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Model Studies of Distortion Condition Resistance Wall from Gabionic Elements

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Retaining walls have been classified and characterized as structures protecting road or railway embankments against landslides. Particular attention was focused on retaining structures classified as light [1-3, 5-12, 14-24], which include, among others, walls made of gabions. Physical models of the gabion retaining wall, prepared on a laboratory scale, test stand and how to perform spatial deformation tests are presented. The models differed in the number and dimensions of gabions. On the basis of measured horizontal deformations of embankment models with a gabion wall, which were subjected to vertical static pressure at the level of the embankment ceiling, the values of basic strength parameters were determined. In particular, the value calculated: horizontal pressure coefficient, shear strength and modulus of deformation. The variability of the values of these parameters was estimated as a function of variable factors related to the gabion wall configuration (determined by the number and dimensions of gabions) and the value of the external load test.

Keywords: gabion retaining wall, deformation state, model tests.

1. RETAINING WALLS AS AN EFFECTIVE STABILIZATION OF ROAD EMBANKMENTS

Retaining walls are structures protecting road embankments against landslides or built to shorten embankment slopes or excavations in mountain areas [6, 8-13].

Traditional massive retaining walls are made of stone, brick, concrete or reinforced concrete [6, 24]. The principle of the so-called lightweight retaining walls consists in including in the cooperation of the soil medium lying outside the wall plane, on the principle of friction on the contact of the wall elements with grains (particles) of the soil medium. These walls began to be widely used since the mid-1960s, that is since the development of reinforced soil technology in the construction of embankments and communication trenches [6, 25]. The use of reinforced soil for the implementation of communication earthworks makes it possible to make vertical walls instead of classic slopes. Over the past few decades, a number of reinforced soil technology variations have been implemented [1, 4, 6, 8-13, 24]. All varieties share a common feature: the modular nature of the elements. The most common solutions include [6, 24]: constructions made of soil reinforced with geotextiles; cribs; quasi-box; coating; terraced prefabricated shells; from GEOWEB cell mats; from T-shaped elements (so-called T-WALL); constructions consisting of prefabricated angles (so-called constructions with many horizontal shelves) and gabion ones. The analysis of the functioning of the last of the specified types of structures is the subject of this article.

2. RETAINING WALLS MADE OF GABION ELEMENTS.

Mesh baskets filled with stone material, called gabions, used as part of the structure protecting the banks of rivers or sea cliffs against erosion, have been known for about two thousand years [2, 6]. The town of Casalecchio near Bologna was considered to be the place of the first applications of modern gabions [6]. Currently, retaining walls made of mesh and stone baskets are quite commonly used as a way of reconstructing damaged traffic embankments as a result of floods or as protection against landslides, or they may constitute in some cases a permanent housing for the unstable slope of the road and railway embankment, sometimes a retaining wall in undeveloped areas and urban agglomerations [1-3, 6, 12, 24]. Cases of constructing gabion abutments for small bridges are known [6, 24]. In Poland, damaged earthworks are eagerly rebuilt for nearly twenty years using gabion walls, guided by the MACCAFERRI® technology [2, 3, 6, 7, 10, 24]. Numerous examples of the use of these structures can be found on the Gdańsk Coast (Jastrzębia Góra, Hel Peninsula - as protection of the sea shore against erosion) and in Lower Silesia (especially in the Kłodzko Valley) as part of the reconstruction of communal roads damaged and in many cases damaged during the 1997 flood. [24]. Technology seems to be particularly useful in crisis situations. Its reliability is determined by: insensitivity to uneven subsidence (susceptibility), water permeability, durability, resistance to mechanical damage, environmental friendliness. These are advantages of undoubted significance for civil engineering.

The filling of gabions (mesh baskets), which are a special kind of porridges, is essentially stone material (most often crushed stone, pebbles, field stone, etc.). In practice, the coating is a metal mesh with hexagonal eyes and a special double wire weave, protecting the entire mesh against propagation of damage in the event of local wire damage [2, 3, 5-7, 24].

3. MODEL TESTS OF THE STATE OF DEFORMATION OF A RETAINING WALL FROM GABION ELEMENTS

3.1. GENERAL THOUGHTS

The issue of gabion walls in the aspect of dimensioning and stability is quite intensively discussed in the country and abroad [1, 5, 6, 8-24]. Basically, work is undertaken in the field of theory and simulation tests of gabion structures, based on numerical models. The subject of this article is the experimental analysis of the deformation state of the gabion wall modelled on a laboratory scale. However, this type of approach (burdened with technical conditions, such as: fairly limited dimensions of the test stand, disturbances resulting from measuring equipment and techniques, etc.) could, according to the authors, partially fill the so-called a gap in a fairly wide field of research in this area.

3.2. PHYSICAL MODELS ON A LABORATORY SCALE, POSITION AND TEST METHOD

Research models with 0.54 x 0.54 m plans and 0.42 m height (cuboid) consisted of a retaining wall (also called curtain wall) 0.42 m high, made of gabion baskets and sand backfill, constituting a ground massif. Gabion baskets were made of FORTRAC® system geogrids type R 90 / 90-20T [4] with technical characteristics: plastic-polyester, polymer coating, tensile strength in longitudinal and transverse direction $R_r \geq 90$ kN / m, square mesh size 10 x 10 mm. The baskets were filled with basalt grit 8/16 mm (angle of internal friction in a medium compacted state $\varphi = 37.9^0$).

Three types of retaining wall models were constructed, shown in Figure 1 [21-24]: A - a model constituting a system of three gabions, filling the surface of the measuring wall (vertical axis *z*), B - a system of four gabions, C - a system of seven gabions. Research models differed in dimensions and number of gabions enclosing the measuring wall. The variable parameter were the dimensions of the gabions in the horizontal (the so-called depth, characterized by the y axis) and vertical (the so-called gabion element height), while the second dimension in the horizontal direction (being the width of the container 0.54 m, reduced by the value of the so-called backlash) remained unchanging and amounts to 0.52 m. In view of the above, individual wall types contain gabions with dimensions (height x depth x length):

- in type A: 0.12 x 0.10 x 0.52 m; 0.18 x 0.15 x 0.52 m; 0.12 x 0.20 x 0.52 m,
- in type B: 0.12 x 0.10 x 0.52 m; 0.12 x 0.15 x 0.52 m; 0.12 x 0.20 x 0.52 m; 0.06 x 0.25 x 0.52 m,
- in type C: 0.06 x 0.10 x 0.52 m; 0.06 x 0.15 x 0.52 m; 0.06 x 0.20 x 0.52 m.

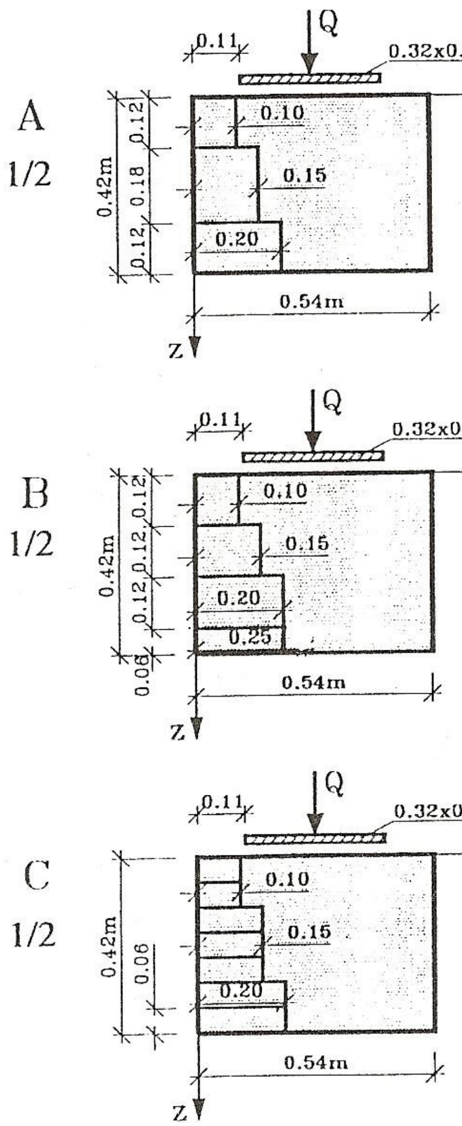


Fig. 1. Research models [24]: A - three gabion system, B - three gabion system on a gabion mattress (four elements in total), C - a system of seven gabion mattresses, 1 - gabions without interconnection, 2 - gabions interconnected

In addition, two variants of the wall casing were considered:

1 - gabions without interconnections (loosely spaced on individual floors - measurement levels);

2 - gabions connected together with metal "paper clips" (combined).

The models were located in a steel rectangular container (Fig. 2). The walls and bottom of the container are adapted to measure horizontal and vertical deformations (displacements) generated from the inside. The wall structure reflects the resistance of the soil centre zones surrounding the gabion wall model including the massif outside the wall, while the bottom is a modelled single-parameter substrate. The external test load (which is a mapping of the operational and service load) was carried out with a vertically directed static pressure evenly distributed at the floor level, centrally, with a value in the range of $0 \leq q \leq q_{limiting} = 239.5$ kPa. The maximum value of $q_{limiting}$ load determined the limit state of active horizontal pressure of the massif without a gabion wall. The subject of the study were horizontal and vertical displacements of the model, measured in the planes of the walls and bottom of the container. The displacement values were transformed into stress values (horizontal - the so-called horizontal pressure and vertical - pressure on the ground) based on the product of the displacement values recorded with special sensors and the known constant elasticity of the sensors C [kN / m3]. The research was comparative - the results of the measurements were compared with the results obtained on the reference models, which constituted the solid of the soil medium without a gabion wall. Two states of density of the massif adjacent to the gabion retaining wall were considered: the condition of loosely packed massif (phase I of tests) and the state after initial compaction carried out in the load process to the maximum value $q_{limiting} = 239.5$ kPa and relief to zero (phase II of tests).

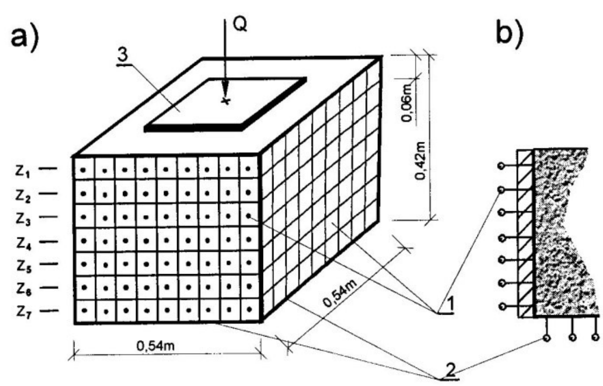


Fig. 2. Test stand [13, 24]: a - general view, b - vertical section through the measuring wall, 1 - horizontal pressure sensor, 2 - vertical pressure sensor, 3 - plate 0.32x0.32 m transferring the test load to the model, z_1 = 0,03 m, z_2 = 0,09 m, z_3 = 0,15 m, z_4 = 0,21 m, z_5 = 0,27 m, z_6 = 0,33 m, z_7 = 0,39 m - measurement levels

Designations were adopted for models (gabion wall + massif) and container:

k = 1, 2, ... 7 - depth indicator in the model, which is equivalent to numbering of measurement levels;

z_k [m] = [z_1 = 0,03; z_2 = 0,09; z_3 = 0,15; z_4 = 0,21; z_5 = 0,27; z_6 = 0,33; z_7 = 0,39], - depth in the gabion wall model with a massif;

p_{yk} [kPa] - unit lateral pressure of the model (horizontal stress);

q [kPa] - static unit load of the model in the range of 0.0-239.5 kPa (at maximum load, the active limit state of horizontal pressure was obtained in the massif model without a gabion wall);

L. S. - ground centre (massif) loosely packed, w.z. - pre-compacted massif.

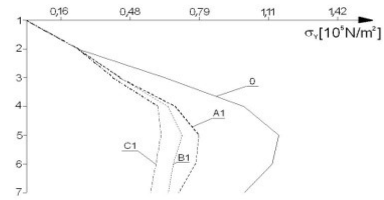
4. TEST RESULTS OF UNIT HORIZONTAL PRESSURE

The results of unit horizontal pressure tests of individual models are shown in Figure 3a, b, c, d, e [24]:

- Fig. 3a illustrates the distribution of unit horizontal pressure for individual models of the massif with the gabion wall (models A1, B1, C1; i.e. the gabions are not connected with each other),
- Fig. 3b: unit lateral thrust for model A with gabions loosely laid and bound;
- Fig. 3c: unit lateral pressure for model B with loosely connected and bound gabions;
- Fig. 3d: unit lateral pressure for model C with gabions loosely laid and bound;
- Fig. 3e: distribution of unit horizontal pressure for individual models of the massif with a gabion curtain wall (models A2, B2, C2; i.e., the gabions are joined together).

It can be seen from the charts that the dimensions of the gabion baskets and the configuration of the retaining wall elements as a complex whole have quite a significant impact on the value of the horizontal pressure of the massif and the form of the pressure curve (the charts refer to the active limit state). In the case of linking (joining) of gabion baskets forming a retaining wall, a reduction in horizontal pressure (measured with horizontal deformations of the measuring wall of the test container - vertical axis z) was obtained from a few to over 10%. Generally, it was found possible to reduce horizontal deformations of the massif model supported by a gabion wall (and at the same time an increase in load capacity) by up to about 50% depending on the gabion wall structure, i.e. dimensions, shape and arrangement of gabion baskets.

a



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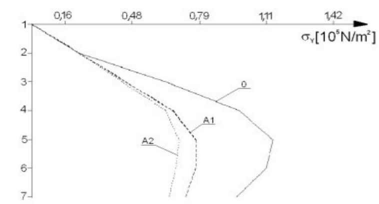

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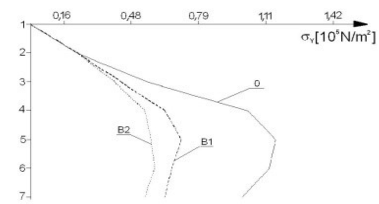

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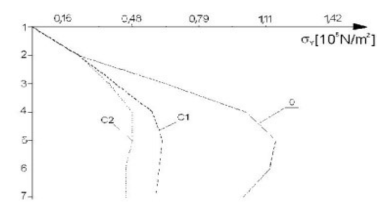

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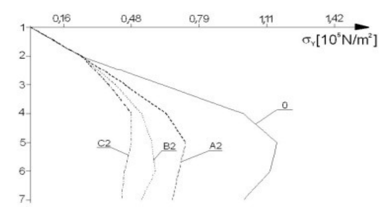


Fig. 3. Graphs of unit horizontal pressure of individual models gabion retaining wall [22, 24]: a - models with non-aggregated gabions, b - model A with gabions loosely laid and bound, c - model B with loosely laid and bound gabions, d - model C with loosely connected and bound gabions, e - models with combined gabions.

5. THEORETICAL GENERALIZATIONS OF RESEARCH RESULTS

5.1. INTRODUCTORY REMARKS

Theoretical generalizations of experimental research results were made by treating linear horizontal and vertical displacements of models as a measure of the spatial state of deformation. An analysis of changes in the value of the horizontal pressure coefficient and shear strength of the massif enclosed with a retaining wall of gabion elements was presented. The effect of increasing the load capacity was demonstrated, being a representative of the general state of strengthening the soil massif with a gabion wall, depending on the number of gabions and their geometric dimensions.

Data from laboratory tests regarding models [18, 24]:

1) Data for the massif model without a gabion wall (pattern)
   $[z_k] = [z_1, z_2, z_3, z_4, z_5, z_6, z_7]$
   * l.s. centre (loosely packed):
   $[p_y]$ = [0.8; 30.4; 84.8; 134.4; 140.0; 134.3; 124.8]
   ** centre of hot water (pre-concentrated):
   $[p_y]$ = 0.8; 12.8; 44.0; 80.0; 96.0; 104.2; 96.32].

2) Data for massif models with A-type gabion wall
   Note: p_{yk}^z - unit horizontal pressure, the index z denotes the number of gabions
   * l.s. centre: p_{yk}^3 = [1.8; 28.8; 56.0; 40.0; 60.0; 96.1; 98.4]
   ** w.z. centre: p_{yk}^3 = [0.9; 10.4; 28.0; 28.8; 48.0; 63.2; 80.0]

3) Data for the massif model with gabion wall type B
   * l.s. centre: p_{yk}^4 = [1,4; 28.8; 24.8; 33.6; 36.0; 48.0; 52.0]
   ** w.z. centre: p_{yk}^4 = [3.2; 14.4; 17.6; 28.8; 33.6; 40.0; 41.6]

4) Data for the massif model with a C-type gabion wall
   * l.s. centre: p_{yk}^7 = [0.96; 16.0; 30.4; 28.8; 30.4; 29.6; 20.8]
   ** w.z. centre: p_{yk}^7 = [0.9; 11.2; 25.6; 28.8; 30.4; 28.7; 25.7]

The maximum q = 239.5 kPa was assumed as the design load.

5.2. THE HORIZONTAL PRESSURE COEFFICIENT KA AND THE EFFECT OF INCREASING THE LOAD CAPACITY Δp_z

The values of the pressure coefficient *Ka* in the active limit state were the basis for estimating the effect of increasing the load capacity and calculating the shear strength of the massif with a gabion wall.

Horizontal pressure coefficients were calculated according to the relationship [18, 24]:

$$K_a = p_y \cdot (p_z)^{-1} = p_y \cdot (q_{limiting})^{-1} \tag{1}$$

for the reference model (without a gabion wall) and massif models supported by gabion walls.

As a measure of the increase in the load capacity of the massif supported by a gabion wall in relation to the reference mass, the possibility of increasing the vertical load q and admitting appropriately higher values of vertical stress $p_z = f(q)$, with a determined value of horizontal stress originating from the pressure of the ground massif dust p_y, was considered. The size of the external load introduced in the tests guarantees the occurrence of uniform stresses at the height of the model and therefore $p_z = q_{limiting}$ was assumed.

For example, for the gabion massif model, loosely loaded, loaded with maximum test pressure $q_{limiting}$ = 239.5 kPa, the average lateral pressure $p_{y,average}$ = 92.8 kPa was obtained from the test tests. Then the experimental pressure coefficient receives the value:

$$K = p_{y,average} \cdot (p_z)^{-1} = 0.387$$

for the loosely massif (l.s.) model with gabions type A, the following were obtained from tests with a load $q_{limiting}$ = 239.5 kPa:

$p_{y*,average}$ = 54.44 [kPa] and pressure coefficient $K^* = p_{y*,average} \cdot (p_z)^{-1} = 0.227$.

Assuming the horizontal lateral pressure for the reference model (without gabions) $p_{y,average}$ = 92.8 kPa as the basis, a relationship can be constructed that shows an increase in the range of external load capability (Δp_z) in the massif with gabions in relation to the reference:

$p_z^* = p_{y,average} \cdot (K^*) - 1 = 92.8 \cdot (0.227) - 1 = 408.81 > p_z = 239.5$ kPa.

The effect of load capacity increase generated by installing a gabion wall was expressed by the difference between the maximum load of the massif with gabions and the maximum load of the reference mass:

$$\Delta p_z = p_z^* - p_z = 408{,}81 - 239{,}5 = 169{,}31 \text{ kPa}$$

or otherwise:

$$dp_z = p_z^* \cdot (p_z)^{-1} = 408{,}81 \cdot (239{,}5)^{-1} = 1{,}71 > 1{,}0.$$

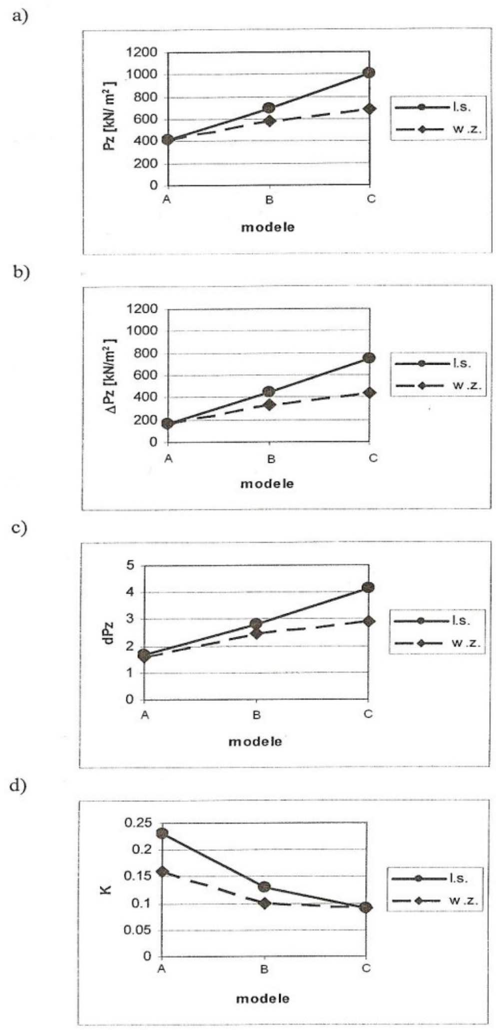


Fig. 4 a, b, c, d. Physical parameters as characteristics of individual massif models loosely packed (l.s.) and pre-compacted (w.z.) [22, 24]: a - vertical stress p_z, b - vertical stress increase (load-bearing effect) Δp_z, c - vertical stress increase dp_z, *d* - horizontal pressure coefficient *K*

Figure 4 shows the calculated values of the parameters: p_z, Δp_z, dp_z and the K_a coefficient in individual research models A, B, C. Note the smaller values of the coefficient of pressure Ka in models with pre-compacted soil, which is in accordance with the basic principles geotechnics.

5.3. SHEAR STRENGTH

A non-coherent soil medium with strengthening can be treated as [11, 18, 24]:

a) medium without coherence, in which the angle of internal friction increased due to reinforcement ($c = 0$, $\Delta\varphi > 0$),
b) soil material, which in its boundary state behaves as anisotropic cohesive, characterized by an angle of internal friction of a value such as in an unreinforced medium, but containing features that indicate coherence, directly proportional to the strength of the reinforcement tensile inserts ($c > 0$, $\Delta\varphi = 0$). In the examined models, gabion walls are an element of strengthening the soil material (Fig. 1).

- **The effect of increasing the angle of internal friction in a reinforced soil medium**

In the research process, active pressure p_y was obtained with a value depending on a number of accompanying factors. If in the case of $q_{limiting}$ the values of p_z and K are treated as extreme, then after substituting them for the classic limit state equation, the effect of increasing the angle φ in reinforced soil can be determined. This procedure was considered correct for qualitative and comparative purposes, regarding the mechanical properties of physical models of unreinforced and strengthened soil, subjected to identical test conditions.

Considering the case of soil without consistency (a), the limit state condition is in the form of a non-reinforced soil sample [18, 24]:

$$p_y / p_z = \mathrm{tg}^2\,(45^0 - 0{,}5\,\varphi) = K_{minimum} \quad (2)$$

and by analogy for soil with reinforcement:

$$p_y^* / p_z = \mathrm{tg}^2\,(45^0 - 0{,}5\,\varphi^*) = K^*_{minimum} \quad (3)$$

There is also a relationship: $\varphi^* > \varphi$ and

$\Delta\varphi = \varphi^* - \varphi$, where φ is the angle of internal friction of the tested soil, $\Delta\varphi$ - the effect of increasing the angle of internal friction.

After substituting the relevant data for the examined medium without reinforcement, we obtain:

$0{,}387 = \mathrm{tg}^2\,(45^0 - 0{,}5\ \varphi)$, the angle $\varphi = 26{,}22^0$ was calculated from this formula (note: this result is approximately consistent with the value of $\varphi = 29{,}5^0$ determined from laboratory tests of a sample of sand mass not reinforced loosely).

For a medium reinforced with e.g. A-type gabion wall:

$0{,}227 = \mathrm{tg}^2\,(45^0 - 0{,}5\ \varphi^*)$, from this formula the angle $\varphi^* = 39^0$, which is much higher than for the model without reinforcement.

After determining the φ and φ^* values from the above formulas, the shear strength of the non-reinforced (comparatively) and reinforced model was then calculated from the conditions:

$$\tau_f = p_z \cdot \mathrm{tg}\varphi \ \text{ oraz } \ \tau_f^* = p_z \cdot \mathrm{tg}\varphi^* \tag{4}$$

For the above dependence $p_z = q_{limiting} = 239.5$ kPa was substituted in accordance with the previously mentioned assumptions. The results of calculating the parameters: φ, $\Delta\varphi$, τ_f are shown in Figure 5.

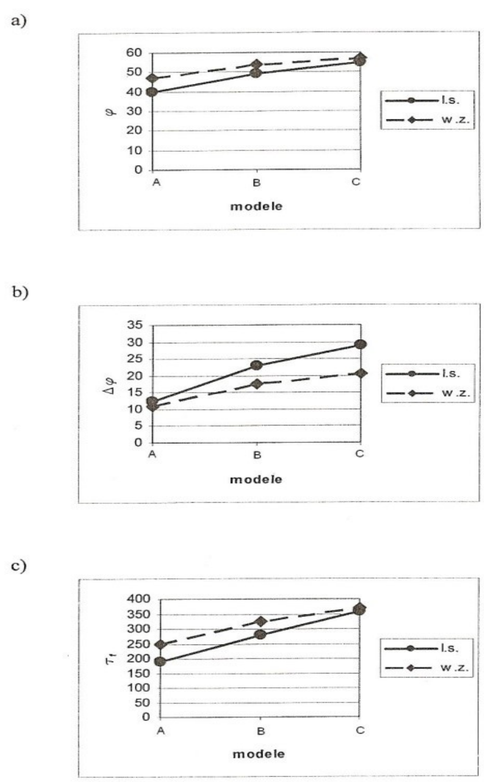


Fig. 5. Physical parameters as characteristics of massif models A, B, C [22, 24]:
a - angle of internal friction φ,
b - increase in internal friction angle $\Delta\varphi$,
c - shear strength τ_f

- **Consistency effect in coherently reinforced soil**

The case (b) of reinforced soil is considered, for which in the system of vertical stresses p_z and horizontal p_y the damage curve is determined by the equation [18, 22, 24]:

$$p_z = p_y \cdot \mathrm{tg}^2\,(45^0 + 0{,}5\ \varphi) + p_0 \tag{5}$$

where:

$$p_0 = 2\ c \cdot \mathrm{tg}\,(45^0 + 0{,}5\ \varphi) \tag{6}$$

is the "initial" stress (when $p_y = 0$), indicating that the reinforced soil behaves as if it had anisotropic coherence. Because there is a limit state (i.e. $p_z = q_{max}$), the value of this consistency c is maximum.

The results of model tests allowed to calculate the element $p_0 = p_z^* - p_z = \Delta p_z$ (effect of increased load capacity). Element p_0 (for different models, i.e. variants of the gabion wall configuration) and the angle of internal friction calculated for the massif not reinforced with the gabion wall $\varphi_{l.s.} = 26{,}22^0$ (for a massif made of loosely packed soil material) and $\varphi_{w.z.} = 36{,}05^0$ (for the pre-compacted massif) was substituted for the formula (6), from which the coherence effect c for individual models was calculated:

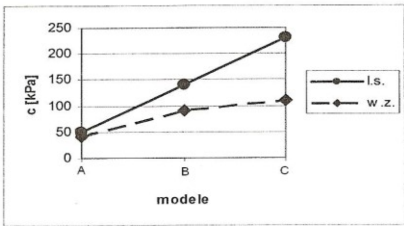
$$c = p_0 \cdot [2\ \mathrm{tg}\,(45^0 + 0{,}5\ \varphi)]^{-1} \tag{7}$$

Then it was assumed that the mechanism of soil sample destruction consists in the slippage of soil medium grains in relation to the insert material (i.e. the reinforcement is not destroyed, and the destruction of the soil sample is symmetrical). The condition of shear reinforced soil is:

$$\tau_f = p_z \cdot \mathrm{tg}\varphi + \Delta\tau_f = p_z \cdot \mathrm{tg}\varphi + c \tag{8}$$

where the first component refers to the massif not reinforced with gabions and the second ($\Delta\tau_f$)is an addition resulting from the strengthening.

The results of calculations of the values of coherence value c and shear strength strength τ_f are f are illustrated in Figure 6. It was found that the consistency obtained as a result of gabion strengthening in the pre-compacted mass (w.z.) is lower in relation to the identical model with loosely packed soil material (l.s) a)



b)

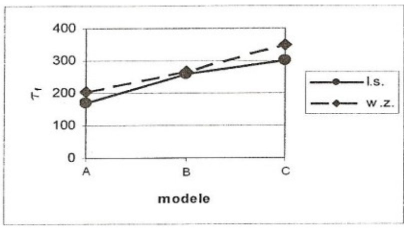


Fig. 6. Physical parameters as characteristics of massif models A, B, C [22, 24]:
a - consistency c [kPa], b - shear strength τ_f [kPa]

6. SUMMARY

As a result of tests carried out on laboratory models of the massif reinforced with a gabion wall using an external static load q [kPa] mapping the operational load, a significant impact of the strengthening on the change (in a positive sense) of the mechanical properties of this centre was found, in particular:

- effects of increased load capacity: with a set value of horizontal deformation (lateral pressure), the permissible external load of the reinforced massif model is a multiple of the permissible load for the massif without reinforcement;
- increase of the soil medium shear strength (effect of increasing the angle of internal friction and the phenomenon of "cohesion" resistance);
- an increase in the deformation modulus and susceptibility modulus (calculated from the theories of elasticity theory): with a continual approach, the non-coherent reinforced medium will be more resistant to deformation.

It seems justified to take into account the structure of the structure (configuration of gabion elements, their geometric dimensions, quality of connections between them) in the process of designing retaining walls composed of gabions.

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