The influence of clay wetting on geomembrane-clay interface strength

L'influence de humidification d'argile sur la résistance à l'interface entre la géomembrane et l'argile

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ABSTRACT

The wetting and saturation of bottom clay liner at waste landfill reduces the shear strength of clay-geomembrane interface. This behavioral aspect was studied as a possible mechanism of local bottom failure of waste landfill in Zagreb.Two series of clay-geomembrane interface strength tests were conducted in large shear box for the range of normal pressures expected in field. In the first series the interface strength was tested in the “as compacted” state in order to obtain the referent values for ordinary conctruction conditions. In the second series the samples were flooded with water and left to saturate before testing. The shear testing was performed in unconsolidated-undrained conditions in order to simulate rapid loading conditions during relatively fast waste filling. The test results clearly showed the significant interface strength reduction in flooded samples.

RÉSUMÉ

L'humidité et la saturation en eau au niveau de la membrane étanche inférieure réduisent la résistance au cisaillement à l'interface entre la géomembrane et l'argile sur les sites d'enfouissement de déchets. Cet aspect de comportement a été étudié puisque il décrit le mécanisme possible de rupture locale au fond du site d'enfouissement de déchets à Zagreb. Deux séries d'essais ont été effectuées dans un grand appareil de cisaillement direct dans la gamme complète des contraintes normales anticipées sur le site afin de déterminer les résistances au contact entre la géomembrane et l'argile. Dans la première série des essais la résistances au contact a été déterminée pour les conditions normales de mise en place, afin de définir les valeurs de référence. Dans la deuxième série, les échantillons ont été complètement immergés dans l'eau et ont ensuite été laissés pour atteindre l'état de saturation avant de procéder aux essais. La résistance au cisaillement a été déterminée dans les conditions non consolidées et non drainées afin d'imiter les conditions de croissance rapide de charge pendant le remplissage relativement rapide du site d'enfouissement. Les résultats obtenus durant ces essais montrent de manière claire que la résistance au contact diminue considérablement dans le cas des échantillons saturés à l'eau.

1 INTRODUCTION

The rehabilitation of Landfill Jakuševec in Zagreb included excavation of about 6.000.000 m^3 of old waste from nearby dumpsite and its placement on engineered bottom barrier. The old waste was dumped for almost 30 years and the reason for its removal was registered contamination of underground water in river valley. The whole project started in late 90-ties. The planned final volume of landfill is about 12.000.000 m^3.

The removal and placement of old waste ended in September, 2003 and in 2004 four Cells were covered, while the remaining part of landfill is used for actual city waste disposal.

The bottom lining system consists of 1m thick clay layer covered with 2.5mm HDPE textured geomembrane, than cushion geotextile, gravelly drainage layer, filter geotextile, protection layer and waste are placed.

In November, 2002 the construction of bottom clay liner in southwest part of Cell 4 was completed and covered with geomembrane and cushion geotextile. The snow stopped further works and the bottom liner remained insufficiently protected during winter period. In early spring 2003 the geomembrane in lowest parts of Cell (near drainage pipe) was raised and it was found that the superficial clay layer under geomembrane had excessive water content. In these parts this clay layer was replaced. Other occasional nearby control did not show unacceptable results and the placement of old waste started very fast to compensate winter delay.

Regular measurements of the leak detection system in March/April 2003 indicated damage of the geomembrane and detection system inside waste body, which was at that time already 28 m high. The visual inspection of temporary slopes did not show any particular cracks or deformations, and there was no waste displacement or movement (Maertens et al, 2004).

However, it was decided to excavate the waste to the damaged part of geomembrane and repair the lining system.

Several possible mechanisms were considered as the possible cause of the damage. One of them was possible undetected wetting of clay layer and, therefore, reducing of interface clay-geomembrane strength. This failure mechanism was attributed cause of Kettleman Hills landfill slope failure (Mitchell et al, 1990). Also, a similar case in UK is reported by Jones and Dixon (2003).

In this paper the investigation works and laboratory testing program for determining the influence of wetting on interface strength are presented.

2 SITE INVESTIGATIONS

In order to reach the damaged area, the corridor in the width of 30m (expected length of damage) was excavated into the slope of waste body. The excavation of waste ended in July 2003, then the protection and drainage layers were removed and the area approx. 25x15 m was examined (Fig 1).

Samples of clay and geomembrane were taken for further testing. After completion of the inspection, the damaged part was remediated, the snapped cables of leak detection were repaired and pit was filled with waste to required height.

The inspection on site revealed the following (Maertens and al, 2004):

Figure 1. Investigation area inside the waste body

- the geomembrane was damaged in a zone of few meters wide in the direction perpendicular to the main waste slope. The damage consisted of several ruptures and cracks. Maximal crack width was about 50 cm. One of the smaller ruptures is shown on Fig. 2.
- in some places the uncracked geomembrane was still under tension force and soon after cutting the fissure of 40 cm opened
- some smaller defects were also observed near welded connections
- clay underneath the geomembrane was somewhere very soft which indicates excessive water content
- the slip plane was observed in the superficial clay layer 1-3 cm under the geomembrane (Fig. 3). When the geomembrane was removed this surface layer mostly remained “stuck” to it

Figure 2. Smaller geomembrane rupture in damaged zone

The clay was sampled to depth of 60 cm at 15 positions: 12 samples from investigated damaged area and 3 samples outside of it (control area). The water content was examined in top 5 cm and at 3 points in 20 cm depth intervals.

The top layer in control area had water content in acceptable range (mean 20.6%, max. 20.97%). The average water content in deeper samples varied from 20.1 to 21.2%.

In damaged area the surface layer (0-5 cm) had significantly higher water content (27 to 30.4%) and in other depths varied from 19 to 23.3% (still in acceptable range).

Since the investigation took place several months after indicated damage, the leachate was leaking through the cracks in ruptured geomembrane, and possibly contributed to higher water content values at clay surface.

The results of all these observations are not quite conclusive towards precise explanation of damage cause. However, the certain facts are:
- the excessive wetting of surface clay under the unprotected geomembrane was found after winter period in some lower parts of Cell 4, while other parts were not examined thoroughly
- after the fast placement of waste, somewhere with relatively steep slopes, the geomembrane suffered excessive deformations and its tensile strength was reached leading to opening of cracks
- the slip plane was somewhere found in the thin softer surface clay layer very close to geomembrane
- the leachate leaking through initial cracks may contribute further wetting of surface clay and induce progressive cracking

Figure 3. Slippage of softer clay layer over firm clay base

3 LABORATORY TESTING PROGRAM

3.1 *Testing conditions*

In order to examine the possible mechanism of interface strength reduction due to clay wetting, a program of limited laboratory testing was proposed. The basic idea of this program was to determine the interface shear strength parameters for the range of clay compaction conditions as found on site in:

I. "as compacted" state (as a referent state) and in

II. saturated undrained conditions (as a possible state after wetting and fast loading).

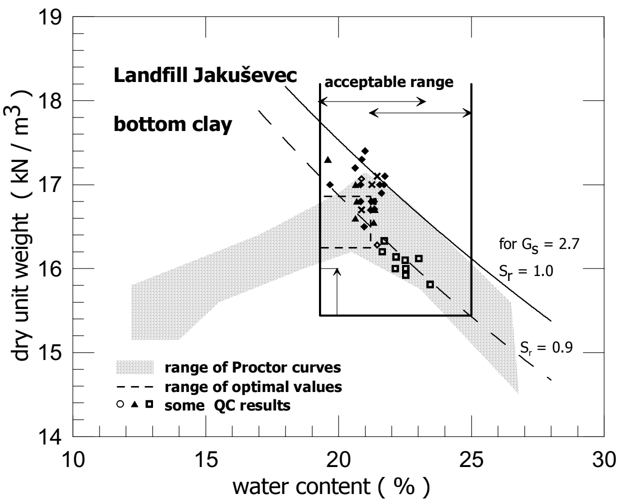

Figure 4. The range of field compaction conditions on Cell 4

The range of clay compaction conditions on site is presented on Fig. 4, using data obtained from quality control (QC) program. The design specifications defined the acceptable field range with dry unit weight greater than 95% of optimal value and field water content, w, between w_{opt} and w_{opt} + 4%. The optimal values were defined individually for each part of clay borrow pit and relevant liner construction sections.

The bottom clay liner was constructed in four layers of 25 cm. Also presented are some control results for top layer of liner (beneath the geomembrane) made by nuclear probe. Other testing methods showed similar results. The results are within acceptable range, although some results show unlikely saturation over 100% when compared with theoretical curve for average clay specific gravity of 2.7.

The conditions for testing were chosen to encounter the presented range and approximately follow the line of 90% saturation, like :

Case a)	water content: 19%	dry unit weight: 17.3 kN / m^3
Case b)	water content: 22.5%	dry unit weight: 16.1 kN / m^3
Case c)	water content: 25%	dry unit weight: 15.3 kN / m^3

These cases are presented in Fig.5 with laboratory Proctor curve for particular clay samples, taken for laboratory testing.

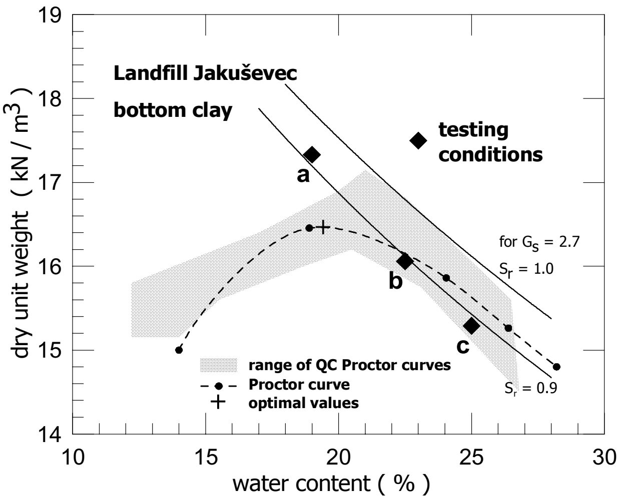

Figure 5. Compaction properties of tested clay (w_{opt} = 19.42%; γ_{dry} = 16.47 kN / m^3)

3.2 *Swelling properties*

The unsaturated samples were placed in consolidation (oedometer) ring and flooded with water. Rapid swelling occurred (actually a heave of compacted sample due to unconfined relaxation of capillary forces and overconsolidation by compaction efforts).

The measured free swelling for the most compacted specimen (case a) was 1.37 mm which gives about 6.9% of vertical and volume expansion.

Also, the pressures which prevent swelling were measured by increasing the vertical load as necessary to maintain the deformation gauge reading within 0.01mm of the corrected zero reading, with recording cumulative magnitude of vertical stress and corresponding elapsed time.

The results are presented in Fig. 6. The range of swelling pressures from 12.5 to 75 kPa means that the heave of bottom clay liner (when wetted) could be prevented by weight of 0.5m of drainage gravely layer and at least 1m of compacted waste for the loosest compaction conditions. For the densest state the heave would be totally prevented with several meters of waste overburden.

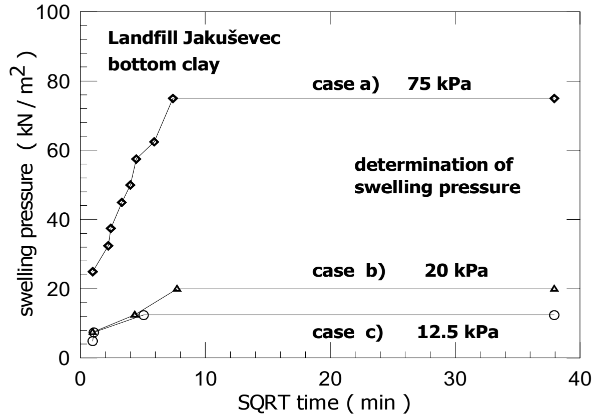

Figure 6. Determination of swelling pressures

3.3 *Interface strength testing*

The clay samples and geomembrane taken from the cell 4 were used. The clay specimens were prepared to densities and water contents for case a) to c) and the clay-geomembrane direct shear testing was done according to ASTM D 5321-92. The dimensions of tested samples were 30x30x4 cm. The shear testing was done on apparatus "GEOTEST", USA at Geotechnical laboratory of Faculty of Civil Engineering, Zagreb. The total of 18 tests was done in following series:

Series I – "as compacted": the specimen were "consolidated" in unsaturated conditions for 24 hours and then sheared at rate of shearing 1mm/min and with normal pressures of 50, 200, and 400 kPa. These conditions simulate relatively optimistic situation where the bottom clay stays in "as compacted" state and filling of landfill is slow enough allowing for some sett-lements and tighter adhesion of geomembrane and clay to occur.

Series II –"soaked": all three specimens were prepared as above, then flooded with water and left to saturate ("soaking") 24 hours, with low normal pressure of 2-5 kPa. During this phase some swelling of sample occurred. The shearing was done in unconsolidated-undrained conditions: after swelling the samples were sheared immediately when the normal pressures (also 50, 200 and 400 kPa) were applied. The rate of shearing was 1mm/min .

The results of tests are presented in following figures. The complete shear stress-displacement curves are presented for each normal pressure and all testing conditions in Figs 7-9. In Figs. 10 and 11 the residual strength parameters (apparent adhesion, a, and friction angle, φ), of each series was obtained.

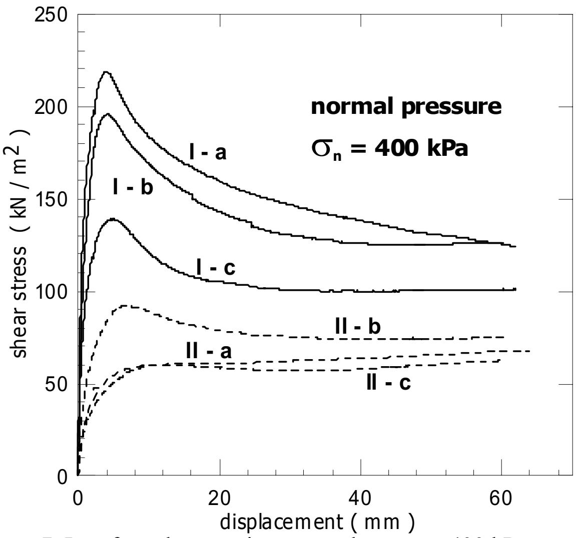

Figure7. Interface shear testing–normal pressure 400 kPa

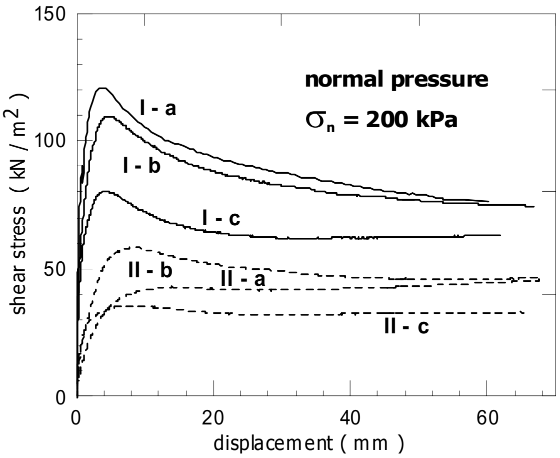

Figure 8. Interface shear testing–normal pressure 200 kPa

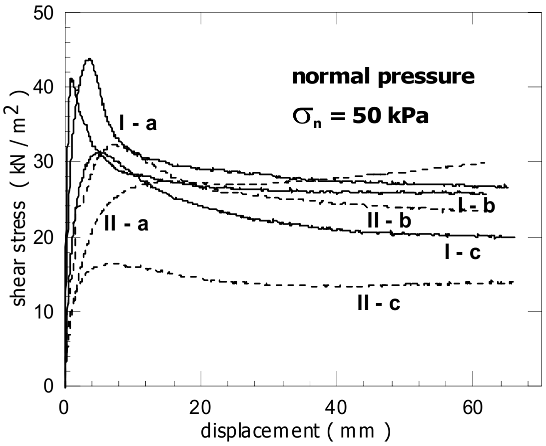

Figure 9. Interface shear testing–normal pressure 50 kPa

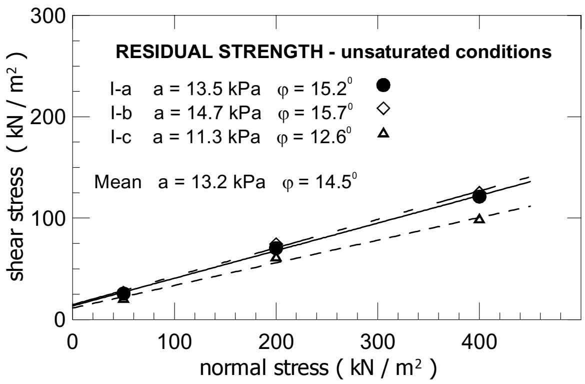

Figure 10. Residual interface strength – unsaturated specimen

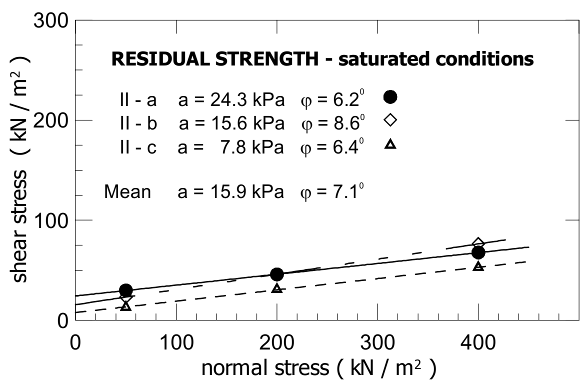

Figure 11. Residual interface strength – wetted specimen

The water content was measured after tests. The unsaturated tests showed negligible change of 0.3-1%, probably due to local inhomogenities in large sample. The flooded specimen ended tests at virtually same water content of 24-25%. The case c) specimen had negligible change of 0.5% and for case a) specimen the change was 5-6%.

The presented test results show that the interface clay-geomembrane strength is significantly affected by moisture and density conditions of clay before testing.

For unsaturated "as compacted" specimen the differences are more pronounced in peak values than in residual. For the samples near Proctor optimum values (cases a and b) the residual strength values are virtually the same, while the looser and wetter case c consistently gives lower values.

For the specimen pre-treated by unconfined flooding and soaking of water the trend of results is similar, except that difference between peak and residual values is not large.

The pre-treatment gave significantly lower strength parameters. The difference is obvious in friction angle which for wetted specimen has value about half the value of "as compacted" specimen. Also, in the same test series the values of friction angle are about the same, while some differences could be found in values of apparent adhesion.

As expected, the case c) gives minimal strength results in all tests, while the residual results for cases a and b for normal pressure of 50 kPa fall in narrow range (probably affecting also the interpreted adhesion value).

4 CONCLUSIONS

The excessive wetting of bottom clay liner in large landfills is not uncommon real-life scenario, although it is not prescribed in design specifications. In unconfined situation the compacted clay layers swell and soak water, which in turn reduces strength of clay and clay-geomembrane interface.

In the performed testing program the main difference between testing conditions was in residual friction angle, while the average apparent adhesion values do not differ significantly.

As expected, the lowest strength gives the specimen with lowest dry density and highest water content, well above the optimal Proctor value.

Since the textured geomembrane used in tests was the same in all tests, all the differences between tests can be attributed to clay conditions and varied properties. This means that, for the given geosynthetic, the complete failure criteria (at interface or in ground) controls the underlying soil, and that this aspect should be emphasized in design specifications and construction control.

The variability of measured geosynthetic interface strength is not an uncommon problem due to various reasons (Stoewachse et al, 2002). Introducing the influences of inherent soil variability or possible soil conditions as variables (for eg. design or acceptance testing), enlarges the number of necessary tests and makes these design procedures impractical. It seems that more detailed studies of interface failure mechanisms are needed in order to improve the understanding of problem and propose more effective testing procedures.

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