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LIMIT STATES OF TRANSPORT EMBANKMENTS ON THE SOILS OF LOW BEARING CAPACITY AND MINING AREA SUBSOIL

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Abstract: The foundation of embankments on the soils of low bearing capacity is one of the basic problems of highway engineering. This article presents some examples of the analyses that allow us to show how the soils of low bearing capacity and the ravelling of mining-area subsoil influence the stability and settlement of road embankments.

Streszczenie: Podstawowym problemem w budownictwie drogowym jest posadowienie nasypów na gruntach słabonośnych. Przedstawiono przykłady analiz wpływu gruntów słabonośnych i rozpełzania podłoża górniczego na stateczność i osiadanie nasypów budowli infrastruktury transportu lądowego.

Резюме: Основным вопросом в дорожном строительстве является основание насыпей на слабонесущих грунтах. Представлены примеры анализов влияния слабонесущих грунтов и ползучести горного основания на устойчивость и оседание насыпей построек инфраструктуры сухопутного транспорта.

1. INTRODUCTION

One of the basic problems of highway engineering is the foundation of embankments on the soils of low bearing capacity. Such cases are encountered more and more often when the construction is erected in the areas that used to be exploited for industrial purposes or in built-up areas characterized by complex geotechnical conditions. In the case of soils that are very compressible, traditional methods of their consolidation that consist in their initial overloading with an embankment require allowing soil to consolidate even for few years. That is why the need for searching for new methods of strengthening the soil under embankments arises, at the same time meeting the requirements for bearing capacity and road use and suggesting new technologies for their foundation.

Technical conditions for designing roads, Polish norms and Eurocodes recommend performing the calucations for all the constructions in accordance with the methods of limit states. Limit load is a load value that provokes the occurrence of the bearing capacity or the serviceability limit state in the construction. If the limit states of bearing capacity are exceeded, the embankments may lose their stability, and the entire construction or its part may be destroyed and the conditions resulting from the plasticization of the material in the body and subsoil may be settled. The bearing capacity of the construction is a limit load value determined by adopting the design characteristics of the material and subsoil. In the serviceability limit states, we approach the characteristic load values, mechanical features of materials and geotechnical parameters. Among their types the overstrains of the construction and subsoil are most frequently mentioned.

Although the assessment of slope and embankment stability constitutes one of the complex issues and some difficulties in determining the parameters charaterizing the slope are encountered, a number of diversified methods of their calculation are really impressive. In literature, substantially less attention has been devoted to the serviceability limit state that should demonstrate that the strain of an embankment caused by anticipated interactions will not lead to loss of its suitability for use.

The settlement of transport embankment resulting from the ravelling of mining-area subsoil is a complex and current problem. The subterranean exploitation of mines is responsible for the unbalance of the orogen and the displacement of rocks in the vicinity of an exploited bed near a post-exploitation cavern.

This article presents some examples of the analyses of how the soils of low bearing capacity and the ravelling of mining-area subsoil influence the stability and settlements of land transport infrastructure constructions.

2. THEORETICAL BASES FOR DESIGNING EMBANKMENTS ON THE SUBSOIL OF LOW BEARING CAPACITY

2.1. BEARING CAPACITY LIMIT STATES

Designing and erecting transport embankmenks on the soils of low bearing capacity constitute one of the most difficult geotechnical problems. Embankments on such soils are subject to substantial deformations as a result of their compressibility and to plastic strains of subsoil [8]. These plastic strains occur when in a subsoil the shear stresses exceed its shear strength. As the load of an embankment increases, the stresses in an infirm ground also increase which is responsible for a greater settlement of an earthen structure. Once an embankment reaches the boundary level characteristic of a given subsoil, an upthrust of weak soil layers to the sides, sliding of embankment slopes and its sudden settlement can occur leading to the degradation of road surface (figure 1).

While building and using such constructions, we often deal with:

• loss of the boundary subsoil bearing capacity and uncontrollable sinking of an embankment in conjunction with an upthrust of weak soil layers and loss of embankment stability;

• prolonged and substantial settlement of the subsoil of low bearing capacity.

The stability of an embankment on a very compressible layer of peat can roughly be assessed by analyzing the subsoil limit load. It is determined depending on the value of an internal friction angle ϕ of the weak soil layer, using Therzaghi's proposition.

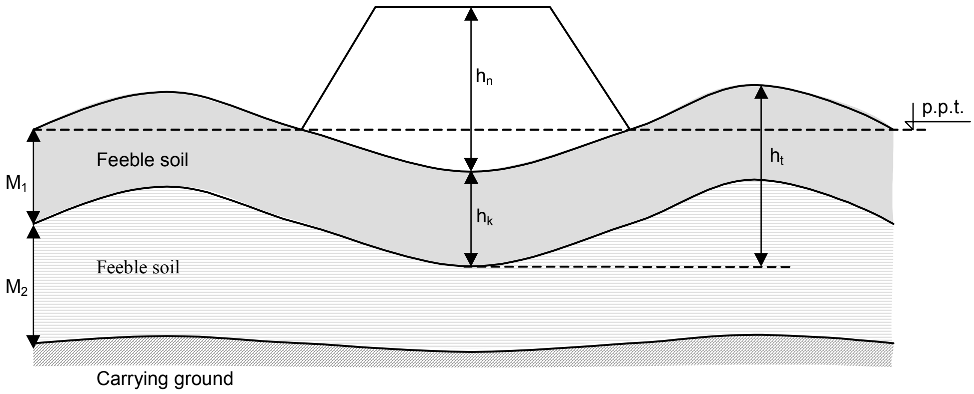

Fig. 1. A model of low bearing capacity subsoil strain

At $\phi \approx 0°$

$$q_f = 5.7c + \gamma_h h_t \, , \tag{1}$$

but at $\phi > 10°$

$$q_f = cN_c + \gamma_h h_t N_q + \gamma b' N_\gamma \, , \tag{2}$$

where:

q_f – the limit load [kPa];

γ_h – the soil bulk density between the terrain surface and the roof of the weak soil layer [kNm^{-3}];

h_t – the hollow in the roof of the weak soil layer in relation to the terrain surface [m];

γ – the bulk density of a weak soil layer (taking into account water displacement) [kNm^{-3}];

b' – the length of the horizontal projection of the embankment slope [m];

c – the cohesion of a low bearing capacity soil [kPa];

N_c, N_q, N_γ – bearing capacity coefficients dependent on an internal friction angle ϕ of soil [12].

The stability coefficient of embankment on the soil of low bearing capacity is defined by the dependence

$$F = \frac{q_f}{\sigma_z}, \quad \sigma_z = q + \gamma_n h_n + \gamma_k h_k, \tag{3}$$

where:

q – the moving load of embankment [kNm^{-2}];

γ_n – the bulk density of embankment soil [kNm^{-3}];

h_n – the height of the embankment erected (taking into account the anticipated settlement) [m];

γ_k – the bulk density of the soil of low bearing capacity (between the bottom of the embankment and the roof of a weak soil layer) [kNm^{-3}];

h_k – the thickness of the soil of a low bearing capacity T [m].

At $F < 1.0$ an upthurst of infirm ground from under the embankment occurs. When the bottom of infirm ground assumes the horizontal direction, an upthrust to two sides occurs, and in the other case – an upthrust to one side, in the direction of a greater depth. The study conducted by the author has shown that the coefficent of embankment stability should range from 1.30 to 1.50 (depending on the importance of the road and the accuracy in determining the cohesion resistance c of the boggy layer).

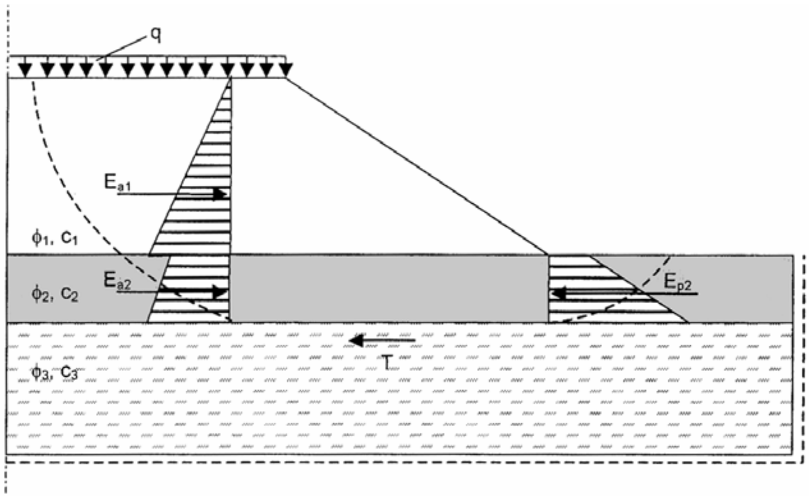

Fig. 2. Design method of stability assessment for embankment on the soil of low bearing capacity according to Kezdi's method

Taking into account a general case of an embankment on the soil of a low bearing capacity, the author recommends the block method for assessing the embankment stability. This method presented by Kezdi posits that the slope block is displaced along the floor of a weaker layer. This pattern of the slope devastation can be employed when $\varepsilon_n > \varepsilon_p$. The sliding-down force is an active pressure from the embankment body, and the holding forces – possible passive pressure from the footing and friction force acting in the foundation of the soil medium body (figure 2). The embankment stability coefficient is determined by the dependence:

$$F = \frac{T + E_{p2}}{E_{a1} + E_{a2}}, \tag{4}$$

where:

T – the horizontal shear resistance of weak layer floor;

E_{a1} – the active pressure in embankment;

E_{a2} – the active pressure in weak layer;

E_{p2} – the passive pressure in weak layer.

Figure 3 provides a sample dependence of the stability coefficient (2) on the diversified soil geotechnical parameter (for the angle of internal friction of embankment soil equal to the angle of its slope).

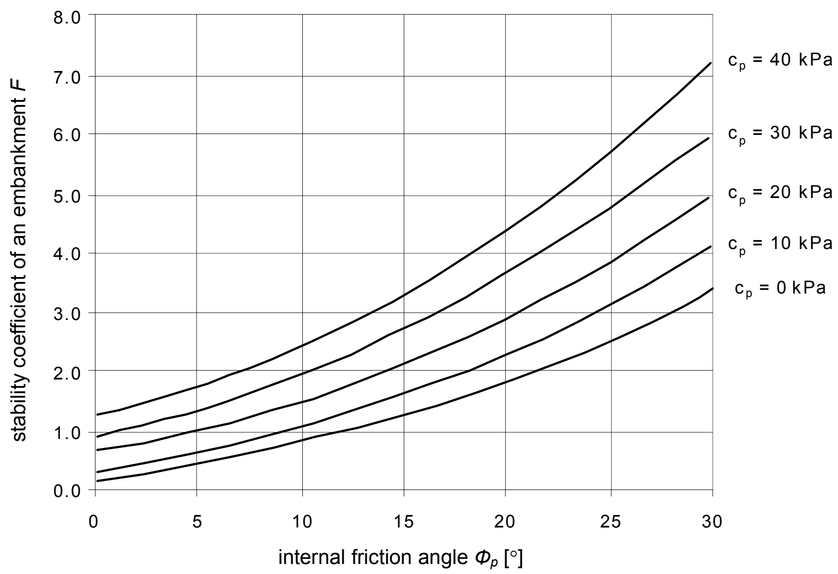

Fig. 3. Dependence of stability coefficient on diversified subsoil geotechnical parameter

This model for the devastation of a slope can be applied when the boundary values of embankment soil strains are greater than boundary values of subsoil strains.

Numerous comparative analyses with the Finite Element Method (MES) carried out by the author have shown that the above-mentioned model representing the slope stability loss is in agreement with the model of the devastation of an embankment on the subsoil of low bearing capacity. The obtained models of embankment bearing capacity loss demonstrate substantial consistency with the above-mentioned block method of stability assessment according to (4).

2.2. SERVICEABILITY LIMIT STATES

A settlement of subsoil is a vertical displacement of a soil layer loaded with an embankment. The total settlement is a sum of intitial, consolidating and secondary settlements. Usually a minor initial settlement is caused by a change in the shape of soil surface under load. Consolidation is the process extended over time. In fine-grained soils characterized by small coefficient of permeability, consolidation fades away when excessive water pressure caused by soil loading is dissipated. As for the secondary settlements, also extended over time, it commences when the process of consolidation has been completed (with constant effective stresses). Because of the changes in compression charateristics occurring in the soil and the stresses that fade away as the depth increases, a settlement of subsoil is calculated by dividing it into layers and adopting in each of them the appropriate stress as well as the compression charateristic. The total settlement is a sum of settlements of individual layers.

Settlements of embankments on the soils of low bearing capacity are calculated by taking into account only these layers that will not be upthrust and will remain under the bottom of the embankment. They are calculated in accordance with the following dependence:

$$s = s_n + \sum s_i = s_n + \sum \frac{\sigma_{zi} \cdot h_i}{M_i}, \tag{5}$$

where:

$s_n = \int_0^{h_n} \frac{\sigma_z dh}{M_n} = \int_0^{h_n} \frac{\gamma_n \cdot hdh}{M_n} = \frac{\gamma_n \cdot h_n^2}{2M_n}$ – own settlement of an embankment [m];

σ_{zi} – the vertical stress in the centre of each subsoil layer analysed [kPa];

h_i – the thickness of layers that have not been upthrust [m];

M_i – the endometric compression modulus of each layer [kPa];

M_n – the endometric compression modulus of embankment soil [kPa].

In order to give a reliable assessement of the serviceable limit state of the subsoil loaded with an embankment, it is indispensable to know the value of the endometric primary compression modulus in various load ranges. Determining it with a classic method (on the basis of endometric research) is carried out over a long period of time. In order to determine the settlement of the soil of low bearing capacity over a period of time we have to find the value of the coefficient of vertical consolidation based on the experimental interpretation of consolidation curves. Recently a methodology for estimating settlement and consolidation parameters on the basis of the research of diversified organic soils has been suggested [13]. The merit of the presented methodology of calculations is the possibility of determining the value of the endometric primary compression modulus based on the correlations found without the necessity for carrying out prolonged endometric research. It is suitable for engineering purposes, most of all for testing organic soils consolidated in a normal way and fully saturated with water (gyttias, lacustrine chalks, peats, aggradate muds).

The choice of the way of foundation work and the technology of making transport embankments on the subsoil of low bearing capacity is influenced by:

- the position of the grade line in relation to the exisiting terrain which determines the adoption of the appropriate height of the embankment;
- the location of the roof of the soils of low bearing capacity in relation to the terrain surface and soil types, the thickness and arrangement of layers in transverse and longitudinal profiles of the embankment;
- the purpose of the embankment (road category, load class);
- access to the building site for construction equipment, the lead time and cost of the investment.

The above-mentioned factors determine the choice of an appropriate, in a given case, technology for the foundation and the construction of embankments on the soils of low bearing capacity.

Figure 4 shows one of the examples of cracks in the surface resulting from a non-uniform settlement of a transport embankment.

Fig. 4. Longitudinal cracks in the surface resulting from a non-uniform settlement of an embankment

While modernizing the road, the width and height of the embankment were increased. The left embankment was widened by 3.0 m, and the right one by 1.0–1.50 m. The embankment of the existing road was firm, but built on the layer of peat left in the subsoil. Deposited peats were firm (w_n < 500%). Underground water was present in the footing of the embankment at a depth of 0.20–1.30 m below the terrain level and was subject to fluctations reaching, on an average, 0.5 m. The widened part of the embankment is founded on the old transport embankment, and the elevation of its top (in the extreme case) reaches a height of approximately 0.5 m above the top of the old embankment. For economical reasons it was a conscious decision not to reinforce the subsoil underneath the embankment. The old embankment was bound to the new one with shelves and a system of geosynthetics. The settlement of the embankment after 43 months of using the road is shown in figure 5.

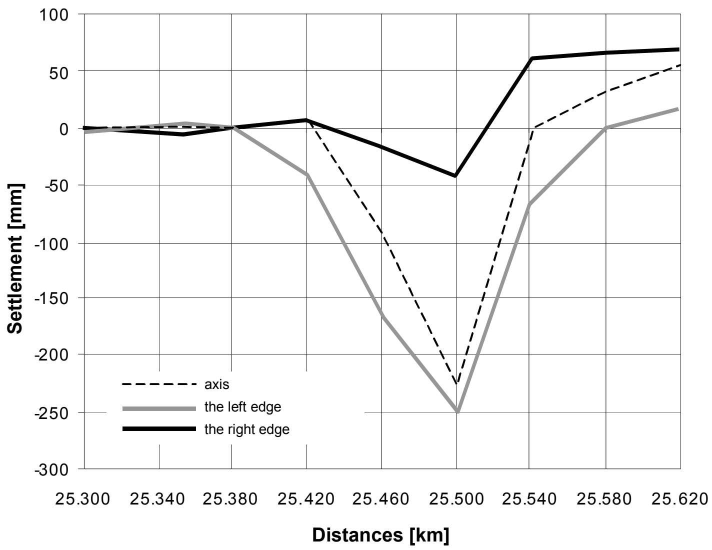

Fig. 5. The results of the measurements of the settlement of the embankment (in its axis, at the left and right edges of the road)

The predicted final settlement of the embankment was 41 cm and the analysis of the results of the measurements showed the maximum settlement of the embankment approaching 25 cm (within ca. 43 months). The highest values of settlement were observed in the axis and at the left edge of the road, and the lowest – at the right edge. This testifies to the possibility of further settlements of the embankment within approximately four years. Once the embankment has become firm, major repairs of road surface are being planned. In the meantime, a maintenance work is being done.

On the one hand, the example presented above shows the effects of the embankment settlement on the condition of road surface; on the other hand, it shows the strategy of investor, who because of substantial costs did not make a decision to reinforce the subsoil. Another factor hindering the reinforcement of subsoil was no possibility of making the detours to avoid the road in question.

3. EMBANKMENTS ON MINING-AREA SUBSOIL

3.1. CONCEPTS FOR MODELLING THE INTERACTION OF AN EMBANKMENT WITH MINING-AREA SUBSOIL

Recently a number of projects have been carried out on a large scale in the field of land transport infrastructure engineering with regard to motorways, roads and bridges as well as accompanying facilities. The construction is often located in the areas that used to be exploited for industrial purposes and are characterized by complex geotechnical conditions. The construction of motorways in mining areas requiring special protection against mining deformations is a new and complex issue. The subterranean exploitation of mines has a negative impact on the surface and the structures erected on it, which results in: a change in water relations, displacements of an orogen and its tremors causing vibrations in subsoil.

What still remains an unresolved question is the interaction of a transport earthen structure with subsoil in mining areas. There are no strict regulations and guidelines for the design of earthen structures (slopes, embankments and earth dams) and their protection against mining deformations. So far no complex approach to the design of embankments in mining areas in accordance with bearing capacity and use limit states has been adopted. Mining effects are especially dangerous if they appear in the conditions of tension strains (in the convex part of the subsiding trough), which causes ravelling of both subsoil and embankment. This leads to non-uniform settlements of the embankment top, local subsidence as well as horizontal displacements of roadways, and ultimately to safety-threatening of local earth slides and the damage to the surface. The settlements of motorway embankment resulting from ravelling of mining-area subsoil is considered to be the current problem.

This study presents the theoretical bases as well as the concepts of modelling the interaction of an embankment with mining subsoil and predicting its deformations. They are mainly focused on examining the effect of mining tension strains and on the safe operation of road constructions.

The considerations concern continuous deformations of the surface. As a result of the exploitation of mines, the so-called subsiding trough is formed on the surface, which is characterized in its every point by: settlement, slope, curvature and soil horizontal strain (figure 6).

The theories of orogenesis are based on geometric models defining a vectorial displacement field of orogen adopted as a continuous medium. In practice, the most frequently applied theory is that of Budryk–Knothe [2], [6]. Extreme values of area deformation index determine the category of a mining-area deformation.

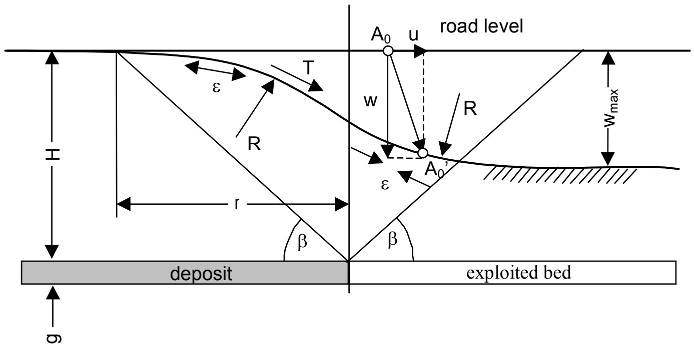

Fig. 6. Model of the subsiding trough established – profile of vertical displacements: w – vertical displacement, T – slope, R – radius of curvature, ε – horizontal strain, H – depth of exploited bed, g – thickness of exploited bed, β – angle of mining effects range, r – radius of principal effect range

The difficulties emerge when one attempts to construct mathematical models for mining-area subsoil. Currently, in order to define the processes occurring in the structure and its foundation, the models of the ground mechanics – elastoplastic models – are used. The computer software packages developed with a view of their application in geotechnics, based on the finite element method, define the soil medium by dint of the linear elastic–perfectly plastic constitutive model. It is impossible to carry out a theoretical analysis of the above-mentioned issues without the conduct of experimental research and its continuous improvement. The interpretation of the findings of numerous field and laboratory tests on cohesive and cohesionless soils, employing the methods of structural soil mechanics, constituted the basis for empirical dependencies that determine the reduction in the strength parameters of soil being subjected to the process of ravelling [3], [6]. Above all, the reduction of soil cohesion, because of its internal friction angle, virtually remained stable. The changes in the soil cohesion occurring during the process of its ravelling can be represented by the following relation:

$$c_R = c_0 \cdot \left(1 - \frac{\varepsilon_R}{\alpha_R + \varepsilon_R}\right) + c_{\mathrm{cr}} \cdot \frac{\varepsilon_R}{\alpha_R + \varepsilon_R}, \tag{6}$$

where:

c_0 – the standard soil cohesion (if $\varepsilon_R = 0$);

c_R – the reduced soil cohesion if ($\varepsilon_R > 0$);

c_{cr} – the critical cohesion when a decrease in the soil resistance is stabilized;

α_R – the coefficient representing the cohesion reduction (when ravelling of infirm made ground $\alpha_R < 1.5$);

ε_R – the horizontal ravelling strain.

3.2. ASSESSMENT OF EMBANKMENT STABILITY AND BEARING CAPACITY

An overview of the methods allowing assessment of slope stability of an earthen structure on mining-area foundation as well as the analysis of the results of calculations have been presented in the authors' previous works [10], [11], in which the boundary strain values for embankment soil (ε_n) have been compared with boundary strain values for subsoil (ε_p) with respect to the predicted mining-area strains. It has been concluded that the block methods used in engineering are ineffective in assessing the embankment stability and that numerical analyses based on the Finite Element Method are of great use. They allow determining simultaneously both the bearing capacity and the strain of an earthen structure (in the flat and spatial states of strain). The patterns established for the embankment bearing capacity loss are substantially consistent with those of the block method of stability assessment presented by Kezdi (4).

The numerical analysis with the Finite Element Method (FEM) is the most general method allowing one to determine both bearing capacity and – at the same time – the strain of an earthen structure [1], [9]. Examples of the available geotechnics-oriented FEM computer programs include HYDRO-GEO [14], PLAXIS and Z_SOIL. Although they allow a number of limitations, e.g. analysis only in the flat state of strain or in axial symmetry as well as no possibility of intervening in the material model, they are appropriate for engineering purposes. Moreover, there are some general programs, such as COSMOS and ABAQUS [15], based on an advanced Finite Element Method. Their rich library of material models and finite elements and the possibility of analysing spatial problems make them widely applicable for the analyses of stress and displacement in numerous static and dynamic problems of mechanics. However, the resulting problem is laborious data input and a huge computing power required. While analysing geotechnical constructions with the ABAQUS program, the most commonly applied soil constitutive models are those of the Coulomb–Mohr and of the modified Drucker–Prager [3], [10], [11].

3.3. SETTLEMENTS OF AN EMBANKMENT AS A RESULT OF THE SUBSOIL RAVELLING

An example of calculating the settlement of an embankment that is being made at the section of the Wrocław–Kraków A4 motorway, between grade-separated intersections at Sośnica and Wirek, founded in the areas that used to be exploited for mining purposes has been presented.

Apart from the effect of a massive embankment on the predicted values of subsidence in the area of mining-area impact and at the same time on the settlements of the embankment top, a crucial issue in the design phase is to allow additional settlements of an embankment due to the ravelling of subsoil. This has been illustrated with the example of calculation to verify the serviceability of II–II section in limit state. In such a case, excessive settlements of the embankment top, while making the structure, have been observed (figure 7 and table 1).

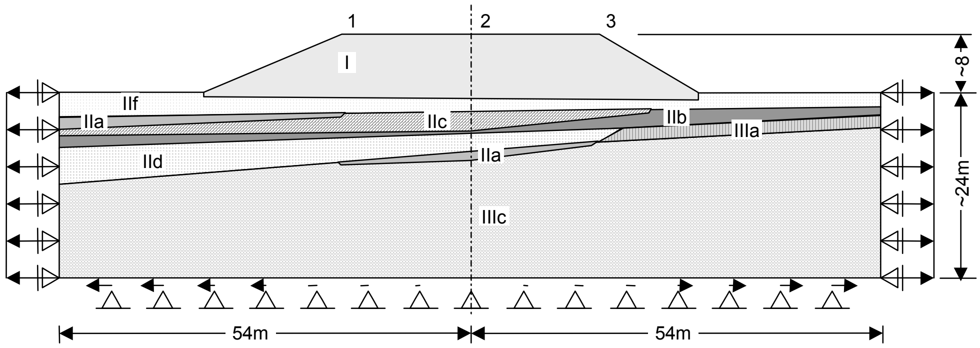

Fig. 7. The static model of an embankment in II–II geotechnical section with modelled ravelling of subsoil (ε = 3 mm/m)

Table 1

A compilation of geotechnical parameters in II–II geotechnical section

Layer	Symbol	Soil	$\gamma^{(n)}$ [kN/m^3]	$c_u^{(n)}$ [kPa]	$\phi^{(n)}$ [°]	$E_0^{(n)}$ [kPa]	ν [–]
I	N	Embankment	18.0	10.00	30.00	60 000	0.30
IIa	Gπ	Dusty clay	19.3	6.50	7.92	8 463	0.32
IIb	Π	Dusty soil	20.6	10.64	11.60	14 899	0.32
IIc, IId	J	Clay	20.5	55.00	14.30	18 800	0.37
IIe, IIe1	Pπ	Dusty sand	19.0	0.00	30.00	38 270	0.30
IIf, IIf1	Ps	Medium-grained sand	20.0	0.00	33.00	66 922	0.25
Iii	Gp	Sandy clay	21.0	27.01	15.84	20 860	0.29
Iii	Gp	Sandy clay	22.0	34.26	19.57	29 593	0.29
Iii	Gp	Sandy clay	22.5	40.00	22.00	48 893	0.29

There is analysed an embankment in the flat state of strain and in the full section because of diversified geotechnical conditions. Its geometrical dimensions are as follows: height – 8.0 m, width of the top – 36.0 m, inclination of the slopes in both sections 1:1–1:2, thickness of subsoil layer – 24.00 m (depth at which additional stresses are by 30% greater compared tothe primary stresses); width of the embankment footing – 108.00 m. The caluclations were performed by using the FEM – ABAQUS computer software package. The stages of FEM analysis with the Drucker–Prager condition of plasticity were as follows: introduction of initial stresses to the subsoil (0); simulation of the embankment erection (I); modelling of the ravelling of subsoil (II); modelling of the subsoil compression (III). In comparative calculations, Coulomb–Mohr's condition of plasticity was taken into account. Characteristic values of material parameters in Coulomb–Mohr's model were transformed into Drucker–Prager's model (table 2). In order to describe the elastic phases of the isotropic activity of the soil medium, a linear dependence of strain on stress expressed by the parameters E and ν has been adopted. The cohesion reduction, which depends on the ravelling of subsoil, cannot be described in this manner, because the parameters of soils were determined on the basis of samples collected following the occurrence of mining deformations. In cohesive soils, with the Drucker–Prager's condition of plasticity taken into account, the protection of the material against shearing was defined, and in the case of cohesionless soils – against compression.

Table 2

Material parameters in Coulomb–Mohr's and Drucker–Prager's models

Layer	Symbol	$c_u^{(n)}$	$\phi^{(n)}$	Non-associated flow rule $\psi = 0$			$E_0^{(n)}$	ν
		[kPa]	[°]	β [°]	d [kPa]	σ_c^0 [kPa]	[kPa]	[–]
I	N	10.00	30.00	40.89	15.00	21.09	60 000	0.30
IIa	Gπ	6.50	7.92	13.42	11.15	12.11	8 463	0.32
IIb	Π	10.64	11.60	19.20	18.05	20.42	14 899	0.32
IIc, IId	J	55.00	14.30	23.16	92.31	107.66	18 800	0.37
IIe, IIe1	Pπ	0.00	30.00	40.89	0.00	0.00	38 270	0.30
IIf, IIf1	Ps	0.00	33.00	43.33	0.00	0.00	66 922	0.25
IIIa	Gp	27.01	15.84	25.30	45.01	53.43	20 860	0.29
IIIb	Gp	34.26	19.57	30.12	55.91	69.32	29 593	0.29
IIIc	Gp	40.00	22.00	32.98	64.24	81.96	48 893	0.29

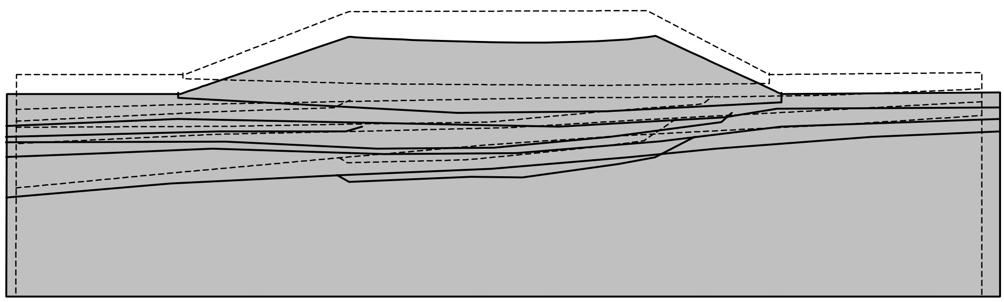

Fig. 8. The deformed model of an embankment in II–II section after the introduction of tension deformations $\varepsilon = 3$ mm/m

An embankment underwent a deformation when it was loaded with its deadweight, and the subsoil in the convex part of the mining trough slope was subjected to the ravelling (figures 8, 9). Against the background of the deformed embankment (broken lines), a flexure of the embankment top and increased vertical displacements of layers under the central section of the embankment are visible. The increment in the settlement of the embankment top on the soil subjected to tension deformations (ε = 3 mm/m, the 2nd category of mining effects) approaches 10 cm.

A total settlement of an embankment is a sum of mining subsidence, settlement occurring in the stage of erection and service of the structure as well as of displacement resulting from the ravelling and compression of subsoil.

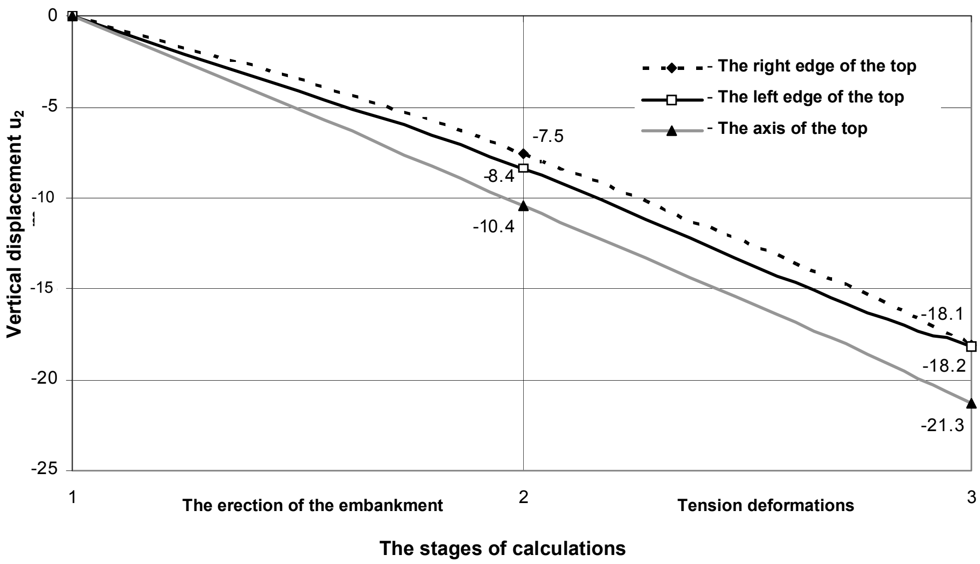

Fig. 9. A pictorial diagram of an increment in the settlement of the embankment top (in the stages of the embankment erection and the subsoil ravelling)

While performing the full static and strength analyses in the assessed section of the embankment (assessment of vertical displacements and the state of stress and plastic strains) it has been observed that:

- An increment in the settlement of the top during the erection stage and an increment in displacements during the ravelling of subsoil stage approached 10 cm in both cases.
- An increment in the settlement in the axis of the embankment is bigger than that at the edges of the top by approx. 2 cm during the erection of an embankment and reaches the value of 3 cm during the ravelling of subsoil.

• An introduction of tension deformations (of the order of 3 mm/m) results in a double increase in vertical displacement (an increase by 10 cm during the erection of an embankment up to 20 cm during the ravelling of subsoil).

• An increment in vertical stress determines an increment in the settlement of an embankment in the elastic phase of soil activity; after erection of an embankment, in the subsoil the plastic zones have not been developed yet, but during the ravelling of subsoil substantial sections of an embankment and subsoil undergo plasticization, and an increment in settlement is inelastic and depends on the value of plastic strains.

• During erection of an embankment an increment in plastic strains is slight; the analysed section II–II (figure 7) comprises the lens and the layer of dusty clay ($I_L = 0.63$) as well as the layer of silt ($I_L = 0.40$) under the central section of the embankment.

• During the stage of modelling the tension deformations, the embankment body and its subsoil undergo total plasticization; the biggest increment in plastic strains is visible at a depth comparable to the height of the embankment (in sandy clay layers). High values of plastic strains also occur in the near-surface layer of dusty sand and in the embankment body.

4. CONCLUSION

The analyses of settlement of transport embankments presented show how the serviceability limit state is important in embankments. This state should also indicate whether the deformation of an embankment will not lead to the loss of its suitability for use. If an embankment is well consolidated and the load applied to the carrying ground is slight, then deformations caused by the embankment deadweight or the load will not exceed the admissible values. It is also necessary to predict the possibility of strains occurring as a result of the changes in water conditions, including prolonged settlements caused by the changes in the moisture content of embankment soil and subsoil.

Embankments erected on the soils of low bearing capacity are subjected to substantial deformations due to their compressiblity and subsoil plastic strains. While building and using them, we have to reckon with the loss of boundary bearing capacity of organic subsoil, sinking of an embankment and upthrust of weak subsoil as well as loss of embankment bearing capacity, but also with a substantial and prolonged settlement of organic subsoil. We can deal with such adverse conditions due to a proper design, the choice of the appropriate way of foundation and the technology of making embankments as well as monitoring of these structures. The methods used to assess the serviceability limit state of these transport embankments should make use of compressibility parametres of oragnic subsoil obtained from properly conducted laboratory tests and formulated correlations.

The settlements of embankments caused by the ravelling of mining-area subsoil is a complex issue. The results of calcutions presented show that an increment in settlement due to the ravelling of subsoil is substantial and cannot be ignored while determining the displacements of the embankment top. High embankments on infirm cohesive soil are especially susceptible to the ravelling of subsoil. What has a major influence on the settlement are mining deformations affecting embankment. Even slight values of horizontal strain (tension and compression) applied periodically can be responsible for substantial increments in the settlement resulting from successive mining effects summing up. What has a slight impact on the settlement is material models of soils adopted during the calculations.

A mattress made of high-density polyethylene geogrids with rigid nodes and filled with stony material takes tension stresses appearing during soil ravelling. A mat under the embankment footing does not, however, reduce the embankment body displacements caused by mining effects.

The FEM ABAQUS software package was used for the numerical calculations which were carried on the computers of the Wrocław Centre of Networking & Supercomputing.

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