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Research paper

Effect of shear rate and saturation on shear strength of mineral and anthropogenic soil

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Abstract: The aim of the study was to determine the shear strength of mineral and anthropogenic soil of similar grain size as a function of the applied shear rate and water saturation. Stability calculations using the finite element method of the road embankment model were also carried out to demonstrate the variation in factor of safety values depending on the adopted values of the angle of internal friction and cohesion. The tests were carried out in a direct shear apparatus in a 100 × 100 mm box with a sample height of 20.5 mm. The samples were formed directly in the apparatus box at optimum moisture content until a compaction index of $I_S = 1.00$ was obtained. Tests were carried out under conditions without and with water saturation at shear rates of 0.01, 0.05, 0.1, 0.5 and 1.0 mm·min^{-1} until 18% horizontal displacement was achieved. The results showed that the effect of shear rate on the strength parameters was not unequivocal and was much smaller than the changes caused by saturation of samples. An increase in shear rate resulted in small changes in the angle of internal friction with a tendency towards a decrease. In contrast, cohesion varied over a much larger range with increasing shear rate, with an apparent initial decrease and subsequent increase. The saturation of the samples resulted in a decrease in the angle of internal friction of the cohesive soil and an increase for the ash-slag mixture. The cohesion of both soils decreased. The results obtained from the road embankment model stability calculations confirmed that soil saturation had a greater influence on the factor of safety values obtained than the shear rate.

Keywords: shear strength, shear rate, mineral soil, ash-slag mixture, direct shear apparatus

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1. Introduction

Soil is a three-phase medium in which the soil skeleton is the dominant phase, followed by water and air, which fill the pores. From a physical point of view, the soil is therefore an arrangement of randomly oriented grains and particles of different shapes and sizes. The deformation of the medium is the result of the deformation of the individual grains and particles and the relative displacements between them. The shear strength is equal to the load causing the relative displacement of the grains and therefore the sum of the strengths of the individual bonds. This strength is frictional, i.e. proportional to the normal load causing the grains to press against each other. The angle of internal friction depends on the shape and surface properties of the moisture-modified grains and the structure of the soil.

In engineering problems, the description of the soil response to loading is most often based on the Coulomb–Mohr boundary condition. According to this condition, failure of the soil will occur when the shear stress across the soil medium exceeds the frictional and cohesion resistance between grains and particles. According to Coulomb's law (1773), shear strength is a linear function of normal stress:

$$\tau_f = \sigma_N \cdot \tan\varphi + c \ \text{[kPa]} \tag{1.1}$$

where: τ_f – shear strength, [kPa], σ_N – normal stresses, [kPa], ϕ – angle of internal friction, [°], c – cohesion, [kPa].

The surface over which the shear stress meets condition (1.1) is referred to as the shear or failure surface. In this equation, the angle of internal friction and cohesion are constants depending mostly on the type of soil. The angle of internal friction is the angle of inclination of the Coulomb straight line with respect to the axis, while the maximum cohesion corresponds to the shear stress in the absence of normal load.

Research has shown that the values of the parameters describing shear strength depend on many factors (e.g., [1, 3, 4]). Slip surface does not always occur along a surface where the Coulomb–Mohr boundary condition is satisfied, and cohesion often does not represent cohesion forces associated with inter-particle interaction, but is only a linear approximation parameter. Soil strength parameters are not material constants, which may lead to a situation where shear strength parameters determined according to different standards, and consequently the results of engineering calculations (e.g. stability) obtained from these determinations, may differ significantly from each other [5]. In engineering practice, the decision to apply appropriate treatments related to, for example, slope protection or ground reinforcement must be made on the basis of reliable and credible data.

Additional factors that affect the values of the shear strength parameters are also the conditions under which the test is performed. These factors can include specimen failures, shear rate and any apparatus modifications that affect the test methodology. Amsiejus et al. [1] showed that only 65 to 85% of the total normal load is transferred to the shear plane of the samples. The determination of the shear strength of the soil depends on the design of the apparatus and therefore on the actual magnitude of the stresses in the shear rate. As indicated by Gruchot [2], Bek et al. [6], Stefanow and Dudzinski [7], the shear strength also depends on the applied shear rate. The problem of the effect of soil shear rate on soil shear strength parameters has been and still is widely discussed in the literature (e.g., [8–10]). It is assumed that this shear rate should be as low as possible to match the loading conditions found under buildings or in the hydrotechnical and road embankments. The low shear rate of the samples allows the pore water pressure to equalize throughout the volume of the samples (e.g., [11, 12]).

Studies of shear strength parameters show opposite or at least inconsistent values for apparently similar conditions, indicating the need for further research. Therefore, the aim of this manuscript was to determine the changes in shear strength and its parameters for mineral and anthropogenic soil of similar grain size composition as a function of the applied shear rate and saturation of samples. In addition, in order to demonstrate the practical significance of the obtained values of the angle of internal friction of cohesion on the values of the factor of safety, stability calculations were carried out using the finite element method for a hypothetical road embankment model.

2. Methodology

The tests were carried out on two soils of similar grain size distribution classified geotechnically as silty soils – a mineral one of natural origin and an anthropogenic one, which was an ash-slag mixture from hard coal combustion. The scope of the study included the determination of grain size composition, density of solid particles, compaction parameters and shear strength parameters.

The grain size composition was determined by a combined method, that is, using wet sieve analysis for grains larger than 0.063 mm and areometric analysis for particles smaller than 0.063 mm [13, 14]. Compaction parameters (optimum moisture content and maximum dry density of solid particles) were determined by Proctor apparatus (II method) at a compaction energy of 0.59 J·cm^{-3} [15, 16].

The determination of shear strength parameters was carried out in a direct shear apparatus (Fig. 1) [17] in which the lower part of the box was movable. Samples with cross-sectional dimensions of 100×100 mm and a height of 20.5 mm were formed directly in the apparatus box at optimal moisture content until a compaction index of $I_S = 1.00$ was obtained. The samples were consolidated for 30 or 60 minutes for mineral and anthropogenic soil, respectively, at a normal stress of 10 kPa. The mineral soil samples were then consolidated for a further 60 minutes at normal stresses of 25, 50, 75, 100, 125 kPa, and for the anthropogenic soil at stresses of 50, 100, 200, 300, 400, 500 kPa. The value of normal stresses was assumed according to the depth from which the mineral soil was collected (about from 1.0 to 2,0 m) and to the potential use of the ash-slag mixture (road embankment). The tests were carried out under conditions without and with the water saturation of the samples by flooding them to their full height. The process of water saturating was carried out during the consolidation of the samples and during their shearing and it was not possible to control the degree of saturation of the soil. The values of the angle of internal friction and cohesion were calculated using the least squares method.

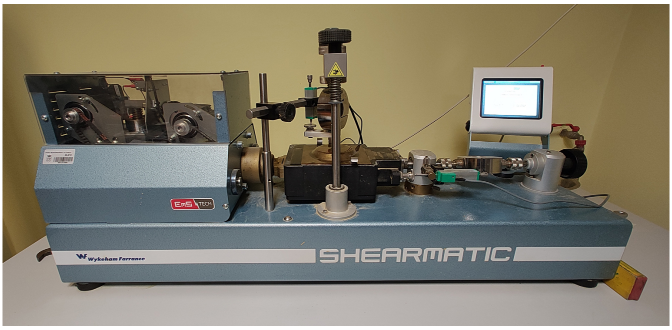


Fig. 1. General view of the Shearmatic direct shear apparatus (photo: A. Gruchot)

For the purpose of determining the appropriate shear rate, the standard [17] proposes, in the case of saturated fine-grained cohesive soils, to use the time dependence of sample settlements (Fig. 2). From this relationship, the value of t_{100}, which is the point of intersection of the linear part of the graph with the almost horizontal line passing through the end points of the initial consolidation, should be read.

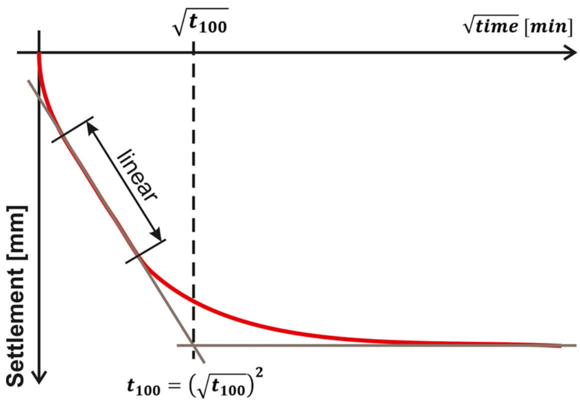


Fig. 2. Graphic representation of determining time [18]

The minimum time to maximum shear strength is calculated from the equation:

$$t_f = 12,7 \cdot t_{100} \tag{2.1}$$

where the coefficient value of 12.7 was derived from consolidation theory and assumes that, at the time of failure, no more than 5% of the pore water pressure remained in the sample. As a consequence of this, the shear rate is:

$$v = \frac{s_t}{t_f} \; [\mathrm{mm \cdot min^{-1}}] \tag{2.2}$$

where: v – maximum allowable shear displacement increment, [mm·min^{-1}] s_t – estimated horizontal shear displacement at failure, [mm], t_f – time to failure, [min].

In this study, due to the high compaction of the specimens and thus the low settlement during consolidation, the shear rate was assumed by ignoring the relationship shown in Fig. 1. Therefore, tests were carried out at shear rates of 0.01, 0.05, 0.1, 0.5 and 1.0 mm·min^{-1} to achieve 18% horizontal displacement.

The shear strength parameters of materials used in earthworks is important for the construction and stability assessment of embankments. Stability analysis of road earthworks is one of the main elements in assessing their safe operation. Eurocode 7 [19] requires the demonstration that the design effects of actions are less than the corresponding design soil resistance determined from the design geotechnical parameters. Stability calculations were carried out for a hypothetical road embankment model with a crest height and width of 10 m each and a slope inclination of 1:1.5 (Fig. 3). The calculations assumed a uniformly distributed vertical load of 25 kPa applied to the crest of the embankment and calculations were also made without a load. Calculations were also carried out with the water table at a height of 1,0 m below its crest and thus the flow established through the earth embankment (Fig. 4). It was assumed that the embankment was made of an ash-slag mixture and that the subsoil was the mineral soil used in the study. The shear strength parameters and the bulk weight were assumed on the basis of the test results obtained. Calculations were carried out in accordance with the assumptions of Eurocode 7, adopting the calculation approach DA3 [19]. The calculations were carried out using the finite element method with the Geo5 program for an elastically perfectly plastic soil model. The obtained results were related to the "probability of landslide occurrence" in accordance with the Wysokiński [20] and the requirements of Eurocode 7. Fig. 3 shows the division of the embankment model and its base into 9026 elements with an edge length of 0.50 m and the number of nodes 16755.

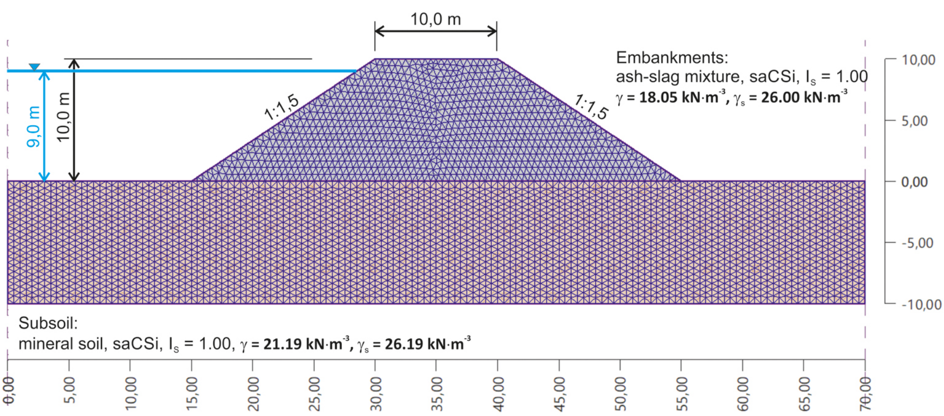


Fig. 3. Diagram of the computational model of a road embankment with an inclination of slope of 1:1.5

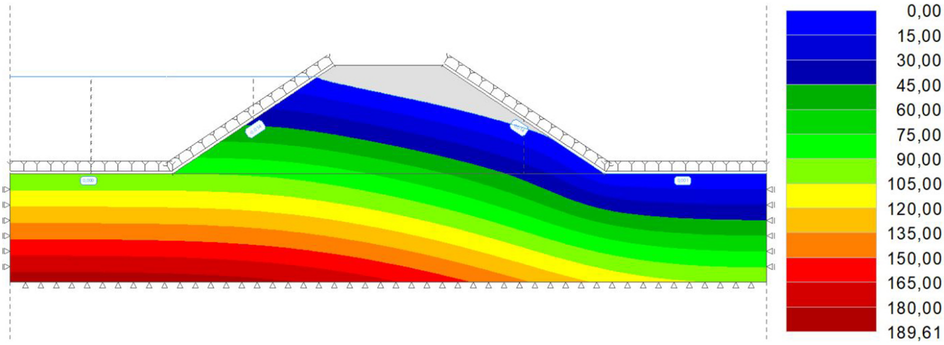


Fig. 4. Pore pressure distribution [kPa] in FEM embankment stability calculations under steady flow conditions

3. Results and discusion

3.1. Granulometric composition and compactability

According to the geotechnical nomenclature, the grain size of both soils corresponded to a multifraction soil of coarse silt with sand (Table 1, Fig. 5). The density of solid particles of the mineral soil was 2.67 g·cm^{-3}, and of the ash-slag mixture was 2.51 g·cm^{-3}.

Table 1. Geotechnical characteristics of tested materials

Parameter	Value for the type of soil	
	mineral	antropogenic – ash-slag mixture
Fraction content [%]:		
– gravel, Gr: 63 ÷ 2 mm – sand, Sa: 2 ÷ 0.063 mm – silt, Si: 0.063 ÷ 0.002 mm – clay, Cl: < 0.002 mm	0.63 45.40 51.22 2.76	6.88 47.08 42.97 3.08
Uniformity coefficient, C_U [–]	7.5	34.0
Coefficient of curvature, C_C [–]	2.0	0.6
Name of soil acc. to [13]	coarse silt with sand (saCSi)	coarse silt with sand (saCSi)
Density of solid particles [g· cm^{-3}]	2.67	2.52
Optimum moisture content [%]	10.2 10.8	1.96 1.92
Maximum dry density of solid particles [g·cm^{-3}]	19.1 20.3	1.55 1.52

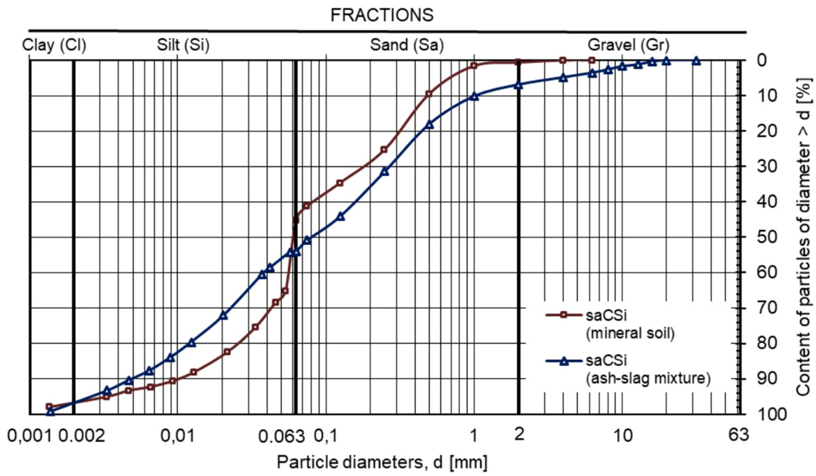


Fig. 5. Grain size curves of the soils

The maximum dry density of solid particles of the mineral soil averaged 1.94 g·cm^{-3} at an average optimum moisture content of 10.5%. In contrast, the maximum dry density of solid particles of the ash-slag mixture skeleton averaged 1.53 g·cm^{-3} at an optimum moisture content of 19.7% (Fig. 6). Similar values for the density parameters of ash-slag mixtures were also found by Gruchot [2]. The values of the maximum dry density of solid particles of both soils meet the condition of adequate compatibility with regard to their applicability in road embankments [21].

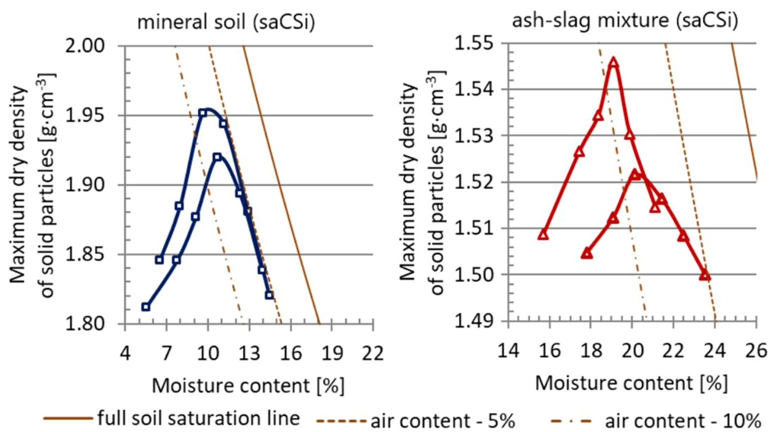


Fig. 6. Compaction curves of the tested soils

3.2. Consolidation settlements

Analysis of the settlement values of the samples during consolidation showed small values, not exceeding 0.15 mm for both soils regardless of normal stresses (Fig. 7). In contrast, quite significant changes in the height of the samples occurred during shearing. Samples of both soils after test at normal stresses lower than 100 kPa without saturation were characterised by a decrease and at higher stresses by an increase in their height. On the other hand, in tests of samples with saturation, an increase in their height was obtained, irrespective of normal stresses. The correlations obtained may be due to the high compaction of the tested soils, which had the effect of reducing consolidation deformation and during shearing, resulted in increased loosening due to grain rotation and displacement.

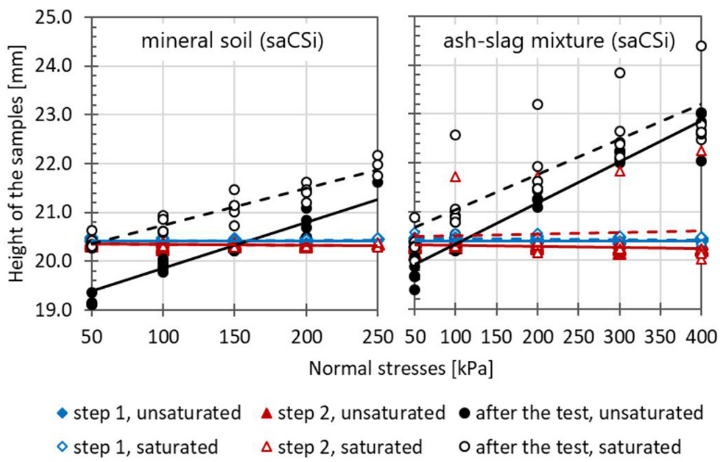


Fig. 7. Changes in the height of test soil samples in relation to normal stresses and their occupation

The shear rate determined on the basis of consolidation settlements and thus the recommendations given in [17], for example, for unsaturation mineral soil samples was approximately 2.4 mm·min^{-1} at normal stresses of 50 kPa and 0.6 mm·min^{-1} at normal stresses of 250 kPa. Correspondingly, for the ash-slag mixture it was also 2.5 mm·min^{-1} at normal stresses of 50 kPa and 1.8 mm·min^{-1} at normal stresses of 400 kPa. Therefore, the shear rates in the presented studies were assumed by resigning from their determination based on the dependence of settlement on the consolidation time.

The specified shear rate values are much higher than the maximum shear rate recommended by [17]. Therefore, the principle of selecting the shear rate according to Fig. 2 and equations (2.1) and (2.2) was ignored in the presented tests.

From the point of view of testing on high porosity samples, where high vertical deformation can be expected, the determination of shear rates according to [17] is appropriate. However, with high soil compaction and therefore low porosity, the maximum settlements occur during the initial consolidation time and the shear rates obtained may be well above 1.0 mm·min^{-1}, which may cause errors in the interpretation of the shear strength parameters obtained.

3.3. Shear strength

Analysis of the dependence of shear stress on horizontal displacement revealed that brittle shear was obtained in the case of tests without sample water saturation (Fig. 8). The maximum value of shear stress was obtained at the initial shear time of the samples with a relative displacement of the samples between 2.5 and 6.5%. After reaching the maximum value, the stresses stabilised after a slight reduction. On the other hand, plastic shear was obtained in the tests of the samples with saturation. Fig. 8 shows an example of the change in shear stresses with increasing horizontal displacement of the samples in tests at shear rates of 0.01, 0.1 and 1.0 mm·min^{-1}.

The values of the maximum shear stresses depended on the type of soil tested, the normal stresses and the saturation of the samples (Fig. 9). The values of the determination coefficient R^2 of the shear strength lines were close to or equal to 1.0, indicating a high fit between the obtained shear stress values and the normal stresses. Both the highest and lowest shear strength values were obtained for the mineral soil in the tests without and with saturation of the samples, respectively.

The range of shear stress values for the mineral soil varied from 74 to 238 kPa and from 47 to 195 kPa without and with saturation of the samples at normal stresses of 50 to 250 kPa, respectively. The saturated of the samples reduced the shear strength from 37% to 18% in relatively to its minimum and maximum values, respectively. On the other hand, for the ash-slag mixture, the range of shear stress values varied from 63 to 390 kPa and from 44 to 390 kPa, respectively, without and with the saturation of the samples at normal stresses from 50 to 500 kPa. The water saturated of the samples reduced the shear strength by 30% to 3% in relative terms to its minimum and maximum values, respectively.

The results showed that the shear stress values varied according to the shear rate. In the case of tests on unsaturated, the range of differences in shear stress values for the same normal stress values was 10 to 18 kPa and 7 to 26 kPa, and in the case of tests on unconsolidated samples it was 3 to 64 kPa and 6 to 120 kPa for cohesive soil and ash-slag mixture, respectively. The nature of the change in shear strength values at the same normal stress values (Fig. 10) depended mainly on the type of soil and also on the saturation of the samples. In general, in the case of tests on mineral soil samples, an initial increase and subsequent stabilisation of shear stresses was obtained. In the case of the ash-slag mixture, the relationship was different – an initial reduction and then a slight increase in shear stress values was obtained as the shear rate increased in the range 0.01–0.1 mm·min^{-1}. The lowest shear stress values for the mineral soil were obtained at shear velocities of 0.01 and 0.05 mm·min^{-1}, and for the ash-slag mixture at 0.05 and 0.1 mm·min^{-1}, respectively, in tests of samples without and with saturation. This relationship may be due to the fact that saturation of the samples at low shear rates increases settlement.

The values of the angle of internal friction of both soils were within a small range. In the case of the mineral soil, the range was from 36.2° to 37.2° and from 34.6° to 35.5° under the conditions without and with the saturation of the samples, respectively (Table 2). On the other hand, the range of angle of internal friction values of the ash-slag mixture was from 32.4° to 33.9° and from 33.6° to 34.8° under conditions without and with saturation of the

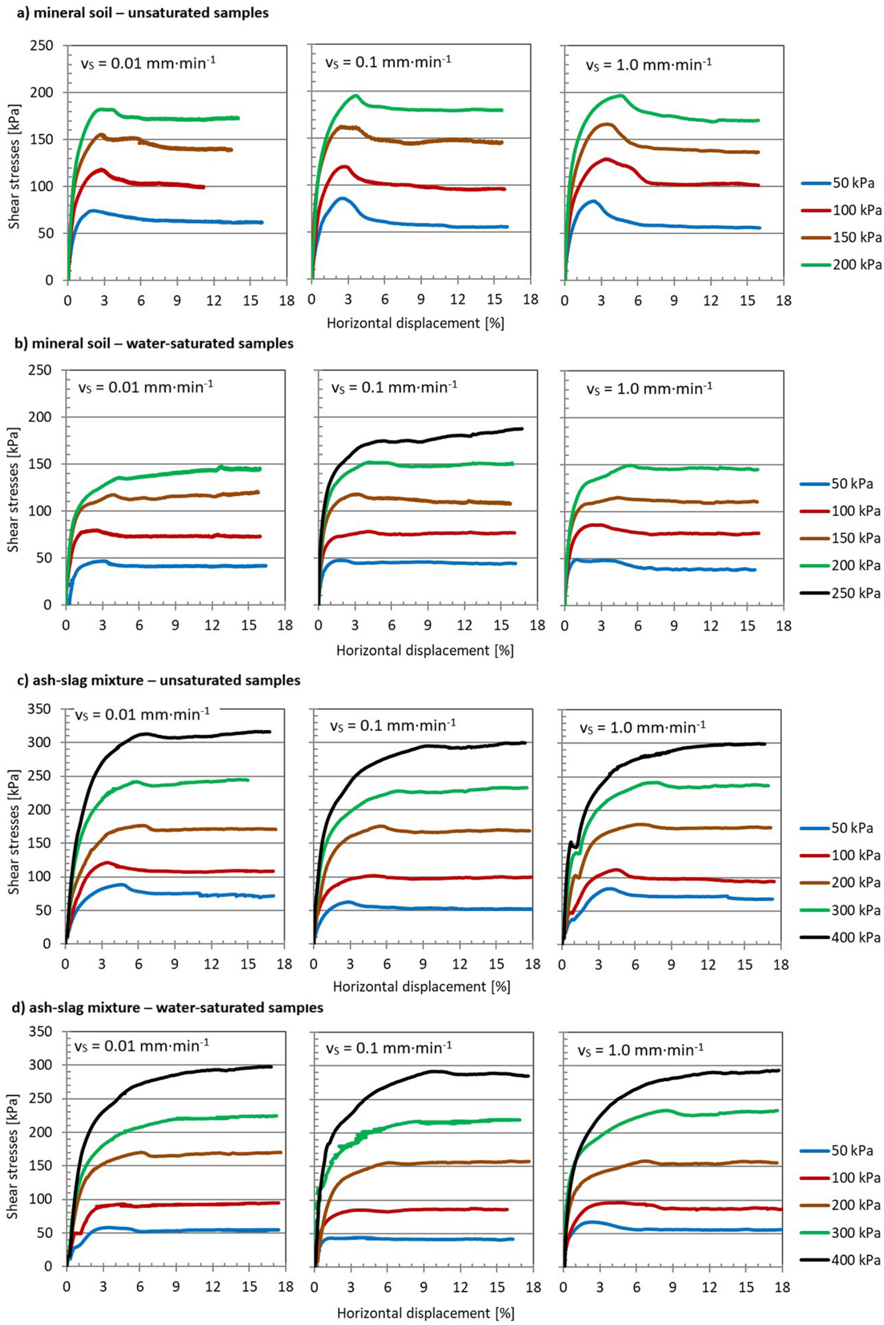


Fig. 8. Changes in shear stresses along with increasing horizontal deformation of samples

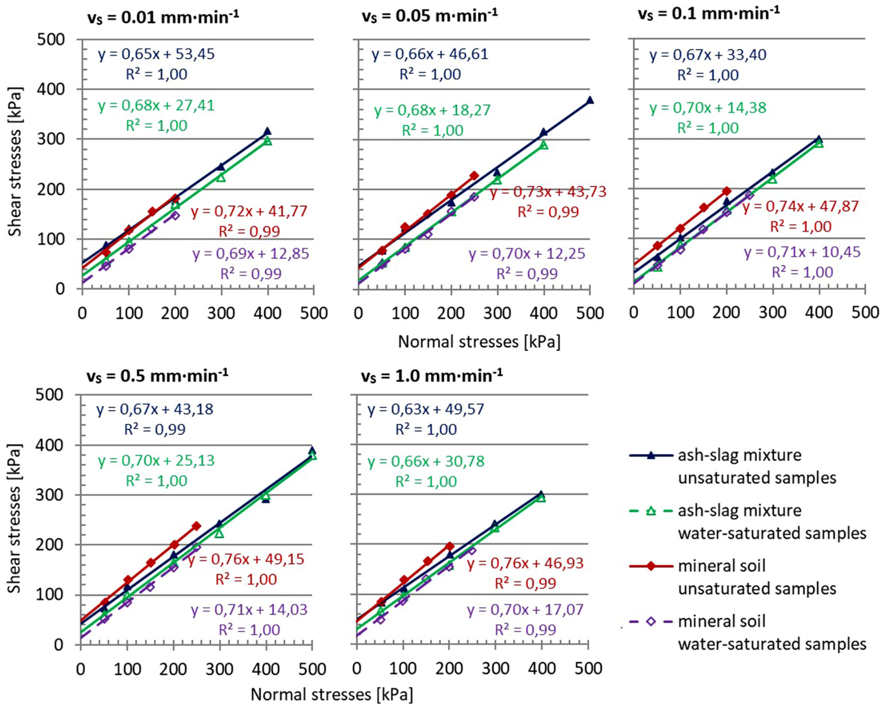


Fig. 9. Shear strength line of tested soils

samples, respectively. On the other hand, the cohesion values of both soils were high and in the case of tests without saturation the range was similar, ranging from 42 to 49 kPa and from 33 to 54 kPa for the mineral soil and the ash-slag mixture, respectively. For the tests with saturation, significant differences in the range of cohesion values were obtained. For the mineral soil, the range was from 10 to 17 kPa and for the ash-slag mixture from 14 to 31 kPa.

The obtained values of shear strength parameters for mineral soil were typical for this type of soil, as indicated, among others, by Zydroń and Dąbrowska [22], who, in their study of sandy clay silt, obtained angle of internal friction values varied from 32° to 40° and cohesion from 9 to 31 kPa. On the other hand, with regard to the ash-slag mixture, Gruchot [2] indicates that these materials are characterised by high values of shear strength parameters, depending on the compaction, moisture content, shear rate, and also on the saturation of the samples. The values of the angle of internal friction range from 32° to 46°, while the high cohesion values, in the range of 12 to 50 kPa, are due to the interlocking of the coarser grains of the ash-slag mixtures [2].

The effect of saturation on shear strength parameters depended primarily on the type of soil tested. It should be noted that the moisture content of unsaturated samples decreased by slightly less than 1% after testing, while it increased by an average of approximately 4.5% in the case of mineral soil samples and 5.5% in the case of ash-slag mixtures in relation to the optimum moisture content.

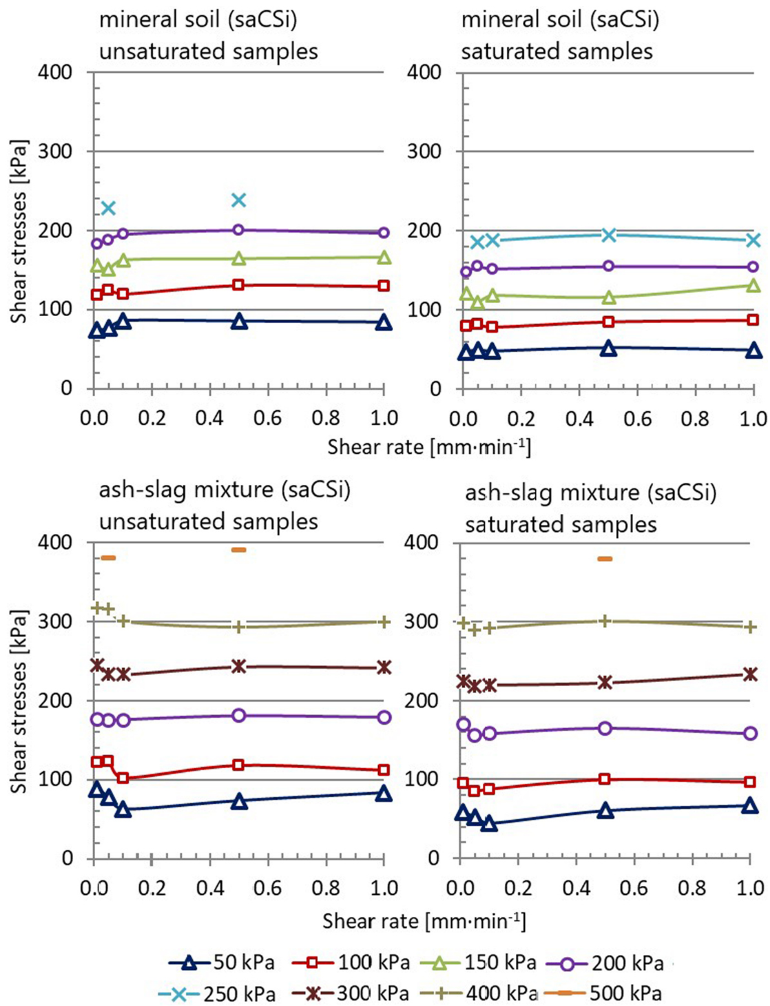


Fig. 10. Changes in shear stress with increasing shear rate for applied normal stresses

An increase in moisture content due to watering of the mineral soil samples resulted in a decrease in the angle of internal friction from about 1° (at v_s = 0.1 mm-min^{-1}) to just over 2° (at v_s = 1.0 mm·min^{-1}), which corresponded to 3% to 6% in relative terms (Fig. 11). In the case of cohesion, on the other hand, it was found to decrease significantly, from about 29 kPa (at v_s = 0.01 mm·min^{-1}) to over 37 kPa (at v_s = 0.1 mm·min^{-1}), which corresponded to 69 to 78% in relative terms. In the case of the ash-slag mixture, the increase in moisture content resulted in an increase in the angle of internal friction averaging about 1° for all shear rates used which was about 2% to 4% relative. In contrast, cohesion similarly to the cohesive soil decreased from about 18 kPa (at v_s = 1.0 mm·min^{-1}) to 29 kPa (at v_s = 0.05 mm·min^{-1}) which corresponded to 42% to 62% relative.

Table 2. Values of shear strength parameters and factor of safety

Shear rate, v_s [mm·min^{-1}]	1.0		0.5		0.1		0.05		0.01	
Type of soil[1)]	A	B	A	B	A	B	A	B	A	B
Tests and calculations for shear conditions without water saturation of the samples										
Moisture content, w [%]	10.1	18.9	10.0	18.7	10.0	18.9	10.0	18.8	9.8	18.8
Shear strength parameters: angle of internal friction (ϕ), cohesion (c)										
ϕ [°]	37.2	32.4	37.1	33.8	36.5	33.9	36.2	33.5	35.9	32.9
c [kPa]	46.9	49.6	49.2	43.2	47.9	33.4	43.7	46.6	41.8	53.5
The factor of safety FS of the embankment under the load of the embankment:										
0 kPa	2.08		2.29		2.20		1.93		2.57	
25 kPa	2.32		2.17		1.98		2.11		1.92	
Tests and calculations for shear conditions with water saturation of the samples										
Moisture content, w [%]	15.0	25.0	15.1	24.9	15.1	25.2	15.6	25.2	15.2	25.1
Shear strength parameters: angle of internal friction (ϕ), cohesion (c)										
ϕ [°]	35.0	33.6	35.5	34.8	35.4	34.8	34.8	34.3	34.6	34.1
c [kPa]	17.1	30.8	14.0	25.1	10.4	14.4	12.2	17.7	12.8	27.4
The factor of safety FS of the embankment under the load of the embankment:										
0 kPa	1.54		1.60		1.35		1.50		1.54	
25 kPa	1.23		1.14		0.90		1.11		1.23	
25 kPa with steady flow	1.07		1.11		0.73		0.73		1.07	

[1)] Type of soil:
A – mineral soil (saCSi), in stability calculations this soil builds the subsoil,
B – ash-slag mixture (saCSi), in stability calculations this soil builds embankment.

Zawisza and Zydroń [23], who carried out tests an ash-slag mixture from the landfill of the "Kraków" Power Plant, point out that moisture content is also important for shear strength parameters in addition to compaction. The authors find that the angle of internal friction decreases with increasing moisture content, with the range of changes being small and varying from about 1° to over 3°. Changes in cohesion have an inverse relationship with moisture content compared to the internal friction angle. At the optimum moisture content, higher cohesion values ranging from 4 to nearly 15 kPa were obtained (at $I_S = 0.90$ and 1.00, respectively) compared to moisture content lower than the optimum by 14%. On the other hand, Gruchot and Łojewska [24] and Gruchot and Resiuła [25] studied in a direct shear apparatus the shear strength parameters of an ash-slag mixture from the "Skawina" Power Plant landfill at different moisture contents of the samples – lower, equal and higher than the optimum, and at different compaction and shear rate. These authors found that the highest values of the angle of internal friction and cohesion were obtained at a moisture content 5% lower and the lowest at a moisture content 5% higher than the optimum. The reduction in the angle of internal friction ranged from 3° to 7°, and in cohesion from about 2 to 23 kPa as moisture content increased [24].

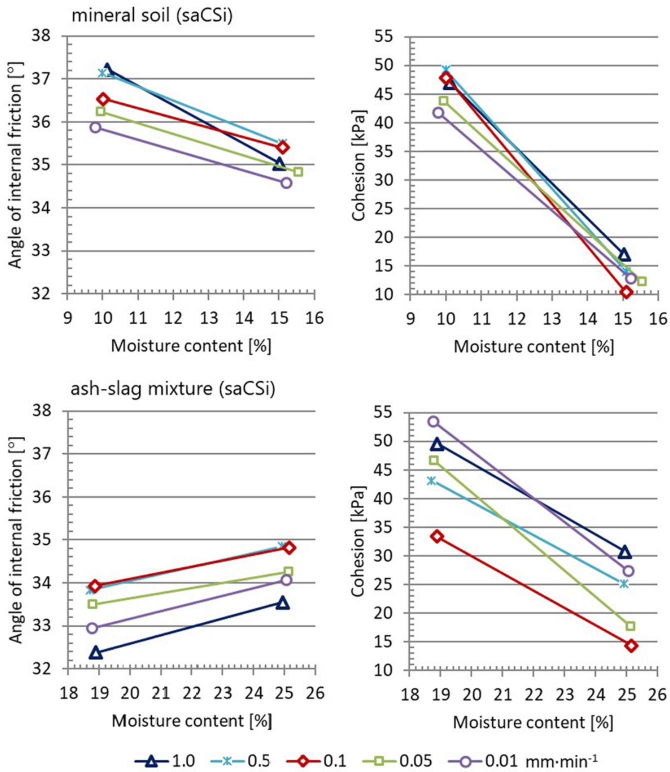


Fig. 11. Influence of moisture on internal friction angle and cohesion

The effect of shear rate on the shear strength parameters varied for both soils and depended on the saturation of the samples (Fig. 12). Only the mineral soil tests showed an increase in the angle of internal friction of 1.3° (4% relative) over the full range of shear rates used. On the other hand, in the tests with mineral saturated soil, as well as in the tests of the ash-slag mixture without and with saturation of the samples, an increase in the internal friction angle of approximately 1.0° (3% relative) was obtained with an increase in shear rate of 0.01 to 0.1 mm·min^{-1}, respectively. Subsequently, at a shear rate of 0.5 mm·min^{-1}, a negligibly small increase or decrease in its value was found. A further increase in shear velocity to 1.0 mm·min^{-1} resulted in a decrease in the internal friction angle of about 0.5° (1% relative) of the mineral soil with the infiltration of the samples and of about 1.4° (4% relative) of the ash-slag mixture under conditions without and with infiltration. It should be noted that the values of the internal friction angle of the ash-slag mixture obtained at a shear rate of 1.0 mm· min^{-1} were on average about 0.5° (2% relative) lower than the values at a shear rate of 0.01 mm·min^{-1}.

The cohesion of the mineral soil in tests without saturation of the samples with an increase in shear velocity from 0.01 to 0.5 mm·min^{-1} increased by 7.4 kPa (18% relative), a further increase in shear rate resulted in a decrease of 2.3 kPa (5% relative). On the other

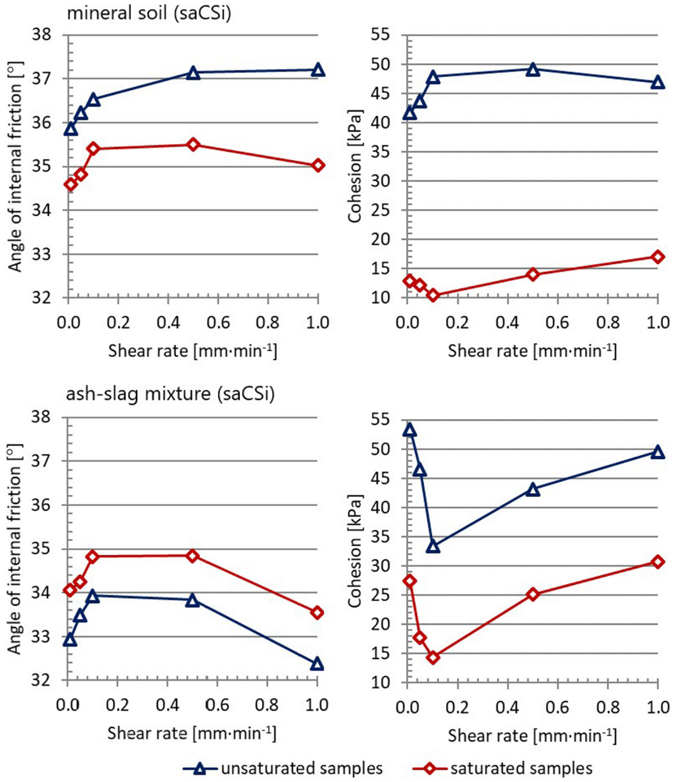


Fig. 12. Relationship of the angle of internal friction and cohesion to the shear rate

hand, in mineral soil tests with slurry and in ash-slag mixture tests without and with saturated samples, an increase in velocity from 0.01 to 0.1 mm·min^{-1} was found to reduce cohesion by 2.4, 20.1, 13.0 kPa (19, 38, 48% relative), respectively. Further increases in shear rate to 0.5 and 1.0 mm·min^{-1} resulted in an increase in cohesion values of 2.3 kPa (5% relative) in the mineral soil tests and an average of 6.0 kPa (18% relative) in the ash-slag mixture.

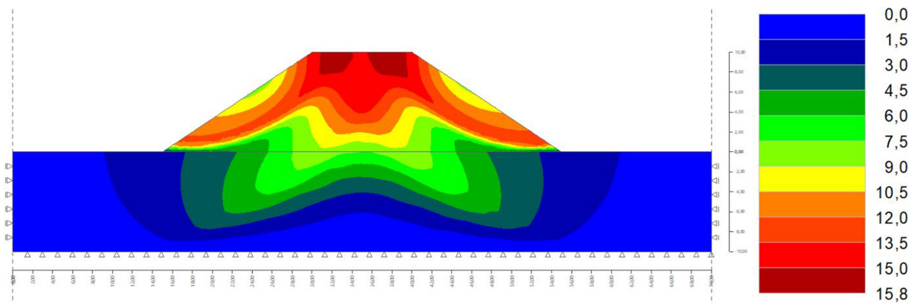
Zydroń and Dąbrowska [22] studied the shear strength parameters of silty soils sampled from landslide sites in southern Poland with three shear rates (0.1, 1.0 and 10 mm·min^{-1}). In general, an increase in shear rate resulted in a decrease in the angle of internal friction and cohesion. The largest changes in these parameters were for samples sheared at 10 mm·min^{-1}. The reduction in the value of the angle of internal friction for the soil with close to 30% sand fraction content was 24° which corresponded to an 85% reduction in the angle of internal friction, and for the silty soil it was 21° (54%), while the reduction in the cohesion value for these soils was 51 and 31 kPa respectively (average 90%). The effect of shear rate on the residual shear strength of cohesive soil-sand mixtures was also investigated by Scaringi and Di Mai [3]. These authors demonstrated the influence of shear rate, but also point to other factors (NaCl content or sand content) having a greater influence on the values of shear strength parameters. Similarly, studies of Bek et al. [6] on the effect of shear rate on the shear strength of cohesive soil showed that an increase in shear rate increased cohesion and decreased the angle of internal friction. In contrast, Deying Li et al. [26] investigating the effect of the degree of consolidation (OCR) and shear rate (0.06–30 mm·min^{-1}) on the residual strength of landslide soils of the Three Gorges Reservoir showed that the reconsolidation of the samples did not have a significant effect, and the shear rate did not show regularity. They attributed the decrease in residual strength at higher shear rate to an increase in water content in the shear plane and an increase in finer particles due to their mechanical disintegration. Quite different conclusions were reached by Beren et al. [27], who, using shear rates ranging from 0.05 to 5 mm·min^{-1} in their study of sand samples without and with saturation, showed that the internal friction angle and peak shear strength increased with increasing velocity, particularly for velocities greater than 1 mm·min^{-1}. Radaszewski and Stefaniak [28] compared the results of field tests of undrained shear strength and shear strength from the direct shear apparatus of silty soils (saSi, clSi) taken from around Poznań (Poland). They showed that the deformation range of up to 10% used in the direct shear apparatus is insufficient to obtain the peak value of the soil shear strength. They also showed that shear strength obtains higher values at lower shear rates. Interesting conclusions regarding the speed of penetration of the probe cone in intermediate soils were reached by DeJong et al. [29]. They analysed errors due to partial consolidation during standard cone penetration (2 cm·s^{-1}) on the interpretation of soil behaviour, as well as pore pressure dissipation data. They demonstrated the necessity of adjusting the penetration rate according to the measurements made under conditions without and with drainage.

The influence of shear rate on the shear strength parameters of the ash-slag mixture from the "Skawina" Power Plant was also dealt with by Gruchot and Łojewska [24], Gruchot and Resiuła [25]. These authors showed that internal friction angle values were higher by about 1° to 5° at higher shear rates (1.0 mm·min^{-1}). However, the effect of shear rate should also be linked to moisture content, as greater values of internal friction angle by approximately 1° to 6° were obtained at lower shear speeds of 0.1 mm·min^{-1}, with a compaction index of I_S = 1.00 and moisture content lower and higher than the optimum. Larger cohesion values were obtained at the lower shear rate and the differences between the values obtained at the two rates ranged from about 6 to 14 kPa.

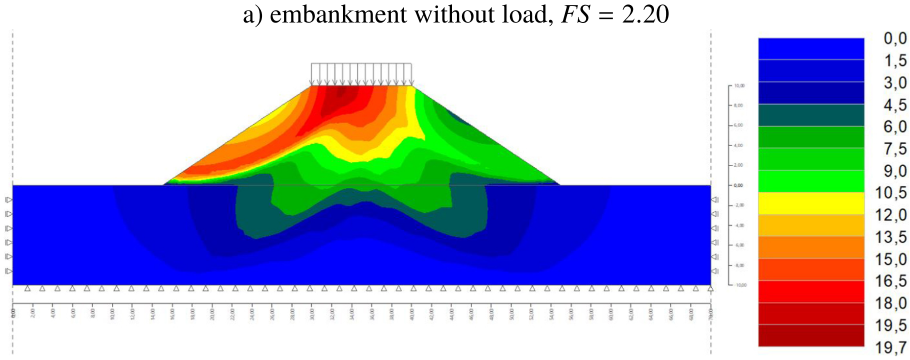
4. Stability calculations results and discussion

The factor of safety values of the road embankment model obtained from the finite element calculations depended on the shear strength parameters used in the calculations, as well as the embankment loading and the occurrence of water seepage. The main extent of displacement (deformation) occurred in the body of the embankment, thus ditto the ash-slag mixture, and hence the shear strength parameters of this soil played a decisive role in the calculation results. In addition, when analysing the influence of the shear strength parameters, it should be clearly emphasised that the value of the factor of safety was significantly influenced by the unsaturation of the samples and the shear rate. In the case of calculations carried out with the assumption that the embankment was not saturated, the factor of safety ranged from $FS = 1.93$ to 2.57 (Table 2), and with load of the crest ranged from $FS = 1.92$ to 2.32. Thus, it can be concluded that the probability of slope failure of the embankment was unlikely [20]. The high values of the factor of safety obtained may be due to the high values of the shear strength parameters and, above all, cohesion, which were generally high (greater than 40 kPa). The problem arising from the use of such high cohesion values in the calculations was pointed out by Gruchot and Zydroń [30], indicating the need to reduce this parameter in the case of anthropogenic soils by up to 50%. The results of the calculations also indicate that using the shear strength parameters from the tests at 1.0, and 0.05 mm·min^{-1}, higher values of the factor of safety at the loading of the embankment crest were obtained, which may also be due to the reinforcement of the embankment during its deformation. Fig. 13 and 14 show the resultant displacements from the embankment model calculations using the shear strength parameters obtained at a shear rate of 0.1 mm·min^{-1}.

Assuming that the embankment was partly saturated and therefore assuming that the shear strength parameters corresponded to the results of the tests with water saturated samples, the factor of safety values decreased and ranged from $FS = 1.35$ to 1.60 and $FS = 0.90$ to 1.23 under conditions without and with embankment loading, respectively

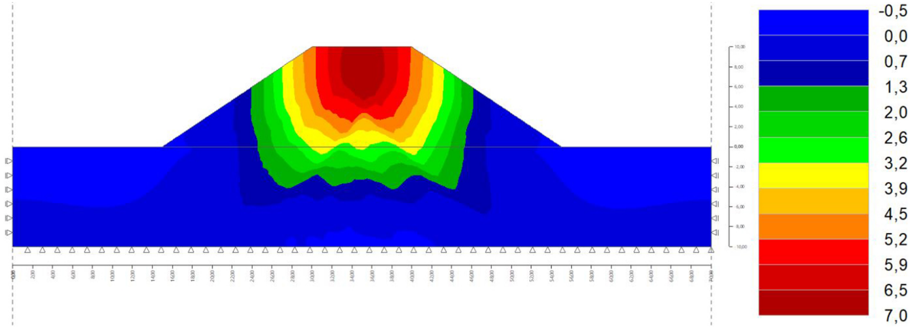


a) embankment without load, $FS = 2.20$

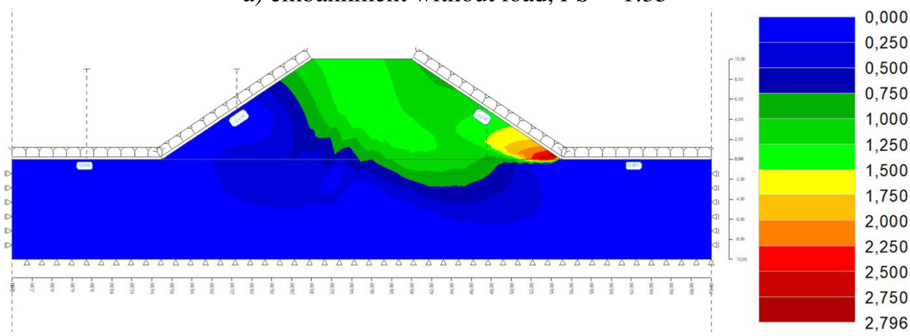


b) embankment with load of 25 kPa, $FS = 1.98$

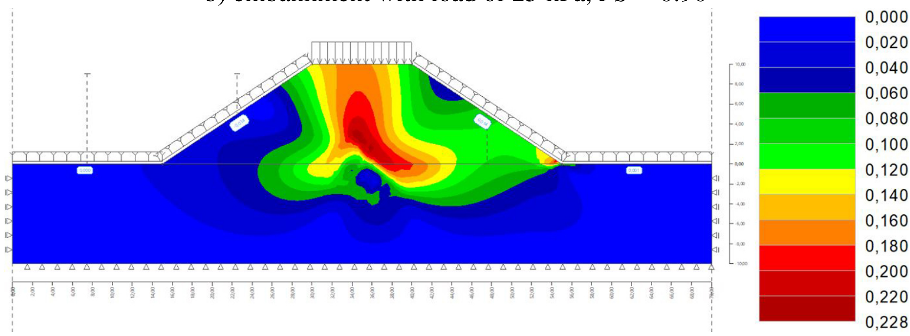
Fig. 13. Displacement [mm] of the embankment from stability calculations including shear strength parameters obtained from tests without water saturated of the samples at a shear rate of 0.1 mm·min^{-1}



a) embankment without load, $FS = 1.35$



b) embankment with load of 25 kPa, $FS = 0.90$



c) embankment with load of 25 kPa with steady flow, $FS = 0.73$

Fig. 14. Displacement [mm] of embankment from stability calculations including shear strength parameters obtained from tests with water saturated of the samples at shear rate of 0.1 mm$\cdot$min^{-1}

(Table 2). In this case, the probability of loss of stability was very likely (parameters at a shear rate of 0.1 mm$\cdot$min^{-1}, with embankment loading) to probable and unlikely. Assuming additionally that the embankment was damming water (so it can be assumed that it was a road embankment periodically fulfilling the role of e.g. a levee) and consequently there was a steady seepage through the loaded embankment, the values of the factor of safety ranged from $FS = 0.73$ to 1.07, indicating a very likely or probable slope failure [20].

In summary, it can be concluded that the factor of safety of an embankment made of an ash-slag mixture is significantly reduced under saturated conditions. Only if the embankment is protected from its saturation and there is no water filtration through it can it be considered that the risk of loss of stability will be unlikely. Therefore, all engineering treatments should be used to limit water infiltration into the embankment, and ash-slag mixtures should not be used in the area of periodic flooding by surface water.

5. Summary

Determining the correct values of the parameters characterising the shear strength of soils is important in the design and construction of all types of earth embankments. The test procedure has a significant impact on the results obtained, so it is very important to choose it correctly depending on the purpose of determining the shear strength parameters.

The results show that the effect of shear rate in the range of 0.01 to 1.0 mm·min^{-1} on the shear strength parameters resulted in much smaller changes in them compared to the changes caused by water saturation of the samples. An increase in shear rates resulted in slight changes in the angle of internal friction with a tendency towards a decrease. In contrast, cohesion varied over a much larger range with increasing shear rate, with an apparent initial decrease and subsequent increase. The saturation of the samples resulted in a decrease in the angle of internal friction and cohesion of the mineral soil, which would indicate a reduction in frictional resistance due to increased moisture content. In the case of the ash-slag mixture, the saturation of the samples caused a slight increase in the value of the angle of internal friction and a clear decrease in cohesion, which could be due to the specific structure of this type of material, as well as its chemical composition.

It should be made clear that increasing the shear rate may result in an excessive increase in pore water pressure, which may reduce frictional resistance, and the application of these test results in design calculations may underestimate the stability results and/or bearing capacity of the subsoil. If the shear rate is too low, the values of the shear strength parameters may be unrealistically favourable, which is also not conducive to the required safety. Of course, from a practical point of view, a low shear rate increases the execution time of such a test. For these reasons, the selection of an appropriate shear rate is of great importance and it is suggested, for tests on highly compacted samples, to use a shear rate that allows the required horizontal deformation to be achieved in a time not exceeding 4–6 hours.

The results obtained from the stability calculations of the road embankment model showed that the adoption of shear strength parameters from tests with saturation of the samples of the soils in question had a greater influence on the factor of safety values obtained than the shear rate used. On the other hand, the most unfavourable values of factor of safety were obtained using shear strength parameters determined at a shear rate of 0.1 mm·min^{-1}. Thus, it can be concluded that, in the case of tests on ash-slag mixtures, this shear rate allows a safe estimation of the strength parameters.

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Wpływ prędkości ścinania i zawodnienia na wytrzymałość na ścinanie gruntu mineralnego i antropogenicznego

Słowa kluczowe: wytrzymałość na ścinanie, prędkość ścięcia, grunt mineralny, mieszanina popiołowo-żużlowa, aparat bezpośredniego ścinania

Streszczenie:

Celem pracy było określenie wytrzymałości na ścinanie gruntu mineralnego i antropogenicznego w zależności od zastosowanych prędkości ścięcia i zawodnienia. Wykonano również obliczenia stateczności metodą elementów skończonych modelu nasypu drogowego w celu wykazania zmian w wartościach współczynnika stateczności w zależności od przyjętych wartości kąta tarcia wewnętrznego i spójności. Badania przeprowadzono w aparacie bezpośredniego ścinania w skrzynce o wymiarach 100×100 mm i wysokości próbki 20,5 mm. Próbki formowano bezpośrednio w skrzynce aparatu przy wilgotności optymalnej do uzyskania wskaźnika zagęszczenia $I_S = 1{,}00$. Badania przeprowadzono w warunkach bez i z zawodnieniem przy prędkości ścięcia 0,01, 0,05, 0,1, 0,5 i 1,0 mm·min^{-1}. Wyniki badań wykazały, że wpływ prędkości ścięcia na parametry wytrzymałości nie był jednoznaczny i był znacznie mniejszy niż zmiany spowodowane zawodnieniem próbek. Wzrost prędkości spowodował niewielkie zmiany kąta tarcia wewnętrznego z tendencją na jego zmniejszenie. Natomiast spójność wraz ze wzrostem prędkości ścięcia wahała się w znacznie większym zakresie, z widocznym początkowym jej zmniejszeniem, a następnie zwiększeniem. Uzyskane wyniki obliczeń stateczności modelu nasypu drogowego potwierdziły, że większy wpływ na uzyskiwane wartości współczynnika stateczności miało zawodnienie gruntu niż prędkość ścięcia.

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