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INTERACTION BETWEEN FLY ASH AND BOTTOM ASH MIXTURE AS A SEALING LAYER OF WASTE DISPOSAL SITES AND HDPE GEOMEMBRANES

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Key words: fly ash, fly ash/bottom ash mixture, waste disposal, sealing layers, HDPE geomembrane, interface shear strength.

INTERAKCJA POMIĘDZY MIESZANINĄ POPIOŁOWO-ŻUŻLOWĄ JAKO WARSTWĄ USZCZELNIAJĄCĄ SKŁADOWISKA ODPADÓW I GEOMEMBRANAMI HDPE

W pracy określono wytrzymałość na ścianie kontaktu międzyfazowego, pomiędzy geomembranami o różnych fakturach stosowanymi do wykonywania sztucznych uszczelnień składowisk odpadów, a zagęszczoną mieszaniną popiołowo-żużlową. Badania przeprowadzono w klasycznym aparacie bezpośredniego ścinania, wykorzystując zmodyfikowaną cylindryczną skrzynkę aparatu. Skrzynkę wyposażono w dodatkową część, umożliwiającą badania interakcji pomiędzy zagęszczonym popiołem lotnym i geomembraną HDPE. Stwierdzono, że ocena wytrzymałości kontaktu nie zależy od zagęszczenia próbki. Na wartość wytrzymałości na ścinanie pomiędzy próbką popiołową i geomembraną ma wpływ jedynie struktura powierzchni geomembrany. W przypadku geomembran o zróżnicowanej teksturze otrzymuje się wyższe wartości kąta tarcia międzyfazowego, a dla geomembrany gładkiej – adhezji.

Summary

Interface shear strength between geomembranes with various textures, which are used for carrying out the artificial sealing of waste disposal, and compacted fly ash/bottom ash mix, was determined in the paper. The tests were conducted in a classical direct shear apparatus, with the use of a modified cylindrical box. The box was equipped with an additional part, which enabled interaction testing between compacted waste and HDPE geomembrane. It was found that interface strength estimation does not depend on sample compaction. Only geomembrane structure has an effect on shear strength between waste sample and geomembrane. In the case of geomembranes with diverse structure greater values of interface friction angle are obtained, and for smooth geomembrane – greater values of adhesion.

INTRODUCTION

The most significant element of the municipal landfill and hazardous waste disposal site construction, and existing storage yard modernization and development as well, is storage yard leak-proof assurance, which allows reducing the negative waste influence on the environment. Waste disposal site tightness is achieved by means of independently working protective barriers in the form of: geological barriers, artificial sealing layers (geomembranes), natural soil liners and covers, and sidewall sealing.

Geological barriers, natural soil liners and covers are appropriately built-in cohesive soil layers with coefficient of permeability k lower than 10^{-9} m/s [1, 19], which are characterized by long-lasting ability of bonding and interrupting chemical compounds from leachate landfill. Mineral layers are the cheapest and the most stable element of storage yard sealing. It is determined by its properties such as: lower value of permeability coefficient, swelling ability, as well as ability of self-sealing, possibility of forming layers of unrestricted volume, resistance to both chemical compounds existing in leachate and temperature. The research, which has been done up till now, proves that although mineral sealing layers do not eliminate the whole leak soaking from storage yards, their effectiveness repeatedly exceeds the reliability of artificial insulating layers (geomembranes). Geomembrane damage causes unsealing of the whole construction. The leakage through a hole in geomembrane is minimized by placing mineral liner beneath the membrane. Calculated flow rates through the composite liner are at least 100 times less than through the geomembrane or mineral liner alone [1].

Most commonly applied materials for building the mineral sealing layers are: clay, boulder clay (which is improved by addition of bentonite, cement or silica) and fly ash.

The possibility of utilizing of fly ash as a material for mineral sealing layers is justified by its chemical, physical and mechanical properties. Fly ash is characterized by its ability of absorbing and stopping a leak, because of its significant water-absorption (up to 80%) and small ability of filtering off (2–16%). The values depend on the depth of the tested layer and its density [3, 4]. Power industry wastes possess the ability of stopping various contaminants and also, heavy metals. It can be stated that they are characterized by neutrality with reference to earthen foundation. According to the author's research of fly ash and fly ash/ bottom ash solubility, chemical compound contents in water solutions of both tested wastes do not exceed the concentration of those compounds in natural soil solutions, however, trace element concentration is greater. Permeability test results which are obtained for power industry wastes from several electric-power plants show that water-permeability is not large and is decreasing in time. The values of permeability coefficient, which are published in a great number of papers, range from 10^{-3} to 10^{-10} m/s, but the test method and the way of sample preparation have not been announced. Compacted power industry wastes shear strength and bearing capacity, determined on the basis of California Bearing Ratio, are significantly greater than those obtained for mineral soils corresponding with them in terms of graining, at similar values of consolidation test results [22]. In the author's research the occurrence of shrinkage cracking caused by flay ash desiccating was not noted, which is one of the main problems appearing when mineral sealing layers are built from cohesive soils. The rapid desiccation of the low-permeability soil layer beneath geomembrane can cause mineral liner cracks up to 300 mm deep and 25 mm wide [5]. Building of power industry wastes into liners, as a non-cohesive anthropogenic soil, is relatively easier than spreading and correct compaction of cohesive soils.

HDPE geomembrane (*high density polyethylene*) is one of synthetic materials used for making artificial sealing barriers in geotechnical engineering structures. One of HDPE geomembrane disadvantages is its smooth surface, which results in a low value of interface shear strength obtained for multilayered liner system. A particular note of the fact was taken after the slope-stability failure of a Class I hazardous waste landfill at Kettleman Hills in California [11, 15, 16]. The failure was caused by insufficient shear strength between layers of mixed storage seals. Interaction between geosynthetics and compacted clay layer was characterized by very low value of interface friction angle, which was equal to 8°. Nowadays, geomembranes with textured surfaces are produced in order to preserve a slippage along phases within mixed seal system.

The aim of the paper was to determine the shear strength interface between fly ash/ bottom ash mix, as a material built in the insulating layer of storage yards, and HDPE geomembranes with various textures. The textured HDPE geomembranes had roughened top surface that could increase the shear resistance between fly ash and geosynthetic. The influence of geomembrane texture on interface shear strength was shown in the paper to better characterize the behaviour of a potentially weak surface within multilayered liner system.

THE PROPERTIES OF THE TESTED POWER INDUSTRY WASTES

Laboratory tests were done on the example of fly ash and bottom ash mixture from dry storage yard, and fly ash sampled directly from storage reservoir. Both waste came from hard coal combustion in Białystok Thermal-Electric Power Station.

Chemical properties

The basic chemical composition and trace element content were investigated for averaged samples of fly ash taken directly from storage reservoir and fly ash/bottom ash mix from dry storage yard. The content of unburnt coal was determined as a loss on ignition at a temperature of 600–800°C. The main chemical compound fraction and trace element contents in investigated wastes, in comparison to their content in unpolluted natural soils, were shown in Table 1. The analysis of chemical composition of both waste water solutions, in comparison to natural soil solutions, was shown in Table 2.

On the basis of the data presented in Table 1, it can be stated that the greatest differences in percentage fraction of individual compounds in fly ash directly from storage reservoir and fly ash and bottom ash mix were observed in the case of SiO_2 and Al_2O_3. The content ranges obtained for those compounds do not overlap. A wider range of unburnt carbon in fly ash/bottom ash mix is caused by various bottom ash contents in stored wastes. Fly ash includes more microelements than fly ash/bottom ash mix from dry storage yard, except zinc, cadmium and potassium. By comparing the quantitative setting-up of main chemical compounds and trace elements in tested wastes and their average contents in unpolluted soils [7], it should be stated that power industry wastes from Białystok Thermal-Electric Power Station include more aluminium, iron, magnesium, titanium, zinc (fly ash/bottom ash mix), chromium (fly ash) and lithium. The determined quantity of microelements does not exceed the acceptable values for these elements in arable land [7].

It is stated (Table 2) that fly ash water-solubility is greater than fly ash/bottom ash mix, independently of the test method, and fly ash stronger alkalize solution obtained by static method. Chemical compound and trace element concentrations in the tested waste water extract are almost in all the cases greater for fly ash water extract (except iron, nickel and potassium). It should be especially noted that sulphate leaching from „fresh" waste is twice as big as from stored waste, and chloride leaching is considerably greater. The contents of individual chemical compounds in tested water extract from both wastes do not exceed

Table 1. Element fraction in wastes from Białystok Thermal-Electric Power Station in comparison to element content in soil

Designation	Unit	Content in tested wastes		Average content in unpolluted soils [7]*)
		Fly ash	Fly ash/bottom ash mix	
Si as SiO_2	%	33.68–41.73	44.28–47.44	X–X0
Al as Al_2O_3	%	21.19–24.48	17.85–20.86	1–3.5
Fe as Fe_2O_3	%	5.19–9.40	5.18–5.43	0.8–3.0
Ca as CaO	%	2.43–5.04	3.04–4.48	l.d.
Mg as MgO	%	1.46–3.62	0.73–2.01	0.1–0.9
S as SO_3	%	0.605–0.674	0.496–0.585	l.d.
P as P_2O_5	%	0.094–0.259	0.082–0.430	l.d.
Ti as TiO_2	%	1.10–1.80	1.04–1.40	0.1–0.6
Mn as Mn_3O_4	%	0.15–0.17	0.035–0.110	0.01–0.13
Na as Na_2O	%	0.022–0.485	0.093–0.202	l.d.
K as K_2O	%	0.055–0.510	0.078–0.604	l.d.
Free CaO	%	0.85–1.8 ± 1.0	<1.0	l.d.
C as a loss of ignition	%	5.0–10.0	7.6–15.0	l.d.
N	ppm	5 250–15 000	4 500–17 000	l.d.
Co	ppm	6.45–20	1.75–15	0.1 > 100
Zn	ppm	51.6–88.96	52.6–165.76	30–125
Cu	ppm	48–55.85	40–40.85	1–140
Pb	ppm	3–150	1.5–100	25–40
Cr	ppm	39.9–230	12–130	7–150
Ni	ppm	3–17.75	2.5–3.35	4–50
Cd	ppm	0.1–0.21	0.7–0.84	0.2–1.05
Mn	ppm	214–575.15	144–205.25	100–1 300
Li	ppm	28–82.25	38–92.35	1.3–56
Na	ppm	165.75–3 600	692.35–1 500	l.d.
K	ppm	460–4 231.7	650–5 011.45	l.d.

ppm = mg/kg, 1% = 10 000 ppm

*) l.d. – lack of dates

Table 2. Tested compound concentration in fly ash water extract in comparison to soil water extract, mg/dm^3

Designation	Fly ash		Fly ash/bottom ash mix		Concentration published for natural soil water extract [7] *)
	method 1)	method 2)	method 1)	method 2)	
SiO_2	48.0	120.0	48.0	54	1–200
Al_2O_3	256.87	113.02	143.8	102.7	l.d.
Al^{3+}	68.0	29.9	38.1	27.2	100–5 700
SO_4^{2-}	594.32	452.44	291.78	277.51	l.d.
Ca^{2+}	292.58	248.5	116.23	112.22	l.d.
Mg^{2+}	12.02	33.67	4.81	2.405	l.d.
PO_4^{3-}	0.07	0.15	0.05	0.04	l.d.
Fe	–	0.12	–	0.22	0.03–2.0
N	5.0	6.0	3.0	5.0	l.d.
Co	0.04	0.08	0.03	0.05	0.0003–0.087
Zn	0.09	0.18	0.13	0.04	0.06–2.2
Cu	2.07	2.69	1.34	1.22	0.003–0.135
Pb	1.00	1.20	0.4	0.8	0.0001–0.010
Cr	0.07	0.07	0.02	0.02	l.d.
Ni	0.144	0.16	0.84	0.24	0.00X–0.0X
Cd	0.0006	0.0008	0.0003	0.0004	0.0002–0.006
Mn	0.2	1.0	0.3	0.4	0.03–0.76
Li	0.4	0.3	0.3	0.2	l.d.
Na^+	40.0	39.0	14.4	15.2	l.d.
K^+	10.0	9.0	35.0	31.0	l.d.
Na_2O	54.0	52.0	19.0	20.0	l.d.
K_2O	12.0	10.8	42.0	37.0	l.d.
$N\text{-}NO_3^-$	0.2	0.1	0.2	0.2	l.d.
$N\text{-}NO_2^-$	0.011	0.011	0.020	0.017	l.d.
NH_4^+	0.07	0.42	0.03	0.17	l.d.
Cl^-	45.5	41.0	0.5	0.6	115–10 000
HCO_3^-	1.0	–	2.0	1.2	l.d.
CO_3^{2-}	6.0	8.0	0.0	0.8	l.d.
OH^-	–	4.0	–	–	l.d.

1) method I – fly ash water extract was obtained by dynamic method during 3-hour washing under pressure,

2) method II – fly ash water extract was obtained by static method after 24 h (4 h of shaking).

*) l.d.– lack of dates

values published for natural soil water extract; however, microelement concentrations are greater in the case of copper, lead, nickel and manganese (static method). It is necessary to say that power industry waste solubility is decreased in time, which is the consequence of acidity increase during waste solution and bonding of many elements in sparingly soluble chemical combinations [9, 13]. After compaction of waste built in an earthen structure, leaching processes of chemical compounds become limited by decreasing permeability coefficient of compacted waste [1, 20].

Both power industry wastes are characterized by alkaline reaction, but fly ash/bottom ash mix reaction is closer to neutral.

Graining

Tested wastes, fly ash and fly ash/bottom ash mix, correspond in terms of graining, with the sandy silt (πp). The grain-size distribution curves, obtained for averaging samples of power industry wastes, were presented in Figure 1. Using the determined curves, the coefficients of uniformity C_U and curvature C_C were calculated (Table 3). These coefficients characterize mineral soils grain-size distribution and their usefulness to compaction.

On the basis of determined values of uniformity coefficient C_U, fly ash was rated among uniformly graded soil, and fly ash/bottom ash mix among variety graded. Coefficients of curvature C_C classify tested wastes as well graded only in the case of fly ash/bottom ash mix, because mix $C_U \geq 6.0$.

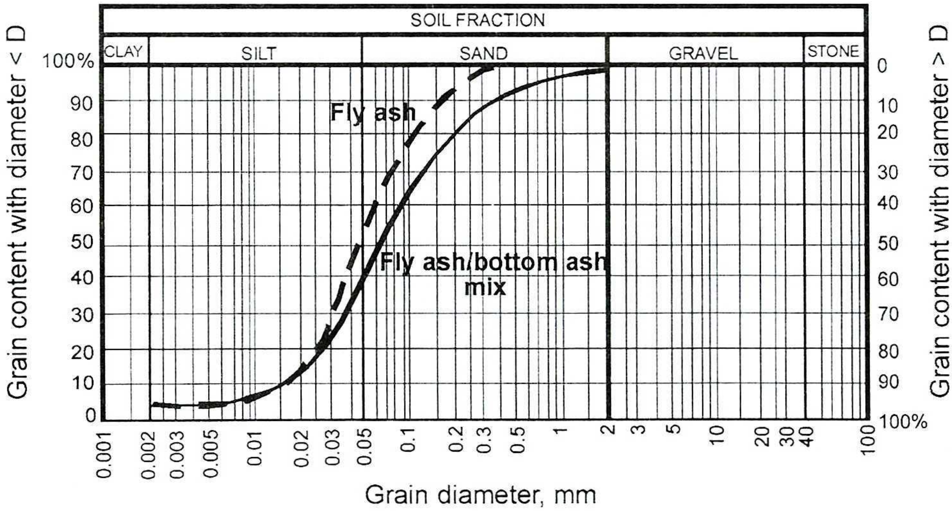

Fig. 1. Graining curves obtained for averaging waste samples on the basis of sieve and hydrometer analysis

Table 3. Graining coefficients of tested power industry wastes

Kind of waste	The effective size, mm				Graining coefficient	
	D_{10}	D_{30}	D_{50}	D_{60}	$C_U = \frac{d_{60}}{d_{10}}$	$C_C = \frac{{d_{30}}^2}{d_{10} \cdot d_{60}}$
Fly ash	0.015	0.03	0.045	0.06	4.0	1.0
Fly ash/bottom ash mix	0.015	0.038	0.075	0.09	6.0	1.07

The effective size D_n is the grain size corresponding to n percent of the passing by weight (n-percent of particles are smaller than D_n).

California Bearing Ratio and swelling

Bearing capacity, determined by means of California Bearing Ratio, is the ability to transfer load without generating excessive deformations. CBR is measured by the formula:

$$CBR = \frac{p}{p_s} \cdot 100\%,$$

where: p – unit load required for soil normalized penetration,

p_s – standard unit load (the unit load required to cause the same piston to penetrate into a sample of normalized compacted crushed rock).

The tests were conducted on samples directly after compaction and samples soaked for four days in water. Samples had been compacted in CBR moulds by Standard Proctor method (dynamic method – compaction energy 0.59 J on 1 cm^3 of soil) at moisture content approximately equalled optimum water content. This way of compaction enabled maximum sample compaction. All the samples were penetrated under loading 2.44 kPa.

Swelling is an increase in soil volume as a result of water access. Soil swelling tests are carried out in oedometers or in CBR moulds, under single-stage load of water-flooded samples. Swelling measurements are done until sample height stops increasing, but at least for four days. Soil swelling is determined by swelling index I_{pc}:

$$I_{pc} = \frac{\Delta h}{h} \cdot 100\%,$$

where: Δh – sample height gain after maximum swelling,

h – initial sample height before soaking.

The values of swelling index I_{pc} were obtained for power industry waste samples compacted by Standard Proctor method in CBR moulds at optimum water content (when compaction is the greatest). All research was done after soaking in water for four days, under consolidation load 2.44 kPa, recommended as minimum load.

Results of CBR and swelling index tests were presented in Table 4.

Summarizing, fly ash/bottom ash mix from dry storage yard is characterized by better geotechnical properties in comparison with fly ash directly from storage reservoir. Fly ash/ bottom ash mix is a more neutral material with relation to subsoil. It possesses greater resistance to mechanical penetration than fresh fly ash, both tested directly after compaction

Table 4. CBR and swelling index I_{pc} values, obtained under loading 2.44 kPa

CBR	Fly ash		Fly ash/bottom ash mix	
	Unsoaked	Soaked	Unsoaked	Soaked
CBR average value, %	22.40 ± 2.23	18.68 ±2.68	37.44 ± 4.43	27.94 ± 2.51
Obtained range, %	20.2 ÷ 24.9	16.1 ÷ 21.2	32.1÷ 41.7	25.0 ÷ 30.4
Standard deviation	1.7986	2.1603	3.5669	2.0219
Variation coefficient, %	8.0295	11.5649	9.5271	7.2365
Swelling index I_{pc}	Fly ash		Fly ash/bottom ash mix	
I_{pc} average value, %	0.188 ± 0.016		0.094 ± 0.011	
Obtained range, %	0.17 ÷ 0.20		0.08 ÷ 0.10	
Standard deviation	0.0130		0.0089	
Variation coefficient, %	6.9353		9.5152	

and soaking, at similar values of swelling index. It is better graded, so consequently – it can achieve better compaction. It is established, that fly ash/bottom ash mix from dry storage yard of Białystok Electric-Power Station is more useful to construction mineral sealing layers of waste landfills. In the event of insufficiently low value of permeability coefficient for determined waste shipment, it can be improved by adding calcium bentonite to increase the workability in the compaction and to obtain low hydraulic conductivity, without affecting the mechanical properties [13].

Laboratory research soil-geomembrane interaction was conducted only on the example of fly ash/bottom ash mix.

INTERACTION TEST METHODS

Soil shear strength can be defined by means of generalized classic Coulomb's condition:

$$\tau_f = \sigma_n \cdot tg\,\Phi + c,$$

where: τ_f – soil resistance at the moment of shearing,

σ_n – shear stress to destruction plane (normal stress),

c – cohesion (soil cohesion resistance),

Φ – apparent angle of international friction (angle of shearing resistance).

Shear strength is usually calculated as a maximum value corresponding, in the case of non-cohesive compacted soil shearing, with the peak value on the stress-strain curve. After reaching the peak soil resistance shearing, soil is getting weakened to a residual value (steady-state value). Residual states are determined at significant sample deformations and at stable stress state. Coulomb's condition presents linear envelope of stress limit state in Cartesian coordinate system τ, σ. The simplest and oldest laboratory test of soil shear strength is direct soil shearing, where the sample is placed in a two-part box with square cross-section and is sheared along horizontal plane of the box division. In direct shear apparatus the dimension of shear surface is not constant – it decreases during the test. A modification of this device is torsional-ring shear apparatus, where the tested sample is ring-shaped. In the torsional-ring shear apparatus there is no displacement value limit because the dimension of shearing surface does not undergo a change. The soil sample is being sheared in a torsional way. Both the device schemes were presented in Figure 2.

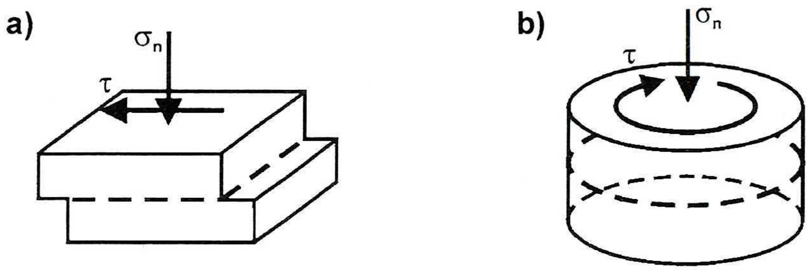

Fig. 2. Shearing scheme in direct shear apparatus: a) classical-box, b) torsional-ring

In the event of research in the direct shear apparatus of the shear strength of fly ash – HDPE geomembrane interface, Coulomb's formula assumes the form:

$$\tau_f = \sigma_n \cdot tg\,\delta + c_a,$$

where: τ_f – soil – geomembrane contact resistance at the moment of shearing,
σ_n – normal stress,
c_a – adhesion,
δ – interface friction angle (geomembrane and soil are treated as a homogenous physical parts of the system, bounded by separation surface – phase limit).

Test results of shear strength of the contact soil – geomembrane are usually presented for the peak strength (maximum contact resistance at the moment of shearing) and a residual strength (steady-state value of shearing resistance).

Most shear resistance tests between mineral sealing layer or the soil built in an embankment and underlying geomembrane are done in an adapted direct shear apparatus. Koerner [8] pointed out that most test results reported in literature were based on the peak strength, and not on residual strength. In order to reach residual state, a large shear displacement may be required, so a shear box larger than used in classic tests should be recommended, as in normalized method ASTM D 5321 [23]. ASTM D 5321 specifies box dimensions with plan area 30 cm by 30 cm. The large displacement is also possible during interface contact tests in the torsional-ring shear apparatus, where residual strength is obtained at a displacement amounting even to 60 cm. A shear displacement of 40–60 cm is typically required to mobilize residual interface strength in the ring shear tests [17]. Generally it is considered that classic direct shear apparatus provides good peak strength estimation because the peak strength is mobilized at a shear displacement of 0.5 cm. In the classic apparatus, modernized to interface contact tests of compacted clay – HDPE geomembrane, the peak strength is reached at about 10% of the sample displacement. The peak strength occurs at lower values of the displacements than in torsion-ring apparatus [18]. Some researchers [12] say that during torsional sample shearing the peak value of shear resistance is underestimated; real values should be higher than observed.

Summarizing, it is considered that the classic direct shear apparatus is better to determine the peak strength, and the torsional-ring apparatus for the residual resistance.

It was stated that geomembrane hardness could play an important role in the mechanism of interface shearing [14]. Relatively hard HDPE surfaces would promote soil grain sliding, whereas relatively soft PVC would promote particle rolling. Mean grain diameter D_{50} (the grain size corresponding to 50% of the passing by weight of finer particles) of soils used for sealing layers also pointed at the possibility of applying boxes with conventional dimensions. Geomembranes are characterized by uniform surface structure in comparison to geotextiles, so the scale effect is minimized [8]. Since soil – geomembrane contact tests in the modernized direct shear apparatus are economically justified; research is carried out in conventional direct shear apparatus with dimensions 10 cm x 10 cm or even 6 cm x 6 cm. The smallest boxes meet requirements concerning coarse grains in sample in the event of fly ash tests.

SHEAR STRENGTH TESTS OF FLY ASH/BOTTOM ASH MIX AND HDPE GEOMEMBRANE CONTACT

Interaction tests between compacted fly ash/bottom ash mixture and HDPE geomembranes were conducted in classic direct shear apparatus, which was equipped with a cylindrical shear box. The box, with 65 mm in inner diameter and 20 mm high, enables compacted soil testing. Soil samples are compacted in bipartite device, which was designed for non-cohesive soil sample forming, and next relocated to the apparatus box. Samples can be also compacted directly in the cylindrical box [21]. The cylindrical box eliminates stress concentration and greater compaction of sample material, which is obtained in the square box corners.

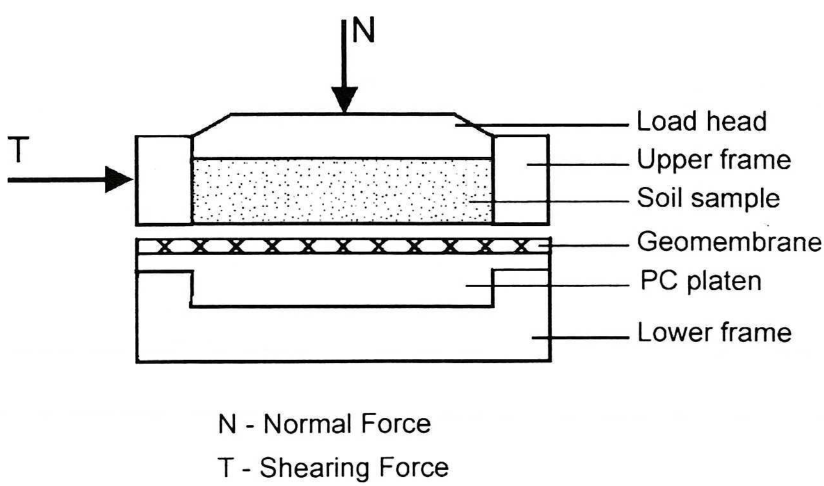

Fig. 3. Scheme of direct shear apparatus modified for interface contact tests

In order to determine interface friction the bottom box frame was equipped with a platen made of polycarbonate (PC) plate, which enables geomembrane fixing. A scheme of the direct shear apparatus, which is adapted to contact strength tests, is presented in Figure 3, and the box with its additional equipment is shown in Figure 4. HDPE geomembrane

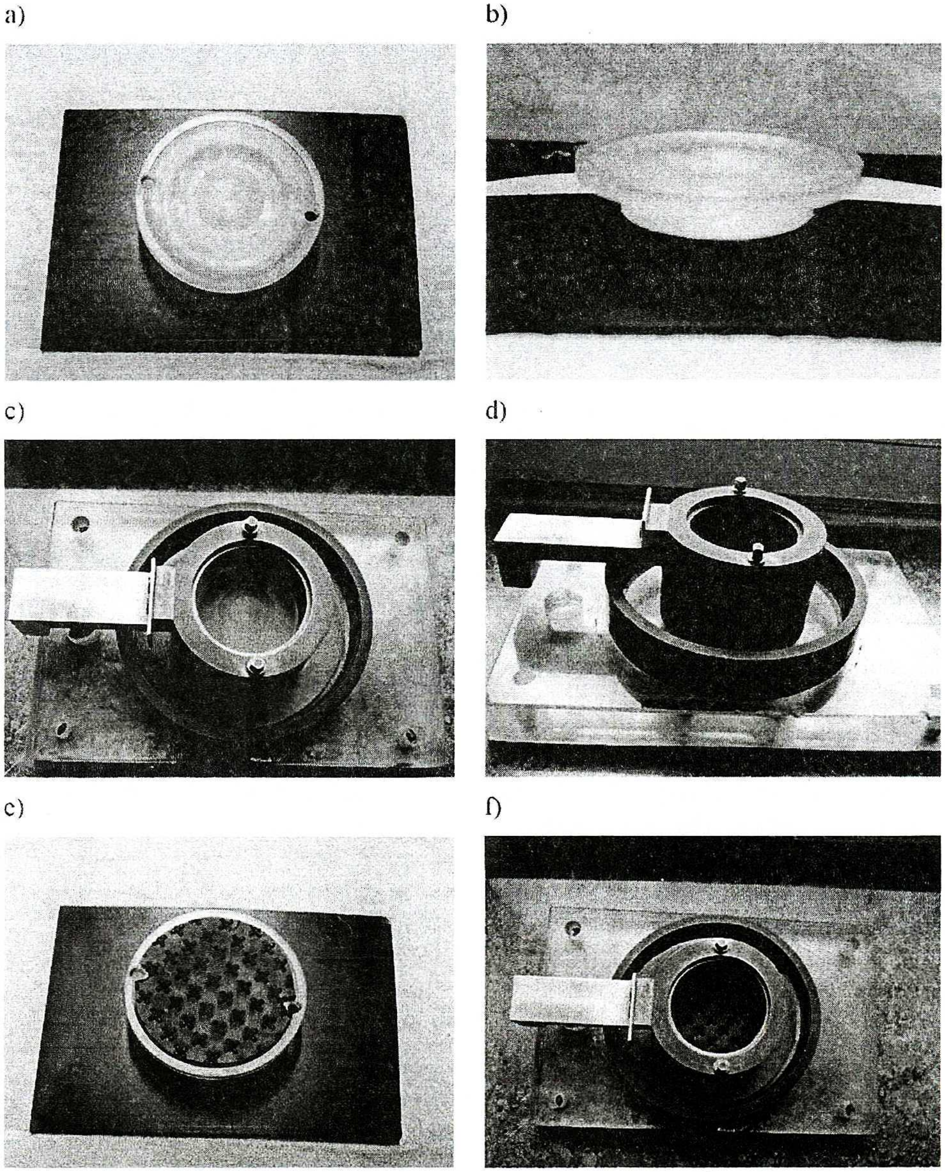

Fig. 4. Cylindrical box of direct shear apparatus with additional instrumentation: a) top view of PC platen for geomembrane, b) PC platen fixed in lower frame of apparatus box – end view, c) cylindrical box with PC platen – top view, d) end view of box, e) PC platen with crossed geomembrane, f) top view of box with geomembrane

was cut to 9.0 cm in diameter and was glued on the upper surface of the PC platen placed in the bottom box frame. The upper box frame was put on the PC platen and the whole box was bolt-clipped. Fly ash/bottom ash sample, compacted in bipartite device, was relocated to the upper box frame and covered by porous stone and load head. The diameter of the PC platen is greater by 25 mm than the upper box frame, in order to keep constant surface contact during shearing test.

Fly ash and bottom ash mixture was compacted at moisture contents close to optimum water contents in order to obtain densities corresponding to maximum compacted sample densities by Modified (compaction effort 2.65 J/cm^3) and Standard Proctor methods (0.59 J/cm^3). Samples were sheared at normal stresses σ equalled: 50, 100, 150, 200 and 300 kPa, with a shear displacement rate of 1 mm/min, without soaking samples in water.

Shear strength test results were presented for peak strength (maximum shearing resistance) and residual strength (established value of shearing resistance).

Geomembranes with various textures next called: smooth, textured and crossed, were used in the interaction tests between compacted waste samples and geomembranes.

TEST RESULTS

Test results, which are presented in Figure 5, justify accepting the research box of dimensions lower than 30 cm. Most tested interface contacts reached the peak strength at very low displacement of two independent box parts with respect to each other – up to 2 mm. The more diverse geomembrane texture is the greater displacement is achieved for mobilized peak strength, amounting to about 3 mm in the case of crossed geomembrane. In the event of smooth geomembrane and waste compacted by modified method the peak strength was reached at a 0.5–1.0 mm displacement.

In all the cases after reaching the peak strength, the contact strength reduction to residual value takes place, which is reached at a displacement of about 6 mm; which determines approximately 10% of tested sample diameter.

In many cases test results carried out in the direct shear apparatus show that the relationship between the interface shear strength τ and normal stress σ, can be nonlinear. It very often occurs in compacted clay – geomembrane contact test, performed under great values of normal stress [2]. Relationship graphs $\tau_f = f(\sigma_n)$ obtained for compacted fly ash/bottom ash mix and HDPE geomembranes were unambiguously linear for a mix compacted by two different efforts and smooth and crossed geomembranes. In the author's research, only in the case of textured geomembrane, the curvilinear graph was observed (Fig. 6). For the sake of statistically well representation of relationship by means of straight line, a search for curvilinear relationship was given up.

It can be concluded from Figure 7 that the strength of tested interface contact practically does not depend on a fly ash compaction method, but depends on HDPE geomembrane texture. In shear strength tests of compacted fly ash/bottom ash mix and geomembranes with clear texture (textured and crossed) nearly twice as big values of shear strength were obtained (at a value of shear stress σ_n of 300 kPa), as in the case of smooth geomembrane.

The relationship of the obtained strength parameters of compacted fly ash/bottom ash mix – HDPE geomembrane contact, depending on the kind of HDPE geomembrane texture, was presented in Figure 8. The interface friction angle δ increases along with texture diversity, however, the adhesion c_a distinctly decreases. Geomembrane texture influences

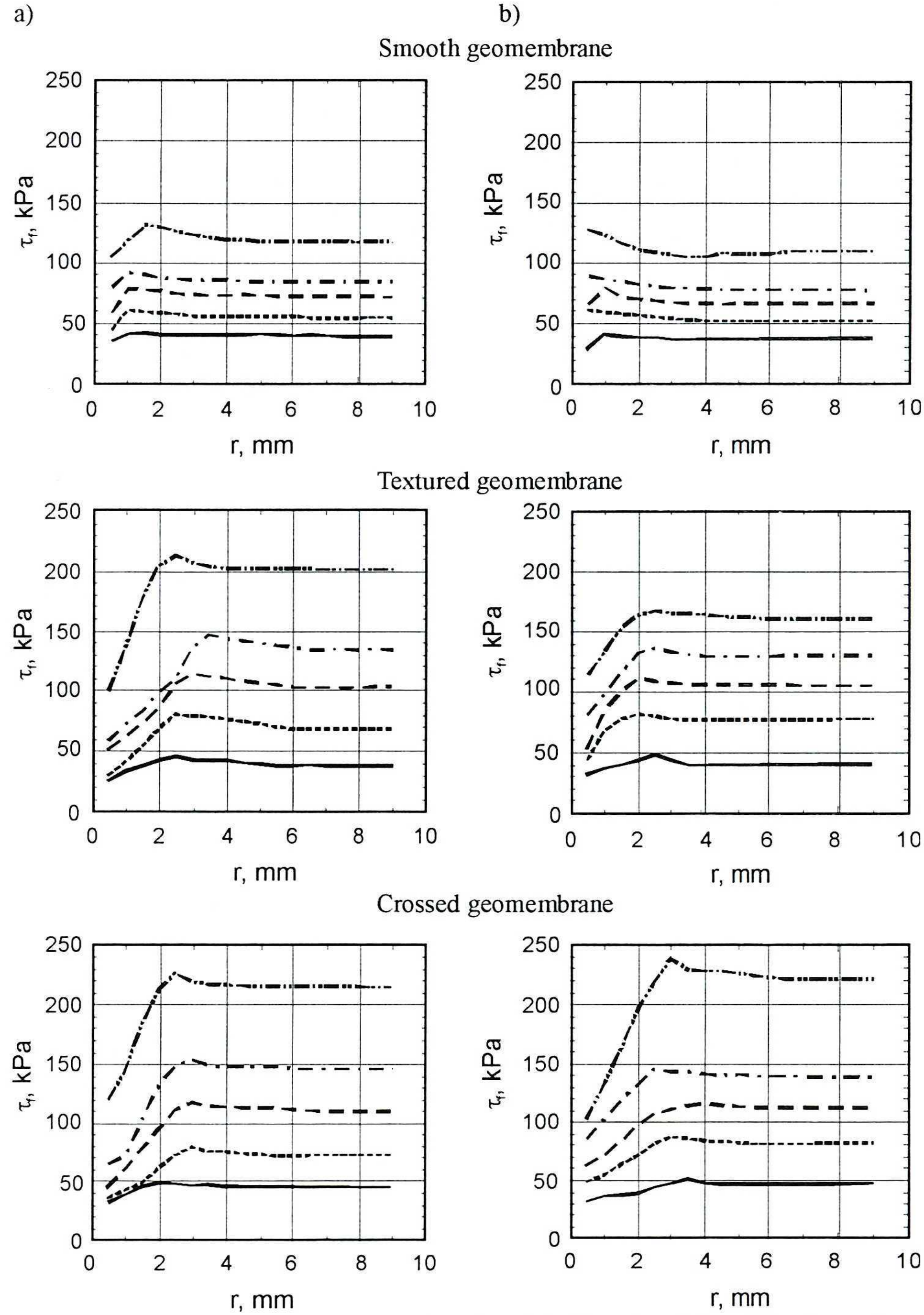

Fig. 5. Relationship graphs of fly ash/bottom ash mix – HDPE geomembrane shear strength in dependence on sample displacement r, tested at values of normal stress σ equalled 50, 100, 150, 200 and 300 kPa, for samples compacted by Standard or Modified Proctor method:
a) standard mix compaction, b) modified compaction

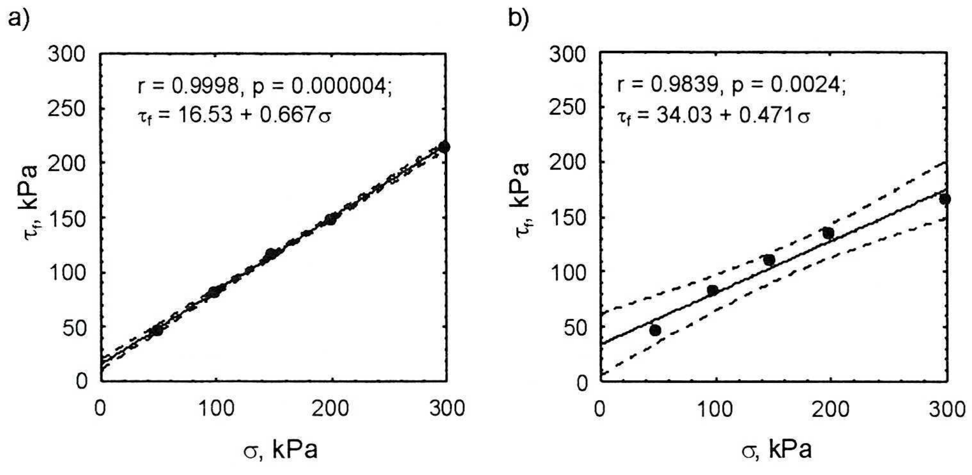

Fig. 6. Shear strength graphs of compacted fly ash/bottom ash mix – textured geomembrane contact: a) standard mix compaction, b) modified compaction

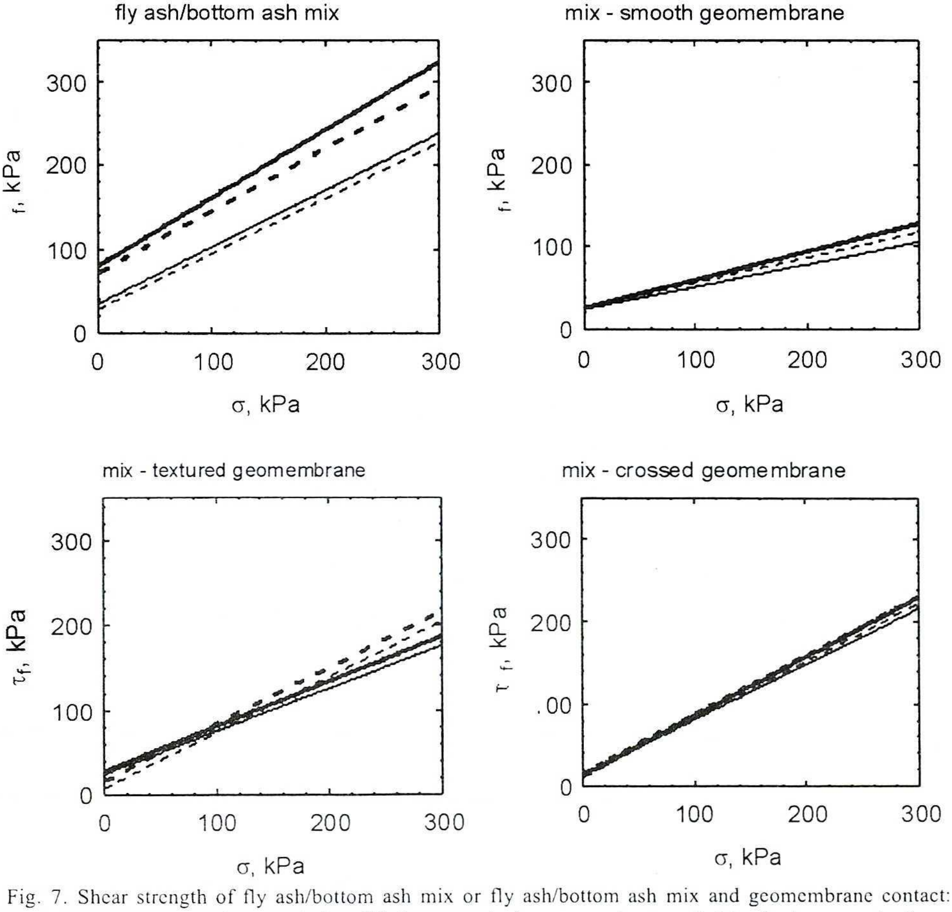

Fig. 7. Shear strength of fly ash/bottom ash mix or fly ash/bottom ash mix and geomembrane contact: ——— peak strength (modified compaction), – – peak strength (standard compaction), ——— - - - - appropriate residual strength.

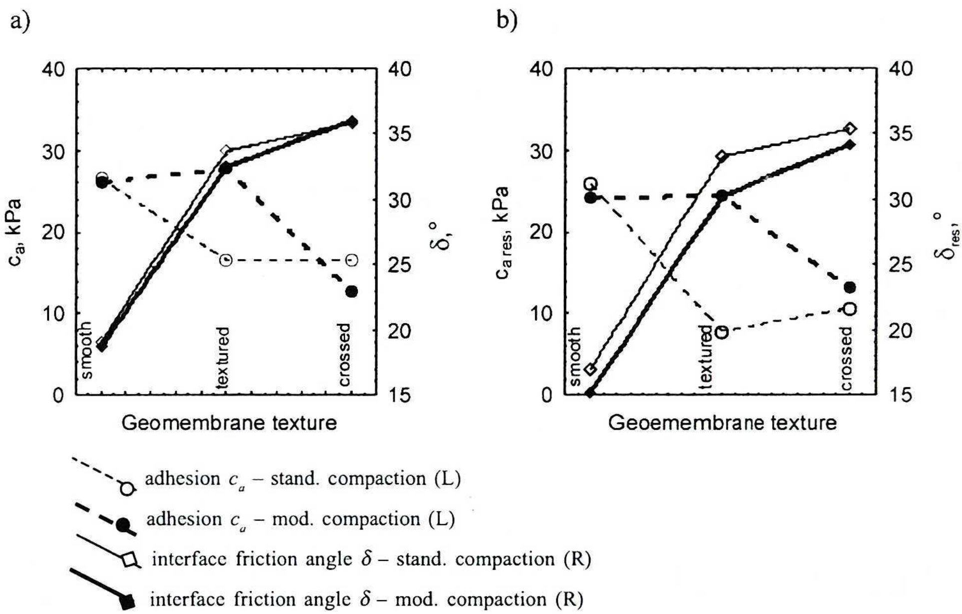

Fig. 8. Relationship between interaction friction angle δ or adhesion c_a and tested geomembrane texture: a) peak parameters, b) residual parameters

interface friction angle δ of fly ash/bottom ash mix – HDPE geomembrane contact (especially residual values) more than it was stated for sand – HDPE geomembrane contact [6], increasing by 19° for a mixture compacted by means of Modified Proctor method and smooth and crossed geomembranes. Those values were obtained for both peak and residual contact shear strength.

The sample compaction methods have little influence on values of tested interface shear strength. However, one can see the distinct impact of HDPE geomembrane texture on shear strength of flý ash/bottom ash mix – geomembrane contact.

CONCLUSIONS

On the basis of the test results of interface contact between compacted fly ash/bottom ash mix and HDPE geomembranes with various textures, which were obtained in modernized direct shear apparatus, it can be stated that:

1. The estimation of the interface contact between fly ash/bottom ash – HDPE geomembrane must be carried out on the basis of contact shear strength, and not only values of strength parameters δ and c_a. In the case of geomembranes with diverse texture the greater values of interface friction angle δ are obtained, and for smooth geomembrane – greater values of adhesion c_a.
2. The HDPE geomembrane texture diversity has an effect on interface shear strength. Taking into consideration textured and crossed geomembranes, twice as big value of peak and residual strength as for smooth geomembrane was received. In the case of

textured and crossed geomembranes the peak shear strength of contact is similar to residual value, for smooth geomembrane the peak strength is greater by 25% than residual strength. It should be stated that contact strength, to be more exact the value of interface friction angle δ, can decrease after geomembrane exposition to leak effect [10].

3. The tested mixture compaction method does not have much influence on interface shear strength value, peak values or residual values.
4. The contact residual strength is not much lower than the peak value, in contrast to fly ash shear strength, where the difference between peak and residual shear strength of tested waste amounts to 30%.

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