Scientific Journal of the Military University of Land Forces
ISSN: 2544-7122 (print), 2545-0719 (online)
2020, Volume 52, Number 1(195), Pages 135-153
DOI: 10.5604/01.3001.0014.0269

Original article

Selected safety issues in designing engineering structures

Jacek Ryczyński[1]*, Piotr Saska[1], Andrzej Surowiecki[2], Krzysztof Ksiądzyna[2]

[1] Faculty of Management,
General Tadeusz Kosciuszko Military University of Land Forces, Wroclaw, Poland,
e-mail: jacek.ryczynski@awl.edu.pl; piotr.saska@awl.edu.pl

[2] Faculty of Security Sciences,
General Tadeusz Kosciuszko Military University of Land Forces, Wroclaw, Poland,
e-mail: andrzej.surowiecki@awl.edu.pl; krzysztof.ksiadzyna@awl.edu.pl

INFORMATIONS

Article history:

Submited: 2 August 2018

Accepted: 26 October 2018

Published: 16 March 2020

ABSTRACT

The article discusses the safety requirements specified in pertinent standards and recommendations for designing civil engineering objects, with particular emphasis on earth structures intended for vehicle traffic. The focus was on the following issues: the essence of reliability and durability of the structure and ensuring safety at the stage of designing vehicle traffic embankments with a slope supported with the use of a retaining wall and embankments placed on a substrate characterized by insufficient bearing capacity. The procedure for designing traffic embankments on weak ground, reinforcing weak ground and designing retaining structures (on the example of a reinforced soil massif) was carried out in accordance with the calculations pertaining to the field of geo-engineering, applying general analytical dependencies.

KEYWORDS

engineering structures, retaining structures, design, operational safety

* Corresponding author

Introduction

Engineering structures are a large group of structures including: bridges, culverts, tunnels, railways, roads, airports and retaining structures. When designing the structure, attention should be paid to the important time factor. The durability of any structure is limited and depends on the conditions of use, the value and nature of the applied loads (static, dynamic), changes in the properties of materials resulting from ageing, fatigue, creep and corrosion. The durability of the structure can be ensured through proper maintenance, repairs and compliance with the conditions for use, recommended in the design.

An important group of engineering structures are earth structures designed for vehicle traffic, e.g. road and railway embankments. Ground and coating bridge structures are an extraordinary structures of mixed design. The main focus of this work was on the last group – earth structures.

1. The essence of safety and durability of the structure

An engineering structure can be considered safe if it has been designed and built in accordance with the rules of construction art, meets relevant technical guidelines and standards, and is properly used [1-8]. The basic condition for the design and use of the structure can be formulated as not exceeding the load capacity of the N_k structure by the permanent and operational loads Q_e included in the formula:

$$Q_o \cdot \gamma_o + Q_w \cdot \gamma_w + Q_e \cdot \gamma_e < N_k \quad (1)$$

where:

Q_i are the internal forces applied by the natural load system, the equipment, and operational loads, while γ_i are the corresponding (standard) load factors.

The essence of structural dimensioning methods is to meet the above condition, with the safety reserve expressed on the left (1), in the values of load multipliers, and on the right (1) – in the material factors. Their values take into account the random nature of the permanent and external loads and the bearing capacity of the whole structure and its elements. Three conditions must be met in order to ensure the safety of the structure: strength, stiffness and stability. According to the effective dimensioning method, structural elements must be designed so that they do not exceed the limit bearing capacity and service states.

The strength condition given in (1) aims to check whether a given load of the structure does not cause damage to the whole or a part of the structure (e.g.: does not cause a rupture, crushing, fracture, cut). The method, in which a axial tensile force P_r has acted destructively on a rod, is good illustration of this condition. When the value of this force increases to the destructive value of P_{rn}, the rod will be torn apart. Theoretically, the rod will not be destroyed if $P_r < P_{rn}$. In practice, one should expect random deviations: of shape, dimensions, strength characteristics of the rod material and inaccuracy in the determinations the values of P_r and P_{rn} forces. The safety factor $n_b > 1$ must therefore be taken into account in the design process, and the strength condition, which is also a condition for the safety of the tensile bar, is recorded as:

$$P_r \leq P_{rn} \cdot (n_b)^{-1} \quad (2)$$

The value of the safety factor n_b depends on a number of factors, e.g. the calculation methods used, the type of load, the structural material and the reliability category of the structure. In the current method of limit states, the safety factors are separated and are treated as so-called partial coefficients relating to:

- the expected loads,
- the bearing capacity of the structural element, depending on the dimensions of the element and the strength characteristics of the material.

The strength condition in the limit state method is described by the following inequality:

$$\sigma_{obl} \leq R_{obl} \quad (3)$$

where:

σ_{obl} – stress determined taking into account the design load values,

R_{obl} – design strength of the material (designations for particular types of material, e.g. wood, steel, reinforced concrete are given in structural standards, e.g. for steel – f_d).

The stiffness condition concerns the limitation of the deformation of the structure. Movements exceeding limit values can lead to structural damage or hinder the use of the building (e.g. complicating the operation of machinery and equipment set up on the structure). The design standards specify the permissible value of vertical displacements, e.g. floor deflections or deflections of bridge beams f_{gr}. Fulfillment of the stiffness condition is determined by the following inequality:

$$y_{max}=f\leq f_{gr} \quad (4)$$

where:

y_{max} – the largest structural deflection due to load,

f_{gr} – limit deflection, considered acceptable.

For example, in the case of a simply supported, single-span beam, loaded evenly with a vertical pressure of q [kN/m], the above condition takes the form of:

$$y_{max}=5ql^4\cdot(384EI)^{-1}\leq f_{gr} \quad (5)$$

where:

the product of EI is the bending stiffness of the beam (E [kN/m^2] – modulus of elasticity of the material, I [m^4] – moment of inertia of the cross-section of the beam with respect to the main axis perpendicular to the load plane q). The permissible movements of f_{gr} are defined in standards, regulations and technical guidelines. For example, deflections of L/300 road steel bridges, where L is the span length. For bridge structures, the structural stiffness is determined [9].

The permissible movements of f_{gr} are defined in standards, regulations and technical guidelines. For example, the f_{gr} deflections of some steel elements are (as per PN-90/B-03200): main joists l_0/350, where l_0 is the span of the element.

The stability condition is a stand-alone structural safety control tool applied to some structures, e.g. industrial stacks, towers, retaining walls. It is verified on the assumption that a given structure meets the conditions of strength and stiffness. The simplest illustration of the stability control can be presented on the example of a retaining wall stabilizing the slope of a road or railway embankment. The retaining wall, treated as a rigid body, is loaded with a vertical force G_V (derived from its own weight and possible operational load) and a horizontal force P_H, which is the resultant of ground pressure (and, in some cases, ground water pressure).

Load P_H tends to shift the wall in the plane of the foundation, which is counteracted by the horizontal friction force T_H at the point of contact of the wall foundation with the ground. Theoretically, the shift will not occur if the following condition is met:

$$P_H<T_H \quad (6)$$

In practice, the $n_{bp}>1$ displacement factor per shift is used. Therefore, the stability condition due to displacement takes the form of:

$$P_H\leq T_H\cdot(n_{bp})^{-1} \quad (7)$$

where:

$T_H=\varkappa\cdot G_V$, where $\varkappa$ is the friction coefficient.

The horizontal load of P_H generates a moment which rotates the structure M_w=P_H·r_o relative to the external, lower edge of the wall (r_o – P_H force arm). The M_w moment is opposed by the holding moment M_u=G_V·r_u relative to the same edge (r_u is the G_V force arm). Theoretically, there will be no wall rotation if the following condition is met:

$$M_w < M_u \tag{8}$$

After entering the safety factor for capsizing $n_{bw}>1$, the condition in (8) takes on the following form:

$$M_w \le M_u \cdot (n_{bw})^{-1} \tag{9}$$

When designing the structure, attention should be paid to the important time factor. The durability of any structure is limited and depends on the conditions of use, the value and nature of the applied loads (static, dynamic), changes in the properties of materials resulting from ageing, fatigue, creep and corrosion. The durability of the structure can be ensured through proper maintenance, repairs and compliance with the conditions for use, recommended in the design.

2. Notes on structural dimensioning

The dimensioning of a structure consists in verifying if the structural members are safely designed with the calculated internal forces. In general, the load capacity condition (1) having the following form is analyzed:

$$\sum Q_i \cdot \gamma_i \le R_{mk} \cdot A_p \tag{10}$$

where:

$\sum Q \cdot \gamma$ – internal forces occurring in the analyzed cross-section of the element,

N_k=R_{mk}· A_p – section load capacity, resulting from the strength of the material of the R_{mk} structure and the geometric aspect ratio A_p. e.g. the bending indicator.

As mentioned above, structural systems and their components are dimensioned applying the limit state method. Limits are defined as those states, in which a structural system or a part of it ceases to fulfil the tasks assumed in the project, i.e. it threatens safety or does not meet specific performance requirements. Structures and their elements should be checked for the possibility of two groups of limit states: resistance and use.

Ultimate limit states include [2-4, 6, 8-10]:

- destruction of the most exerted sections of the structure,
- loss of stability of part or the whole the structure treated as a rigid body (system),
- destruction of the elements or progressing destruction of the whole structure,
- transformation of the structure into a geometrically variable system due to plasticization or material breakage in certain sections, or loss of stability in the shape of certain structural members,
- structural conditions resulting from the plasticization of the material or substrate of the structure, leading to the destruction of the structure or an unacceptable (for operational safety reasons) change in its shape.

Verification of limit states of load capacity consists in determining measurable cross-sections of the structural system and proving that the values of internal forces occurring in them from the design value of loads are not greater than the load capacity of these cross-sections, resulting from the design strength or other design mechanical properties of the structural material. This control is defined by condition (10).

The limit states of use include [2-4]:

– unacceptable deformations (displacements) of the structure or the substrate, on which the structure is founded,
– excessive scratching of the structure,
– excessive vibrations of the structure.

The verification of the limit states of use is intended to demonstrate that the structural deformations, the apertures of the cracks and other undesirable effects due to the loads, calculated taking into account the characteristic values of the mechanical properties of the materials, are not greater than the values which are considered acceptable (also called limit values). For instance, in the case of a bridge beam, the limit state of deflections is checked by demonstrating that internal forces applied by loads do not cause deflections greater than those considered acceptable according to the intended use of the structure and the safety of the users.

In the European Union Member States, structures (including engineering structures) are erected in accordance with the requirements specified in the Directive of the Council of Europe 89/106/EEC [1]. These requirements are included in the Construction Eurocode Program, which is a set of standards which defines the rules for the design of structures with regard to the requirements of the Directive concerning structural and fire safety.

The actions stipulated in the Eurocodes, which act on the structure, include [2-4, 6, 8-10]:

– loads,
– deformations caused e.g. by the difference in the settlement of particular elements of the structure and external factors, e.g. temperature change.

3. Safety in the design of earth structures on weak ground

The designed land transport route is sometimes, out of necessity, routed in areas with weak soils. Embankments erected on a weak-bearing substrate are subject to significant deformations which result from their compressibility and the plastic deformations of the substrate [7, 11-14]. These deformations result from the fact that the shear strength of the substrate is exceeded by the values of shear stresses, which are the sum of embankment load and operational load. After the subject embankment has reached the limit height (for a given substrate type), the following destructive phenomena may occur: catastrophic displacement of weak layers to the sides (Fig. 1) or a landslide of embankment slopes and its rapid settlement. This undesirable phenomenon may be counteracted by, among others, placing earth buttresses at the foot of the embankment, made of, for example, the spoil extracted from the cuts during the process of building the traffic route (Fig. 2) [7].

As we know, deformations of the substrate, in excess of the limit values, are unacceptable during the lifetime of the earth structure. For this reason, it is recommended to remove weaker layers when constructing highways and expressways. When building roads of lower

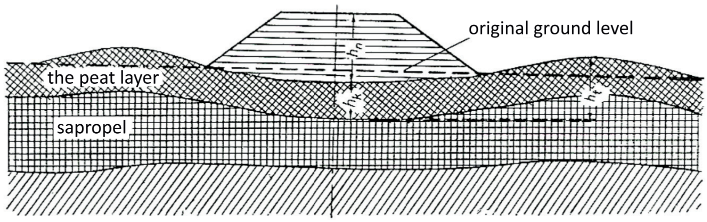

Fig. 1. Settlement of the embankment on muddy ground with simultaneous displacement of weak soil from under the embankment: h_n – embankment height, h_k – thickness of the peat layer, h_t – ceiling recess in the weak layer
Source: [7].

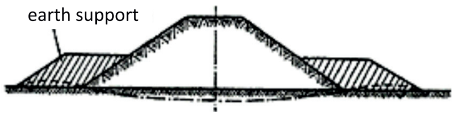

Fig. 2. Symmetrical system of earth supports used to reduce the settlement of an embankment founded in weak ground
Source: [7].

technical classes, weak layers 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 is not exceeded due to the possibility of catastrophic soil displacement from the embankment during the road operation period.

The stability of the embankment placed on weak substrate is checked by comparing the limit load for the weak-bearing layer q_f [MPa] with the vertical stress acting in the ceiling of this layer σ_z (derived from the dead weight of the embankment and the operational load). Z. Wiłum recommends to determine the limit load using formulas derived from the Terzaghi-Schultze formula [7]. However, if the angle of friction of the inner weak-bearing layer is close to zero ($\Phi \approx 0$), he simplified formula [5, 7, 12] may be used:

$$q_f = 5.7c + \gamma_h h_t \tag{11}$$

where:

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

c – cohesion of the weak layer [MPa],

γ_h – volumetric weight of soil located between the ground surface and the ceiling of the weak layer [kN/m^3],

h_t – depression of the ceiling of the weak layer (the sapropel in Fig. 1) in relation to the ground surface [m].

If the angle of friction of the weak inner layer is quite large ($\Phi > 10°$), the following formula is proposed [5, 7, 12]:

$$q_f = cN_c + \gamma_h h_t N_q + \gamma b' N_\gamma \tag{12}$$

where:

γ – volumetric weight of the soil in the weak layer, taking water buoyancy into account,

b' – length of rectangular projection of the embankment slope on a horizontal plane,

N_c, N_q, N_γ – load capacity coefficients according to K. Terzaghi, found in tables formulated by Z. Wiłun [7],

other designations as in formula (11).

Vertical stresses σ_z acting in the ceiling of the weak layer are calculated using the following formula [5, 7, 12]:

$$\sigma_z = q + \gamma_n h_n + \gamma_k h_k \tag{13}$$

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/m^3],

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/m^3],

h_k – thickness of the peat layer [m].

If q_f and σ_z are calculated, the safety factor is determined [5, 7, 12]:

$$F = q_f \cdot (\sigma_z)^{-1} \geq 1.0 \tag{14}$$

When $F < 1.0$, weak soil is displaced from underneath the embankment. Displacement may take place in the following forms:

- two-sided displacement, if the bottom of the mud layer is horizontal,
- if the bottom of the mud layer is not horizontal, one-sided displacement occurs in the direction of the greater depth.

The soil pushed out from under the embankment causes the area next to the embankment to rise, which leads to an increase in the dimension of h_t (Fig. 1). As a result, the limit load for the weak-bearing layer q_f is slightly increased, which is a positive phenomenon. Z. Wiłun [7] recommends to consider that, in the soil displaced above the water level, the volumetric weight increases by approx. 10 kN/m^3.

On the basis of the above specified formulas, the design of embankments is carried out in two versions:

- version I, if it is possible to leave a peat layer and a weak marshy layer (sapropel) under the embankment (then the weak soil will serve as the base of the embankment),
- version II, when it is necessary to remove the sapropel.

Here are the details of the procedure for designing embankments on marshy soil according to [7]:

- in the first version, the stability factor $F = 1.2$-1.5 must be obtained, depending on the technical class of the road and the accuracy of the designation of the cohesion resistance *c* of the marshy layer,
- when calculating stability, in $h_t = h_k$ should be adopted in formulas (11) and (12),
- if the stability factor F is found to be too low, lateral pressure slabs should be designed at the bottom of the embankment (the so-called buttresses), the structure of which is presented in the manual [7]. The width of the slabs l should be equal

to the depth of the marsh (sapropel) h_s; in this case, the depth of the ceiling of the weak layer h_t to formulas (11) and (12) is determined relative to the level of the slab top, i.e. [7]:

$$h_t = h_k + h_l \tag{15}$$

where:

h_l – the height of the pressure slab,

- in the second version, when it is necessary to remove a weak sapropel layer only by means of the weight of the embankment, this is only possible if $F < 0.6$,
- peat extrusion is facilitated by ditches cut in the peat layer near the base of the embankment, so, using formula (11) for limit stress, the depression of the weak-bearing layer's ceiling should be substituted for limit stress, not however relative to the ground surface level h_t, but relative to the bottom of the ditches cut.

K. Towpik [14] drew attention to the problem of the need to strengthen or rebuild the subgrade and substrate on high-speed railway routes. At speeds exceeding 100 km/h, vibrations in the track and in the ground increase significantly, leading to the loosening of the soil grains and vertical deformations of the track. The intensity of this phenomenon depends on the speed at which surface waves propagate (so-calle "surface waves"). Rayleigh waves). The critical speed of the Rayleigh waves $v_{R,kr}$ is a function of the type of ground base (i.e. the technical and operational characteristics of the ground, e.g. the load capacity measured by the limit load (q_f) and which determines the undesirable limitation of the running speed of trains. K. Towpik [14] provides some characteristic values of $v_{R,kr}$:

- $v_{R,kr}$ = 29-73 m/s and permissible driving speed of $v_{dop} \leq 121$ km/h in case of loose soil (silt, dust, plasticised clay),
- $v_{R,kr} \approx 115$ m/s and permissible driving speed of $v_{dop} \leq 269$ km/h for an average substrate,
- $v_{R,kr}$ = 146-258 m/s and permissible $v_{dop} \leq 320$ km/h for substrate with good characteristics.

Exceeding the critical speed $v_{R,kr}$ by 50% results in excessive track deformation. In turn, exceeding the $v_{R,kr}$ by 70% may result in deformations which cannot be removed by increasing the thickness of the subcrust layer. It is then necessary to reinforce or rebuild the subgrade (the layer of soil lying directly under the railway track, the thickness of which is usually 1 m) and the substrate.

4. The problem of strengthening weak soil in the substrate of earth structures designed for vehicle traffic

Referring to the previous chapter, an embankment built on a weak, marshy ground, with e.g. peat bog layers, is analyzed. One of the most prominent features of peat is the ability to significantly increase its strength after consolidation [5, 7]. As a result of the consolidation process, loaded peat experiences a decrease in porosity and a simultaneous increase in density and strength. This type of soil density improvement, called consolidation, is so significant in organic soils (as opposed to mineral soils) that it is worth taking into account in practical applications. Therefore, soil improvement with the applying consolidation, i.e. through embankment loading is usually addressed at organic soils. The embankment is built in stages, at a controlled speed of the programmed load, in such a way that the increasing strength of the peat is always greater than the actual working load of the embankment.

The reinforcement of the marshy soil through consolidation in question results from the increase of effective stress (i.e. stresses transferred by the ground frame) [5, 7]. Therefore, the strength of organic soils can be described by the Coulomb-Hvorslev equation:

$$\tau_f=(\sigma_n-u_{(t)})\cdot\tan\varphi_e+c_e \tag{16}$$

where:

σ_n – total normal stress,

$(\sigma_n-u_{(t)})$ – effective stress,

φ_e – effective internal friction angle,

c_e – effective cohesion.

Due to difficulties in describing the strength of organic soils applying effective stresses, this description relies on total stresses. In the case of fast soil loading, the $\sigma_n=u_{(t)}$ equation can be adopted. Then equation (16) takes the form of [5, 7]:

$$\tau_f=c_u \tag{17}$$

where c_u is cohesion.

With this assumption, the value of $\tau_{f,max}$ determined through field studies, applying a cross probe corresponds to the cohesion of c_u, specified in the triaxial apparatus. The shear strength of peat, determined in the field using a cross probe, depends, among others, on the degree of structural heterogeneity of the peat. It is therefore recommended to adopt the calculated value of organic soil strength $\tau_f^{(\mathrm{r})}$ [5, 7]:

$$\tau_f^{(\mathrm{r})}=\mu\cdot\tau_{f,max} \tag{18}$$

where:

μ – correction factor of 0.45 for poorly distributed peat and 0.55 for medium distributed peat,

$\tau_{f,max}$ – peat shear strength, determined in the field using a cross probe.

The essence of the method of strengthening marshy soil by loading the embankment is presented below in relation to:

1) assessment of the increase in strength parameters of organic soils after consolidation,
2) assessment of the stability of an embankment erected on organic soils.

The proposal of the French Central Road and Bridge Laboratory (Laboratoire Central des Ponts et Chaussees – LCPC) concerning the assessment of the increase of strength parameters of organic soils after consolidation is presented below [5]. The assessment is carried out on the basis of the so-called angle of consolidation growth of α_{cu} (Fig. 3). The formula used to calculate the cohesion of consolidates substrate c_{uk} has the following form [5]:

$$c_{uk}=c_{uo}+\sigma_k\cdot\tan\alpha_{cu} \tag{19}$$

where:

c_{uo} – cohesion of unconsolidated soil,

σ_k – consolidating stress (σ_{k1}, σ_{k2}, σ_{k3}), generated by the weight of an embankment built in three stages,

α_{cu} – angle of consolidation growth, determined from the graph in Figure 3c.

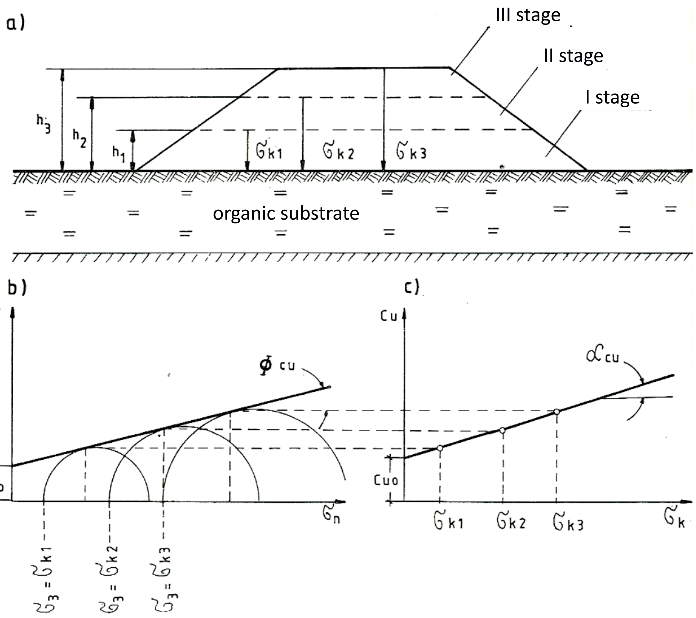

Fig. 3. Method of determining the angle of consolidation growth of αcu applying the LCPC method: a) diagram presenting the load of weak (organic) substrate with the embankment erected in three stages (I, II, III), b) graphs of Mohr's wheels at the stress limit state and a straight line serving as their envelope, as test results in a triaxial apparatus for three organic soil samples, c) graph indicating the value of the consolidation growth angle α_{cu}
Source: [5].

An assessment of the stability of an embankment placed on an organic substrate is presented in accordance with [5, 7] using an approximate (Swedish) method, in which the circular slip surface is assumed in the case of structural damage (Fig. 4). The safety of design of this embankment is determined by the coefficient value according to the following formula [5, 7]:

$$F=[0.005\cdot(\pi R^2)\cdot\Sigma c_{ui}\cdot\delta_i]\cdot(\Sigma Q_i\cdot x_i)^{-1} \tag{20}$$

where:

- R – radius of circular slip surface,
- c_{ui} – cohesion of marshy soil in different layers and sectors,
- δ_i – angles for the circular segment of the "i" layer and the sector,
- Q_i – weights of lumps of soil from the sliding part of the embankment
- x_i – distance between the center of gravity of the ground and the center of rotation "0".

In formula (20), the numerator is the moment of holding forces, while the denominator is the moment of rotating (sliding) forces. The stability of the embankment is conditioned by the safety factor of $F>1.0$.

The settlement of the organic substrate s_{po} is calculated according to the formula given in the geotechnical standard [5]:

$$s_{po}=\Sigma(\sigma_z\cdot h_i)\cdot(M_0)^{-1} \tag{21}$$

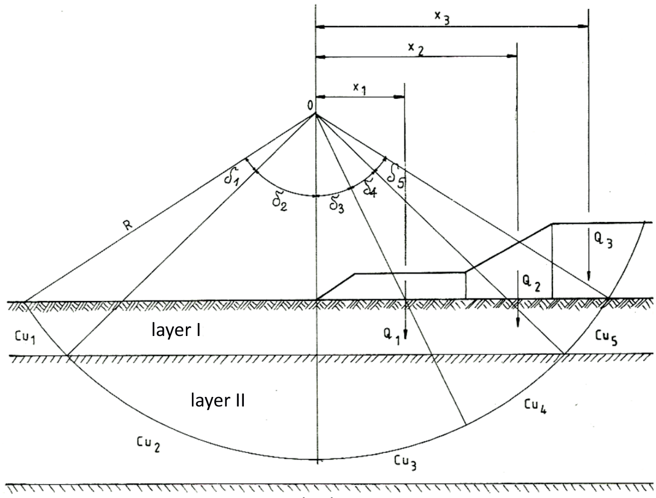

Fig. 4. Diagram used for the assessment of the stability of an embankment placed on a marshy substrate, applying the simplified (Swedish) method. Designations: Q_1, Q_2, Q_3 – weights of soil lumps, Q_1 is the buttress, x_1, x_2, x_3 – distance between the center of gravity of the ground and the center of rotation, c_{u1}, c_{u2}, c_{u3}, c_{u4}, c_{u5} – cohesion in particular layers and sectors of the organic substrate, taking into account the effect of increasing the organic strength of the substrate under the embankment, e.g. in sectors 3, 4 and 5, cohesions c_{u3}, c_{u4}, c_{u5} increased under the load of the embankment
Source: [5].

where:

σ_z – vertical stress in the middle of the thickness of each considered layer of marshy ground of the embankment base [MPa],

h_i – thickness of a single layer of marshy substrate [m],

M_0 – edometric module of primary compressibility [MPa].

When determining the M_0 value in a laboratory, it should be assumed that the test performed in an edometer should be treated as a model of the real process taking place in the ground.

5. General principles of designing the stabilization of earth structure slopes at risk of landslide

Apart from the necessity of overcoming problems with the operational quality of the ground surface of earth structures, engineering practice has shown that embankments or cuts are often affected with the problem of landslides [15-18]. As we know, the phenomenon of landslides leads to a loss of stability of the earth structure and threats to the safety of operation. Among the systems of retaining structures, used not only to support unstable slopes, but also to avoid or shorten earth slopes, attention should be paid to so-called lightweight retaining structures [19]. According to [19], “lightweight” structures include those which use the ground material to the greatest possible extent to cooperate in the transfer of horizontal forces, resulting from the pressure of the ground mass and the operational load. A special type of lightweight retaining walls are reinforced soil structures which contain elements forming the wall and are connected with them, placed in the soil backfill with horizontal layers of the reinforcing insert (e.g. flat bars) made of a material of adequate tensile strength. One of the advantages of reinforced soil structures is their uniform (i.e. favorable) distribution of the load transmitted onto the ground (Fig. 5), which is particularly important in the case of substrates displaying e.g. heterogeneous strength properties as a function of the length of the retaining wall.

Designing reinforced soil structures applying the limit state method, e.g. according to the French standard NFP 94-200 [19, 20], requires checking three conditions of stability: external, internal and general. Figure 6 presents the design diagram for the structure [19].

External stability is checked for slippage along the base and for the bearing capacity of the substrate.

The stability of the structure due to slip along the base is conditioned by two inequalities [19, 20]:

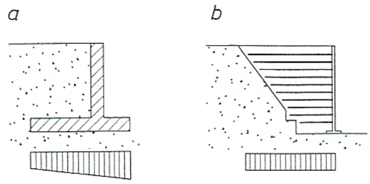

Fig. 5. Distribution of loads transmitted by the retaining structure to the ground: *a* – under a classic retaining structure (concrete or reinforced concrete), *b* – under the reinforced soil mass
Source: [19].

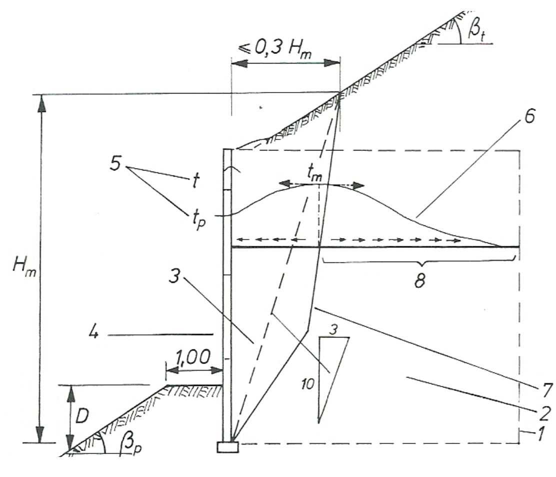

Fig. 6. Design diagram for reinforced soil structure: *1* – reinforced soil mass, *2* – reinforcing insert anchoring zone, *3* – active zone of reinforced soil, *4* – casing of the front of the structure, *5* – tensile values of the reinforcement insert (*t* – to the length of the insert, t_p – at the joint of the insert with the casing of the retaining wall), *6* – distribution of tensile forces along the length of the reinforcement insert, *7* – line of maximum tensile strength at the height of the retaining wall, dividing the massif into active and passive zones, *8* – length of the insert's anchoring, *D* – depth of the massif's foundation, *D* ≥ 0.4 m – basic depth, *D* = 0 – when the structure is supported on a rock or concrete bench
Source: [19, 20].

$$R_h \gamma_{F3} \leq [R_v \tan\varphi_{1k}(\gamma_{m\varphi})^{-1} + L c_{1k}(\gamma_{mc})^{-1}], \text{ [kN/m]} \tag{22}$$

and

$$R_h \gamma_{F3} \leq [R_v \tan\varphi_{fk}(\gamma_{m\varphi})^{-1} + L c_{fk}(\gamma_{mc})^{-1}], \text{ [kN/m]} \tag{23}$$

where:

R_h, R_v – horizontal and vertical component of the resultant load in the structure base [kN/m, along the structure casing].

φ_{1k}, c_{1k} – characteristic values of internal friction angle [°] and soil cohesion [MPa] in reinforced massif,

φ_{fk}, c_{fk} – characteristic values of internal friction angle [°] and substrate soil cohesion [MPa],

L – length of reinforcement layers with rectangular cross-section [m],

γ_{F3} – method coefficient,

$\gamma_{m\varphi}$ – partial safety factor used for the internal friction angle; it is 1.20 for the standard load combination and 1.10 for an exceptional combination,

γ_{mc} – partial safety factor used for effective cohesion; it is 1.65 for a standard load combination and 1.50 for an exceptional case.

The bearing capacity of the floor is ensured if the following inequality [19, 20] is fulfilled:

$$q_{ref} \leq q_{fu}(\gamma_{mq})^{-1} \text{ [kN/m}^2\text{]} \tag{24}$$

where:

q_{fu} – limit load capacity of the substrate [MPa],

γ_{mq} – partial safety factor concerning the resistance of the reinforced massif base, assumed as 1.50,

q_{ref} – load of the substrate with reinforced soil massif [MPa].

The load on the substrate q_{ref} is calculated using the following formula:

$$q_{ref} = \gamma_{F3} R_v [L - 2 M_b (R_v)^{-1}] \tag{25}$$

where:

γ_{F3}, R_v, L – as in formulas (22) and (23),

M_b – sum of the static moments of the vertical and horizontal components of the load about the axis parallel to the length of the resistance structure and passing through the center of gravity of the base of the structure (static resultant moment) [kNm/m].

Internal stability is verified to determine whether two conditions are met [19]:

1) the tensile stress in the reinforcement of the subsoil mass is within the permissible range,
2) the anchoring resistance of the inserts is correspondingly higher than the forces pulling the reinforcement out of the ground.

Figure 7 presents a diagram adopted for internal stability control according to [19, 20], developed on the basis of theoretical analyses, model tests of reinforced soil massifs and monitoring of real structures.

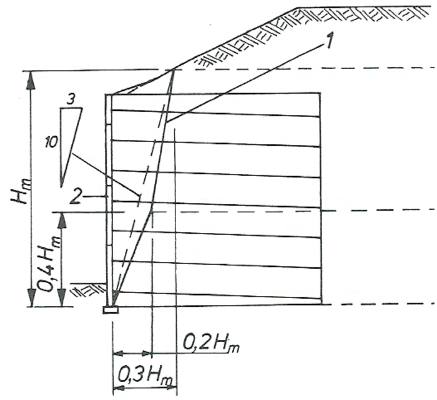

Fig. 7. A diagram of internal stability control of reinforced soil structure – vertical cross section through reinforced massif with height H_m: *1* – dashed line showing the location of maximum tensile stresses in the reinforcement, *2* – structure casing
Source: [19, 20].

The sequence of calculations is as follows:

– determination of the line of maximum tensile stress in the reinforcement t_m at height H_m of a massif of reinforced soil (Fig. 6, 7), based on the calculation of the maximum horizontal stress σ_h in each reinforcement layer,
– calculation of tensile stress t_p next to the structural casing (Fig. 6),
– calculation of the characteristic value of friction r_f mobilised in each reinforcement layer,
– check of the safety of each reinforcing layer for breakage,
– verification of the anchoring condition in each reinforcing layer,
– verification of the strength of the structure's casing made of reinforced soil.

The safety of each reinforcing layer due to tearing is ensured if the following inequalities are fulfilled (the first inequality concerns the reinforcement insert, the second inequality concerns the connection between the insert and the housing [19, 20]:

$$\gamma_{F3} t_m \leq r_{ck} (\gamma_{mt})^{-1} \tag{26}$$

$$\gamma_{F3} t_p \leq r_{ak} (\gamma_{mt})^{-1} \tag{27}$$

where:

γ_{F3} – method coefficient,
t_m, t_p – tensile stresses specified above [MPa],
$r_{ck}=\sigma_r \cdot A_{cd}$ – characteristic strength of reinforcement layer [kN/m],
$r_{ak}=\sigma_r \cdot A_{ad}$ – characteristic strength of the reinforcement layer at the point of connection to the housing [kN/m],
A_{cd} – cross-sectional area of the reinforcement layer [m^2/m],
A_{ad} – cross-sectional area of the reinforcement layer [m^2/m],
σ_r – tensile strength of the reinforcement [MPa],

γ_{mt} – partial safety factor due to breaking the reinforcing layer, assumed as 1.50 for standard structures and 1.65 – for extraordinary structures (specified in [19]).

The condition for proper anchoring of inserts in each reinforcing layer is expressed by the following inequality:

$$\gamma_{F3} t_m \leq r_f (\gamma_{mf})^{-1} \tag{28}$$

where:

γ_{F3}, t_m – as specified above,

r_f – characteristic value of friction mobilized in each reinforcement layer [kN/m],

γ_{mf} – partial safety factor for anchoring reinforcement inserts; 1.20 for standard structures, 1.30 for extraordinary structures.

The strength of the structure's casing consisting of reinforced soil is verified by applying the following inequality [19, 20]:

$$\gamma_{F3} t_p \leq r_{pk} (\gamma_{mp})^{-1} \tag{29}$$

where:

γ_{F3}, t_m – as specified above,

r_{pk} – characteristic value of housing strength in connection points with the reinforcement [kN/m],

γ_{mp} – partial safety factor relating to the strength of the casing, assumed as 1.65 for concrete casings and 1.50 for metal casings.

The general stability of the structure from reinforced soil is verified by considering [19]:

- all potential surfaces of damage to the ground mass,
- possibilities of counteracting landslides based on the shear strength of the soil along the surface of the destruction,
- increasing the effect of the ground mass stability phenomenon due to the installation of reinforcement inserts in layers crossing the destruction surfaces.

Figure 8 illustrates the cylindrical shape of a potential failure surface in case of homogeneous soil in a reinforced soil massif [19, 20].

Tensile forces mobilized in the reinforcement influence the stability of the structure, generating stresses in the massif of reinforced soil. The contribution of soil reinforcement to overall stability is expressed in tensile forces in each layer of reinforcement which cuts through the destruction surface. The maximum value of these forces at the point of intersection with the destruction surface is limited [19]:

- by friction in the point of contact between the ground base and the reinforcement,
- by the tensile strength of the reinforcement,
- by the strength of the housing at the points of connection to the reinforcement, increased to account for the value of friction along the reinforcing elements, which is mobilized between the connections to the housing and the destruction surface.

The designed shear strength of the soil in the massif τ_d along the destruction surface (at each point) is calculated according to [19]:

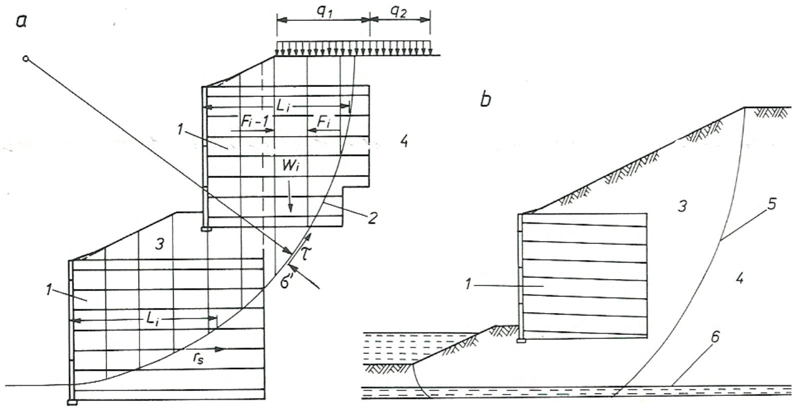

Fig. 8. General stability control diagrams: *a* – destruction along the cylindrical surface, *b* – extraordinary massif damage case with a weak layer in the substrate, *1* – reinforced soil massif, *2* – circular line illustrating massif destruction, *3* – destruction zone, *4* – zone outside the destruction area, *5* – line (non-circular) of massif destruction, *6* – weak layer in the substrate
Source: [19, 20].

$$\tau_d = (c_d + \sigma_n \tan \varphi_d) \text{ [MPa]} \tag{30}$$

where:

c_d, φ_d – design values of shear strength parameters, in order: consistency [MPa], angle of internal friction [°],

σ_n – stress in the reinforced soil mass, normal to the destruction surface [MPa].

Figure 8 presents only one potential damage area. In practice, different locations of these surfaces are analyzed, usually applying the Bishop method [19, 7].

Fulfilling the condition of equilibrium of static moments is the final stage of general stability verification [19]:

$$M_w \leq M_u \tag{31}$$

where:

M_w – structure capsizing moment generated by constant, variable and exceptional loads [kNm/m],

M_u – structure holding moment, resulting from the shear strength of soil along the destruction surface [kNm/m].

Summary

Attention was drawn to the current problems in civil engineering, related to the design on weak substrate and the stabilization of slopes of earth structures threatened by landslides. Selected methods of solving geoengineering problems are provided. The principle of dimensioning engineering structures has been displayed on selected examples. The aim of the article was to provide the Reader with general information about the assumptions for the design of reinforcement of weak substrate for earth structures designed for vehicle traffic, and the stabilization of massifs with retaining walls (technically and economically rational), design procedure based on general analytical dependencies, taking into account the current procedures (Eurocode 7 and the French standard, applied in the design of contemporary retaining structures from reinforced soil using the method of limit states).

The authors have at their disposal numerous design examples of lightweight retaining structures of various types. In this article, however, the general analytical dependencies are intentionally used, since the detailed solution of e.g. a given type of retaining structure requires a comparative analysis in relation to other examples of structures (with different, inter alia, structure or configuration of reinforcement), while ensuring the necessary number of identical parameters, e.g. height of the massif, strength features of the ground base.

It should be emphasized that engineering practice currently has quite a wide spectrum of structural solutions in the field of light retaining structures, including methods of their dimensioning developed in the form of numerical algorithms. However, these matters go beyond the publishing framework of this article.

The summary of the issues raised in the article is accompanied by the awareness of the current technological development of resistance structures, based on the achievements of material engineering and the pursuit of the use of soil as a material for these structures. In view of the dynamically appearing new material and technological possibilities (e.g. sintering of weak soils), it should be expected that they will lead to the generation of optimal forms of retaining structures.

Acknowledgement

No acknowledgement and potential founding was reported by the authors.

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.

ORCID

Jacek Ryczyński https://orcid.org/0000-0002-5501-8609

Piotr Saska https://orcid.org/0000-0002-9760-856X

Andrzej Surowiecki https://orcid.org/0000-0003-4080-3409

Krzysztof Ksiądzyna https://orcid.org/0000-0001-8820-1533

References

1. Dyrektywa Rady 89/106/EWG z dnia 21 grudnia 1988 r. w sprawie zbliżenia przepisów ustawowych, wykonawczych i administracyjnych Państw Członkowskich odnoszących się do wyrobów budowlanych (Dz. Urz. WE L 30 z 11.02.1989).

2. *Eurokod 7. Projektowanie geotechniczne. Część 1: Zasady ogólne PN-EN 1997-1*. Warszawa: Polski Komitet Normalizacyjny; 2008.
3. *Eurokod 7. Projektowanie geotechniczne. Część 1: Zasady ogólne PN-EN 1997-1:2008/Ap2*. Warszawa: Polski Komitet Normalizacyjny; 2016, Załącznik krajowy.
4. *Eurokod 7. Projektowanie geotechniczne. Część 2: Rozpoznawanie i badanie podłoża gruntowego PN-EN 1997-2*. Warszawa: Polski Komitet Normalizacyjny; 2009.
5. *Poradnik wzmocnienia podłoża gruntowego dróg kolejowych.* Biedrowski Z (ed.). Politechnika Poznańska: Poznań; 1986.
6. Świeca M. *Zasady projektowania geotechnicznego w nawiązaniu do Eurokodu 7 z zastosowaniem programów numerycznych*. Warszawa: Instytut Techniki Budowlanej; 2011.
7. Wiłun Z. *Zarys geotechniki. Podręcznik akademicki.* 9th ed. Warszawa: Wydawnictwa Komunikacji i Łączności; 2010.
8. Wysokiński L, Kotlicki W, Godlewski T. *Projektowanie geotechniczne według Eurokodu 7. Poradnik*. Warszawa: Instytut Techniki Budowlanej; 2011.
9. Machelski C. *Sztywność obiektu mostowego jako parametru użytkowego konstrukcji inżynieryjnych*. Przegląd Komunikacyjny, 2016;2:27-32.
10. Starosolski W. *Konstrukcje żelbetowe według Eurokodu 2 i norm związanych.* Warszawa: Wydawnictwo Naukowe PWN; 2012.
11. Skrzyński E. *Podtorze kolejowe*. Warszawa: PKP Polskie Linie Kolejowe; 2010.
12. Stilger-Szydło E. *Posadowienia budowli infrastruktury transportu lądowego*. *Teoria, projektowanie, realizacja.* Wrocław: Dolnośląskie Wydawnictwo Edukacyjne; 2005.
13. Towpik K. *Infrastruktura transportu szynowego*. Warszawa: Oficyna Wydawnicza Politechniki Warszawskiej; 2017.
14. Towpik K. *Koleje dużych prędkości. Infrastruktura drogi kolejowej*. Warszawa: Oficyna Wydawnicza Politechniki Warszawskiej; 2012.
15. Bzówka J, Juzwa A, Knapik K, Stelmach K. *Geotechnika komunikacyjna*. Gliwice: Wydawnictwo Politechniki Śląskiej; 2012.
16. Grabowski Z, Pisarczyk S, Obrycki M. *Fundamentowanie*. Warszawa: Oficyna Wydawnicza Politechniki Warszawskiej; 2009.
17. Pisarczyk S. *Geoinżynieria. Metody modyfikacji podłoża gruntowego.* Warszawa: Oficyna Wydawnicza Politechniki Warszawskiej; 2014.
18. Pisarczyk S. *Mechanika gruntów*. Warszawa: Oficyna Wydawnicza Politechniki Warszawskiej; 2010.
19. Jarominiak A. *Lekkie konstrukcje oporowe.* Warszawa: Wydawnictwa Komunikacji i Łączności; 2010.
20. NFP 94-200. Renforcement des sols par inclusions. Ouvrages en sols rapportes renforces par armatures ou nappes peu extensibles et souples. Dimensionnement.

Biographical note

Jacek Ryczyński – DSc, Eng. The author of scientific publications on broadly understood military safety and engineering connected with fuels. His main area of scientific interest are problems connected with assessment of the suitability of fuel delivered to fuel bases and forecasting the maximum time of safe storage in specific storage conditions and also impact of selected factors on the increase in the intensity of fuel ageing.

Piotr Saska – LTC DSc, Eng. Author and coauthor of over 50 scientific publications on broadly understood military engineering and safety. His main area of interest is the influence of an explosion wave on the environment and military vehicles, problems associated with transport infrastructure, engineering, in particular the construction of roads and railways.

Andrzej Surowiecki – DSc, Eng. Associated Professor, he graduated University of Technology in Wroclaw, the Faculty of Civil Engineering. Currently, he is the professor at the Faculty of Civil Safety Engineering at the General Tadeusz Kosciuszko Military University of Land Forces. His scientific interests are focused on following fields: mechanics of surface and subgrade, roads and railways, as well as design of communication earthen structures in terms of operational reliability.

Krzysztof Książdzyna – MSc. He's academic lecturer at Faculty of Security Sciences on the General Tadeusz Kosciuszko Military University of the Land Forces. His scientific interests are focused on computer modeling, process safety management and risk analysis.

Wybrane problemy bezpieczeństwa w projektowaniu obiektów inżynieryjnych

STRESZCZENIE Tematem artykułu są wymogi bezpieczeństwa podane w normach i zaleceniach projektowania obiektów inżynierii lądowej, ze szczególnym uwzględnieniem komunikacyjnych budowli ziemnych. Uwagę skupiono na zagadnieniach dotyczących: istoty niezawodności i trwałości konstrukcji oraz zapewnienia bezpieczeństwa na etapie projektowania nasypów komunikacyjnych ze skarpą podpartą z zastosowaniem ściany oporowej, a także nasypów posadowionych na podłożu o niedostatecznej nośności. Procedurę postępowania dotyczącą projektowania nasypów komunikacyjnych na słabym podłożu, wzmacniania słabego podłoża oraz projektowania konstrukcji oporowych (na przykładzie masywu z gruntu zbrojonego) przeprowadzono zgodnie z obowiązującymi w geoinżynierii obliczeniami, z wykorzystaniem ogólnych zależnościach analitycznych.

SŁOWA KLUCZOWE obiekty inżynieryjne, konstrukcje oporowe, projektowanie, bezpieczeństwo eksploatacji

How to cite this paper

Ryczyński J, Saska P, Surowiecki A, Książdzyna K. *Selected safety issues in designing engineering structures*. Scientific Journal of the Military University of Land Forces. 2020;52;1(195):135-53.

DOI: http://dx.doi.org/10.5604/01.3001.0014.0269