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Original article

Technical aspects of decision-making in communication engineering

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ABSTRACT

The subject of the article are problems of safety in the operation of earth structures designed for vehicle traffic in the conditions of threats of loss of stability. In particular, the following issues were discussed: design and construction of embankments on weak, marshy soil, safety of earth structures in floodplains, safety of earth structures in landslide areas, earth structures located in areas of mining damage.

Examples of solutions to particular problems are given. The summary contains statements of utilitarian and cognitive significance.

KEYWORDS

transport earth structures, threats, operational safety

Introduction

The safety of earth structures' transport operation depends to a large extent on the technical quality of the ground substrate, characterized by adequate shear strength, which is a parameter used to determine the load capacity. In geotechnics, however, the bearing capacity of the substrate is expressed in an analytical way as a limit load. A derivative of the application of boundary (destructive) load is the occurrence of a boundary stress state in the substrate and significant settlement of the structure [1-4]. The value of settlement of a building in this (boundary) state of the substrate is the sum of two components: $s = s_w + s_p$, where s_w is settlement due to the compressibility of the soil, s_p – settlement due to the plasticization of the substrate under the foundation and the associated phenomenon of displacement of the ground from under the structure (e.g.: the embankment or road paving).

1. Introduction to the issue of soil bearing capacity

The stress limit state condition at any point of the substrate is determined according to the Coulomb-Mohr hypothesis by the value of tangential stress τ equal to the ground shear resistance τ_f [5]:

$$|\tau| = \tau_f = \sigma \tan \varphi + c \tag{1}$$

where:

σ – normal stress at the analyzed substrate point,

φ – internal ground friction angle,

c – cohesion.

Depending on the principal stresses, the limit state condition at a given point in the substrate is determined by the following formula (designations as in the previous formula) [5, 6]:

$$0.5\,(\sigma_1 + \sigma_3)\sin\varphi = 0.5\,(\sigma_1 - \sigma_3) - c\cos\varphi \tag{2}$$

For cohesive soil, when $c \neq 0$ and $\varphi = 0$, the above formula takes the following form: $\sigma_3 = \sigma_1 - 2c$.

Another form of the above formula is known [5, 6]:

$$(\sigma_1 - \sigma_3)\,(\sigma_1 + \sigma_3 + 2\,\sigma_c) = \sin\varphi \tag{3}$$

where:

$\sigma_c = c\,\mathrm{ctan}\,\varphi$ – the so-called initial stress at the stress limit state in cohesive soil, when $c \neq 0$ and $\varphi \neq 0$.

For non-cohesive (loose) soil, when $c = 0$ and $\varphi \neq 0$, the stress limit state condition takes the following form [5, 6]:

$$(\sigma_1 - \sigma_3)\,(\sigma_1 + \sigma_3) = \sin\varphi \tag{4}$$

Determination of the substrate limit loads q_f is performed on the basis of the equation for the stress limit state in the ground base. An approximate formula coined by L. Prandtl, A. Caquot and J. Kerisel [5, 6] is known:

$$q_f = [\gamma_0 D\, e\pi^{\tan\varphi} \tan^2(0.25\,\pi + 0.5\,\varphi)]\,[1 + 0.25\,B\,(D)^{-1}\,e^{0.5\,\pi\tan\varphi}] +$$

$$+\,c\cot\varphi\,[e^{\pi\tan\varphi}\tan^2(0.25\,\pi + 0.5\,\varphi) - 1] \tag{5}$$

where:

γ_0 – volumetric weight of the soil (e.g. an embankment) above ground level,

D – height of the ground embankment above the ground level or depth of foundation of the road structure in the trench,

B – width of the structure transferring the load to the ground (usually 1 linear meter for a road or a railway track).

Length L of the structure transferring the load to the ground (L is taken to be the width of the roadway or a railway track) is given in a formula developed by K. Terzaghi-E. Schultze [5]:

$$q_f = [1 + 0.3\,B\,(L)^{-1}]\,c\,N_c + \gamma_D\,D\,N_q + [1 - 0.2\,B\,(L)^{-1}]\,\gamma_B\,B\,N_\gamma \tag{6}$$

where:

N_c, N_q, $N\gamma$ – coefficients depending on the value of the internal friction angle of the ground under the structure.

In the case of a non-homogeneous substrate, the following volumetric weights of the soil should be taken into account: γ_D – next to the structure loading the substrate γ_D and γ_B in the substrate below the structure. Other designations – as in the previous model.

The empirical formula proposed by A.W. Skempton [5, 7] is also worth noting:

$$q_f = c\,N_c^* + \gamma_D D \tag{7}$$

where:

- c – cohesion of soil under a loading structure,
- N_c^* – coefficient depending on the quotient of D/B (adopted on the basis of Z. Wiłun's tables [5]; B – as in the above formulas),
- γ_D – volumetric weight of the soil next to the loading structure,
- D – recess in the ground.

According to Z. Wiłun [5], the formula proposed by A.W. Skempton should be used for cohesive soils, characterized by the value of $\varphi \approx 0°$, whereas the formula proposed by K. Terzaghi-E. Schultze is valid for soils with an angle of internal friction $\varphi > 0°$.

Earth structures (embankments, cuts) founded on the ground or in the ground constitute the foundation of the road/railway paving, which transfers static and dynamic loads and cooperates with the substrate indirectly. At present, the operational quality of the substrate is crucial in the upgrading of transport routes, e.g. on high-speed railway lines where small tolerances are permitted in track positions, and on high-traffic lines, where substrate and track deformations increase rapidly.

Quite often road and railway tracks are inevitably built in areas with poor substrate conditions. These structures may be built only after a technical assessment of the substrate has been carried out, and under specific conditions, a decision is made to use one of the many available systems ensuring the safe operation of these structures [1-3, 8-13]. Several such cases are noted in this article.

2. Stability of embankments on weak, marshy substrate

Embankments erected on a weak, marshy substrate are subject to significant deformations due to their compressibility and plastic deformations of the marshy soil. Plastic deformations are the result of shear stresses in a weak substrate, the value of which exceeds their shear strength. As the load on the embankment increases, stresses in the weak substrate increase and cause the plasticization zone to increase, which results in an increase in the settlement rate of the embankment. When an embankment has reached a certain height limit for a given weak substrate, destructive phenomena may occur: catastrophic displacement of weak layers to the sides, landslide of embankment slopes and rapid settlement of the embankment (Fig. 1) [5].

Deformations of the substrate, in excess of the limit values, are unacceptable during the lifetime of the earth structure. However, they can be beneficial during the construction period if, due to their occurrence, the embankment base reaches the level of a load-bearing subsoil at the bottom of the marsh. In the case of highway and expressway construction, it is recommended to remove the weak bearing layers, e.g. by having them be pushed aside under the weight of the embankment [5, 9, 10, 13, 14]. When building secondary roads, weak layers

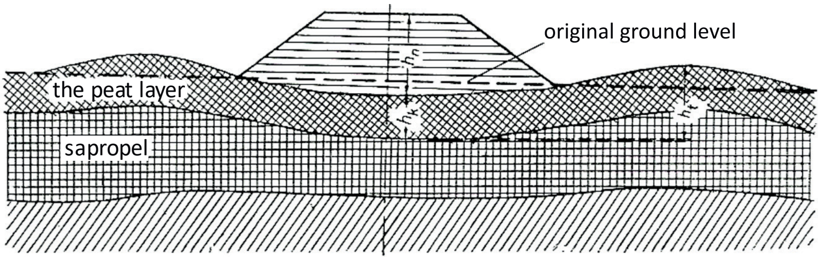

Fig. 1. Settlement of the embankment on muddy ground with simultaneous displacement of weak soil under the embankment
Source: [5].

can be allowed to remain in the substrate, in which case the long-term settlement of the embankment is to be expected. It is then necessary to check whether the permissible value of the safety factor F_w is not exceeded due to the possibility of catastrophic soil displacement from the embankment during the road operation period [5, 9, 10, 14]:

$$F_w = q_f(\sigma_z)^{-1} \geq 1.0 \tag{8}$$

where:

q_f – limit load of the weak layer situated directly under the embankment [MPa],

σ_z – vertical stresses σ_z acting in the ceiling of the weak layer are calculated using the following formula [5, 14]:

$$\sigma_z = q + \gamma_n h_n + \gamma_k h_k \tag{9}$$

where:

q – uniformly distributed vertical load, derived from the dead weight of the embankment and the operational load [MPa],

γ_n – volume weight of the ground material of the embankment [kN/m3],

h_n – height of the finished embankment, taking into account the expected settlement [m],

γ_k – volumetric weight of the peat layer situated between the bottom of the embankment and the ceiling of the weak layer (Fig. 1) [kN/m3],

h_k – thickness of the peat layer [m].

The weak soil underneath the embankment is displaced at $F_w < 1.0$.

Figure 2 presents an embankment erected on mineral substrate at the bottom of the marsh (completely filled with peat of solid consistency) after the complete removal of the peat deposit [9]. This foundation method is recommended when the thickness of the peat layer does not exceed 2-3 m. The solution guarantees maximum stability of the embankment, as it will be based on a stable, load-bearing substrate.

Figure 3 illustrates the solution for crossing marshes such as in Figure 2, for embankments $H \leq 8$ m high and for peat layer thickness of $h > 3$ m [9]. This is an example of an embankment partially submerged in peat deposits, after its local removal.

Building an embankment on irrigated marshes, with floating peat carpets, it is deposited at the bottom of the peat bog. There are two technological possibilities [5, 9]:

Fig. 2. Embankment founded on mineral soil at the bottom of a marsh completely filled with peat of solid consistency after the previous, complete removal of the peat layer
Source: [9].

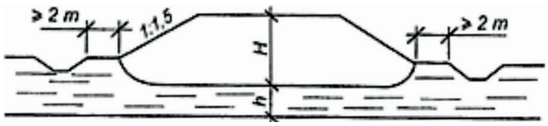

Fig. 3. Embankment which is partially submerged in peat deposits, after its partial removal. The solution is applied when crossing marshes filled with peat of solid consistency: *H* – height of the embankment with its part below ground level, *h* – thickness of peat layer, left under the bottom of the embankment
Source: [9].

- the embankment is placed on the mineral bottom, after the peat layer has been removed (Fig. 4a),
- the embankment is placed on a peat layer, previously cut on both sides of the embankment base to allow vertical movement of the carpet towards the bottom of the mineral marsh (Fig. 4b).

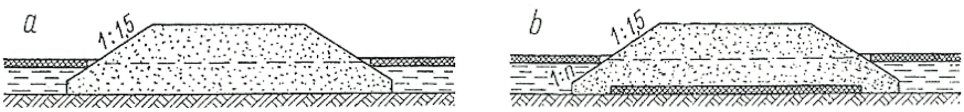

Fig. 4. Cross-sections of embankments founded on irrigated marshes:
a – embankment on the mineral marsh bottom, after removing the peat carpet,
b – embankment on a peat carpet lowered to the mineral mud bottom (under the weight of the embankment)
Source: [5, 9].

Figure 5 shows an example of partial removal of weak bearing soil in marshes which are completely filled with solid peat, by making side cuts to the bottom of the mineral mud along both edges of the embankment base. This treatment prevents the peat from being displaced sideways [5, 9]. Instead of this solution, a series of such longitudinal cuts can be made in the base of the embankment spanning approximately three peat thicknesses. This will facilitate the rapid drainage of water from the peat deposits. These cuts, both lateral and longitudinal, in the base of the embankment, are filled with sand directly after the execution, before the embankment is built.

Apart from classic methods of embankment construction on organic soils, special technologies are used, e.g. [5, 8, 9]:

- the use of explosives to loosen peat deposits and plunge the embankment to the mineral bottom,
- building vertical sand drains into the base of the embankment.

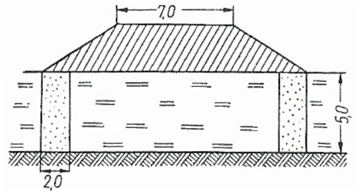

Fig. 5. Cross section through the embankment with vertical side cuts to the mineral mud bottom. A solution facilitating quick drainage of water from peat layers
Source: [5, 9].

The essence of sand drainage is to shorten the water filtration path in the soil by introducing vertical drains spaced at short distances from one other (3-4 m). In this way, the subsidence and stabilization of embankments can be accelerated by dynamizing the process of self-compaction of the embankment under its own dead weight (Fig. 6).

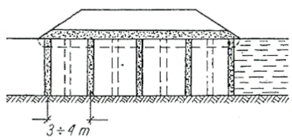

Fig. 6. Cross section through the embankment with vertical sand drainage installed in a staggering pattern in peat substrate, spaced 3-4 m apart
Source: [5, 9].

The precursor of the vertical drainage method has been pioneered in the US (California) [5], where a 1 m high road embankment was built on peat bog soil drained by means of sand drainage.

3. Earth structures on flood plains

Traffic embankments built on floodplains, which include the vicinity of rivers and water reservoirs, directly next to the earth body, are at risk of being washed out and soaked. Water seepage into the embankment reduces adhesion and friction between soil particles, causing a reduction in the stability of slopes and the embankment body. Seasonally, several times a year, rising water levels cause of a ground water depression curve to be formed inside the embankment (Fig. 7) [15]. The shape of this curve can be concave or convex, depending on whether the outside water level on both sides of the embankment rises or falls. Flood water is particularly dangerous and creates a significant difference in groundwater levels between the embankment and the outside water. The water pressure generated by this difference can cause the embankment slope to “wash away” (slip). For these reasons, embankments built on floodplains are erected according to special rules [13, 15, 16]:

– a variable inclination of the slope is designed (Fig. 8), milder on the side of the inflowing water (from 1:2 to 1:3) and normal on the opposite side (1:1,5); the underwater part of the embankment has a milder slope than the above-water part and is protected by a stone floating; the protruding crown of the embankment above the great water is placed at the height of min 0.5-1.2 m,

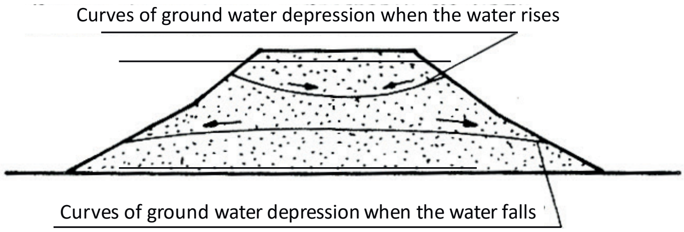

Fig. 7. Curves of ground water depression in embankment in floodplains:
when the water rises on both sides; when the water falls on both sides
Source: [15].

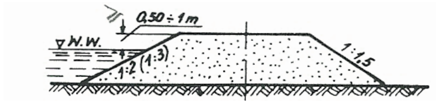

Fig. 8. Cross section through the embankment in a floodplain area.
The inclination of slopes is more gentle on the incoming water side (from 1:2 to 1:3),
and normal on the opposite side (1:1.5)
Source: [15].

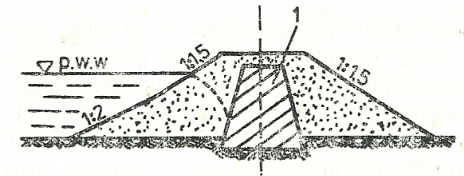

Fig. 9. Cross section through the embankment in a floodplain area.
The body of the embankment contains a core from impermeable soil:
1 – embankment core, p.w.w – great water level
Source: [16].

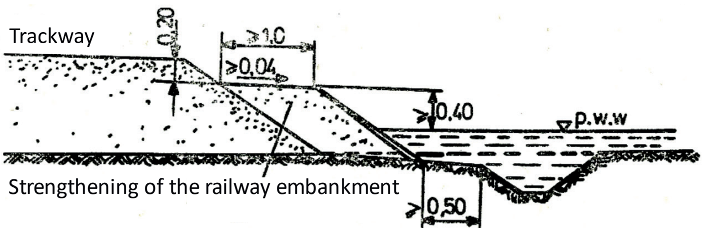

Fig. 10. Railway embankment built on flood plains with a buttress,
which strengthens the embankment
Source: [16].

- core of impermeable soils, e.g. clay, loam, is installed inside the earth body (Fig. 9) which prevents water from seeping to the other side of the embankment,
- in embankments, buttresses are made on access roads to bridges to strengthen embankments; the crown of the bench should rise at least 0.4 m above the level of great water (Fig. 10); buttresses protect the embankment from flood water infiltrating into the embankment and ensure greater stability of the embankment,

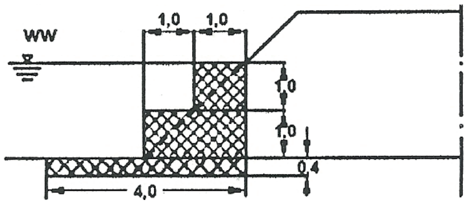

Fig. 11. The embankment slope is permanently or periodically under water, protected by gabions
Source: [17].

– slopes of embankments which are permanently or periodically under water can be protected against washing out using gabion structures, i.e. mesh baskets filled with stone material (Fig. 11).

4. Earth structures in landslide areas

The risk of landslides of embankments and excavations is often present in cases [6, 13, 14]:

– an excavation in interlayered soil, inclined towards the excavation (Fig. 12a),
– excavation in permeable soil, on a slope of impermeable soil (Fig. 12b),
– an embankment made of permeable soil, on a slope made of impermeable soil, in which no indentations have been made.

The likelihood of a landslide can be reduced if measures and technological solutions are implemented to prevent water from entering the slip area. For example, rainwater can be taken in by means of slope trenches installed above the slope of the excavation or embankment and routed out of an area where there is a risk of slipping (Fig. 13) [13]. Groundwater flowing down the slip surface should be intercepted by means of drains penetrated several

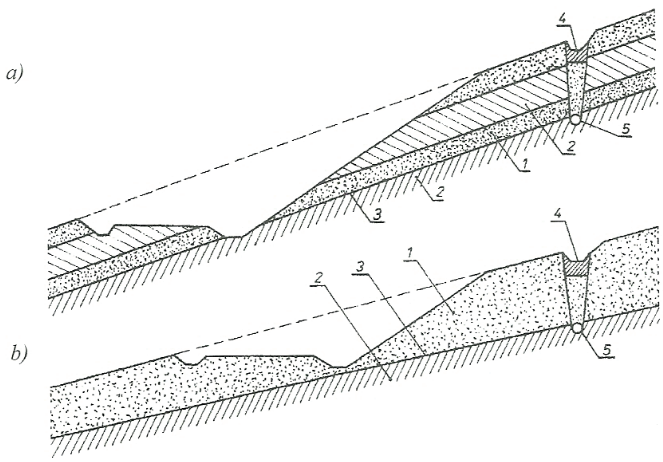

Fig. 12. Road excavations threatened by landslides: a) excavation in interlayered soil, inclined towards the excavation, b) excavation in permeable soil, on a slope of impermeable soil, 1 – permeable ground, 2 – impermeable ground, 3 – slip surface, 4 – slope trench, 5 – drainage pipe
Source: [13].

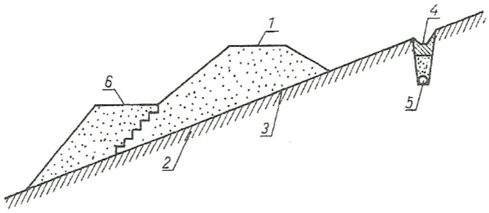

Fig. 13. Embankment in a landslide area, protected by a buttress: 1 – embankment made of permeable soil, 2 – impermeable ground substrate, 3 – slip surface, 4 – slope trench, 5 – pipe drainage, 6 – buttress made of permeable soil, anchored in the embankment using so-called cuts
Source: [13].

centimeters in an impermeable layer (Fig. 12). The water relations on the slope of the earth's structure can be regulated e.g. by the use of horizontal drains (Fig. 14), thus preventing the risk of landslide [14]. The most frequently used methods of road structure protection also include the following systems: stabilization with gabions, forming a retaining wall (Fig. 15) and using horizontal geosynthetic 'mattresses' (Fig. 16) [14].

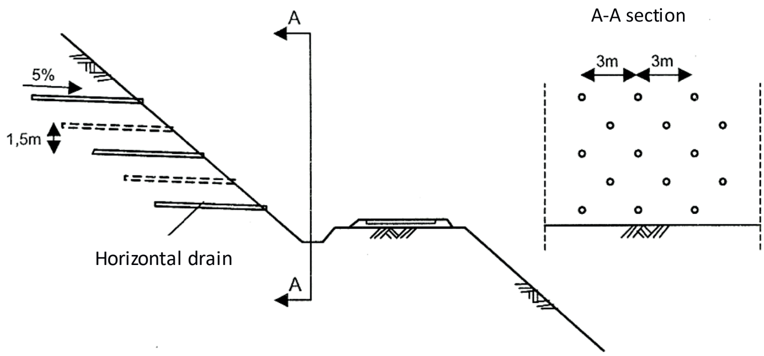

Fig. 14. Horizontal drains in the slope of a railway trench, made to arrange the water conditions, securing the slope against a landslide
Source: [14].

The risk of landslide is assessed on the basis of the minimum value of the equilibrium factor F_{min} as follows [5, 14, 18, 19]:

- very unlikely, when $F_{min} > 1.50$,
- unlikely, when $1.50 > F_{min} > 1.30$,
- likely, when $1.30 > F_{min} > 1.0$,
- very likely, when $F_{min} < 1.0$.

The safe values of the F_{min} coefficient, according to [14], are assumed:

- not less than $F_{min} = 1.30$ for building characteristics,
- for important structures $F_{min} > 1.50$,
- when $F_{min} = 1.50$, the slip area at the top of the road can be considered as the slip area.

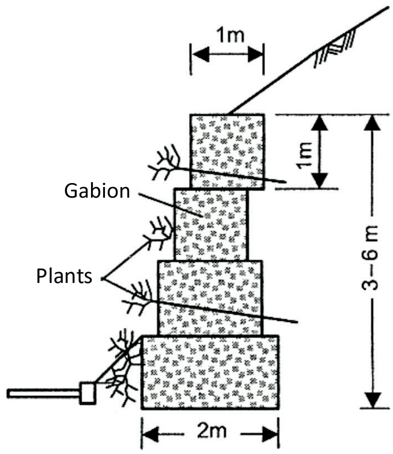

Fig. 15. Example of a retaining structure made of gabions protecting the slope of a road excavation against a landslide
Source: [14].

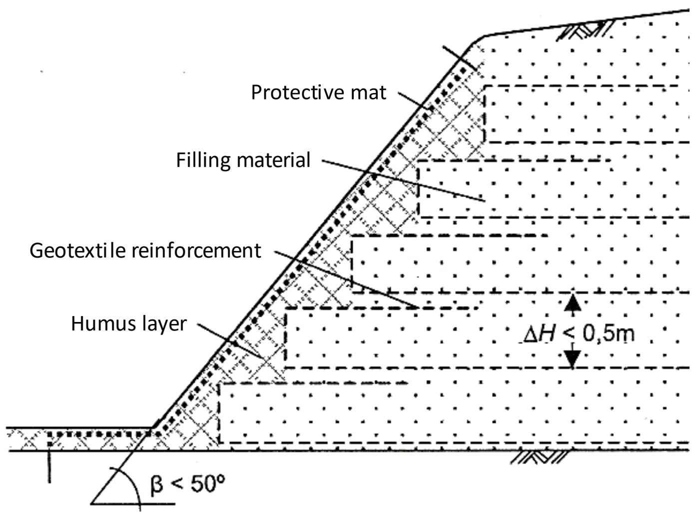

Fig. 16. Horizontal geosynthetic "mattresses" filled with granular material as a way of securing a road embankment against a landslide
Source: [14].

The appropriate number and location of geosynthetic layers, which will ensure the assumed value of the F_{min} safety factor (Fig. 16) may be designed using one of the block methods by which the cylindrical surface of the slip is assumed (Fig. 17) [14, 20]. In the "block" method of assessing slope stability, the sliding forces are [14, 20]: soil self-weight Q_i, ground water discharge pressure forces and operational load. Holding forces include frictional resistance and soil cohesion resistance T_i (Fig. 18). The geosynthetic insert "*j*" is inserted into the earth structure and properly anchored (Fig. 18), and cuts through the slip surface, creating an additional holding force G_j. The value of G_j is equal to the numerical strength of the design tensile strength of the geosynthetics.

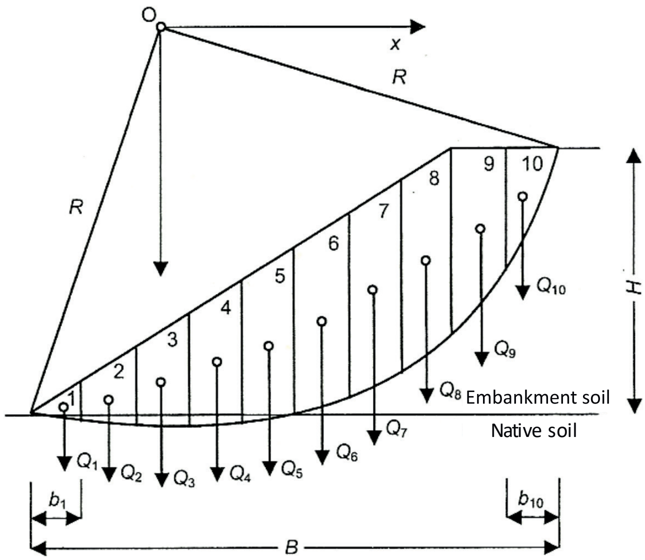

Fig. 17. Division of the earth structure into elements 1÷10 in case of using the block method for the assessment of stability of the embankment reinforced with geosynthetics: Q_1, Q_2, … – block weights
Source: [14, 20].

The weight of the Q_i block is distributed into the normal component N_i and the S_i tangent (Fig. 18):

$$Q_i = N_i + S_i = Q_i \cdot \cos \alpha_i + Q_i \cdot \sin \alpha_i \tag{10}$$

Force G_j applied by the geosynthetics ae distributed onto the normal component $G^N{}_j$ and the tangent $G^T{}_j$ (Fig. 18):

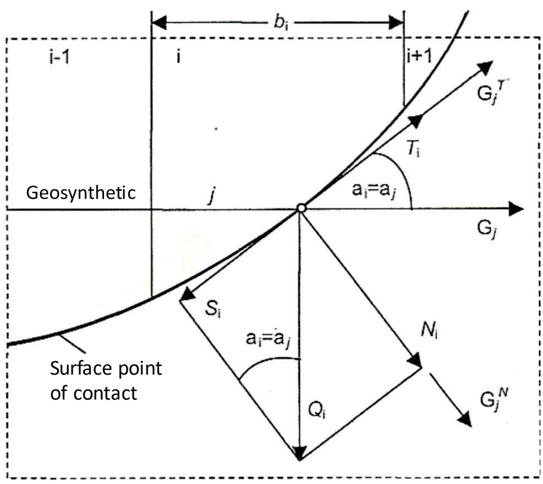

Fig. 18. Distribution of forces at the geosynthetic-surface point of contact: G_j – holding force generated by geosynthetics, T_i – resistance to friction and soil cohesion
Source: [14, 20].

$$G_j = G^N{}_j + G^T{}_j = G_j \cdot \sin \alpha_i + G_j \cdot \cos \alpha_i \tag{11}$$

The design of a ground structure reinforced with geosynthetics can be performed in two stages [14]. The first stage consists in the assessment of the stability of the slope without reinforcement. The minimum value of the safety factor is calculated using the following formula:

$$F_{min} = M_u (M_0)^{-1} = \sum T_i \cdot R \cdot (\sum Q_i \cdot x_i)^{-1} \tag{12}$$

where:

$M_u = \sum T_i \cdot R$ – holding moment which is formed by the friction resistance force and soil cohesion T_i relative to 0,

$M_0 = \sum Q_i \cdot x_i$ – rotating element generated by the weight of the Q_i block on the x_i arm relative to 0.

The second stage takes into account the increase of the holding moment after the introduction of the reinforcement. Then the value of the stability factor is:

$$F_{min,z} = M_{u,z} (M_0)^{-1} \geq F_{dop} \tag{13}$$

where:

$M_{u,z} = M_u + \Delta M_u = (\sum T_i \cdot R + R \cdot \sum G^T{}_j)$ – increased holding moment due to installed geosynthetic reinforcement,

$R \cdot \sum G^T{}_j$ – increase in holding moment due to the presence of geosynthetics.

According to [14], after taking a given F_{min} value, the tensile force transferred by the geosynthetic material should meet the following condition:

$$\sum G^T{}_j \geq (F_{min,z} - F_{min}) \cdot (\sum T_i)^{-1} \tag{14}$$

The required anchorage length of the geosynthetic insert to eliminate slippage is calculated as follows [14, 20]:

$$L_j = G_j (1.8 \cdot \gamma_0 \cdot z_j \cdot \tan \varphi)^{-1} \tag{15}$$

where:

G_j – design tensile strength of geosynthetics contained in layer j [kN/m],

γ_0 – volumetric weight of embankment soil [kN/m3],

z_j – depression of the geosynthetic insert in layer j [m],

φ – internal ground friction angle [°].

5. Engineering structures in mining damage areas

The main elements of the Polish transport system located in Upper Silesia, i.e. A1 and A4 highways and E30 and E65 railroad mains, are exposed to the effects of the impact of mining operations, which intensify certain serious problems related to the occurrence of mining damages. The coexistence of mining activities, combined with the need to maintain continuity and safety of road and rail traffic, constitutes a technical, technological and organizational challenge [10, 11, 16, 17, 21, 22].

The nature of the terrain deformations resulting from the mining of hard coal seams depends on many factors. Approximately 80% of these deformations are due to:

– mining with a ceiling infarction (i.e. without the use of a backfill) of shallow deposits,
– reactivation of old excavations under the influence of various factors, such as: changes in water conditions, surface loads, mining works in the deeper parts of the rock mass.

Deformations occurring in the areas covered where underground mining (above the mine workings) is conducted, are divided into [16, 17]:

– continuous, in the form of slowly increasing ground displacements, transmitted to the road or railway track surface (Fig. 19) [21],
– discontinuous deformations (Fig. 20) [17], in the form of local, rapid subsidence above the mine workings.

Continuous deformations cause the formation of a continuous depression, which is described by five deformation indicators (Fig. 19) [21]: terrain lowering *w*, terrain slope *T*, horizontal deformation ε, horizontal displacement *u*, curvature with the radius of *R*. These deformations pose a direct threat to the ground surface of traffic structures. They result in landslides, uneven subsidence of road or rail surface, transverse cracks in embankments, etc.

Discontinuous deformations are characterized by a sudden appearance of sinkhole funnels with a diameter of up to 10 m, landslides or, less frequently, linear deformations in the form

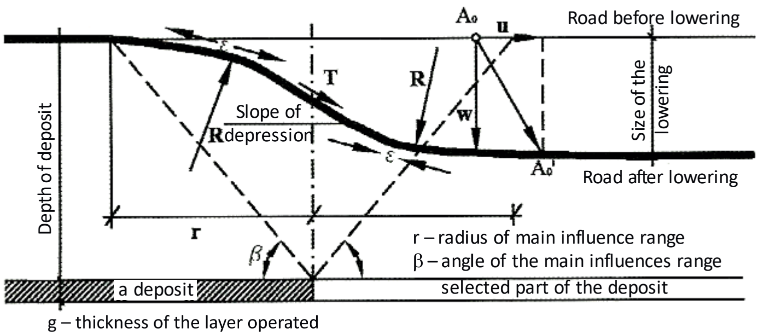

Fig. 19. A depression characterizing continuous deformations: *w* – vertical displacement [m], *T* – terrain slope [mm/m], *R* – radius of curvature [km], *ε* – horizontal deformation [mm/m], *u* – horizontal displacement [m]
Source: [21].

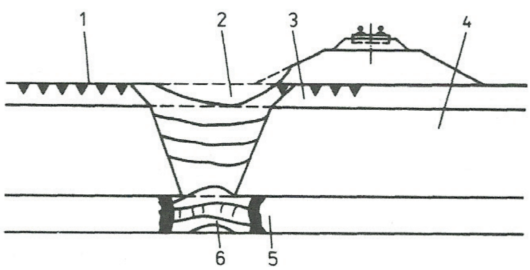

Fig. 20. Non-continuous deformation above mine workings. Cave-in settlement as a result of mining damage: *1* – ground surface, *2* – a sinkhole, *3* – a surface layer, *4* – native rock, *5* – a deposit, *6* – part of the rock mass filled with a caved cover
Source: [17].

of cracks, crevices, thresholds and humps. Deformations most often occur above shallow mining excavations, located above the so-called critical depth. This depth depends on the local geological conditions and usually reaches 50-100 m below the surface. Non-continuous deformations can also occur as a result of so-called rock bumps, or underground quakes, caused by cracking of rocks as a result of disturbing the balance of forces in the rock mass, tectonic movements or mining of deposits. Incontinuous deformations in the rail or road belt cause a safety hazard for the user. Article [21] draws attention to the most frequently observed, so-called linear discontinuous deformations of the terrain surface occurring in the road construction industry (Fig. 21) [21], associated with long-term mining, concentrated in one area. Such changes on the surface require immediate intervention by limiting speed and profiling the surface as quickly as possible. In extreme cases, the road section must be taken out of service until repair work has been carried out.

K. Kłosek [10, 11], among others, has been conducting advanced studies of the influence of discontinuous deformation of the mining substrate on the load-bearing capacity and stability of railway lines. Article [10] gives an example of discontinuous mining deformations in the area of railway line no. 149 (C-E65), located in the Upper Silesian Industrial District. A shaft sinkhole occurred at a distance of 80-100 m from this double-track railway line. As a result, the track bed was located in the coastal zone of the local depression basin. As a result, the technical condition of the substrate was significantly weakened, which led to the loss of the required load-bearing capacity. Deformations loosening the surface of the terrain (creep) transformed into a series of deep cracks and cracks reaching the direct zone of the railway track (Fig. 22) [10]. Before such alarming deformations occurred in the railway area, discontinuous deformations of the tracks had previously been recorded, as shown in Figure 23 [10]. The possibility of superpositioning horizontal compaction deformations of the mining ground with thermal deformations of the track, which may lead to buckling of the track in the vertical plane is highly undesirable (Fig. 24) [10].

If a disaster of a transport structure in the area of mining damage is imminent, structural safety devices are used to absorb the load in the event of loss of support for the road surface or railway track structure.

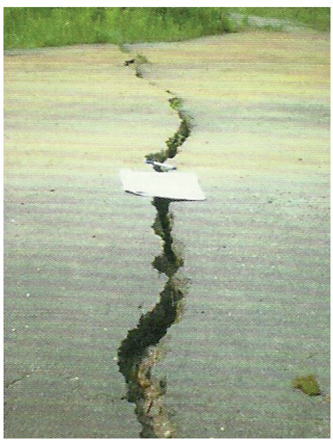

Fig. 21. Example of linear discontinuous road deformation of cement concrete pavement
Source: [21].

Fig. 22. Location of the PKP railway line no. 149 (CE-65) in relation to the range of discontinuous deformations
Source: [10].

Fig. 23. Track discontinuous deformations as a local fault under the railway line PKP No. 149 (CE-65)
Source: [10].

Fig. 24. Buckling of the track in the vertical plane as a result of superposition of horizontal deformations compacting the mining ground with thermal deformations of the track
Source: [10].

Figure 25 presents the design scheme adopted by K. Kłosek [11] for an embankment located on a mining base, with reinforcement at the level of the base. This diagram is an auxiliary element of the numerical model of the earth structure, founded in a mining area. It was assumed that a double-track railway line is located on the embankment. The separation layer between the mining substrate and the embankment is a mattress made of PVA geogrid (with a tensile strength of 400 kN/m), filled with stone material. Studies have shown the effectiveness of this type of reinforcement, e.g. after the application of a double geogrid located at the base of the embankment and at its top level, i.e. under the surface, the embankment together with the surface retains the features of elastic material in the area of horizontal loosening deformations (dispersions), the values of which range from 1.5 to 4.5 mm/m. The effectiveness of these reinforcements depends on a proper static analysis of the structure and selection of the geosynthetic reinforcement, taking into account its long-term strength, which guarantees the reliability of the structure's operation during its service life, i.e. not less than 50 to 120 years.

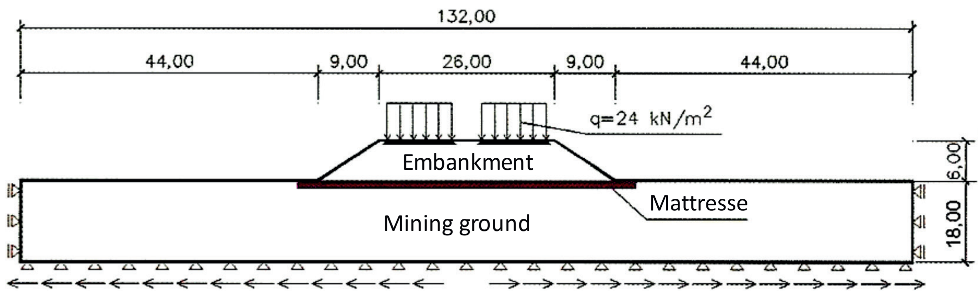

Fig. 25. Calculation diagram for a railway embankment located on a mining substrate, with horizontal reinforcement at the base level
Source: [11].

Summary

The problems of decision making in hazardous situations presented in the article are addressed only to selected operational instances of traffic earth structures (located on poor ground, floodplains, landslides) and engineering structures located in mining damage areas.

As we know, the spectrum of crisis events related to traffic engineering is much wider. For example, in relation to railway structures, the following procedures are important:

- after a flood (cases of damage to the ground, flooding and scouring of the track, destruction of engineering structures),
- in case of special events, threats to traffic safety and railway accidents (e.g. train rupture on track, fire in train, rail rupture, track deformation, electrical traction damage).

The subject therefore requires continuation in the scope of analysis of documentation of events and assessment of the conduct of relevant services and simulation tests.

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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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Techniczne aspekty podejmowania decyzji w inżynierii komunikacyjnej

STRESZCZENIE Tematem artykułu są problemy bezpieczeństwa eksploatacji komunikacyjnych budowli ziemnych w warunkach zagrożeń utraty stateczności. W szczególności omówiono następujące zagadnienia dotyczące: projektowania i budowy nasypów na słabym podłożu bagnistym, bezpieczeństwa budowli ziemnych na terenach zalewowych, bezpieczeństwa budowli ziemnych na terenach osuwiskowych, budowli ziemnych zlokalizowanych na terenach szkód górniczych.

Podano przykłady rozwiązania poszczególnych problemów. Podsumowanie zawiera stwierdzenia o znaczeniu utylitarnym i poznawczym.

SŁOWA KLUCZOWE komunikacyjne budowle ziemne, zagrożenia, bezpieczeństwo eksploatacji

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