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EFFECT OF MINING DEFORMATIONS ON STABILITY OF TRANSPORT EMBANKMENTS

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Streszczenie: Przeprowadzono analizę sprężysto-plastyczną metodą elementów skończonych w płaskim stanie odkształcenia, z uwzględnieniem wzmocnienia podłoży nasypów drogowych na terenach górniczych poduszkami z materiału ziarnistego, zbrojonymi geokratami. W analizie wytrzymałościowej przyjęto zmodyfikowany warunek plastyczności Druckera–Pragera ze wzmocnieniem zależnym od spójności.

W rozpatrywanych przykładach wyznaczono wartość obciążenia granicznego przyłożonego w koronie nasypu, stan naprężenia i przemieszczenia oraz zasięgi stref plastycznych w korpusie i podłożu.

Abstract: The elastoplastic analysis was performed, using the Finite Elements Method, in flat state of strain, including improvement of road embankment subgrades in mining areas using granular cushions reinforced with geogrids. For the purpose of resistance analysis, modified Drucker–Prager condition of plasticity was adopted, where reinforcement depends on the cohesion.

In the analysed examples, we determined the limit load value imposed along the embankment top, state of stress and displacement as well as the ranges of plastic zones in the embankment body and subsoil.

Rezume:

1. INTRODUCTION

In mining areas, geotechnical parameters of constructions are of a crucial importance. Underground mining adversely affects the ground surface and the structures built on it. Formation displacements (continuous and discontinuous deformations), changes in the water regime (leading to further deformations) and underground shocks causing subsoil tremors occur in mining areas. Therefore the foundations of engineering structures erected there must be specially designed to withstand such adverse effects.

Recently large-scale construction projects, including the building of motorways, roads, bridges and associated facilities, have been launched. They are often located in large derelict industrial areas or built-up areas characterized by complicated geotechnical conditions. In such cases, special methods of reinforcing the subgrade and new foundation technologies must be used [3], [7].

The fundamentals of founding structures and analyses of their service conditions in mining areas can be found in the literature, but there are no standards for founding transport embankments in such areas and as yet no ways of protecting the structures against mining damage have been proposed. Because of mining damage such embankments are subject to large horizontal and vertical deformations and loosening.

The aim of this paper is to present the main problems relating to the interaction of earthen structures (slopes, embankments and earth dams) with the subsoil in mining areas. The presented models of the interaction between the structures and the subsoil as well as the computing algorithms used for assessing their stability can be helpful in developing new ways of protecting earthen structures and the subsoil in mining areas.

2. MODELS OF INTERACTION BETWEEN STRUCTURE AND SUBSOIL IN MINING AREA

First driving (deposit draining, overburden panelling and removing) and mining result in deformations of both ground surface and subsurface layers. A subsiding trough (at each point characterized by subsidence, gradient, curvature and horizontal deformation of the ground) forms in the area. Nonuniform vertical subsidence resulting in the curvature of the terrain and nonuniform horizontal displacements of soil particles responsible for horizontal deformations of the ground belong to the principal factors which affect the interaction between an engineering structure and the subsoil in a mining area. Depending on the location of the working face, deformations of the subsoil can cause its loosening or compression (creep).

Attempts at developing mathematical models and representing the totality of the processes which take place in the subsoil still encounter considerable difficulties. Different models of the interaction between an engineering structure and the subsoil in a mining area – from the absolutely rigid body model for both the structure and the ground through the elastic body model and the Winkler model for the structure and the elastic half-plane model for the subsoil to the elastoplastic models currently proposed – have been tested. Recent experience in this field has shown that deformations of the surface in mining areas can be classified as rheological phenomena because of formation creeping and the time-variable mining conditions.

Currently, the final version of Eurocode EC7 [8] dealing with geotechnics and tentative standards ENV (Polish geotechnical standards harmonized with the system of EU standards, preceding proper European standards) is being completed. Still there are no separate guidelines or specifications of the changes in the properties of the subsoil in mining areas. The interaction of earthen structures with the subsoil, assessment of such structures and ways of protecting them against mining damage have not been dealt with at all.

Still only the results of preliminary field and laboratory tests and tentative empirical relations for a reduction in the cohesion or internal friction angle of the subsoil are used [2], [5]. For example, a reduced soil cohesion, corresponding to the expected mining strains/ deformations, may be described by relation:

$$c_R = c_o - (c_o - c_{kr})\frac{\varepsilon_R}{\alpha_R + \varepsilon_R}, \tag{1}$$

where:

ε_R – horizontal loosening strain,

c_o – standard soil cohesion if $\varepsilon_R = 0$,

c_{kr} – critical cohesion when a decrease of soil resistance is stabilized, if $\varepsilon_R \geq \varepsilon_{kr}$,

α_R – coefficient of cohesion reduction; $0.5 \leq \alpha_R \leq 4.0$ (when loosening makes ground weak, $\alpha_R < 1.5$).

3. COMPUTING ALGORITHMS FOR ASSESSING STABILITY OF EMBANKMENTS ON SUBSOIL IN MINING AREA

Numerous investigations and observations have shown that horizontal mining-area subsoil deformations ε_R associated with the convex part of the subsiding trough's edge determine the stability of embankments. The deformations lead to the loosening of the subsoil structure and to the changes in the stress and strength parameters of soil medium. The loosening of the soil medium causes nonuniform subsidence of the earthen structure and earth slides. The boundary value of horizontal loosening deformations, whose exceeding leads to a state of limit stress equilibrium, is crucial for an analysis of the interaction between an earthen structure and the subsoil.

By comparing the boundary values of soil deformations (ε_n) in embankment with the boundary values of subsoil deformations (ε_p) in regard to the anticipated mining deformations (ε_R) one can assess the stability of earthen structures founded on subsoil in a mining area. However, very few such analyses exist and none of them has been applied in practice. In the literature on the subject, one can find rare theoretical attempts to determine the global stability index of an embankment on the basis of local stability indices calculated for arbitrary cross-sections of the earthen structure body. In the case of an embankment founded on strong ground ($\varepsilon_n < \varepsilon_p$), the Kandaurov–Müller discrete medium model is used [5]. Then it becomes necessary to determine experimentally the variation of lateral thrust coefficient $\xi = \xi(\varepsilon_R)$ (dependent on the horizontal deformations of the subsoil in a mining area) included in the system of differential equations for the medium equilibrium.

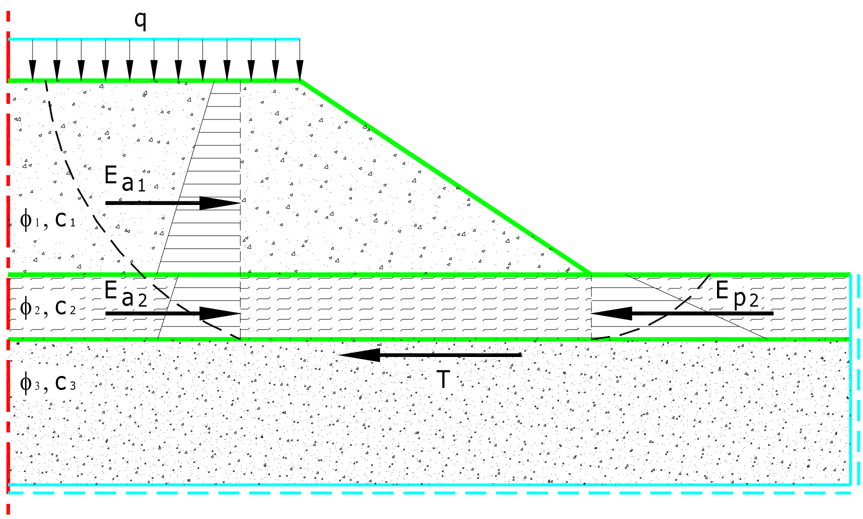


Fig. 1. Computational scheme for the Kezdi method of assessing stability of embankment founded on ground with low bearing capacity

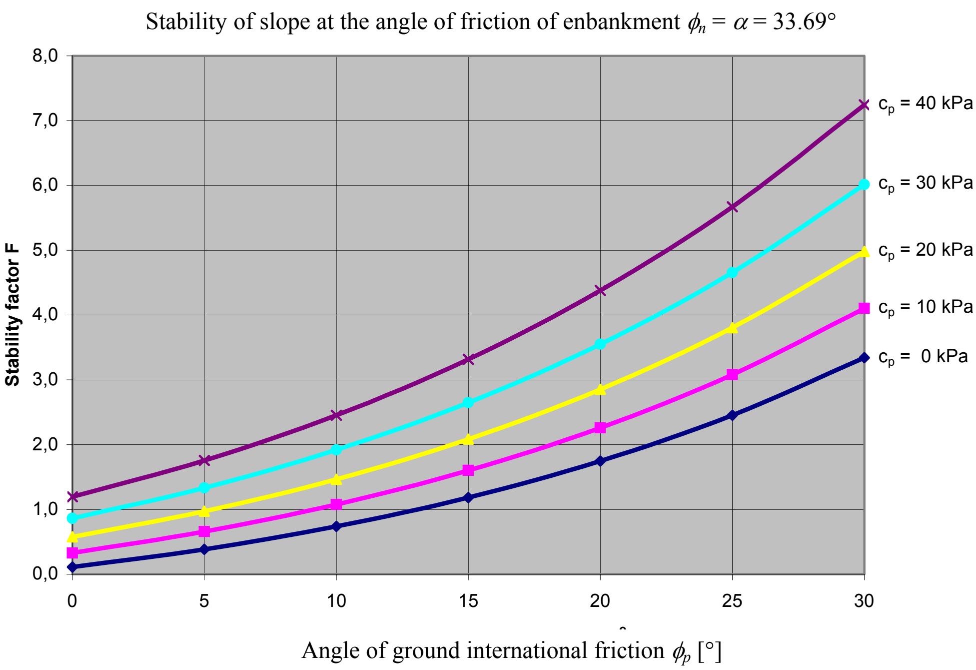


Fig. 2. Stability coefficient depending on different values of subsoil geotechnical parameters for the embankment whose angle of internal friction is equal to its angle of slope (computational scheme as in figure 1)

If $\varepsilon_n > \varepsilon_p$, one can use the computational scheme for an embankment founded on soil with poor bearing capacity shown in figure 1 and the Kezdi relation for the embankment stability index:

$$F = \frac{T + E_{p2}}{E_{a1} + E_{a2}}, \tag{2}$$

where: T is shear resistance at the floor level of weak layer, E_{a1} is active soil pressure in the embankment, E_{a2} is active soil pressure in the weak layer and E_{p2} is the passive soil pressure of the weak layer.

The stability index calculated from formula (2) versus different values of subsoil geotechnical parameters at the embankment's angle of internal friction equal to its angle of slope is represented in figure 2.

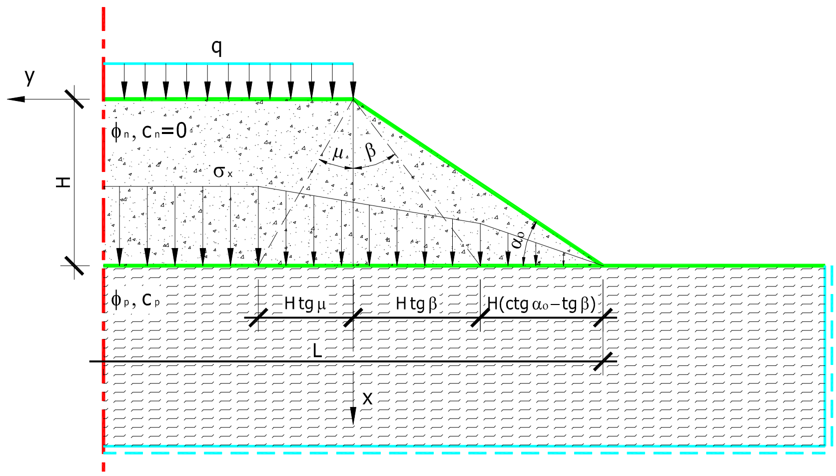


Fig. 3. Computational scheme for embankment slope failure according to the theory of limit equilibrium states

If an embankment made from incohesive soil is founded on cohesive soil whose strength is such that $\varepsilon_n = \varepsilon_p$, then one can use limit state theory solutions [6] to describe a failure of the embankment. The following relations follow from an analysis of the state of stress in the three zones of the computational scheme shown in figure 3:

- In zone I (the minimum Rankine state of stress)

$$\sigma_x = \gamma x, \quad \sigma_y = \gamma x \frac{1 - \sin\varphi}{1 + \sin\varphi}, \quad \tau_{xy} = 0. \tag{3}$$

- In zone II (intermediate), the stresses in the base of the slope follow from the solution of limit state equations (hyperbolic quasi-linear partial differential equations) and change linearly ranging from the values given by relations (4) to the following values:

$$\sigma_x, \sigma_y = \frac{\gamma x}{\cos^2 \varphi}(1-\lambda)(1 \pm \lambda), \quad \tau_{xy} = \frac{\gamma x}{\cos^2 \varphi} \tan\alpha (1-\lambda)^2,$$

$$\lambda = \sin^2 \alpha + \cos\alpha \sqrt{\sin^2 \varphi - \sin^2 \alpha}\,. \tag{4}$$

• In zone III (the maximum Rankine state of stress), the state of stress at the slope base level changes linearly ranging from the value at point D of the slope to $\sigma_x = \sigma_y = 0$. The stability of the slope is determined by the strength of soil at the slope base level within zone III.

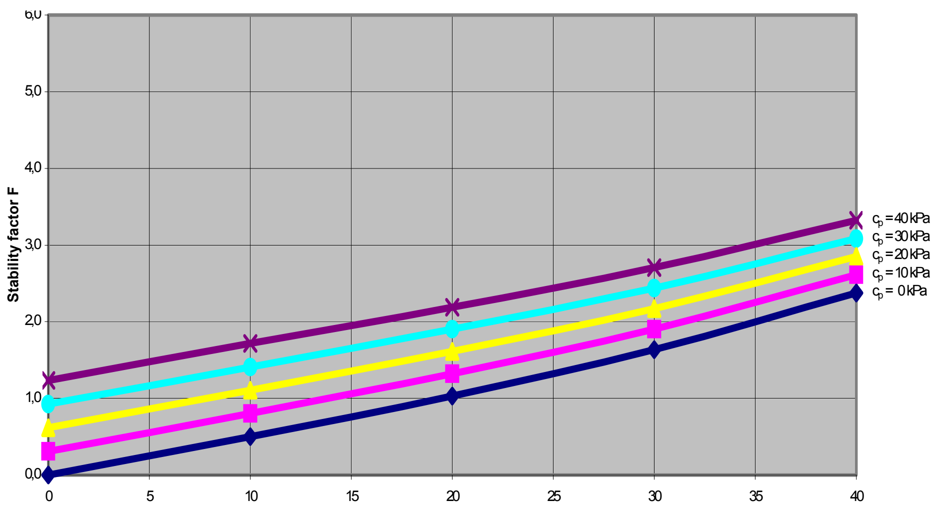
The stability index in this case is given by a ratio of shear resistance along the section DB to the stress which occurs there:

$$F = \frac{(1-\lambda^2)\tan\varphi_p}{(1-\lambda)^2 \tan\alpha} + \frac{2c_p \cos\varphi_p}{\gamma H (1-\tan\alpha \cdot \tan\beta)(1-\lambda)^2 \tan\alpha}, \tag{5}$$

where: $\lambda = \sin^2 \alpha + \cos\alpha \sqrt{\sin^2 \varphi_n - \sin^2 \alpha}$,

$$\beta = \arctan\left[\frac{(\tan\mu + \tan\alpha)(\lambda^2 - \sin^2 \varphi_n)\text{ctan}\alpha}{\text{ctan}\alpha \cos^2 \varphi_n - (\tan\mu + \text{ctan}\alpha)(1-\lambda^2)}\right], \quad \mu = 45^0 - \frac{\varphi}{2}.$$

Stability of slope at the angle of friction of enbankment $\phi_n = \alpha = 33.69°$



Angle of ground international friction ϕ_p [°]

Fig. 4. Stability coefficient depending on different values of subsoil geotechnical parameters for the embankment whose angle of internal friction is equal to its angle of slope (computational scheme as in figure 3)

The relationship between the stability index calculated from formula (5) and different values of the subsoil's geotechnical parameters at the slope's internal friction angle equal to its inclination angle is illustrated in figure 4.

The stability index of an earthen structure can be determined by approximate engineering methods only when numerical values of the geotechnical parameters of the base soil of structure body commensurate with the mining deformations of the site are known. If the Fellenius block method is used, the stability index of an embankment is calculated from this relation

$$F = \frac{\Sigma[W_i(\gamma) + W_i(\gamma')]\cos\alpha_i \tan\varphi_i' + \Sigma c_i' \, l_i}{\Sigma[W_i(\gamma) + W_i(\gamma_{sr})]\sin\alpha_i}, \tag{6}$$

where: $W_i(\gamma)$ is the weight of the i-th soil block above the water level; $W_i(\gamma')$ – the weight of the i-th soil block below the water level calculated assuming γ'; $W_i(\gamma_{sr})$ – the weight of the i-th soil block below the water level calculated assuming γ_{sr}; φ_i', c_i' are effective strength parameters; and l_i, α_i are, respectively, the length and inclination of the slide surface in the i-th block. The calculations yield a minimum stability index value regarded as a measure of the slope safety margin and the landslide hazard.

The above analysis can be used to assess an embankment stability if:

a) examination of actual geotechnical profiles shows that no privileged slide surfaces occur and there is no danger that the embankment structure will slide over the subsoil;

b) it is possible to predict changes in the strength of soil in the soil base and the embankment, taking into account the predicted mining deformations;

c) numerical values of geotechnical parameters of the soil in embankment body commensurate with the mining deformations of the ground are known.

In the present study, we assume that approximation methods used in engineering (the so-called "methods of stripes") are of limited usability for the purposes of estimating the stability of embankment on mining foundation, as it is not possible to include the mining strains in calculations. We done a number of calculations to evaluate the embankment stability by means of the Fellenius and Bishop's method, taking into account the reduction of soil cohesion of foundation and embankment. The analysis of calculation results proved that cylindrical slip surface adopted in these methods did not precisely describe the mechanism of earthen structure stability loss. It concerns the embankment that is not reinforced at the footing against mining effects as well as the embankment that is strengthened by means of a mat made of grainy material, reinforced with geo-grids. Out of the methods for stability evaluation discussed above the Kezdi method (2) proved to be the best, as it presents the most realistic description of devastation.

4. NUMERICAL ANALYSES PROPOSED

4.1. EMBANKMENT STRENGTH ANALYSIS BY FINITE ELEMENT METHOD

The processes set in motion by underground mining and the formation of a subsiding trough cause changes in the stress and strain states. The changes are proportional to the severity of the mining effects, expressed in mining area categories (I–V), and also depend on the direction of deformation propagation relative to the construction axis and the rate of mining.

In the present study, we assume that approximation methods used in engineering (the so-called "methods of stripes") are of slight usability for estimating the stability of embankment on mining foundation, as it is not possible to include the mining strains in calculations. We performed a number of calculations to evaluate the embankment stability by means of the Fellenius and Bishop's method, taking into account the reduction of foundation and embankment soil cohesion. The analysis of calculation results proved that cylindrical slip surface adopted in these methods did not precisely describe the mechanism of earthen structure stability loss. This concerns the embankment that is not reinforced at the footing against mining effects as well as the embankment that is strengthened by means of a mat made of grainy material, reinforced with geo-grids. Out of the methods for stability evaluation discussed above the Kezdi method proved to be the best, because it presents the most realistic description of devastation.

Contemporary development of numerical methods, above all the finite elements method (MES) and constitutive soil models (elastoplastic "stress–strain" relations), makes calculation analyses the basic research tool applied to evaluate how an earthen structure mates with subsoil. MES numerical analyses enable us to define both a bearing capacity and – at the same time – strain of earthen structures. Existing MES software packages may be divided into two groups: geotechnics-oriented programs (e.g. HYDRO-GEO, PLAXIS, Z_SOIL) and all-purpose programs (e.g. ABAQUS, COSMOS). Only programs belonging to the first group are applied to engineering purposes, though these are limited only to solving problems of flat state of strain or axial symmetry. Moreover, they offer no possibility to intervene in a material model. The programs belonging to the second group, based on the advanced MES method, are employed for the purposes of scientific studies that aim to solve complex engineering questions. They are supplied with a rich library of material models and at the same time enable us to perform the analysis of spatial questions. However, the resulting problem is laborious data input and high required capacity of computers.

In the present study, the earthen structure was modelled, by means of MES, until it had reached the limit point, by:

• reduction of soil resistance parameters c/R_F and ϕ/R_F, arriving at (by means of trial-and-error method) such a maximum value of reduction coefficient R_F that provoked substantial displacements of characteristic points of the structure (substantial displacement of the point located on the embankment top indicated the loss of slope stability, and substantial displacement of the point situated near the embankment foundation suggested soil upthrust adjacent to the structure foundation, resulting from the loss of bearing capacity of the ground);

• incremental increase in the structure load over the nominal value (e.g. service load imposed along the embankment top or load of embankment kerb weight) and simultaneous monitoring of characteristic points of the structure – there was reached such a limit load value q_{limit} that provoked substantial increase in displacements.

In this research, displacements and stresses in an embankment founded on mining area subsoil were assessed based on an elastoplastic analysis performed using software packages MES: HYDRO-GEO ver. 3.0 [10] and ABAQUS [9]. The aim of simulations for the plane state of strain in the ground was to analyze the limit bearing capacity of the embankment for the successive embankment construction stages, changes in the embankment geometry, loading with vehicles and mining displacements.

4.2. THE EMBANKMENT IN PLANE STATE OF STRAIN – UPPER ESTIMATION OF STABILITY COEFFICIENT

The earthen structure was modelled with triangular six-node finite elements by means of the HYDRO-GEO program. The boundary conditions of numerical computing model (unslidable pivot bearings at the bottom edge of the model and slidable pivot bearings along the symmetry axis of embankment and the vertical edge of subsoil) and the case of mining deformations in the direction perpendicular to the longitudinal axis of embankment were assumed.

An elastoplastic soil model was used for the computations. The analysis was performed for the plane state of strain, using the Drucker–Prager yield criterion:

$$\overline{F} = \alpha\sigma_m + \left(3J_2\right)^{1/2} - k\,, \tag{7}$$

$$\sigma_m = \frac{I_1}{3}, \quad \alpha = \frac{6\sin\varphi}{3-\sin\varphi}, \quad k = \frac{6c\cos\varphi}{3-\sin\varphi}\,,$$

where: $\overline{F}$ is the surface of plasticity, φ – an internal friction angle, α and k – coefficients assumed in accordance with the definition of a surface type (here outer surface).

An elastoplastic soil model and a reduction of soil cohesion to 70% of the initial value were used for the computations ($c_r = 0.7\ c$). The embankment located on the axis of the planned section (from the Sosnica junction to the Wirek junction) of the Wroclaw–Cracow motorway A-4 was analyzed. The entire section passes through an area where coal is mined underground. Owing to restrictions put on mining, the current mining damage is within mining area category II (gradient $T \leq 5$ mm/m, radius of curvature $R \geq 12$ km, horizontal strain $\varepsilon \leq 3$ mm/m).

Table 1

Soil strength parameters assumed for calculations

Soil	Symbol	Condition of soil		Unit weight of soil	Cohesion	Internal friction angle	Modulus of compressibility	Poisson's ratio
		Compaction factor	Plasticity factor					
		I_D	I_L	$\gamma^{(n)}$ [kN/m^3]	$c^{(n)}$ [kPa]	$\varphi^{(n)}$ [°]	M_0 [MPa]	ν
Fine sand (rainy)	P_d	0.45	–	19.0	0.0	30.0	60	0.30
Clay (dusty)	G_p	–	0.40	18.0	10.0	11.5	18	0.32
Embankment – sand	P_d	0.60	–	17.0	0.0	30.0	80	0.30

The geotechnical characteristics are shown in table 1. Humus loam in the hard-plastic state (degree of plasticity $\boldsymbol{I}_L^{(n)}$ = 0.40 and maximum shear strength $\tau_{f\ \max}$ = 0.046 MPa), containing 4.1% of organic matter and 60% of clay and dust fraction, is deposited in the subsoil. This 4.0 m thick layer of cohesive soil is unsuitable as the subgrade for a road surface and when it is wet or loaded, it is highly susceptible to sliding and surface erosion. Below there are watered fine sands (of medium density $I_D^{(n)} = 0.45$) whose bearing capacity makes them suitable for a road.

The computations comprised the following stages: I – determination of the subsoil geotechnical layers, II – addition of the transport embankment body, III – application of a service load to the top edge of embankment, IV – modelling the horizontal tensile deformations.

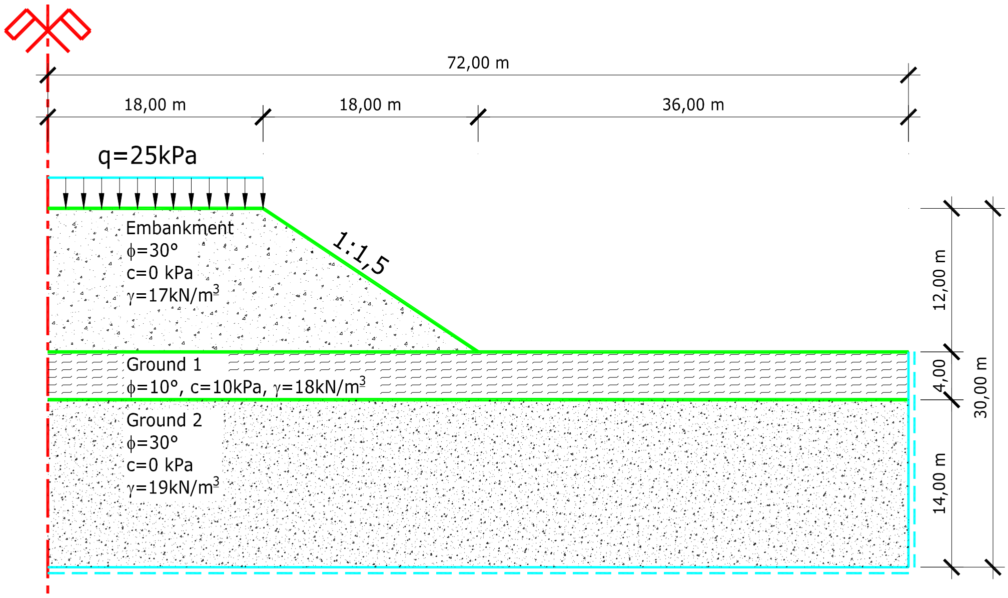
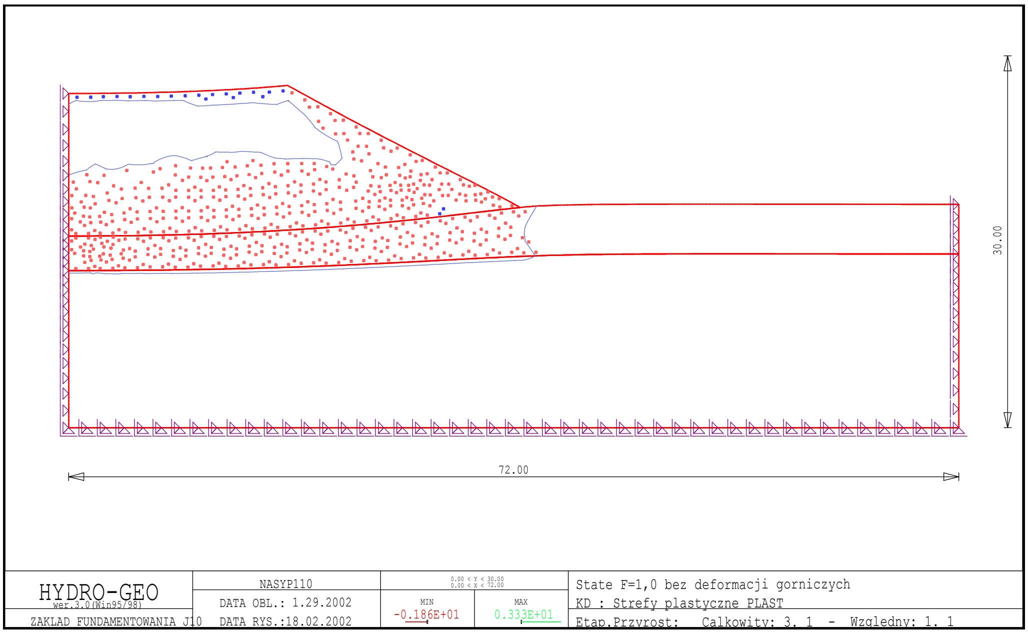


Fig. 5. Static scheme of embankment

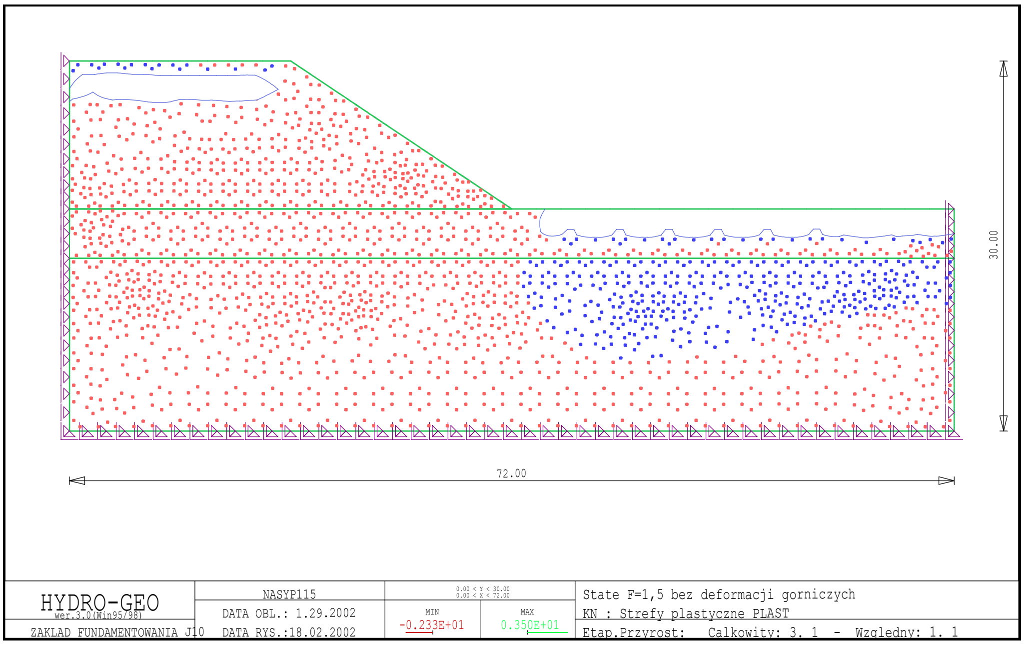
The arrangement of the geotechnical layers, the embankment dimensions and the load scheme are shown in figure 5 (slopes 1:1.5, motor vehicle load q = 25.0 kPa on the overburden).

Figure 6 shows the extension of the plastic zones at unreduced (R_F = 1.0) and reduced (R_F = 1.5) geotechnical parameters.

The method for the reduction of resistance parameters causes that the range of zones of flow is extended up to the areas where the soil actually maintains elasticity. For example, it concerns the area adjacent to the right edge of the model (figure 6), which remains beyond the influence of embankment interactions. The HYDRO-GEO software did not lead to determination of plastic strains in the slope. The software only proved helpful while describing the range of plastic zones. In this case, we did not manage to determine the reliability coefficient, as the displacements were nearly alike in the case of reduction coefficients of R_F = 1.0 and R_F = 1.5. It was only possible to estimate the upper value of the coefficient $F < R_F$ = 1.5, at which a total plasticization of the model occurred.



a)



b)

Fig. 6. Plasticization zones for embankment on mining area subsoil:
a) extent of plastic zone for $R_F = 1.0$; b) extent of plastic zone for $R_F = 1.5$

In the course of the simulations, the top edge of embankment buckled, because the cohesive soil in the central part under the embankment base plasticized first. The difference in the vertical displacements between the embankment axis and the edge amounted to 8.0 cm. When the tensile mining deformations were taken into account (stage IV), the embankment together with the subsoil became totally plasticized.

The analysis indicates that the base of the embankment should be reinforced to reduce the settlement of its central part and to increase the stability coefficient. As a result of mining deformations, the bearing capacity of the subsoil (particularly the compact subsoil) has decreased. The current version of the program does not support the simulation of plastic strain increase.

4.3. EFFECT OF SUBTERRANEAN EXPLOITATION OF MINES ON THE STATE OF STRESS AND DISPLACEMENTS OF TRANSPORT EMBANKMENTS

Subsequently, a stress–displacement analysis of the embankment founded on a weak mining-area subsoil was performed using the ABAQUS software. The program is widely used in science and belongs to the most powerful numerical tools capable of solving a large range of static and dynamic problems in mechanics.

The second method of modelling a transport embankment by means of MES is presented below. The embankment was brought to a limit state by increasing the service load imposed along the embankment top. The limit load value q_{limit} was calculated, at which we observed the substantial increment of its characteristic points' displacements.

In the numerical computations, the soil medium was described by the model of elastoplastic body, and the limit state of a medium was expressed by the Drucker–Prager linear condition of plasticity, with stabilization dependent on cohesion. The non-associated flow rule was adopted.

The Drucker–Prager linear condition of plasticity is shown in figure 7: in the plane *p*–*t* in figure 7a and in the deviator plane in figure 7b.

The linear model is defined as the condition with three invariants of stress:

$$F = t - p\tan\beta - d = 0\,, \tag{8}$$

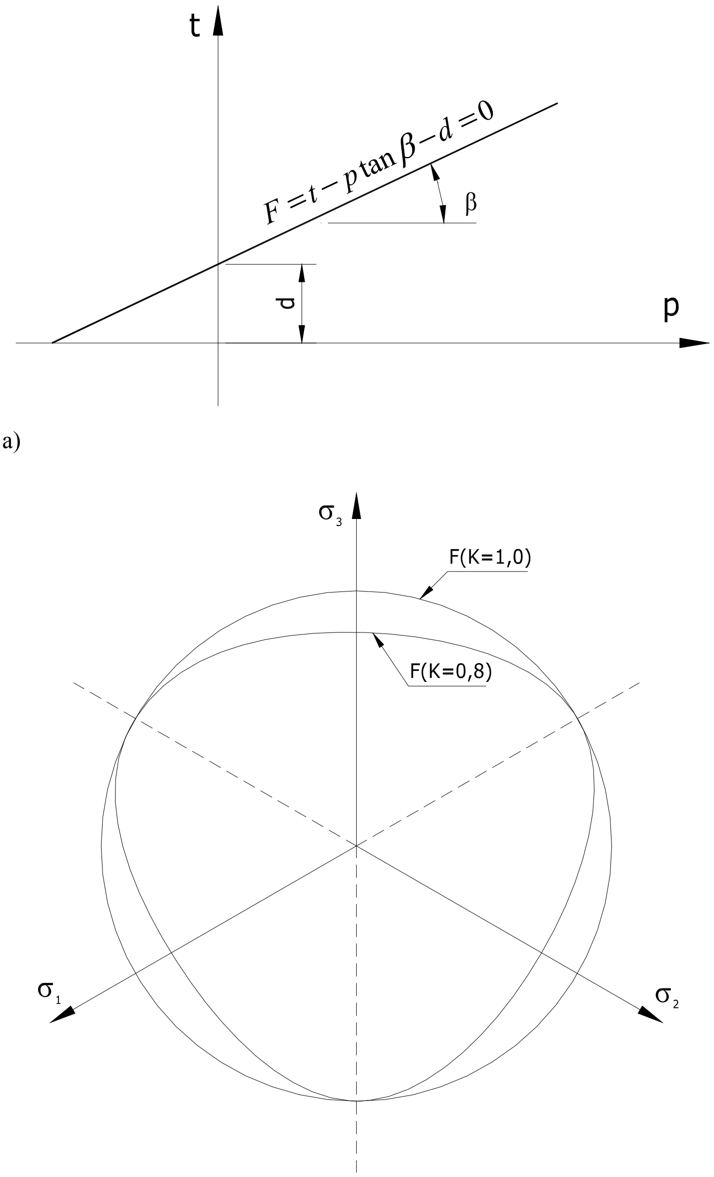
where:

$$t = \frac{1}{2}q\left[1 + \frac{1}{K} - \left(1 - \frac{1}{K}\right)\left(\frac{r}{q}\right)^3\right],$$

β – the slope of the linear surface of plasticity in the plane *p*–*t*, related to an angle of internal friction of material (figure 7a),

d – the material cohesion,

K – the ratio of plasticizing stresses in triaxial tension state to plasticizing stresses.



a)

b)

Fig. 7. The Drucker–Prager linear condition of plasticity:
a) yield surface in the p–t invariants plane; b) yield surface in the deviatoric plane

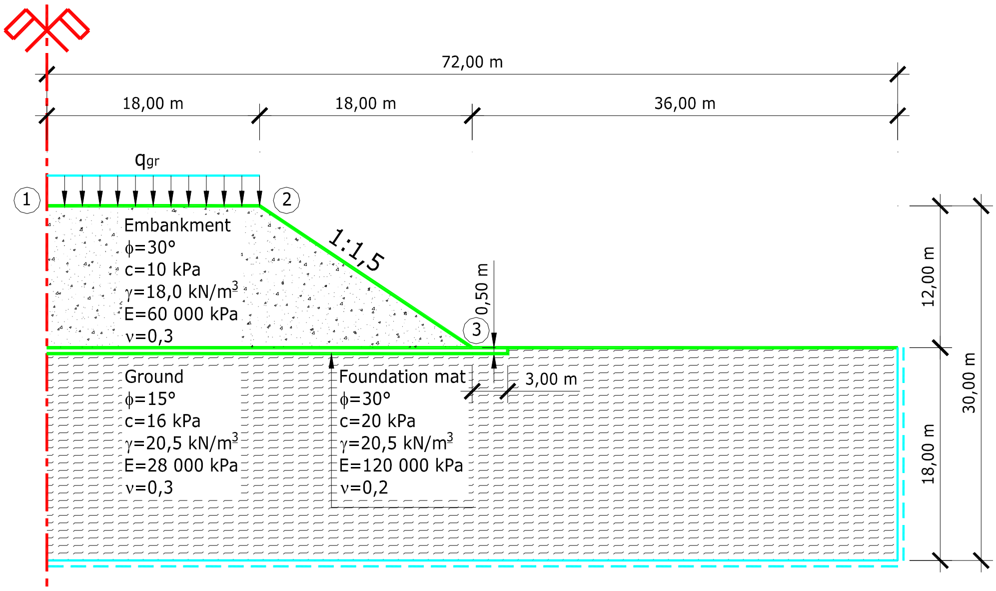


Fig. 8. The static model of the embankment, adopting the subgrade reinforcement of a cushion

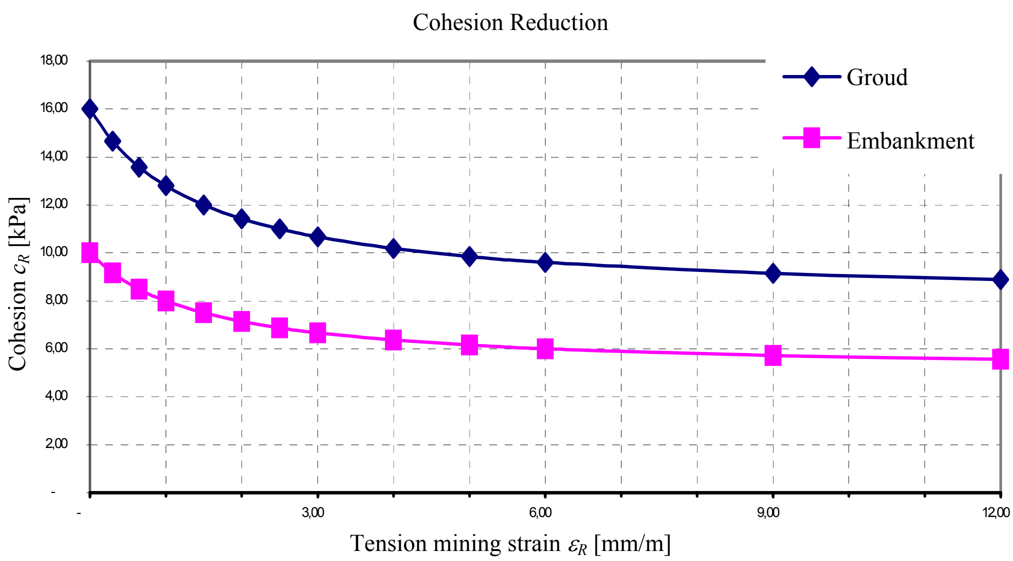


Fig. 9. Reduction of soil cohesion in the subsoil and embankment

The static model of the embankment, included in calculations adopting the foundation reinforcement of a cushion (made of granular material, reinforced with geogrids), is presented in figure 8. The embankment is made of natural loose ground with aggregate supplement obtained from burnt shale. Shale is mining waste material demonstrating cohesion. The resistance of cohesive soil that is subject to stretching deformations decreases due to reduction in soil cohesion of foundation and embankment (figure 9).

The results of the numerical simulations for the particular calculation stages (computational schemes) are shown in table 2. These are horizontal and vertical displacements at characteristic points of the embankment (in the case of a slope without reinforcement and with a cushion).

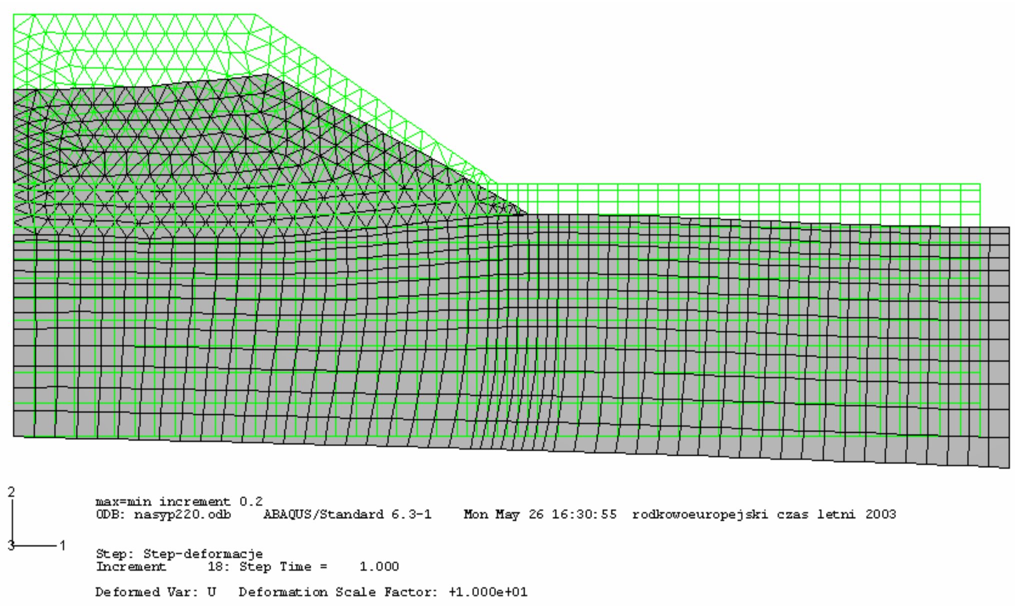


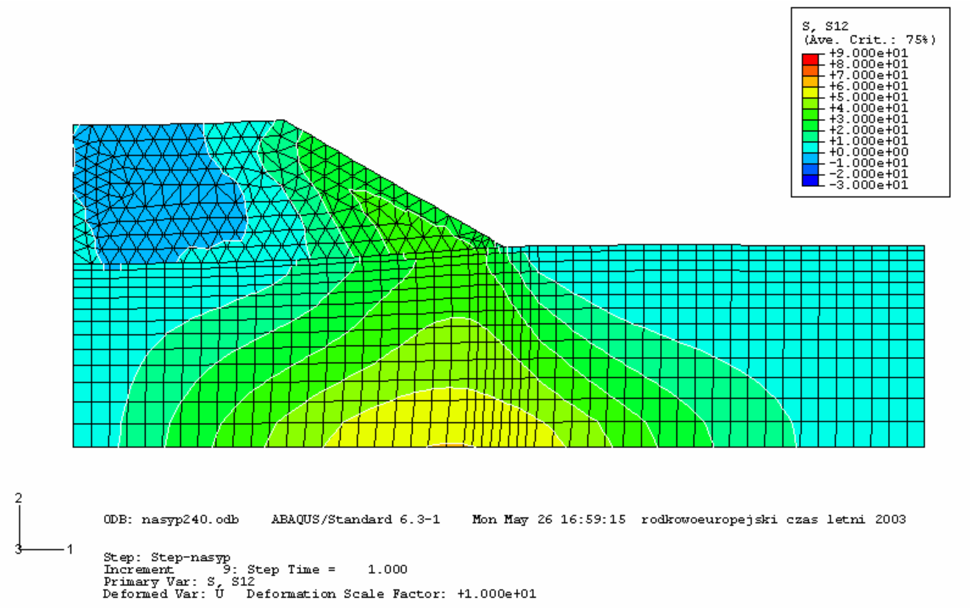
Fig. 10. A deformed transport embankment after forced mining strains

Figure 10 presents a deformed transport embankment after forced mining strains. A decrease in tangent stress in the embankment slope is shown in figure 11 (embankment without mining strains – figure 11a, embankment after forced mining strains – figure 11b).

The following conclusions can be drawn from the computations:

• The loss of stability by a transport embankment on the subsoil of a mining area (noncohesive soil) is indicated by a divergence of the solution and the stability coefficient of embankment (determined by reducing the parameters of soil strength) equal to 1.25.

• Large plastic strains precede the loss of stability by an embankment made from cohesive soil. Large displacements of the embankment and a distinct slide surface can



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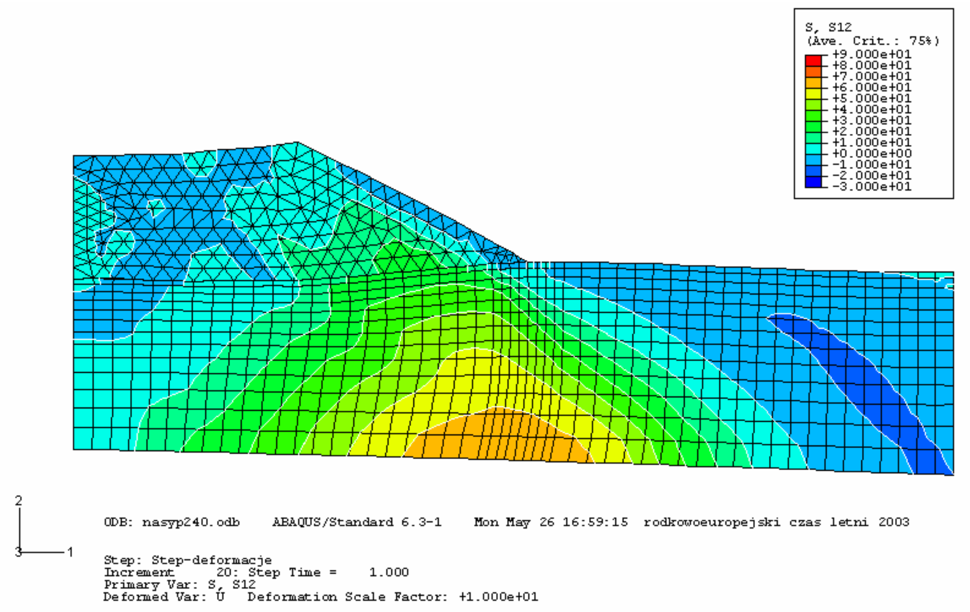

b)

Fig. 11. Decrease in tangent stress in the embankment slope:
a) embankment without mining strains; b) embankment after forced mining strains

be seen on the diagrams showing the maximum plastic strains in the principal plane. The coefficient of stability (determined by reducing the parameters of soil strength) is 1.35.

• If the contacts between the embankment and the subsoil (without a rigid connection between the two finite element meshes) are modelled, the stability of an embankment without mining deformations decreases; however it considerably increases for subsoil tension. For a lower coefficient of friction between the subsoil and the embankment the stability of the embankment increases at the stage with tensile deformations. This is so because the loosening of the embankment is reduced at these parameters.

• Tensile mining deformations result in an increased (as much as 10.0 cm) vertical settlement of embankment, particularly in the central part of its crown. Tensile stresses are concentrated closer to the embankment axis and they and the shear stresses on the slide surface of soil wedge do not add up – owing to the contact elements between the embankment and the subsoil.

• To reduce the settlement and to increase the stability of the embankment, it would be advisable to reinforce the subsoil with synthetic insertions.

5. CONCLUSION

The crown of transport embankments is subject to nonuniform settlement and their slopes are susceptible to earth slides. Loosening of the subsoil (and thus of the embankment body) occurs in the tensile strain zone (in the subsiding trough's convex part). Large limit-state areas appear, which may cause the embankment to lose its stability and lead to increased local subsidence. In the design of transport embankments, the following should be included:

• an analysis of displacements and stresses in the embankment subject to deformations for a particular category of mining area,

• an assessment of the subsoil bearing capacity and the stability of the earthen structure with and without reinforcement against mining effects,

• a monitoring system allowing quick comparison of the actual mining deformations with the ones predicted for the transport structure and assessments of the impact of changes in the deformations on drainage, surfaces and engineering structures.

The assessment of the stability of transport embankments in mining areas is one of the key measures adopted in order to ensure their safety. FEM-based solutions yield a broader range of results than block methods do – besides the stability coefficient values, they also provide information on stress distributions, deformations, pore pressures, the degree of ground effort, the extent of plasticization zones and the increase in plastic strains. This is especially true in the case of the ABACUS program designed for solving strongly nonlinear problems. The program's large library of material models and finite elements allows one to perform the stress–displacement analyses of static and dynamic problems in mechanics. Different general soil models (the Coulomb–Mohr model, the modified Druker–Prager models, the Cam–Clay model, the Cap model) are used for modelling geotechnical structures. But the limit bearing capacity theorems, on the basis of which the most accurate estimate can be obtained by the Fellenius method, apply only in the case of block methods.

The MES ABAQUS software package was used for the numerical calculations which were carried with the computers of the Wroclaw Centre of Networking & Supercomputing.

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