Geosynthetics - 7th ICG - Delmas, Gourc & Girard (eds) © 2002 Swets & Zeitlinger, Lisse ISBN 90 5809 523 1

Shear strength testing on a GCL

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ABSTRACT: Study of the internal shear strength of geocomposite clay liner has been performed. Laboratory testing program consisted of several one-dimensional swelling and consolidation tests and five series of direct shear tests. Influence of the specimen hydration procedure and horizontal displacement rate on the shear test results has been determined. Shear strength criterion and the appropriate laboratory testing procedure are proposed. The results also demonstrate that the direct shear tests on nonreinforced GCLs can be performed with a modified standard direct shear testing equipment, thus avoiding the need for the substantial investment in a specialized apparatus.

1 INTRODUCTION

Geocomposite clay liners (GCLs) belong to the large family of geosynthetic materials. Geosynthetics are used in the environmental, hydraulic, transportation and geotechnical engineering. Koerner (2000) gives a broad overview of the various possibilities of the engineering application of geosynthetics. The primary functions of GCLs in various applications are separation and containment. Today they are predominantly used as a hydraulic barrier layers within the landfill liner and cover systems.

The first use of GCLs was in 1988 at one landfill in the USA. Since then they are used also for the prevention of seepage from surface impoundment containing contaminated liquids, as a secondary containment for underground storage tanks, for the groundwater protection against deicing chemicals at airports, for the subsurface protection from pollution caused by track spillage and accidents, or salt used on roadways, for the control of seepage through dams, as canal liners, and, for the vertical cutoff walls (Koerner 1997, Heerten 1995, Schmidt 1995).

There is a large number of test methods that can be used for the characterization of GCLs. One of the most relevant design parameters is the shear strength, which is for GCLs mainly obtained by the direct shear test. Stability analyses of hydraulic barriers at waste disposal sites are one of the critical design issues. If barrier liners contain GCLs, internal and interface strengths have to be properly determined.

Considering the issue of internal shear strength, GCLs are generally divided into two main groups: reinforced and nonreinforced GCLs. Nonreinforced GCLs can only be used on flat slopes as they have low shear strength especially in hydrated conditions which prevail in environmental applications. Reinforced GCLs use stitch-bonding, needle-punching or thermal-bonding for the improvement of their internal shear strength. It is however known that reinforced GCLs have larger peak strengths but the residual strength is the same as for nonreinforced ones (Gilbert et al. 1996).

GCLs are composite products consisting of geological material (predominantly bentonite or other low permeable clay) and synthetic materials (geotextiles, geomembranes). The internal shear strength of reinforced GCLs will be influenced by the presence of synthetic yarns penetrating the bentonite. Laboratory tests on reinforced GCLs measure the simultaneous contribution of all components to the shear strength, but they do not provide a clear understanding of internal mechanisms and interactions (Richardson 1997).

It was our opinion that in order to understand the shearing behavior of GCLs, it is necessary to investigate in detail the behavior of bentonite itself avoiding the influence of synthetic yarns. Several other reasons influenced our decision, too. Large specimens are necessary for testing reinforced GCLs, as they have to contain a representative pattern of synthetic yarns. For common geotechnical laboratories large samples are not practical because the tests are time-consuming and specialized apparatus has to be used. Specimen size is not a critical issue in testing the nonreinforced GCLs as is the case for the reinforced ones (Gilbert et al. 1997). Therefore, we could use the modified standard direct shear testing equipment.

By reviewing previously published data on shear strength properties of nonreinforced GCLs, a huge scatter of measured values is noticed. There are several reasons for that. First, until 1998 the established test methods or standards for the internal shear strength determination of GCLs did not exist. Internal shear properties of GCLs were mainly determined in accordance with ASTM D5321 standard which was primarily written for the determination of interface friction of all kinds of geosynthetics. As a consequence, the majority of data were obtained for the default strain rate of 1 mm/min. Bentonite as a key element of GCLs, has a very low permeability, thus available data most likely represent the undrained conditions. By the detailed analyses of published results it was also found that the laboratory procedures differ significantly with respect to the following issues: test configuration, specimen size, hydration procedure, normal stress range and strain rate (Koerner et al. 1998). Based on these results, we have concluded that the two most important parameters for which the influence should be clearly determined are: specimen hydration procedure and shear strain rate.

Laboratory testing program in our research was thus planed in two phases. First phase consisted of several one-dimensional oedometer tests in order to investigate swelling and consolidation properties of GCLs. It also enabled us to plan direct shear tests in a proper way concerning two issues: specimen preparation and shear strain rate. In the second phase of our research, five series of direct shear tests were conducted, using two procedures of hydration and four different shear strain rates. Modified shear box has been constructed for the performance of tests on GCLs in the standard direct shear apparatus.

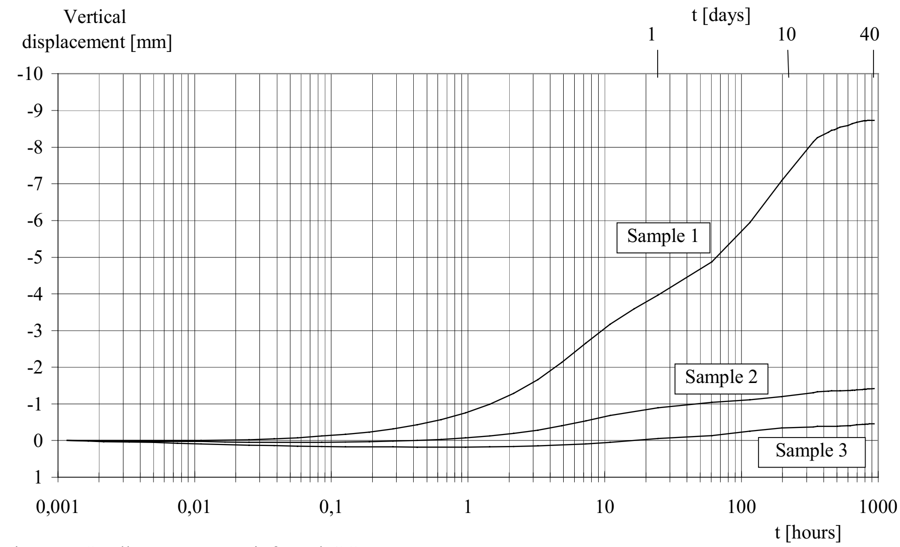
2 OEDOMETER TESTS

Laboratory testing program was performed to investigate the swelling and consolidation properties of GCLs. Several one-dimensional oedometer tests were conducted on the specimens of a GCL known under commercial name Claymax 200R. It is one of the few currently manufactured nonreinforced GCLs in the world produced by CETCO, USA. Natural sodium bentonite (approximately 5 kg/m^2) is adhesively bonded to upper and lower geotextiles.

Specimens with the diameter of 70 mm were placed in a standard oedometer cell. Two procedures of swelling were adopted:

- free swell, under zero confining stress, and,
- constrained swell, under two different values of confining stress: 25 and 100 kPa.

The specimens were allowed to swell for 40 days. Vertical displacements were monitored continuously during the first 8 days. Afterwards, they were measured periodically every 24 hours until the end of the tests. Results of these tests are shown in Figure 1.



Sample	σ_v [kPa]
1	0
2	25
3	100

Figure 1. Swell tests on nonreinforced GCL.

Initial and final heights of specimens are presented in Table 1. Final height of specimens depends on the applied confining stress. As it was expected, specimen 1 under zero confining stress swells the most (Fig. 1). In the case of constrained swell tests (specimens 2 and 3), swelling started later on after the initial compression stage. An equilibrium height was attained approximately in the same time for the tested range of normal stresses. In other words, the time required for the equilibrium state was not dependent on the confining stress.

Table 1. Swell test results.

Sample	Confining stress [kPa]	Initial height [mm]	Final height [mm]
1	0	5.46	14.2
2	25	5.40	6.82
3	100	5.10	5.56

Swell test results were used later in planning of direct shear tests. Depending on the applied normal stress and the duration of hydration stage, the initial height of specimens prior to shearing was estimated. The vertical location of the specimen within the shear box was adjusted to force the failure plane through the bentonite.

On the same samples that were left in oedometer cells at the end of swelling, consolidation tests were conducted with three different loading-unloading-reloading cycles. These tests enabled the determination of the coefficient of consolidation, c_v, which was used for the calculation of the displacement rates in the direct shear tests. Coefficient of consolidation was determined using the log-time or Casagrande's method.

The appropriate displacement rates necessary for the realization of drained conditions were determined according to the ASTM D 3080 standard (Eq. 1) and the relation (Eq. 2) given by Gibson & Henkel (1954):

$$r = d_f / t_f \tag{1}$$

where r = displacement rate; d_f = estimated horizontal displacement at failure; and t_f = total estimate elapsed time to failure,

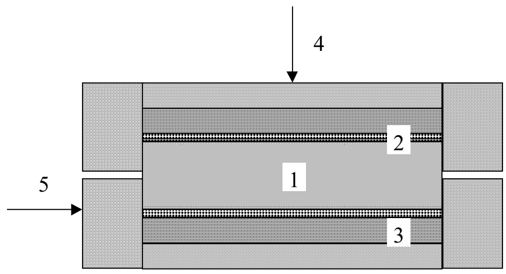
$$t_f = \frac{H^2}{2 c_v (1 - U_c)} \tag{2}$$

where H = drainage path; c_v = coefficient of consolidation; and U_c = degree of consolidation. It was estimated that for the anticipated normal stress range in direct shear tests, the displacement rate of 0.001 mm/min would be necessary for drained conditions.

3 DIRECT SHEAR TESTS

3.1 *Modified shear box*

In order to perform direct shear tests on GCLs by using a standard shear apparatus, some modifications were necessary. Shear box was enlarged to 100 × 100 mm comparing to the standard boxes used for soil specimens, which are 70 × 70 or 60 × 60 mm large. Additional porous plates of different thickness were used depending on the height of hydrated GCL specimens. Finally, gripping of the specimens is solved by using teethed metal plates. Details of a modified shear box are shown in Figure 2.



1 – GCL, 2 – Teethed metal plate, 3 – Porous stone, 4 – Normal force, 5 – Shear force.

Figure 2. Cross-section sketch of modified shear box.

3.2 *Laboratory testing program*

Direct shear tests were conducted on the same product that was used for oedometer tests i.e. GCL. Short description of that type of nonreinforced GCLs is already given.

Laboratory testing program for shear strength determination consisted of five test series. The controlled test parameters along with the specimen codes are shown in Table 2. Two procedures of specimen hydration were used. Standard hydration procedure was applied for the series I-IV. Specimens were placed into the direct shear box, and immediately after the application of respective normal load, tap water was added to the box. Normal loads corresponded to the values that were applied in the shearing stage of tests. During a period of 24 hours vertical displacements were measured and recorded continuously. This procedure will be hereafter called normal hydration procedure. During the shearing stage, which started at the end of the hydration stage, four different shear strain rates were applied. Depending on the applied strain rates, shear tests lasted from 17 minutes to approximately 9,5 days. Time intervals for recording the measured output data were adapted to different test duration.

For the series V the so-called extended hydration procedure was applied. It means that the specimens were hydrated for 9 days prior to shearing. In the shearing stage, the same displacement rate as for the series I was used.

Shearing stage of all tests was stopped after the relative displacement of 15% was achieved. This was determined by the limits of our shear apparatus. Testing program was planned in that way to find out what is the influence of specimen hydration procedure and displacement rate on the shear strength of GCLs.

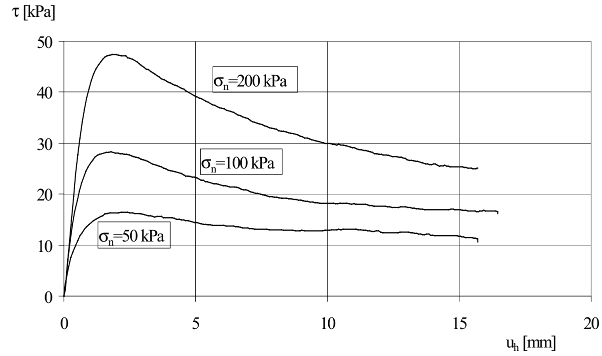
Table 2. Direct shear testing program.

Series*	Displacement rate [mm/min]	Normal stress, σ_n [kPa]			Test duration
		50	100	200	
I	1.219	I-1	I-2	I-3	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

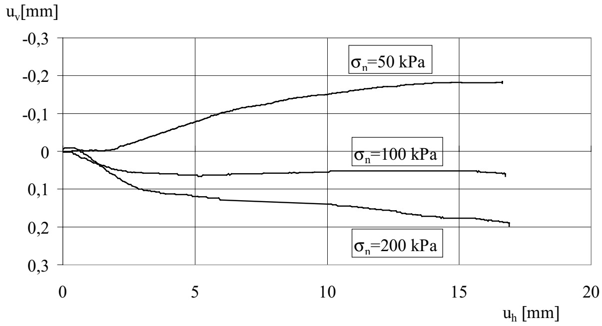
* Normal hydration: I-IV; Extended hydration: V.

3.3 *Test results*

During the direct shear testing the following parameters were measured and recorded in an output file: shear stress, horizontal, and vertical displacement. As the total time required for the determination of residual strength varied considerably for different displacement rates (Table 2), time intervals for recording the output data were adapted accordingly. On the basis of this data, stress-displacement curves and the relationship between vertical and horizontal displacements were created. Respective results for one series of direct shear tests are shown in Figure 3.



a) Shear stress vs. horizontal displacement



b) Vertical displacement vs. horizontal displacement

Figure 3. Direct shear test results for series III.

It can be seen from the Figure 3 that nonreinfoced GCLs exhibit in shearing behavior similar to the behavior of overconsolidated clays i.e. the difference between peak and residual values is significant. What was more important for the interpretation of our results is the development of vertical displacements in shearing. It was seen that in some cases samples show further consolidation during shear, and in the other cases samples started to swell, depending on the applied normal stress and horizontal displacement rate. The summary of the results for all direct shear tests is presented in Table 3.

Table 3. Summary of direct shear test results.

Series – Sample	Displacement rate [mm/min]	Normal stress [kPa]	Peak strength [kPa]	Residual strength [kPa]	Strength ratio	Vertical displacement mode*	Final water content [%]
I – 1		50	24.4	20.4	0.84	+	80.88
I – 2	1.219	100	46.1	37.6	0.82	+	66.44
I – 3		200	71.5	59.5	0.83	+	57.67
II – 1		50	19.7	14.5	0.74	+	85.32
II – 2	0.1219	100	36.3	25.7	0.71	+	64.37
II – 3		200	62.5	42.8	0.68	+	53.00
III – 1		50	14.7	7.9	0.54	–	85.41
III – 2	0.01219	100	30.4	19.3	0.63	+	71.82
III – 3		200	57.8	36.9	0.63	+	58.97
IV – 1		50	16.5	11.7	0.71	–	122.61
IV – 2	0.00146	100	28.3	16.9	0.60	–	78.97
IV – 3		200	47.4	25.3	0.53	0	60.25
V – 1		50	22.5	14.9	0.66	+	124.51
V – 2	1.219	100	37.2	27.3	0.73	+	93.10
V – 3		200	66.7	45.1	0.68	+	71.76

* Consolidation +, Swelling –, No change 0.

By reviewing data in Table 3 one can conclude the following:

- for the same normal stress, peak and residual shear strength values are lower for lower displacement rates (series I – IV),
- strength reduction from peak to residual values are higher for lower displacement rates i.e. strength ratio is lower,
- extended hydration procedure produces lower strength values compared to the values obtained by normal hydration procedure (series I and V),
- final water content is dependent on the normal stress and total duration of tests i.e. on the type of preparation procedure and on the displacement rate.

Looking at the results presented in Table 3, it was obvious that influence of the hydration procedure and the displacement rate should be explained in a more detailed way.

3.4 *Shear strength*

Peak and residual shear strength envelopes are shown in Figures 4 and 5. The obtained values of strength parameters: cohesion, c, and friction angle, ϕ, are extracted in Table 4.

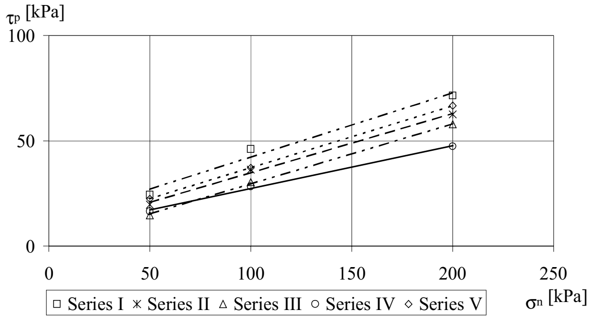


Figure 4. Peak strength envelopes.

Table 4. Shear strength parameters.

Series	Displ. rate	Peak parameters		Residual parameters	
	[mm/min]	c [kPa]	ϕ [°]	c [kPa]	ϕ [°]
I	1.219	11.7	17.0	9.5	14.3
II	0.1219	6.6	15.7	6.0	10.5
III	0.01219	1.0	15.9	0.0	10.5
IV	0.00146	7.0	11.5	7.5	5.1
V	1.219	7.8	16.4	6.0	11.2

It can be seen that the obtained values of cohesion and friction angle show huge variability. Displacement rate and hydration procedure both affect the strength parameters but their influences overlap and are not yet clearly differentiated.

3.4.1 *Influence of the hydration procedure*

The influence of the hydration procedure can be seen by the comparison of the results for test series I and V. In these series the same displacement rate was applied in the shearing stage after the completely different hydration procedure. In the series I the normal hydration was applied during the 24 hours. On the other hand, for the series V the so-called extended hydration stage lasted 9 days.

Looking at Table 3 it can be seen that the total values of peak and residual strength are lower for the series V. It seems that these differences are predominantly caused by higher values of final water contents for the samples from series V in comparison to the samples from series I.

Peak strength envelopes are almost parallel for series I and V that means that they have similar friction angle but different cohesion. In case of residual values, both strength parameters, c and ϕ, are lower for series V.

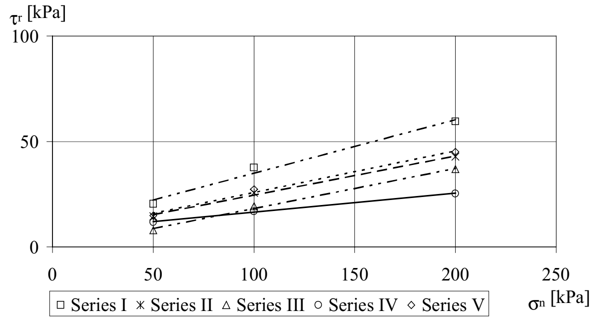


Figure 5. Residual strength envelopes.

It is also interesting to compare the results from series IV and V. Although these series differ in hydration and shearing stage, they have almost identical total duration of tests. It can be seen that they have very similar cohesion and quite different friction angles. It points out that total duration of tests determines the expressed cohesion due to the similar values of realized final water contents i.e. void ratio. On the other hand, it seems that friction angle is much more affected by the displacement rate.

3.4.2 *Influence of the displacement rate*

The influence of horizontal displacement rate can be better seen if the results are presented as it is shown in Figures 6 and 7. The dependence of peak and residual strengths on the displacement rates for the same normal stress values are shown.

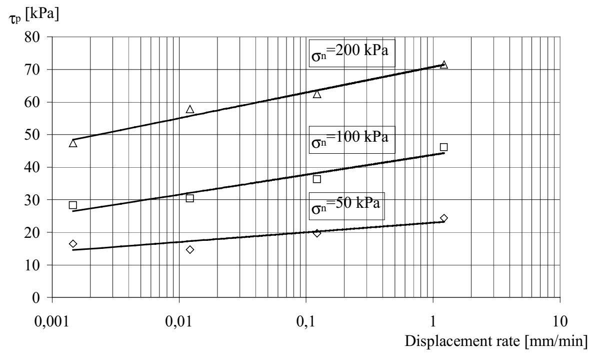


Figure 6. Peak strength vs. displacement rate.

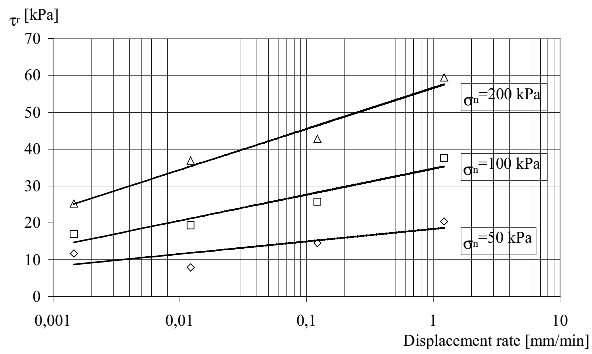


Figure 7. Residual strength vs. displacement rate.

The decreasing displacement rate produces lower values of measured strengths. This trend is more emphasized for residual strength than for peak strength values. Comparing the values for the series I and IV with maximum and minimum displacement rate values (1,219 mm/min vs. 0,00146 mm/min) one can see that the strength reduction is approximately 35% for the peak values and about 52% for the residual values. In the literature this phenomena is explained either by the changes of effective stress on the failure plane or by the rate effects i.e. creep (Fox et al. 1998).

3.5 *Discussion of test results*

From previous research, it was already known that the measured values of shear strength depend on the way of performing the laboratory tests. This research was directed towards the determination of the influence of two parameters in the direct shear tests: hydration procedure and displacement rate, for which we believed that they are the most important ones.

For the discussion purposes, we will take as a reference value the shear strength for a series I (normal hydration, fast shearing). It is obtained under the displacement rate similar to the one that was mainly used before the ASTM D 6243 was established, and which rate was prescribed in fact for the interface shear strength determination according to ASTM D 5321.

From our results, it looks that the influence of the displacement rate is much more pronounced than the influence of hydration procedure on the total shear strength (Table 3). It can be seen that for the series V (extended hydration) peak strength is lower by approximately 11% and residual strength by approximately 26% from the reference value. For the series IV (slow shearing) peak strength is lower by about 35% and the residual strength by about 52% when compared to the reference value.

Comparison of the results in a more detailed way, considering the influence of the testing procedure on cohesive and frictional part of strength, will more clearly differentiate the influence of hydration procedure and displacement rate. It was already mentioned that the total test duration determines the obtained cohesion due to the achieved final water content. Friction angle is controlled, on the other hand, much more by the displacement rate. In the literature, the reduction of measured strength with the reduction in the displacement rate is in most cases explained by changes of effective stress on the failure plane. Some authors (Fox et al. 1998) pointed out that the rate effects i.e. creep of the bentonite may also influence the observed behavior.

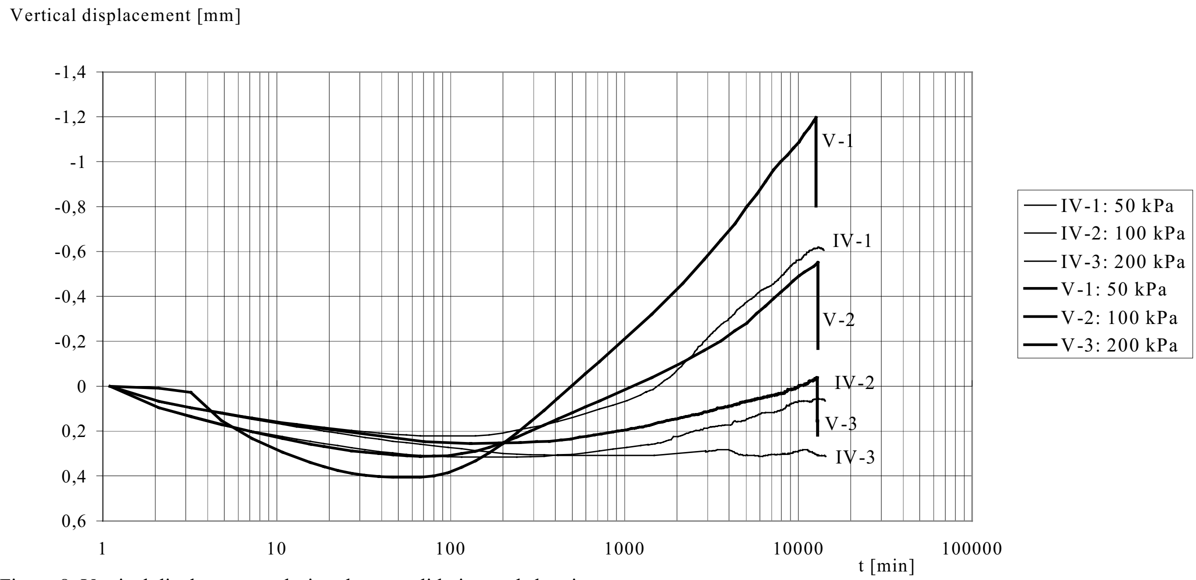


Figure 8. Vertical displacements during the consolidation and shearing stage.

For the explanation of these phenomena it was very important that we have measured vertical displacements continuously from the beginning of tests (hydration stage) till the end of tests (shearing stage). If we assume that undrained conditions are present in series I, than for series II to IV we should have more and more drained conditions. As a consequence of that the samples from series IV should experience the largest vertical deformations (settlement). But, what we have obtained were swelling deformations in shearing stage (Fig. 8). If we have swelling behavior we should expect the negative pore pressure on shearing plane, and it should cause higher values of expressed strength not the reduced values as we obtained for series IV.

At this point of research we can only conclude that there is not enough data for the conclusions about the pore pressure or effective stress distribution in the direct shear tests. But what we have found out is the fact that the total test duration determines the cohesive part of the shear strength due to the higher values of the final void ratio. Samples from series IV and V have similar final vertical displacements, i.e. void ratio, although they come to this state in different ways: continuous swelling for the series IV and swelling followed by settlement for the series V (Fig. 8). These two series show almost the same cohesion.

It also seems that the mode of the vertical displacements in shearing stage (settlement or swelling) influences the obtained friction angle. Only for the series IV we have obtained swelling behavior in shearing and significantly smaller friction angle. For the series I-III and V, settlement occurred during the shearing stage. Friction angles for these series are similar and much higher than friction angle for series IV.

3.6 *Shear strength criterion*

On the basis of presented laboratory test results and current knowledge, shear strength criterion of GCL is proposed as it is shown in Figure 9. The functions presented in Figures 6 and 7 are extrapolated to the displacement rate of 0.001 mm/min for all three normal stress values. These values are redrawn in Figure 9 giving the peak and residual strength envelopes.

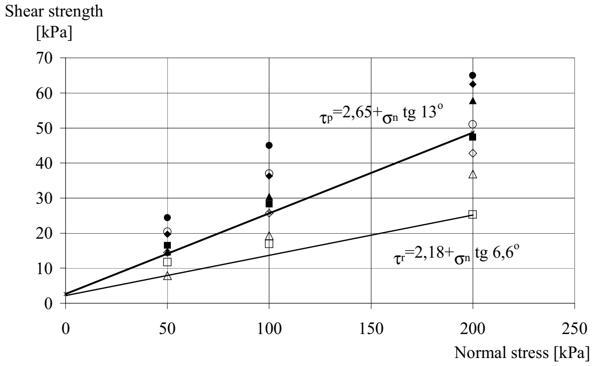


Figure 9. Shear strength envelopes.

Real values of measured peak and residual strengths are shown by full and blank symbols, respectively. Shear strength envelopes created in that way from the available data enables the design of new facilities in a safe way.

4 CONCLUSIONS

It is shown that using the modified standard direct shear apparatus, shear strength of the nonreinforced GCLs could be obtained. The influence of the hydration procedure and displacement rate is obviously demonstrated. There are however still some doubts in the interpretation of obtained laboratory data.

In the time of carrying out our tests ASTM D 6243 standard did not exist. Nevertheless, we have applied some procedures that are accepted in the standard now. In this standard a lot of issues are still left to the judgment of design engineers, because of site-specific and product-specific conditions. The only statement in the standard with which we do not agree is the recommended specimen size in case of examining the nonreinforced GCLs. Comparable results can be obtained by the use of smaller samples (100 ×100 mm), thus avoiding the need for the substantial investment in a specialized apparatus.

It is undoubtedly better for design purposes to perform direct shear tests on GCLs according to the mentioned standard than to perform tests as it was done earlier by following the procedure of ASTM D 5321. For the research purposes it is still necessary to understand some issues in a more detailed way. It is our opinion that the discussions about the influence of hydration procedure, shear strain rate, drained-undrained conditions and creep effects are not yet concluded.

In order to obtain a clear view of the influence of different parameters it will be necessary to satisfy the following conditions:

- specimens should have equal initial water contents,
- specimens should be completely hydrated prior to shearing,
- shearing should be performed with the displacement rates, which will uncouple creep effects.

The first condition will enable better interpretation and lower scatter of test results. In our research specimens had very different initial water contents although they were cut from the same lot. Final water contents are not very reliable factors if this requirement is not met.

By allowing the specimen to fully hydrate prior to shearing, we will be able to distinguish the vertical deformations caused by swelling properties of bentonites and dilation caused by shearing.

It is known that reliable pore pressure measurements are not possible in direct shear test. We can only assume their values based on oedometer test results. In order to avoid swelling deformations in shearing stage, displacement rates should not be too low.

It is therefore our proposal to perform the direct shear tests on GCLs in three stages:

- Stage 1: homogenization of specimens,
- Stage 2: hydration of specimens for 7,14 and 40 days,
- Stage 3: shearing by displacement rates 0,1 and 0,01 mm/min.

With the homogenization of specimens the first condition of equal initial water content should be met. It can be performed in a specially designed container with humidity and temperature control.

During the hydration for 40 days, complete swelling should occur according to our test results, and these specimens will give us the reference value with which two other series should be compared, to find out what is the influence of hydration procedure.

Using the proposed displacement rates in stage 3 and the procedure of data extrapolation to the displacement rate of 0.001 mm/min, creep effects should be minimized. Also, shearing stage is short relatively to the duration of hydration stage, thus eliminating the additional swelling in shear and interference of the effects of pore pressure distribution and creep.

Current research is ongoing on the specimens of nonreinforced GCLs according to the proposed procedure. We believe that the results obtained in that way will take us to the better understanding of the behavior of bentonites and nonreinforced GCLs.

REFERENCES

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

ASTM D 5321 – 92. Standard Test Method for Determining the Coefficient of Soil and Geosynthetic or Geosynthetic and Geosynthetic Friction by the Direct Shear Method.

ASTM D 6243 – 98. Standard Test Method for Determining the Internal and Interface Shear Resistance of Geosynthetic Clay Liner by the Direct Shear Method.

Fox, P.J., Rowland, M.G. & Scheithe, J.R. 1998. Internal shear strength of three geosynthetic clay liners. *ASCE 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 liner. *ASCE Journal of Geotechnical Engineering* 122(4): 259-266.

Gilbert, R.B., Scranton, H.B. & Daniel, D.E. 1997. Shear Strength Testing for Geosynthetic Clay Liners. In Larry W. Well (ed.), *Testing and Acceptance Criteria for Geosynthetic Clay Liners, ASTM STP 1308.* American Society for Testing and Materials.

Gibson, R.E. & Henkel, D.J. 1954. Influence of duration of tests at constant rate of strain on measured drained strength. *Geotechnique* 4(1): 6-15.

Heerten, G. 1995. Groundwater protection using a GCL at Franz-Josef-Strauss airport Munich, Germany. In R.M. Koerner, E. Gartung & H. Zanzinger (eds), *Geosynthetic Clay Liners, Proc. Intern. Symp., Nurnberg, 14-15 April 1994.* Rotterdam: Balkema.

Koerner, R.M., Soong, T-Y. & Gontar, A. 1998. Selected Aspects of GCL Shear Strength Testing. *Geo-Bento '98, Paris*. Insatec. Publ.

Koerner, R.M. 1997. Perspectives on Geosynthetic Clay Liners. In Larry W. Well (ed.), *Testing and Acceptance Criteria for Geosynthetic Clay Liners, ASTM STP 1308.* American Society for Testing and Materials.

Koerner R.M. 2000. Emerging and Future Developments of Selected Geosynthetic Applications (The Thirty-Second Terzaghi Lecture). *ASCE Journal of Geotechnical and Geoenvironmental Engineering* 126(4): 293-306.

Richardson,G.N. 1997. GCL internal shear strength requirements. *Geotechnical Fabrics Report*, 15(3): 20-25.

Schmidt, R. 1995. GCL installation in a water protection area for the A)& motorway near Leutkirch, Germany. In R.M. Koerner, E. Gartung & H. Zanzinger (eds), *Geosynthetic Clay Liners, Proc. Intern. Symp., Nurnberg, 14-15 April 1994.* Rotterdam: Balkema.