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DETERMINATION OF INTERNAL SHEAR STRENGTH PARAMETERS OF GEOCOMPOSITE CLAY LINERS

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Abstract: Geocomposite clay liners (GCLs) are used in environmental, transportation and geotechnical engineering applications. Determination of the internal and interface shear strength parameters of GCLs has a huge importance for stability analyses. Therefore, the study of the internal shear strength of one type of nonreinforced GCLs was performed. Laboratory testing programme consisted of five series of direct shear tests. Special attention was given to the influence of the specimen hydration procedure and horizontal displacement rate on the shear test results. The analysis of the direct shear tests, presented in the paper, clearly demonstrate that the measured values of internal shear strength depend on the way of performing the laboratory tests. The internal shear strength envelopes for nonreinforced GCLs are proposed on the basis of obtained results.

Key words: landfills, bentonite, geocomposite clay liners (GCLs), internal shear strength, direct shear test, nonreinforced GCLs.

Introduction

Geocomposite clay liners represent one type of the geosynthetics that have been used more frequently since the 70's. According to ASTM a geosynthetic is "*a planar product manufactured from polymeric material used with soil, rock, earth, or other geotechnical engineering related material as an integral part of a man-made project, structure, or system*". Koerner (2000) gives a broad overview of the various possibilities for their engineering application.

Geocomposite clay liners (IGS 2000) or geosynthetic clay liners (ASTM 1997) are manufactured hydraulic barriers containing a layer of high-quality sodium bentonite clay attached or adhered to geotextiles or a geomembrane. Numerous commercially manufactured products of GCLs are available on the worldwide market. They are used in environmental, transportation and geotechnical engineering applications. In environmental applications GCLs act as a hydraulic barrier component on waste disposal sites as a part of the liner and cover systems. Because of their low permeability they serve very often as a replacement for low permeability soil or clay liners. In comparison with classical clay liners they provide many advantages, including a greater resistance to differential settlements, desiccation, and freeze-thaw deterioration. They also have self-healing characteristics due to their swelling potential. It is also important that their installation is simple, easy and less time-consuming.

From the engineering design point of view, stability analyses of the hydraulic barriers at waste disposal sites are one of the most important issues. Therefore, proper determination of the shear strength parameters of all the barrier components plays a major role in the design process. Internal shear strength parameters of GCLs are generally obtained from laboratory direct shear tests, but the amount of the published data is limited. The principal issues for internal shear strength testing of GCLs are: test configuration, gripping technology, specimen size, degree of hydration and hydration liquid, normal stress range, and shear strain rate. It was found, by reviewing the published data, that there is a significant variability of the test procedures used and the results obtained. Therefore, a study of the internal shear strength of one type of nonreinforced GCL was performed. Special attention was given to the determination of the influence of two parameters: specimen hydration procedure and shear strain rate. The laboratory testing program was planned accordingly and consisted of five series of direct shear tests. Two procedures of specimen hydration and four different shear strain rates were investigated in the program.

The peak and residual shear strength envelopes were determined on the basis of test results. Influence of the hydration procedure and shear strain rate is clearly shown. The appropriate laboratory testing procedure is proposed for the determination of internal shear strength of GCLs. With this procedure the two most relevant parameters are included in the testing programs.

Internal shear strength testing of GCLs

Samples

There are a wide variety of commercially manufactured products of GCLs on the market. From the shear strength point of view, they are divided into two main groups: reinforced and nonreinforced GCLs. In our investigation, one of the very few nonreinforced GCLs known under commercial name Claymax 200R (CETCO, USA) is used. It consists of approximately

5.0 kg/m^2 of adhesive bonded natural sodium bentonite sandwiched between two lightweight woven geotextiles. A cross-section sketch of the nonreinforced GCL is shown in the Fig. 1. In the other type of GCLs called reinforced GCLs, geotextiles are held together, for example, by stitch bonding or needlepunching. The physical bonding of the geotextiles enhances the internal resistance of GCLs to shearing.

Several reasons influenced the decision to test one type of nonreinforced GCL in our investigation. It was our opinion that in order to understand the shearing behaviour of GCLs, it was necessary to investigate in detail the behaviour of bentonite itself. In the case of reinforced GCLs, their behaviour in direct shear test is influenced by the presence of synthetic yarns. Moreover, previous investigations (Gilbert et al. 1996) prove that reinforced GCLs have larger peak strengths, but the residual strength is the same as for nonreinforced ones. Finally, the specimen size is not a critical parameter in testing the nonreinforced GCLs, as is the case for reinforced ones. In the case of reinforced GCLs large specimens are necessary. Large samples are not practical for standard tests in common geotechnical laboratories. Some investigators show that there is also a problem of partial hydration of specimens that leads to inaccurate results in shear (Gilbert et al. 1997).

GCLs are geocomposites consisting of geological material (predominantly sodium bentonite) and synthetic materials (geotextiles, geomembranes). Bentonite is a key component because its function is to maintain low hydraulic permeability. Therefore, it will be presented in some details.

Bentonite is a naturally occurring clay that is extremely hydrophilic (water attracting). In contact with water or even water vapour bentonite attracts the water forming a complex configuration that leaves little free-water space in the voids. This fact explains the resulting low permeability of most GCLs, the most important property of barrier layers.

Bentonites, which are used for the production of GCLs, consist mainly of three-layer mineral montmorillonite of the smectite group. Other ingredients like quartz, christoballite, feldspars, muscovite/illite, and other clay and nonclay minerals are not important for the functionality in waste containment applications (Egloffstein 1997). Because of the high content of montmorillonite (60–90 %), bentonites have desirable properties like swelling, high ion exchange capacity, adsorption capacity against heavy metals and very low permeability.

Most of the commercially available GCLs use sodium bentonites. Calcium bentonites are rarely used because of lower swelling potential and higher permeability. The water adsorption capacity of sodium bentonites is approximately 400–700 % compared with the capacity of calcium bentonites of roughly 200 % (Egloffstein 1995). The permeability of calcium bentonites is $1–5 \times 10^{-10}$ m/s and of sodium bentonites $1–3 \times 10^{-11}$ m/s (Egloffstein 1997). GCLs manufactured in the USA use natural sodium bentonites found in Wyoming. In many European countries natural calcium bentonites are found (Koerner 1997). In order to receive better swelling properties such calcium bentonites are activated with soda (soda activated bentonites). Some properties of natural sodium bentonites are given in Table 1, compared to the properties of a calcium bentonite, for example, from Bavaria.

Fig. 1. Cross-section sketch of nonreinforced GCL.

Table 1: Properties of bentonites (Koerner 1997).

	Na-bentonite	Ca-bentonite
Montmorillonite content [%]	75	66
Specific surface area [m^2/g]	560	490
Exchange capacity [meq/100g]	76	62

In addition to the previously mentioned favourable properties of bentonites for the waste containment applications, there are some critical issues that should be kept in mind. The ion-exchange capability of bentonite under typical use conditions can cause the transformation of original sodium to calcium bentonites. As a consequence of that, some physical characteristics will be changed, like swelling, permeability and self-healing properties (Egloffstein 1997). This can be avoided by the proper installation of GCLs. The design engineer should consider the compatibility of GCL with the adjacent soils or liquids with which it will come into contact. GCLs should not be used if they can come into contact with limestone. Extreme weather conditions (heavy raining, very dry areas) should also be avoided (Mackey 1997).

The most interesting property of bentonites for our research is their shear strength. Previous investigations show that they have very low strength especially in free-swell conditions. Mesri & Olson (1970) and Olson (1974) conducted consolidated-undrained triaxial tests with pore water pressure measurements on homoionic sodium- and calcium-montmorillonites. Gleason et al. (1997) performed consolidated-drained direct shear tests on thin layers of bentonites according to ASTM D3080. The obtained values of shear strength parameters of montmorillonites and bentonites are shown in Table 2.

Shear apparatus

Direct shear tests were performed in a modified shear apparatus. Some modifications of a standard shear box as used in common soil mechanics laboratories were necessary. Standard shear boxes have the following dimensions: specimen size 70×70 mm or 60×60 mm, specimens height 20 mm. GCLs have a height of approximately 5 mm in as-received state, and their height is variable depending on the degree of saturation. Therefore, porous plates of different thickness were added to the apparatus. Specimen size was enlarged to 100×100 mm. In that way, the maximum horizontal displacement was enlarged, too, and the measurement of residual strength was achieved. Finally, gripping of the specimens is solved by using teethed metal plates, containing 112 teeth on the size of 100×100 mm. Details of the modified shear box are presented in Fig. 2.

Laboratory testing program

Published results of the internal shear strength parameters for nonreinforced GCLs demonstrate a huge variety of data

Table 2: Shear strength parameters of montmorillonites and bentonites (Mesri & Olson 1970; Gleason et al. 1997).

	Homoionic Na-montmorillonite	Homoionic Ca-montmorillonite	Na-bentonite	Ca-bentonite
	CU triaxial test		CD direct shear test	
Liquid limit [%]	880–1160	190–220	603	124
Plasticity index [%]	-	160–190	567	98
Cohesion [kPa]	7	14	6	5.8
Friction angle [°]	5	15	12	21

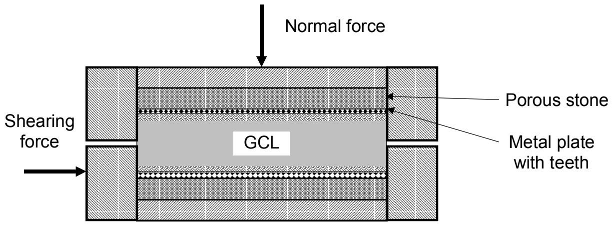

Fig. 2. Cross-section of the modified shear box.

(Daniel & Shan 1991; Daniel et al. 1993; Fox et al. 1998; Shan 1993). The range of the measured values of friction angle, φ, and cohesion, c, are the following:

dry specimens	φ = 22–37°	c = 7–50 kPa,
hydrated specimens	φ = 0–27°	c = 0.2–30 kPa.

There is obviously a huge scatter of the measured values of shear strength parameters. However, by reviewing the published data, it can be seen that laboratory procedures differ significantly concerning the following issues: test configuration, specimen size, hydration procedure, normal stress range, strain rate, and maximum horizontal displacement. One of the reasons for the scatter is that the established test methods or standards for the shear strength determination of GCLs did not exist at that time. We concluded that the two most important parameters for which the influence should be clearly determined are: specimen hydration procedure and shear strain rate. Our laboratory testing program was therefore planned accordingly. It consisted of five series of direct shear tests (Table 3). Two procedures for specimen hydration (series I and V) and four different shear strain rates (series I–IV) were investigated in the program.

Specimens were placed into the direct shear box during the hydration stage. Immediately after the application of normal stress on the specimen, water was added to the shear box. During the hydration for 24 hours (normal consolidation) or 9 days (extended consolidation), vertical displacements were measured and recorded continuously. Shearing of the specimens with different rates of displacements begun at the end of the hydration stage until the relative displacement of 15 % was achieved. Depending on the rate of displacements, shearing lasted from 17 minutes for series I and V to 9.5 days for series IV (Table 3).

Results

Stress and strain components

During the direct shear testing of nonreinforced GCL, shear stress, vertical and horizontal displacements were measured and recorded in an output file. Time intervals for recording output data were adapted to the different test durations given in Table 3. On the basis of this data, stress-displacement curves were created for every series of testing program. Three curves coresponding to three normal stress values (50, 100 and 200 kPa) for a series III are shown on Fig. 3. It can be seen that nonreinforced GCLs demonstrate shearing behaviour similar to that of overconsolidated clay materials exibiting peak and residual strengths. In our case, residual strengths were determined at the displacement of 15 mm, that is at the relative deformation of 15 %. Total values of peak and residual shear strengths along with their ratios are given in Table 4.

By reviewing the data presented in Table 4, we can conclude the following:

— the obtained values of peak and residual strengths are lower for lower displacement rates (comparing series I to IV),
— strength reduction from peak to residual values are higher for lower displacement rates,
— extended consolidation produces lower strengths, too (comparing series I and V).

Table 3: Laboratory testing program.

Series	Displacement rate [mm/min]	Normal stress			Test duration
		σ_n = 50 kPa	σ_n = 100 kPa	σ_n = 200 kPa	
I	1.219	I-1	I-2, I-2_p	I-3, I-3_p	17 min
II	0.1219	II-1	II-2	II-3	3 hours
III	0.01219	III-1	III-2	III-3	27 hours
IV	0.001463	IV-1	IV-2	IV-3	9.5 days
V	1.219	V-1	V-2	V-3	17 min

I-IV: Consolidation stage 24 hours.
V: Consolidation stage 9 days.

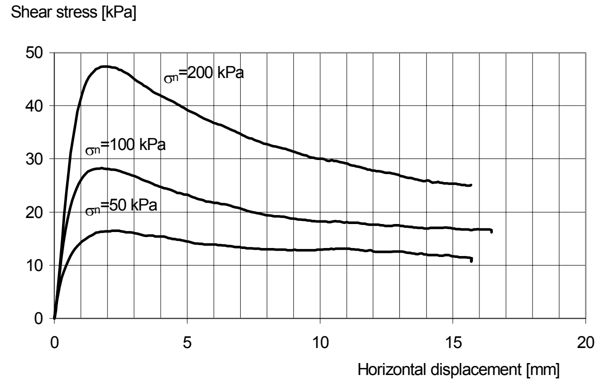

Fig. 3. Shear stress vs. horizontal displacement.

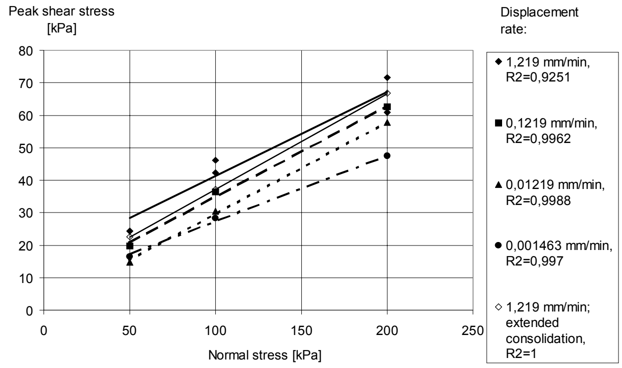

Fig. 4. Peak shear strength envelopes.

Table 4: Results of the direct shear tests.

Displacement rate [mm/min]	Normal stress σ_n [kPa]	Peak strength τ_p [kPa]	Residual strength τ_r [kPa]	Strength ratio τ_r/τ_p
1.219	50	24.4	20.4	0.84
	100	46.1	37.6	0.82
	200	71.5	59.5	0.83
0.1219	50	19.7	14.5	0.74
	100	36.3	25.7	0.71
	200	62.5	42.8	0.68
0.01219	50	14.7	7.9	0.54
	100	30.4	19.3	0.63
	200	57.8	36.3	0.63
0.001463	50	16.5	11.7	0.71
	100	28.3	16.9	0.60
	200	47.4	25.3	0.53
1.219*	50	22.5	14.9	0.66
	100	37.2	27.3	0.73
	200	66.7	45.1	0.68

* Series V — extended consolidation

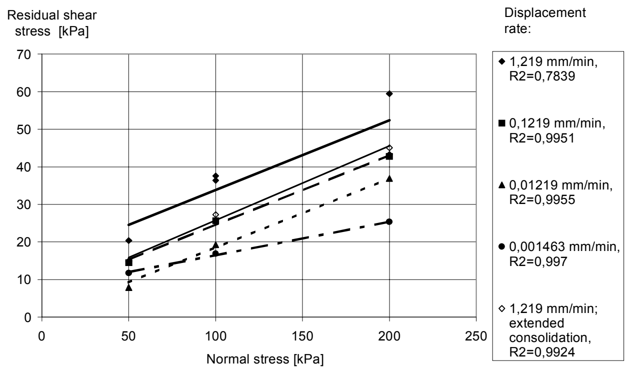

Fig. 5. Residual shear strength envelopes.

Internal shear strength envelopes

Peak and residual shear strength envelopes are shown in Figs. 4 and 5. Note that the scale for the x- and y-axes are different for clarity of presentation. Both figures show that different internal shear strength envelopes will be obtained depending on the way of performing the direct shear test. The strength envelopes for series I–III and V are almost parallel. It means that they have similar friction angle but different cohesion. Series IV with a lowest displacement rate shows significantly smaller friction angle.

The obtained values of friction angle and cohesion for five series of tests are extracted in Table 5. The range of obtained values for friction angle and cohesion are:

peak parameters	φ = 11.5–16.4°	c = 1–15.4 kPa,
residual parameters	φ = 5.1–11.2°	c = 0–15.3 kPa.

According to the ASTM D3080 standard and our oedometer test results, drained conditions were realized only for series IV. Although the range of obtained values is smaller than the previously mentioned range from the published data, it is obvious that the influence of the displacement rate and the hydration procedure should not be disregarded.

a. *Influence of the displacement rate:* The influence of a displacement rate on the measured peak and residual shear strength is demonstrated in Figs. 6 and 7. Note that the results of the series V are represented on both figures by full squares, but they were not included in the trend lines because of different hydration procedure. It can be seen that higher values of displacement rate cause larger values of measured strength. It is interesting to compare the results for series IV and V (Table 5). They have almost identical total duration of test including consolidation and shearing stage and therefore approximately the same hydration conditions. Their displacement rates represent the minimal and maximal values of the applied range. It can be seen that displacement rate influences the value of a friction angle much more than the value of a cohesion. Looking at the total values of measured shear strength (Table 4) for series IV and V, one can see that in spite of the similar hydration, series IV shows lower strength due to a much lower displacement rate. The only explanation for such results could be found in the rheological properties of the material.

b. *Influence of the hydration procedure:* The influence of a hydration procedure can be seen by comparing the results for series I and V (Table 5). The samples from series I and V were

Table 5: Shear strength parameters.

Series	Displacement rate [mm/min]	Peak parameters		Residual parameters	
		Cohesion, c [kPa]	Friction angle, φ [°]	Cohesion, c [kPa]	Friction angle, φ [°]
I	1.219	15.4	14.5	15.3	10.5
II	0.1219	6.6	15.7	6.0	10.5
III	0.01219	1	15.9	0	10.5
IV	0.001463	7.0	11.5	7.5	5.1
V	1.219	7.8	16.4	6	11.2

sheared with the same displacement rate, but their hydration procedure was totally different. The standard hydration and consolidation procedure of 24 hours duration was used for series I. For the series V extended hydration and consolidation lasted 9 days. The cohesion for series V is lower by a factor of 2 or more when compared to the cohesion for series I, looking to the peak and residual values. The friction angle in both cases is somewhat larger for series V than for series I. It can be concluded that hydration procedure affects much more the cohesion than the friction angle of GCLs.

By reviewing all data recorded during our investigation, the influence of hydration procedure is demonstrated through the influence of the final water content of samples on the measured shear strength. The final water content of samples depends on the applied normal stress and the total test duration (Fig. 8). Lower values of applied normal stress and longer test duration cause larger final water content of samples. Achieved values of the final water content affect the measured values of shear strength. This fact is demonstrated in Fig. 9 through the influence of the final water content on the peak shear strength. The influence of the displacement rate, and test duration on the results is also shown in the same figure.

The interpretation of Figures 8 and 9 took us to the following conclusions:

— final water content is inversely proportional to the applied normal stress, that is larger normal stress causes a lower final water content,

— final water content is proportionally dependant on the test duration, that is a longer test cause higher values of final water content,

— longer test duration results in lower shear strengths and this is caused by higher values of the final water contents,

— higher displacement rates on the contrary cause higher shear strengths, that is rate effects exist, too.

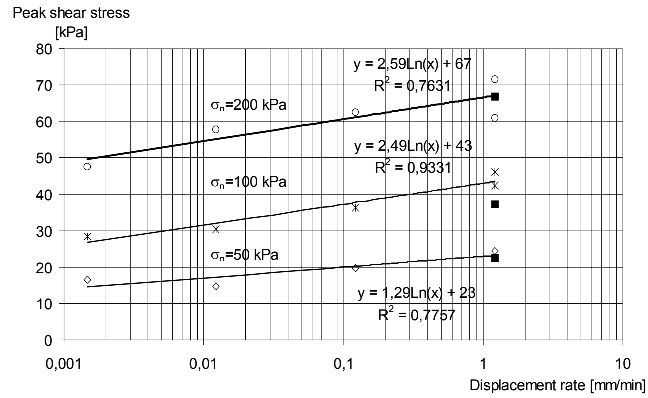

Fig. 6. Peak shear strength vs. displacement rate.

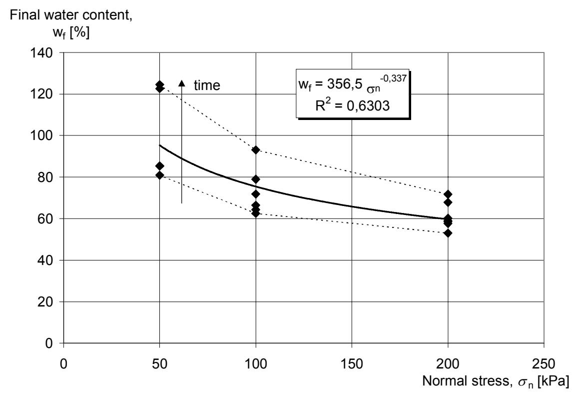

Fig. 8. Final water content of samples.

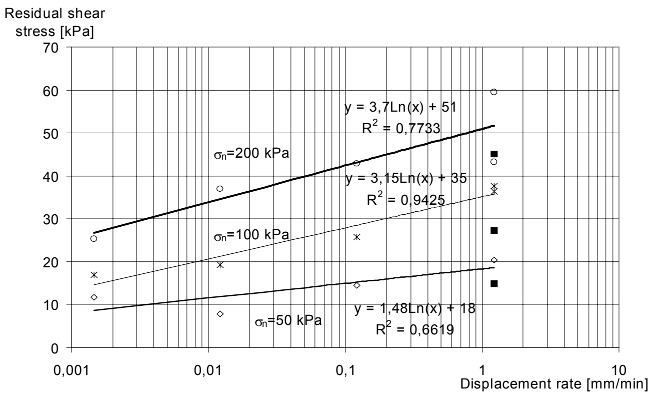

Fig. 7. Residual shear strength vs. displacement rate.

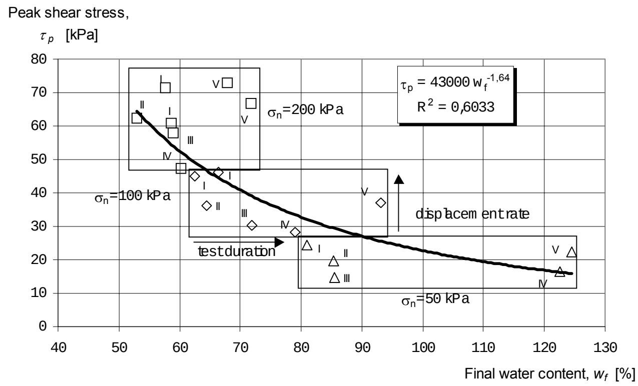

Fig. 9. Peak shear strength vs. final water content.

Proposed internal shear strength criteria

As already mentioned in the introduction to this article, stability analyses of the hydraulic barriers at waste disposal sites are one of the most important issues for design engineers. Therefore, proper values of the shear strength parameters of all barrier components including GCLs have to be chosen.

The analysis of the direct shear tests, presented in the paper, clearly demonstrate that the measured values of the shear strength depend on the way of performing the laboratory tests. The main factors affecting the results are hydration procedure and displacement rate. Therefore, we propose the shear strength envelopes as shown in Fig. 10. The envelopes are created on the basis of the results presented in Figs. 6 and 7. Peak and residual strengths are obtained by the extrapolation of the functions to the displacement rate of 0.001 mm/min for all three normal stress values. These values are redrawn in Fig. 10, giving the proposed envelopes. The real values of measured peak and residual strengths are presented by full and blank marks, respectively. The envelopes defined in that way will give the opportunity for engineers to design waste disposal facilities.

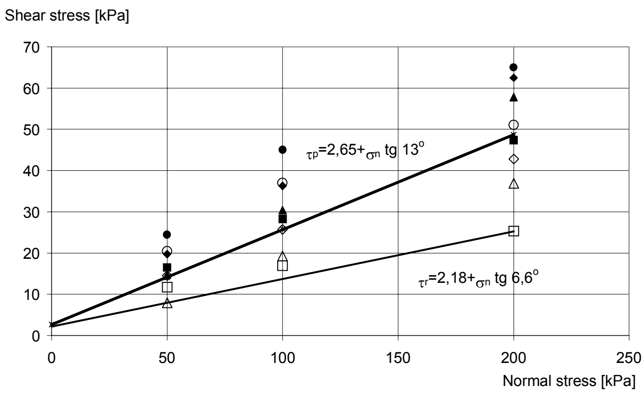

Fig. 10. Proposed shear strength envelope.

Conclusions

A series of direct shear tests were performed in the modified shear box in order to determine the internal shear strength of one type of GCLs. The results showed that by using the modified shear box, peak and residual strength could be obtained for the nonreinforced types of GCLs. The key parameters for performing the direct shear tests are the hydration procedure and the applied displacement rate. The influence of these parameters on the results is demonstrated. On the basis of the obtained results, peak and residual strength criteria are proposed. As the waste disposal facilities have a huge impact on the environment, it is important for design engineers to have reliable shear strength parameters. It is believed that the proposed shear strength criteria enable designing of new facilities in a safe way.

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References

ASTM D 3080–90: Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions.

ASTM D 4439–97a: Standard Terminology for Geosynthetics.

Daniel D.E. & Shan H.-Y. 1991: Results of direct shear tests on hydrated bentonitic blankets. *Project Report, Geotech. Engrg. Ctr.,* The University of Texas, Austin, 1–13.

Daniel D.E., Shan H.-Y. & Anderson J.D. 1993: Effects of partial wetting on the performance of the bentonite component of a geosynthetic clay liner. *Proceedings, Geosynthetics ‘93, Vancouver, B.C., IFAI Publ.,* 1483–1496.

Egloffstein T. 1995: Properties and test methods to assess bentonite used in geosynthetic clay liners. In: R.M. Koerner, E. Gartung & H. Zanzinger (Eds.): Proceedings of an International Symposium: Geosynthetic Clay Liners, Nurnberg, Germany 14–15, April 1994. *Balkema,* Rotterdam, 51–71.

Egloffstein T. 1997: Geosynthetic clay liners. Part six: Ion exchange. *Geotechnical Fabrics Report*, June-July, 38–43.

Fox P.J, Rowland M.G. & Scheithe J.R. 1998: Internal shear strength of three geosynthetic clay liners. *Journal of Geotechnical and Geoenvironmental Engineering* 124, 10, 933–944.

Gilbert R.B., Fernandez F. & Horsfield D.W. 1996: Shear strength of reinforced geosynthetic clay liners. *Journal of the Geotechnical Engineering* 122, 4, 259–266.

Gilbert R.B., Scranton H.B. & Daniel D.E. 1997: Shear strength testing for geosynthetic clay liners. In: L.W. Well (Ed.): Testing and acceptance criteria for geosynthetic clay liners. *ASTM STP* 1308, 121–135.

Gleason M.H., Daniel D.E. & Eykholt G.R. 1997: Calcium and sodium bentonite for hydraulic containment applications. *Journal of Geotechnical and Geoenvironmental Engineering* 123, 5, 438–445.

IGS 2000: Recommended descriptions of geosynthetics functions, geosynthetics terminology, mathematical and graphical symbols.

Koerner R.M. 1997: Perspectives on geosynthetic clay liners. In: L.W. Well (Ed.): Testing and acceptance criteria for geosynthetic clay liners. *ASTM STP* 1308, 3–20.

Koerner R.M. 2000: Emerging and future developments of selected geosynthetic applications (The Thirty-Second Terzaghi Lecture). *Journal of Geotechnical and Geoenvironmental Engineering* 126, 4, 293–306.

Mackey R.E. 1997: Geosynthetic clay liners. Part five: Design, permitting and installation concerns. *Geotechnical Fabrics Report*, January–February, 34–39.

Mesri G. & Olson R.E. 1970: Shear strength of montmorillonite. *Geotechnique* 20, 3, 261–270.

Olson R.E. 1974: Shearing strength of kaolinite, illite, and montmorillonite. *ASCE Journal of the Geotechnical Engineering Division* 100, GT11, 1215–1229.

Shan H-Y. 1993: Stability of final covers placed on slopes containing geosynthetic clay liners. *Dissertation,* The University of Texas at Austin, 1–296.