PP geogrid, R_r = 400 kN/m for the PVA geogrid, R_r = 800 kN/m for the Aramid geogrid,
– aggregate filling of the reinforcing "mattress": Coulomb-Mohr material model E = 120 MPa, c = 80 kPa, φ = 38°.

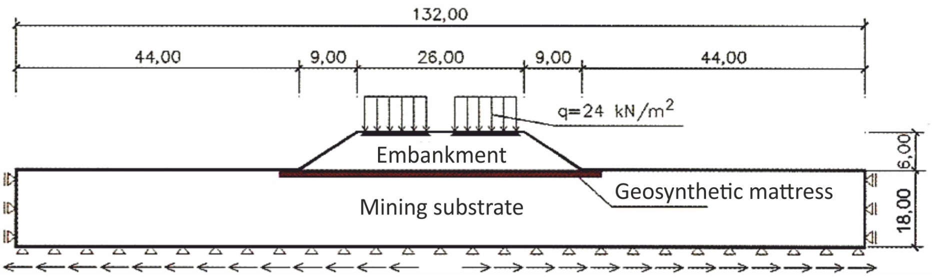

Fig. 12. Calculation diagram for a railway embankment located on a mining substrate, with reinforcement in the form of a geosynthetic mattress, installed at the embankment base level
Source: [22].

Geosynthetic products are characterized by different physical and mechanical properties, so the relevant guidelines require the verification of two conditions: design tensile strength F_d and permissible deformations.

The strength parameter used in the modeling of geogrids was (according to the European guidelines [23]) the design long-term tensile strength F_d calculated from the formula [22]:

$$F_d = F_k \cdot (A_1 \cdot A_2 \cdot A_3 \cdot A_4 \cdot \ldots \cdot A_n \cdot \gamma)^{-1} \text{ [kN/m]} \tag{18}$$

where:

F_k – short-term value of tensile strength [kN/m],
$A_1, \ldots, A_n$ – material factors,
A_1 – coefficient taking into account material creep,
A_2 – coefficient taking into account mechanical damage of material during installation and transport of geosynthetics,
A_3 – coefficient concerning the method of joining geosynthetic layers,
A_4 – coefficient taking into account environmental impact,
A_n – coefficients taking into account other, additional impacts, e.g. shocks caused by mining activity,
γ – material safety factor (γ = 1.75).

Values of material coefficients A_1, A_2, A_3, A_4 for geogrids were adopted in accordance with the provisions of the guidelines [23], depending on the polymer type.

The results of numerical simulations lead to the following observations [22]:

- in the model without a reinforcing mattress installed at the base of the embankment, the development of plasticization zones covers the entire embankment body, even when the horizontal loosening deformations are ε = 1.5 mm/m,
- where the base of the embankment is reinforced with a stone mattress in a polypropylene PP geogrid casing (with significant elongation properties), the plastic zones for ε = 1.5 mm/m cover the upper layers of the embankment, which can lead to damage to the paving; reinforcement with such geogrids is therefore inappropriate,
- switching geogrid raw material to PVA significantly affected the effectiveness of soil strengthening under the embankment; after the introduction of a double mattress consisting of a PVA geogrid, the embankment with railway paving has displayed the features of a resilient material even for cross-creepage of the mining area of ε = 1.5÷4.5 mm/m. When a triple mattress is used, the positive effects of the reinforcement are even more prominent.

A confirmation of the purpose of numerical simulations was obtained on the basis of their verification in field conditions [22].

4.4. Reinforcement of the mining base and embankment slopes in a mining damage area

The structure in question is a section of the A-4 Wroclaw–Krakow highway between the Sosnica and Wirek junctions, which are located in mining areas. In relation to the existing terrain surface, the highway driveway grade is routed on embankments and in excavations. In the process of embankment construction, near-surface weak-bearing soils were replaced and the mining substrate under the embankment was strengthened by using a "mattress" made of stone material surrounded by a layer of geogrids [2, 24] (Fig. 13).

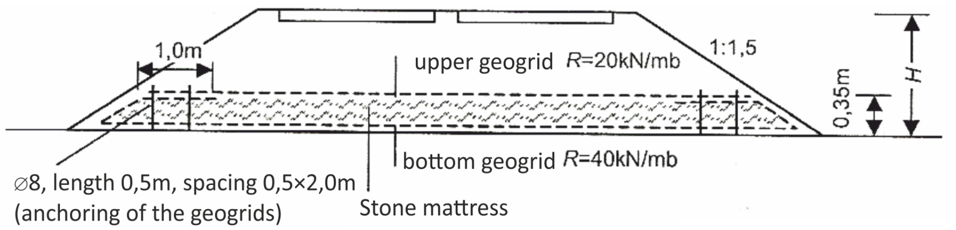

Fig. 13. The embankment of a fragment of the A-4 highway with reinforcement at the level of the mining substrate: H – height of embankment, 0.35 m – height of stone mattress in a geogrid coating with a tensile strength of R

Source: [2, 24].

Based on the Fellenius evaluation of the stability of embankment slopes, the following conclusions were reached [2]:

– increasing the stability of the embankment can be achieved by installing a stone “mattress” in the substrate,
– slopes of embankments of $H \leq 5$ m in height, placed on a foundation reinforced with a stone “mattress”, are stable and do not require reinforcement,
– in the case of embankments of $H > 5.0$ m in height (placed on a substrate reinforced with a stone “mattress”), in view of the calculated value of the stability factor $F_{min} < 1.0$, it is necessary to additionally reinforce the embankments using geogrids (Fig. 14 according to [2, 24]).

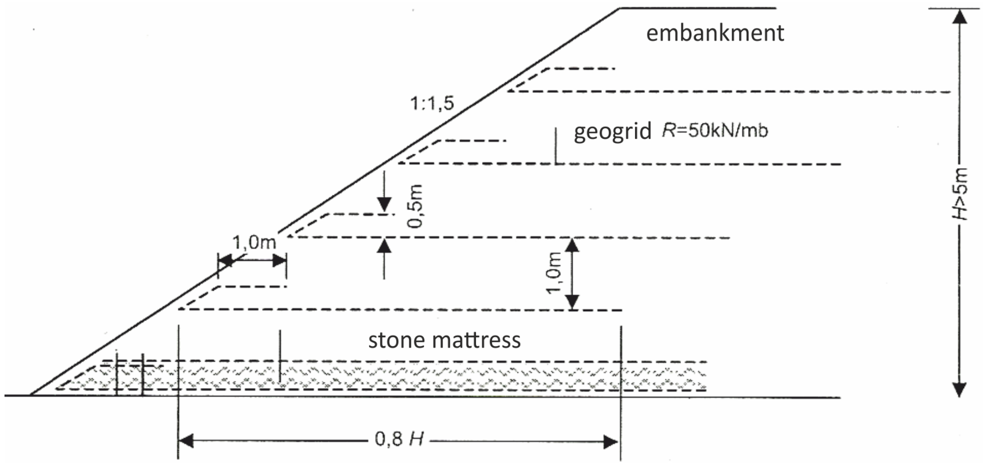

Fig. 14. An embankment of $H > 5$ m in height, situated in a mining damage area, with horizontal reinforcement of the substrate (stone mattress) and additional geogrids in the escarpment area
Source: [2, 24].

4.5. Notes on the technology of manufacturing road embankments in mining areas

Two types of reinforcement layer structures are most often used to strengthen earth structures dedicated to vehicle traffic in mining areas. These are so-called “sandwich” or “mattress” cushions [25, 2]: The former is a structure made of aggregate horizontally stratified with geogrids. In the “mattress” structure, the stone material is wrapped in a geogrid from below and from the top in such a way that the aggregate grains wedge each other, thus limiting the horizontal movement of the stone material under external, vertical load. The “mattress” can therefore act as a three-dimensional independent unit (layer), pre-stressed with a geosynthetic coating, bearing tensile stresses.

Figure 15 shows, according to [25, 2], two structures of a cushion reinforcing the ground under an embankment in a mining area.

In designing building reinforcement systems in mining damage areas, it should be stressed [2] that placing a cushion under the base of the embankment increases the load-bearing capacity of the embankment and causes it to settle evenly. However, it does not reduce the displacement of the embankment body resulting from mining influences. Therefore, a concept of soil reinforcement and embankment body located in the mining areas has been presented in the work [26] (Fig. 16). According to this concept, a model earth structure is as follows (individual layers are discussed from the top, to the embankment base):

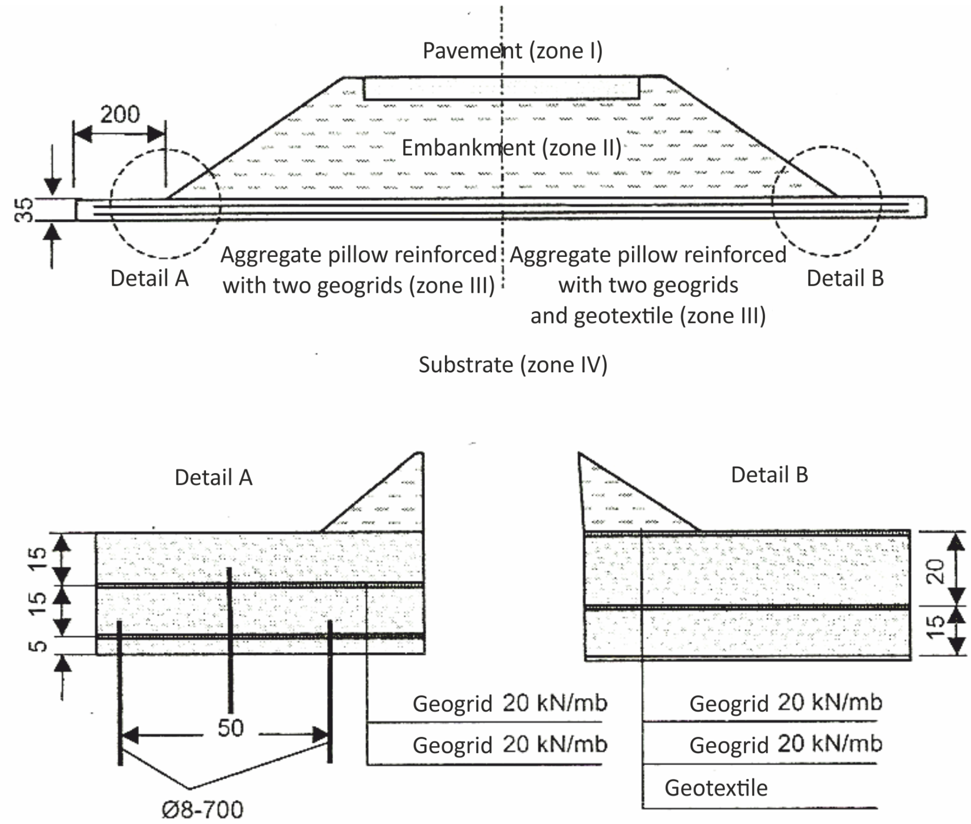

Fig. 15. Structures of a cushion reinforcing the substrate under an embankment in a mining area: detail A – aggregate pillow reinforced with two geogrids, detail B – aggregate cushion reinforced with two geogrids, placed on a geotextile
Source: [25, 2].

- roadway structure consisting of a surface and substructure under the paving,
- spatial geogrid (thickness 0.40 m) filled with mineral aggregate, the purpose of which is to increase the bearing capacity of the road base and assist in the transfer of stretching deformations created in the phase of mining impact,
- the body of the embankment, made of aggregate obtained according to the recommendations in [27] from burnt shale found in the mine dumps,
- reinforcement in the form of geogrids arranged in at least two layers in the lower part of the embankment body; the purpose of reinforcement is to reduce the displacements of the embankment on the basis of bearing additional tensile stresses resulting from the creepage of the ground (the cross-section of geogrid reinforcement depends on the value of projected mining deformations),
- 0.30÷0.50 m thick fine sand backfill placed under the embankment, allowing for partial reduction of horizontal displacements of mining origin,
- spatial geogrid (max. 0.60 m in thickness) filled with mineral aggregate,

– a cushion made of mineral aggregate, serving as a substrate for the geogrid, placed on a double layer of geotextile; the geotextile serves a reinforcing and separating function; the thickness of the cushion depends mainly on the load capacity of the reinforced substrate,
– the native substrate.

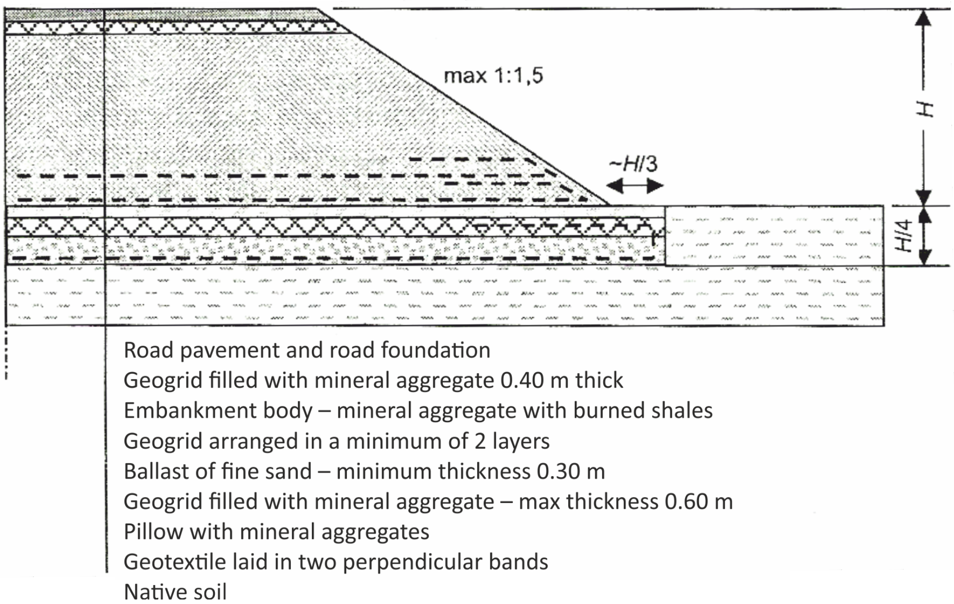

Fig. 16. The system of soil and embankment body reinforcement located in mining areas. The details of the solution are discussed in the text
Source: [25, 2].

Figure 17 presents a view of a spatial geogrid without aggregate filling [28].

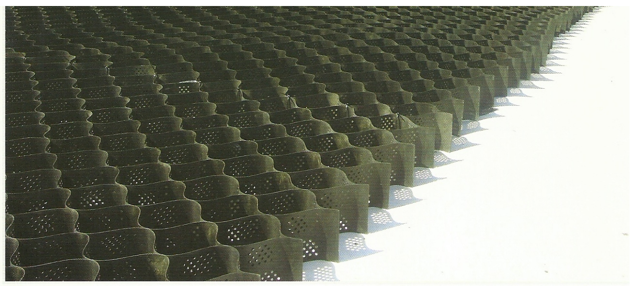

Fig. 17. View of the spatial geogrid without aggregate filling
Source: [28].

Summary

The processes of designing, constructing and maintaining land transport infrastructure facilities in mining areas require access to information on the potential effects of mining operations, the impacts of which are or will be revealed on the surface of the area. During the process of revealing the deformation of the terrain on a driveway or railway track, it is necessary to conduct monitoring, e.g. geodetic monitoring, which will enable current assessment of the compliance of the deformation indices calculated on the basis of these measurements with the predicted indices (being an element of the assumptions to be made for the project).

The process of designing communication structures on mining substrate should include four main stages (according to [2] the Finite Element Method should be applied):

1) identification of mining and geotechnical conditions and preliminary works, which will enable, among others, the assessment of: the category of the mining area, the magnitude of the impact of mining quakes on the structure, changes in water conditions in the area of the structure, caused by mining operations and the projected mining operations,
2) verification of the ultimate limit state of the designed structure (e.g. an embankment) in the selected geotechnical cross-sections, the limit load (taking into account the basic forms of mining deformations) and the selection of the optimum reinforcement of the embankment,
3) verification the limit state of use of the designed structure in selected geotechnical cross-sections (taking into account the basic forms of mining deformations) with a proposal to secure the structure against mining influences – on the basis of a list of results of numerical analyses, taking into account adverse impacts and design situations,
4) monitoring design, i.e. constant control of certain technical parameters of the structures during their construction and operation.

The presented effective methods of using geosynthetics in order to limit destructive changes in the substrate of traffic embankments located in mining areas are particularly noteworthy. The proper selection of polymers, with particular emphasis on their rheological properties, is an important criterion in the application of reinforcement (such as a geosynthetic mattress).

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Conflict of interests

All authors declared no conflict of interests.

Author contributions

All authors contributed to the interpretation of results and writing of the paper. All authors read and approved the final manuscript.

Ethical statement

The research complies with all national and international ethical requirements.

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References

1. Grygierek M. *Problematyka funkcjonowania drog na terenach gorniczych.* Magazyn Autostrady. 2018;1-2:40-5.
2. Stilger-Szydlo E. *Posadowienia budowli infrastruktury transportu ladowego. Teoria–Projektowanie–Realizacja.* Wroclaw: Dolnoslaskie Wydawnictwo Edukacyjne; 2005.
3. Wilun Z. *Zarys geotechniki*. Warszawa: Wydawnictwa Komunikacji i Lacznosci; 2010.
4. *Katalog Przebudow i Remontow Nawierzchni Podatnych i Polsztywnych. KPRNPP-2013*. Warszawa: Generalna Dyrekcja Drog Krajowych i Autostrad. Instytut Badawczy Drog i Mostow; 2013.
5. Grygierek M. *Zmiennosc modulow sprezystosci niezwiazanych warstw nawierzchni drogowej w warunkach gorniczych odksztalcen rozluzniajacych.* Drogi i Mosty. 2010;9(2):17-30.
6. *Instrukcja 364/2000. Wymagania techniczne dla obiektow budowlanych wznoszonych na terenach gorniczych*. Warszawa: Instytut Techniki Budowlanej; 2000.
7. Kwiatek J (ed.). *Ochrona obiektow budowlanych na terenach gorniczych.* Katowice: Wydawnictwo Politechniki Slaskiej; 1997.
8. Popiolek E. *Ochrona terenow gorniczych.* Krakow: AGH; 2009.
9. Skrzynski E. *Podtorze kolejowe*. Warszawa: PKP Polskie Linie Kolejowe; 2010.
10. Grygierek M, Sternik K. *Identyfikacja parametrow modelu nawierzchni drogowej w obszarze nieciaglych deformacji gorniczych – badania wstepne*. Magazyn Autostrady. 2018;5:32-7.
11. PN-82/B-2000. *Obciazenia budowli. Zasady ustalania wartosci*. Warszawa: PKN; 2000.
12. Konderla H. *Badanie wytrzymalosci gruntu spoistego na rozciaganie*. In: Flisiak D (ed.). *Geotechnika i budownictwo specjalne 2000. XXIII Zimowa Szkola Mechaniki Gorotworu, Bukowina Tatrzanska, 13-17 marca 2000*. Krakow: Katedra Geomechaniki Gorniczej i Geotechniki AGH; 2000. p. 161-9.
13. Litwinowicz L. *Wplyw rozluznienia nasypow znajdujacych sie w zasiegu oddzialywania podziemnej eksploatacji gorniczej na ich statecznosc*. Prace Inst. Inzynierii Budowlanej i Sanitarnej Politechniki Lubelskiej. 1982;7.
14. Litwinowicz L. *Wplyw zmian spojnosci gruntu zwiazanych z oddzialywaniem podziemnej eksploatacji gorniczej na statecznosc budowli ziemnych i nosnosc podloza*. In: *X Krajowa Konferencja Mechaniki Gruntow i Fundamentowania, Warszawa, 21-22 pazdziernik 1993 r.* Warszawa; 1993. p. 273-8.
15. Glinko H, Zgodzaj-Blaszczyk M. *Nosnosc rozluznionego podloza gruntowego na terenach gorniczych.* In: *Budownictwo na terenach gorniczych. Konferencja Naukowo-Techniczna*. Katowice–Kamien k/Rybnika; 1985. p. 23-37.
16. Kwiatek J. *Obiekty budowlane na terenach gorniczych*. Katowice: Glowny Instytut Gornictwa 2002.
17. Kwiatek J. *Wybrane problemy geotechniki terenow gorniczych.* Wroclaw: Zaklad Narodowy im. Ossolinskich; 1982.
18. Lambe TW, Whitman RV. *Soil Mechanics*. New York: John Wiley & Sons; 1979.
19. Litwinowicz L. *Wspolpraca nasypu z podlozem gorniczym*. In: *Budownictwo na terenach gorniczych. Konferencja Naukowo-Techniczna*. Katowice–Kamien k/Rybnika; 1985. p. 125-39.

20. *ZSoil.PC v. 2007 3D Academic*, [software]. Lausanne: ZACE Services Ltd.; 2007.
21. Skrzynski E., Sikora R. *Kolejowe budowle ziemne, t. 1, Utrzymanie i naprawy*. Warszawa: Wyd. Komunikacji i Lacznosci; 1990.
22. Klosek K. *Wzmacnianie podtorza gorniczego geosyntetykami*. Przeglad Komunikacyjny. 2016; 71(11):27-30.
23. Rüegger R, Hufenus R. (2003). Bauen mit Geokunststoffen – Ein Handbuch für den Geokunststoff-Anwender. St. Gallen: SVG; 2003.
24. Stilger-Szydlo E. *Projektowanie i wykonawstwo nasypow drogowych na terenach eksploatacji gorniczej.* Drogownictwo. 2003;8:239-43.
25. Gryczmanski M. *Poduszki zbrojone pod nasypami na terenach gorniczych – podstawy projektowania i badania.* In: *V miedzynarodowa konferencja. Trwale i bezpieczne nawierzchnie drogowe. Kielce, 11-12 maja 1999. Referaty*. Warszawa: Instytut badawczy drog i mostow; 1999. p. 55-62.
26. Stilger-Szydlo E, Tutaj W. *Effect of Mining Deformations on Stability of Transport Embankments.* Studia Geotechnica et Mechanica. 2004;3-4.
27. Skarzynska KM. *Odpady poweglowe i ich zastosowanie w inzynierii ladowej i wodnej*. Krakow: Wyd. Akademii Rolniczej; 1997.
28. *GEOKRATA. Systemy stabilizacji gruntu*, [online]. TABOSS GEOKRATA. Available at: http://geokrata.com/index.php?cat=web&id=materialy [Accessed: 18 June 2018].

Biographical note

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Infrastruktura komunikacyjna na terenach górniczych (problemy wybrane)

STRESZCZENIE Tematem artykułu są problemy bezpieczeństwa eksploatacji obiektów infrastruktury transportu drogowego i kolejowego na terenach górniczych. W szczególności omówiono zagadnienia dotyczące: ogólnej charakterystyki górniczych deformacji górotworu i powierzchni terenu, wpływu deformacji ciągłych i nieciągłych na zagrożenie bezpieczeństwa funkcjonowania obiektów infrastruktury transportu lądowego oraz wzmocnienia budowli infrastruktury transportu lądowego na terenach górniczych.

Podano przykłady wzmocnienia budowli transportu lądowego, znajdujących się w zasięgu wpływów górniczych.

SŁOWA KLUCZOWE budowle transportu lądowego, tereny górnicze, wzmocnienie podłoża i nasypów budowli

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