SOILS AND FOUNDATIONS Vol. 47, No. 5, 845–856, Oct. 2007
Japanese Geotechnical Society

EVALUATION OF INTERFACE SHEAR STRENGTH BETWEEN GEOSYNTHETICS UNDER WET CONDITION

M. W. SEO[i], J. B. PARK[ii] and I. J. PARK[iii]

ABSTRACT

This paper presents direct shear testing data for interfaces between a nonwoven geotextile or two types of geosynthetic clay liners (GCL) (reinforced and unreinforced) and two types of geomembranes (smooth and textured). In this study, the effect of moisture on interface shear behavior was investigated by performing shear tests in both dry and wet (or hydrated) conditions because the geosynthetic interfaces in a landfill are easily exposed to rain, leachate and groundwater beneath the liners. The degree of strength reduction with increasing displacement and the effect of the normal stress level on friction angles were examined, and the modified hydration method applied for the GCL was also validated. The test results showed that the normal stress level, interface water presence and hydration methods dominated the interface shear strength and behavior. The relationship between the peak secant friction angle and the normal stress demonstrated that the friction angle decreased with increasing normal stress, implying that the shear strength for safe design should be determined by using the maximum value of the normal stress applied in landfills. Finally, comparisons with a few published test results were presented and some design implications for the geosynthetic-installed landfills were discussed.

Key words: GCL, geosynthetic, hydration, interface shear strength, landfill, wet condition (**IGC**: H11/M9)

INTRODUCTION

Various kinds of geosynthetic such as geotextile, geomembrane, and geosynthetic clay liner (GCL) have been widely installed in waste landfills for different purposes. Geotextiles have been commonly used as separation layers, filtering layers and geomembrane protectors. Geomembranes are commonly used as a liner to minimize leachate migration at the bottom of landfills and to reduce landfill gas escape from the cover system. However, GCL is a relatively new type of geosynthetic that is installed to function as an alternative to compacted clay liners (CCL) in final covers or as an augmentation to CCL in bottom-lining systems of waste containment facilities. GCL is a good alternative due to its low hydraulic conductivity and relatively low cost (Bouazza, 2002). It is generally composed of a thin layer of bentonite (1) mixed with an adhesive that is attached to a geomembrane or (2) sandwiched between two geotextiles.

With the increased application of geosynthetics at the side slopes of landfills, dams, and banks, the stability of geosynthetic-involved slopes has become an important factor for consideration in side slope design. Geosynthetics slippage against soil or other geosynthetic along the weak interface of steep side slopes can induce an excessive local stress that may lead to tear and consequently sliding failure of the slope. Especially for waste containment facilities, interface shear strength of soil/geosynthetic or geosynthetic/geosynthetic is often lower than that of other materials like soil and wastes (Gabr and Valero, 1995; Kavazanjian, 2001). Therefore, structural stability should be considered during all phases of installing a liner or cover system with geosynthetics.

In most landfill sites, the interfaces of soil/geosynthetic or geosynthetics are vulnerable to water from leachates, rain, and groundwater, which can significantly change the interface shear strength. In the failure case of Kettleman Hill waste landfill, 'wetting' of HDPE liner/compacted clay liner was concluded to have contributed considerably to the slope failure (Mitchell et al., 1990; Seed et al., 1990). Subsequently, the effect of water presence on the interface shear strength has been investigated extensively (Yegian and Lahlaf, 1992; Ellity and Gabr, 2001; Triplett and Fox, 2001; Briancon et al., 2002). However, the studies did not perform a variety of laboratory tests with varying interfaces and conditions.

For geotextile (GT)/geomembrane (GM) interfaces, some studies have been carried out to examine the effect of water on the interface shear behavior (Yegian and Lahlaf, 1992; Ellithy and Gabr, 2001; Briancon et al., 2002). Yegian and Lahlaf (1992) performed direct shear tests under static loads and shaking table tests under dynamic

[i] Senior Researcher, Korea Institute of Water and Environment, Korea Water Resources Corporation, Korea.
[ii] Associate Professor, School of Civil, Urban & Geosystem Engineering, Seoul National University, Korea (junbpark@snu.ac.kr).
[iii] Professor, Department of Civil Engineering, Hanseo University, Korea.
The manuscript for this paper was received for review on July 3, 2006; approved on May 29, 2007.
Written discussions on this paper should be submitted before May 1, 2008 to the Japanese Geotechnical Society, 4-38-2, Sengoku, Bunkyo-ku, Tokyo 112-0011, Japan. Upon request the closing date may be extended one month.

loads for both dry and wet conditions. Ellity and Gabr (2001) found that the influence of moisture on interface shear strength varied according to the surface type of geomembrane. Briancon et al. (2002) devised a very large inclined plane testing apparatus ($2.0\,\text{m} \times 1.2\,\text{m}$ for the lower box) to determine the friction characteristics of geosynthetic interfaces in wet conditions. In addition, Briancon et al. (2002) proposed new procedures to simulate the most common hydraulic conditions to which geosynthetics systems are exposed.

In testing reinforced GCL and geomembrane (GM) interface, a variety of hydration methods have been applied (Gilbert et al., 1996; Eid et al., 1999; Triplett and Fox, 2001). Generally, the hydration of GCL is conducted in a shearing machine under shearing normal stress before the actual shearing test is initiated (Gilbert et al., 1996; Eid et al., 1999). However, this process requires considerable cost and testing time which can vary from 3 to 22 days. To overcome these shortcomings, Triplett and Fox (2001) performed GCL/GM shearing tests using the four-day, two-stage procedure proposed by Fox et al. (1998), which requires a significantly shorter period of time of four days: two out of the machine and an additional two in the machine.

Based on the earlier research, this paper focuses on the influence of moisture on the interface shear strength between GT/GM and GCL/GM. Laboratory tests were carried out using large direct shear testing equipment that is capable of measuring peak and residual interface shear strength at 80 mm. The influence of the normal stress level was examined and the effects of interface wetting or GCL hydration methods were discussed in detail. Finally, comparisons were made with the published works and suggestions were provided on the design of geosynthetic-incorporated waste landfills.

LABORATORY TESTING PROGRAM

Materials Used

Three different kinds of geosynthetics, namely geomembrane (GM), geotextile (GT) and geosynthetic clay liner (GCL), were used in the testing program. The basic information for geosynthetics used is furnished in Table 1, including tensile strength and polymer type which may influence the interface shear strength between geosynthetics under wet condition. The GM is a very low permeable synthetic membrane liner or barrier used with some geomaterials to control fluid migration in a man-made project, structure or system. The GT is a permeable geosynthetic comprised solely of textiles. GTs are used with foundation, soil, rock, earth or other geotechnical engineering-related materials as an integral part of a man-made project, structure or system. Also, the GCL is a hydraulic barrier consisting of a layer of bentonite and geosynthetic like geotextiles or geomembranes, mechanically held together by needling, stitching or chemical adhesives (Koerner, 1998). Bentonite in GCL is known as a very highly plastic and swelling clay material. This is an highly colloidal, expansive clay that is an alteration product of volcanic ash (Mitchell, 1975).

One side of the interface consisted of smooth or textured HDPE (high density polyethylene) geomembrane, and the other was made of a nonwoven geotextile or two commercially available GCL products. For two GCLs, GCL(A) is a reinforced liner in which granular bentonite is held between a woven, slit-film, PP (polypropylene) geotextile ($170\,\text{g/m}^2$) and a nonwoven, needle-punched, PP geotextile ($340\,\text{g/m}^2$). To provide reinforcement, PP fibers are needle-punched through the bentonite and geotextiles. GCL(B) is an unreinforced GCL consisting of bentonite mixed with an adhesive and bonded to a geomembrane, which is 2.0 mm thick and made of textured HDPE material. The liquid and plastic limits are 484% and 45% for the bentonite encased within GCL(A) and 453% and 45% for the bentonite included in GCL(B), respectively.

Direct Shear Tests

One of the first experimental studies of the frictional behavior between geosynthetics was conducted by Martin et al. (1984) with a modified direct shear testing apparatus. Since then, the direct shear testing method has been used for geosynthetics interface tests (ASTM, 1998). ASTM D 5321 (1998) requires direct shear tests on geosynthetics to be performed on specimens having minimum dimensions of 300 mm by 300 mm. In this research, direct shear test method suggested by ASTM D 5321 (1998) was applied and shear testing program for the GCLs was designed based on ASTM D 6243 (1999) and Fox et al. (1997).

Table 1. Geosynthetics tested

Notation	Type	Thickness (mm)	Density (g/m^2)	Tensile strength (kgf/m)		Polymer Type
				MD[1)]	CMD[2)]	
S-GM	Smooth geomembrane	2.0	—	6,506	6,119	HDPE[3)]
T-GM	Textured geomembrane	2.0	—			HDPE
GT	Nonwoven geotextile	9.0	1,000	11,629	7,358	Staple fiber PP[4)]
GCL(A)	Woven + bentonite + nonwoven (needle punched)	7.0	4,100	—	—	PP
GCL(B)	Bentonite attached to geomembrane	8.0	6,650	—	—	HDPE

1) MD: Machine Direction, 2) CMD: Cross Machine Direction, 3) HDPE: High Density Polyethylene, 4) PP: Polypropylene

As described previously, large direct shear tests were applied to evaluate interface shear strength between geosynthetics. A large direct shear test has a merit of being used for both soils and geosynthetics with minor modification, and can be handled with easy and simple manner. For this reason, direct shear tests for geosynthetic interfaces have been recommended and used by many researchers for many decades (Martin et al., 1984; Koutsourais et al. 1991; Jones and Dixon, 1998; Goodhue et al., 2001).

The direct shear tests on large (300×300 mm) rectangular geosynthetic specimens were performed with a maximum travel of 100 mm and no loss in the area of the shear plane. Geosynthetic samples were cut into rectangular shape for testing and clamped to the end of the lower and upper box. Then, the upper box was placed on the lower box and two geosynthetic samples were placed next to each other. Five different normal stresses, ranging from 6 kPa to 154 kPa, were applied. The displacement of the lower box was controlled by a precise motor control system with the horizontal movement monitored by a Linear Variable Differential Transformer (LVDT) with a shearing rate of 1 mm/min. The shearing rate was chosen based on Triplett and Fox (2001) who concluded that no consistent trend is observed between displacement rate and measured shear strength of textured geomembrane (T-GM) and GCL. Triplett and Fox (2001) performed direct shear tests over a shearing rate range of $0.01 \sim 10$ mm/min to investigate the effect of horizontal displacement rate on peak and large displacement shear strength of the geomembrane and GCLs. They found no effect of displacement rate on shear strength, which was in agreement with previously published data indicating that shear strength of a T-GM/nonwoven geotextile interface is independent of displacement rate (Stark et al., 1996). The horizontal load required to maintain the chosen shearing rate was measured by a load cell and displayed on a digital transducer readout.

The schematic view of the large direct shear testing machine used in the test is shown in Fig. 1. The shearing machine, originally designed for soil testing, was modified for testing the geosynthetic interfaces and featured the advantages of having a simple structure and being easily controlled. In addition, the shearing machine can be used for various testing conditions including soil/soil, soil/geosynthetic and geosynthetic/geosynthetic.

Dry and tap-water induced wet (or hydrated for GCL) conditions were applied for all the interface shearing tests which investigated six types of interfaces. Two kinds of geomembrane, S-GM (smooth geomembrane) and T-GM (textured geomembrane), had interface combinations with three other geosynthetics, GT (geotextile), GCL(A) and GCL(B), respectively. In case of the GCL(A)/GM interface, the nonwoven part of GCL(A) was in contact with GM, whereas for interfaces involved with GCL(B), the bentonite part of GCL(B) was in contact with GM. Details of the shear testing program are listed in Table 2.

Wetting (hydration) of Geosynthetic Interfaces

Geosynthetics were submerged or hydrated before being sheared to simulate the wet interface condition. The wetting or hydration method differed depending on the geosynthetic type. The geotextiles were just submerged for one day to simulate wet condition, whereas different hydration methods were applied for the two GCLs. GCL(A) was hydrated either under no normal stress (referred to as Free Swelling: FS) or under a normal stress of 6 kPa (referred to as Constrained Hydration: CH). The FS state was considered in the study to simulate the condition where GCL installed in landfills is hydrated before the waste is placed, while the CH condition simulates the condition in which GCL is hydrated with waste filling process. Though CQA (Construction Quality Assurance) programs generally do not allow for FS to occur, GCLs are frequently hydrated without any loading over the GCL, which was one of the reasons that the FS condition was considered in the study.

Following the two-stage hydration procedure proposed by Fox et al. (1998), GCL(A) was hydrated out of the

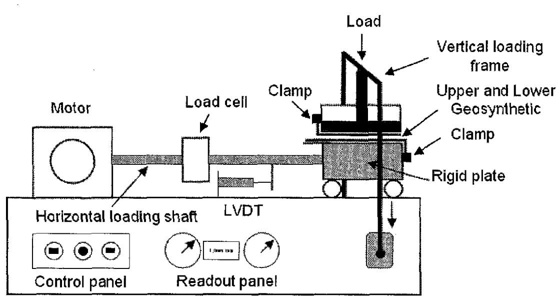


Fig. 1. Schematic view of direct shear testing equipment used

Table 2. List of experimental program

Interface	Condition of the interface	Description of wetting or hydration
GT/S-GM GT/T-GM	Dry, wet	Submerged for one day
GCL(A)/S-GM GCL(A)/T-GM	Dry, hydrated (FS*, CH**)	Hydrated for 10 days
GCL(B)/S-GM GCL(B)/T-GM	Dry, hydrated	Hydrated for 10 minutes with no normal stress

* FS (Free Swelling): hydrated under no normal stress, ** CH (Constrained Hydration): hydrated under a normal stress of 6 kPa

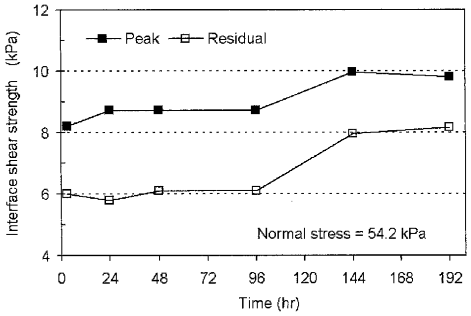


Fig. 2. Interface shear strength of GCL(A)/S-GM with hydration time

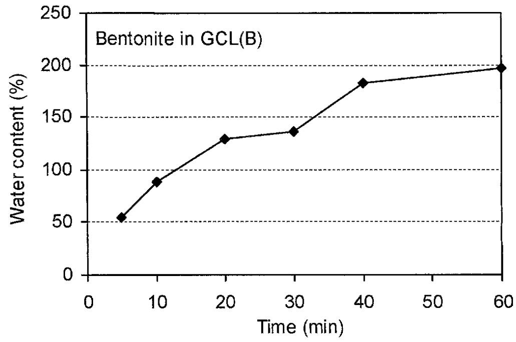


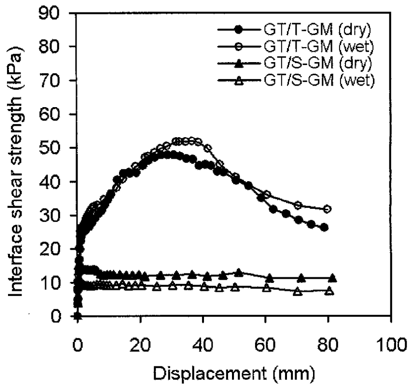
Fig. 3. Water content of the bentonite in GCL(B) with hydration time

shearing machine before the main shear tests, which could reduce hydration time compared to time required when GCL was hydrated in machine. The hydration time for GCL(A) was chosen, referring to the time suggested by Daniel et al. (1998) and obtained from preliminary tests. Namely, interface shear tests with varying hydration times were carried out to determine the time showing a constant shear strength value. The preliminary tests indicated that the interface shear strength of GCL(A)/S-GM in the FS condition started to be stable after a hydration time of 6 days (144 hr), as can be seen in Fig. 2. However, even though after 6 days, peak strength slightly decreased; therefore, hydration time was conservatively chosen as 10 days to get a shear strength in a completely hydrated condition.

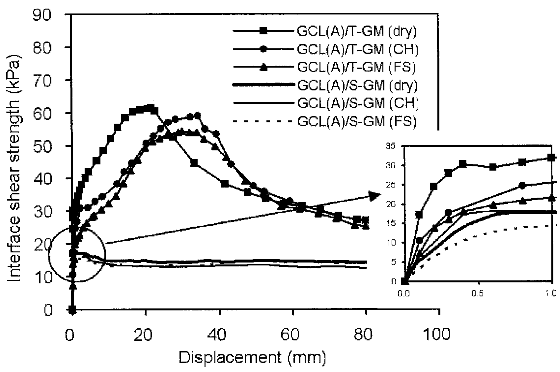
In the case of GCL(B), the bentonite part of GCL(B) was assumed to be dry for all phases of landfilling because an impermeable geomembrane is generally laid above the bentonite part of GCL(B). However, bentonite often becomes hydrated due to unexpected conditions. The testing for the GCL(B)/GM interface was conducted to identify the reduction of strength with the hydration of bentonite included in GCL(B), quantitatively.

Daniel et al. (1993) pointed out that, once water content of bentonite exceeds 50%, interface shear strength does not change too much from direct shear test results with varying water content of bentonite. Namely, the bentonite does not have to be fully hydrated for bentonite's strength to be greatly reduced. It means that, at 50% water content, the bentonite is fully hydrated and very slick. Moreover, the average water content of the failured test plot with same GCL installed (Daniel et al., 1998) was also 60%. Therefore, as water content of bentonite of GCL(B) reached 50% within 10 minutes with no normal stress, GCL(B) was hydrated for 10 minutes in the experiment (Fig. 3).

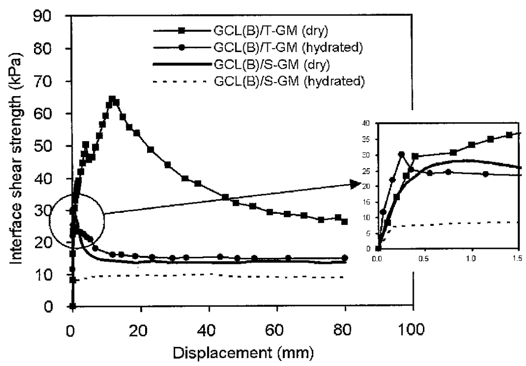
After wetting or hydration is completed out of the shearing machine, the specimens were transferred to a shear box in less than 10 minutes. Then, vertical loading was applied and the shear tests were initiated after vertical displacements were stabilized. In almost all cases, the time for stabilization did not exceed 30 minutes, which was comparable to the case of Fox et al. (1998). More time was required for stabilizing specimens hydrated on the FS condition for GCL(A).



(a) GT/GM interface



(b) GCL(A)/GM interface



(c) GCL(B)/GM interface

Fig. 4. Shear stress and displacement relationships at a normal stress of 100 kPa

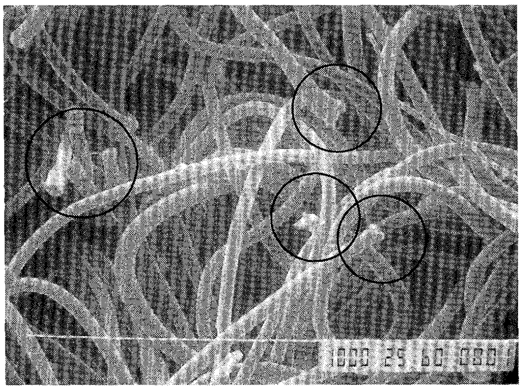
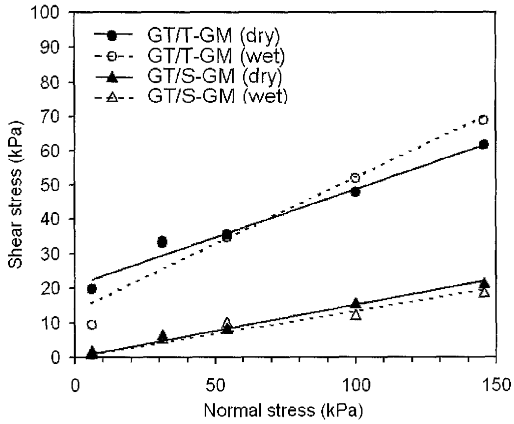
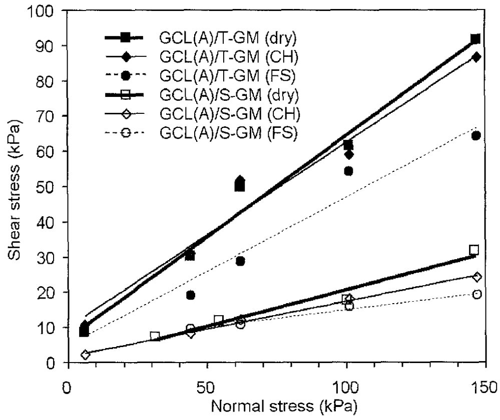


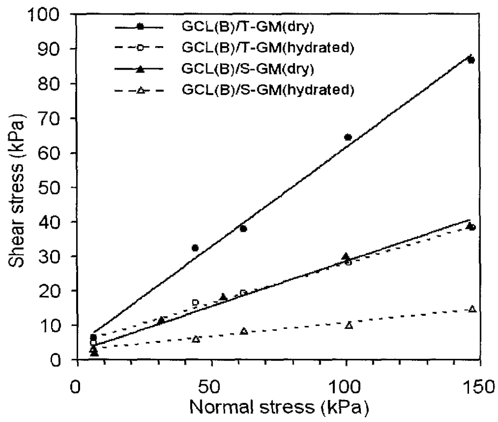
Fig. 5. Breakage of geotextile (GT) filament during shearing of GT/T-GM interface (SEM photo)



(a) GT/GM interface



(b) GCL(A)/GM interface



(c) GCL(B)/GM interface

Fig. 6. Failure envelope for peak shear strength

RESULTS

Shear Stress Behavior Versus Displacement

Figure 4 presents typical shear stress and displacement relationships for six types of interfaces (Table 2) under a normal stress of 100 kPa.

In Fig. 4, the peak strength was followed by significant strength reduction, i.e. post-peak reduction, as shear displacement proceeded for all the interfaces. The peak interface shear stress was usually mobilized at a shear displacement of about 3 mm for interfaces with smooth geomembrane (S-GM). However, the peak interface shear resistance was developed at displacements varying from 10 to 40 mm for interfaces involving T-GM, which implies that greater displacement was required to mobilize the peak shear strength compared with that of S-GM. Such differences in behavior were caused by a difference in the failure mechanism, where T-GM locks into the GT fabric and tears it with sliding. Figure 5 displays SEM (Scanning Electron Microscope) photo showing breakage of geotextile filament caused by T-GM in the GT/T-GM interface test. Namely, the GT/T-GM interface behavior was characterized not by sliding but by the textile fibers pulled by the textured surface of the geomembrane. However, for S-GM involved interfaces, sliding alone is the main failure mechanism and no tearing of geosynthetic filament was found after testing.

Each failure occurred at the geosynthetic/GM interface except for some GCL(B) interfaces. Nonlinear behaviors, especially for the interface between geosynthetic /T-GM, were observed at the region before peak strength. For GT/GM interfaces (Fig. 4(a)), peak shear strength of S-GM decreased with interface wetting, whereas that of T-GM increased at a normal stress of 100kPa. However, these changes caused by interface wetting were not identical over all the normal stresses tested.

In the case of GCL(A)/GM interfaces (Fig. 4(b)), the highest peak shear strength was observed in the dry condition for the T-GM interface and the lowest in the FS condition for the S-GM interface. It was observed that more bentonite acting as lubricating materials were extruded into the interface in FS condition than CH condition and this made the peak shear strength in FS condition show the lowest.

In view of the structure, a high plastic bentonite consists of an octahedral sheet sandwiched between two silica sheets and the bentonite has bonds which are very weak and easily separated by cleavage or adsorption of water. Due to characteristics of the bentonite having a structure easily failed by water, the bentonite extruded into the interface occurs as flakes that are so thin as to appear more like films as the bentonite gets hydrated, resulting in a sig-

Table 3. Peak and residual interface shear strength

Interface	State of interface	Peak shear strength: Friction angle (°)	Peak shear strength: Apparent cohesion (kPa)	Residual shear strength: Friction angle (°)	Residual shear strength: Apparent cohesion (kPa)
GT/S-GM	Dry	8.7	—	6.2	—
	Wet	7.6	—	5.2	—
GT/T-GM	Dry	15.7	20.7	10.3	9.4
	Wet	21.3	13.4	17.2	5.9
GCL(A)/S-GM	Dry	11.6	0.2	7.8	1.6
	FS	6.6	3.4	5.6	3.0
	CH	9.0	1.8	5.4	2.2
GCL(A)/T-GM	Dry	30.0	7.0	16.8	2.4
	FS	22.8	4.9	10.8	2.4
	CH	27.6	10.3	14.1	5.5
GCL(B)/S-GM	Dry	14.6	2.6	7.3	2.5
	Hydrated	4.6	2.7	4.4	1.7
GCL(B)/T-GM	Dry	29.7	4.4	13.1	4.3
	Hydrated	13.0	4.8	4.5	4.5

nificant decrease of shear strength (Mitchell, 1992).

The stress-displacement curves showed that the water existing at the interface and the bentonite extruded into the interface from internal GCL significantly affected the peak strength, peak strength location and stress-displacement curve shape. These results were analyzed in more detail with regard to shear strength, strength ratio and strength reduction in later section.

Finally, for the GCL(B)/GM interface (Fig. 4(c)), peak shear strength in the dry condition was greater than that in the GCL hydration condition. The strength of the T-GM interface decreased considerably with the bentonite part in GCL(B) hydration, which was attributed to the strength loss with bentonite hydration (Daniel et al., 1993). Therefore, as GCLs placed in the landfill can be hydrated easily (Bonaparte et al., 1997; Daniel et al., 1998), GCL(B) is generally not recommended as a side liner on steep side slope.

Peak and Residual (large-displacement) Shear Strength

Peak interface shear strength versus normal stress is plotted in Fig. 6. Failure envelopes were assumed to be approximately linear and characterized using the Mohr-Coulomb failure criterion.

$$\tau = c + \sigma_n \tan\phi \tag{1}$$

where c and ϕ are apparent cohesion intercept (kPa) and interface friction angle determined from linear regression, respectively. Table 3 lists the peak and residual shear strength parameters for each interface. Generally, the failure mechanism of the S-GM interface is characterized by sliding, whereas that of the T-GM interface can be characterized by combinations of sliding, smoothening of T-GM surface and tearing of the textile by the T-GM surface (Fig. 5), thereby increasing the strength.

For the case of GT-involved interfaces, the peak and residual friction angle of the GT/S-GM interface decreased by approximately 1°, as the interface became wet. However, in case of the GT/T-GM interface, the cohesion decreased with the interface wetting (Table 3) in contrast to the friction angle. From Fig. 6(a), it was seen that the shear strength of the GT/T-GM interface in the wet condition have exceeded the strength in the dry condition from certain normal stress (here about 50 kPa), which indicated that the water at the GT/T-GM interface can work as an anti-lubricant on high normal stress (≥50 kPa). One plausible alternative is that the water absorbed by the geotextile fibers causes the fibers to become more pliable and better able to grip the T-GM surface with increasing normal stress. Further explanation regarding the anti-lubricant effect is presented later in the discussion section.

In the case of interfaces involving GCL(A), the shear strength in the dry condition was comparable with that in the CH condition at the range of low stresses. However, with increasing normal stress, the shear strength in the dry condition was slightly higher than that in the CH condition, for both interfaces, S-GM and T-GM. These decreases in the CH condition stress were considered to be caused by lubrication effect of the bentonite extruded from GCL(A) into the interface because of the high normal stress applied.

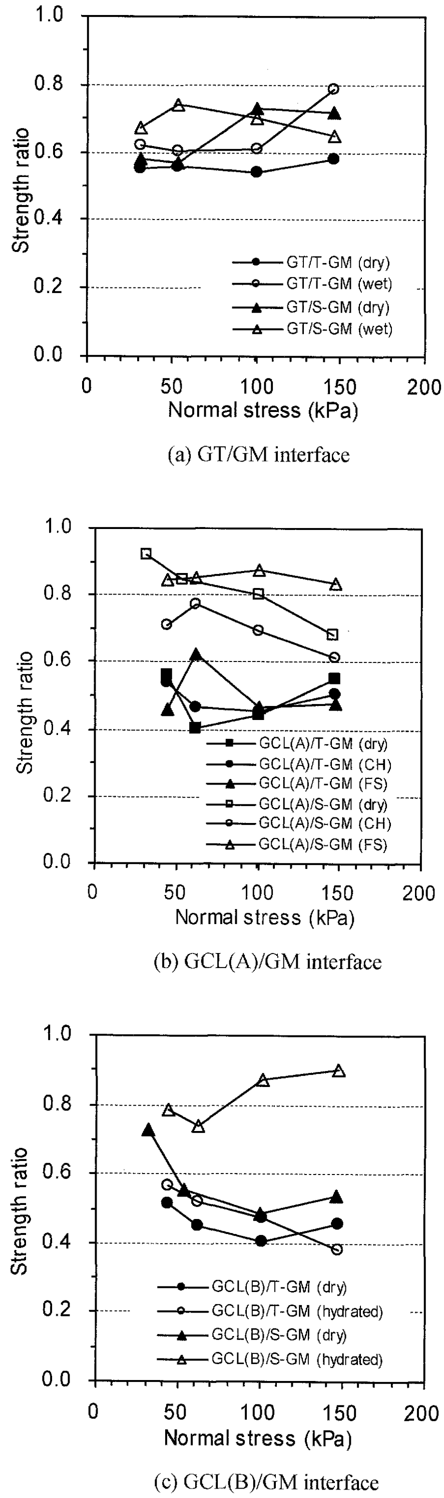
The shear strength in the FS condition was the lowest for both S-GM and T-GM interfaces over the range of normal stress tested in this experiment, due to the amount of bentonite and water extruded from GCL(A) into the interface. The extruded bentonite and water during shearing is known to significantly influence on interface shear strength. The extruded bentonite during shear can reduce the shear resistance significantly. The water content of bentonite in GCL(A) was higher in FS condition than CH condition and the higher water content in the hydrated bentonite results in higher shear reduction in FS condition. This extruded bentonite and water were getting more with increasing normal stress applied and the extruded water also formed a slight water film, which has reduced the strength.

Finally, for the GCL(B)/GM interfaces, significant reduction of peak strength was observed with bentonite of GCL(B) hydrated, as identified from the stress-displacement relationship (Fig. 6(c)). Especially, the shear strength between GCL(B)/T-GM in the hydration condition decreases to the level of GCL(B)/S-GM in the dry condition. This decrease was the result of the loss in strength of the bentonite part with hydration, which was identified by visual inspection after testing. Unlike the other cases, the failure for the GCL(B)/GM interface occurred within the GCL(B) as well as in the interface, which induced further shear strength reduction in the hydration condition. The final water content of GCL(B) was 86% and 73% for S-GM and T-GM, respectively.

Strength Reduction (post-peak strength reduction)

Post-peak strength reduction is prevalent in the interface of many waste containment systems. Most curves identified from this experiment displayed marked strength reduction from peak to residual state (Fig. 4). In order to quantify the level of strength reduction, the strength ratio, i.e. the ratio of residual strength to peak strength, was evaluated for each interface. The strength reduction with increasing displacement is known to be caused by geosynthetic polishing, geosynthetic failure and clay particle reorientation for interfaces involving soil (Gilbert and Byrne, 1996). The strength ratio versus normal stress is plotted in Fig. 7 for each interface. The average values of strength ratio are also summarized in Table 4.

Although the T-GM interface displayed more strength reduction than S-GM, any clear relationship between normal stress and strength reduction or consistent effect of moisture on strength reduction was not found. Simply, some increases of strength ratio caused by water or hydration were observed only for GCL(A)/S-GM in the FS condition and for GCL(B)/S-GM in the hydrated condition. For the GT/T-GM interface, the strength reduction at the displacement of 80 mm was also somewhat mitigated due to the water presence. The values listed in Table 4 can be referred to for the design of the geosynthetic-installed sites where large displacements are expected due to sliding on steep slopes, significant settlements in soft waste and so on. However, it should be noted that additional tests have to be performed using the installed geosynthetics when other geosynthetic is used.



(a) GT/GM interface

(b) GCL(A)/GM interface

(c) GCL(B)/GM interface

Fig. 7. Strength ratio (residual/peak strength)

Effect of Normal Stress on Secant Friction Angle

The interface shear strength parameters, c and ϕ, which are determined by the Mohr-Coulomb failure criterion, are obtained from a best-fit straight line of the shear and normal stresses acting on the interface at failure. However, it has been suggested that the failure envelope cannot be always characterized as a straight line, and that it shows a curved line for many cases, depending on the normal stress level (Wasti and Özdüzgün, 2001). This characteristic is also widely observed in soils, gravel and rock materials. It is acceptable to use a linear failure envelope to analyze the stability of the interface-involved structure. However, the c and ϕ values must be selected for the appropriate range of stress because the interface shear strength shows a dependency on normal stress (Fig. 8). The peak secant friction angle versus the normal stress, plotted in Fig. 8, shows that the friction angle is dependent on the normal stress.

Table 4. Strength ratio of peak to residual state

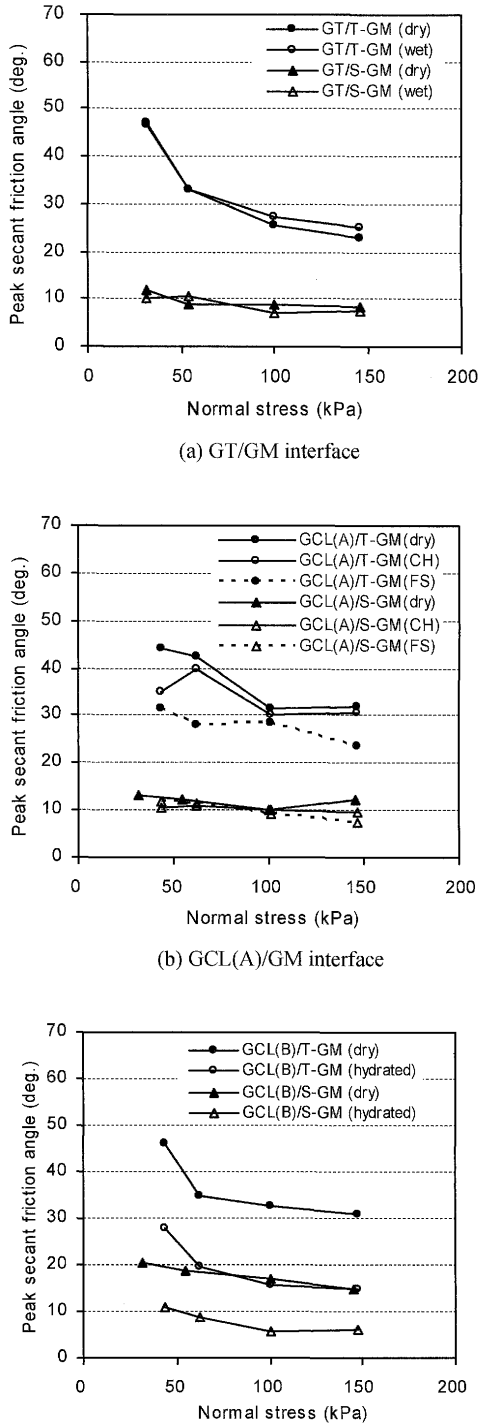
Interface	State of interface	Strength ratio (residual/peak)
GT/S-GM	Dry Wet	0.65 0.69
GT/T-GM	Dry Wet	0.56 0.65
GCL(A)/S-GM	Dry FS CH	0.82 0.85 0.70
GCL(A)/T-GM	Dry FS CH	0.52 0.53 0.49
GCL(B)/S-GM	Dry Hydrated	0.60 0.83
GCL(B)/T-GM	Dry Hydrated	0.55 0.52

In Fig. 8, a term, 'secant friction angle', is used to identify the effect of normal stress. The secant friction angle is calculated using Eq. (2).

$$\phi_{\mathrm{sec\,ant}} = \tan^{-1}(\tau/\sigma_n) \quad (2)$$

where τ and σ_n are the shear strength and normal stress, respectively. The friction angles are summarized in Table 5, together with water condition and range of the normal stress.

The plotting of peak secant friction angle versus normal stress shows that the friction angle decreases with increasing normal stress and that this effect is much more pronounced for T-GM. The secant friction angle at peak is reduced due to the water effect at all interfaces, except for GT/T-GM. The overall trends are comparable to the results of interface shear strength according to the normal stress, as shown in Fig. 6.



(a) GT/GM interface

(b) GCL(A)/GM interface

(c) GCL(B)/GM interface

Fig. 8. Peak secant friction angle with normal stresses

DISCUSSIONS

Comparison with Published Test Results

Geotextile/Geomembrane

Yegian and Lahlaf (1992) conducted static and dynamic shear tests (20.3 × 30.5 cm) to evaluate the interface shear strength between GT/S-GM in dry and submerged conditions. The friction angles corresponding to the submerged condition were consistently smaller by about 1–2° than those corresponding to the dry condition, which is consistent with the results obtained from this research. As expected, the peak dynamic friction coefficient in the submerged conditions was slightly lower by 0.6° than that in the dry conditions.

In addition to GT/S-GM, Ellithy and Gabr (2001) examined the effect of wetting on the interface shear strength for GT/T-GM. They performed direct shear tests under normal stresses ranging from 25 kPa to 500 kPa and found that the cohesion decreased while the friction angle increased for interface wetting condition. This change coincides very well with the testing results obtained from this experiment. They stated that both cohesion and friction angle were affected by wetting and that the shear strength of the interface increased by 73% at the normal stress of 250 kPa due to submergence. However, the interface shear strength could decrease on low normal stress due to the reduction of cohesion. They assumed that the function of water between geosynthetics changes from lubricant to anti-lubricant with increasing normal stress. However, more tests are required to explain the

Table 5. Range of peak secant friction angle

Interface	State of interface	Range of peak secant friction angle (°)	Range of normal stress (kPa)	Final average water content (%)
GT/S-GM	Dry Wet	8–12 7–10	31–146	—
GT/T-GM	Dry Wet	23–47 25–47		—
GCL(A)/S-GM	Dry FS CH	10–12 8–12 9–11	31–146 44–146	— 230 130
GCL(A)/T-GM	Dry FS CH	32–44 24–31 31–40		— 221 140
GCL(B)/S-GM	Dry Hydrated	15–21 6–11	31–146 44–146	— 86
GCL(B)/T-GM	Dry Hydrated	31–46 15–28		— 73

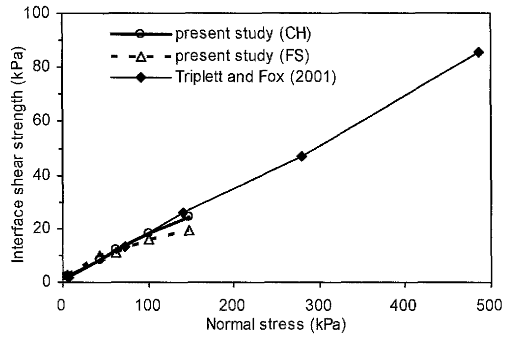
mechanism for this effect in greater detail.

Briançon et al. (2002) performed an inclined plane test, known to be more appropriate for the cases of low normal stress, to verify the influence of water on GLS (Geosynthetic Lining System) stability. They found that the decreasing level of the interface friction angle due to water presence was usually inconsistent, depending on the kinds of geotextile and geomembrane. The difference of friction angle from dry to wet condition varied from one interface to another: 1.5° for smooth PP (polypropylene) GM interface with the GT for reinforcement, 1.3° for HDPE GM interface with the GT for protection, and 4.3° for PP GM interface with the GT for protection. Girard et al. (1990) also carried out tilting table tests (1.0 × 1.0 m) to measure interface friction angle between PVC (polyvinyl chloride) GM and nonwoven GT. They observed that the water presence reduced the angle by between 2.5° for S-GM and 5.0° for T-GM. The decrease of friction angle for T-GM is consistent with the testing results of this research under low normal stress level (Fig. 6(a)).

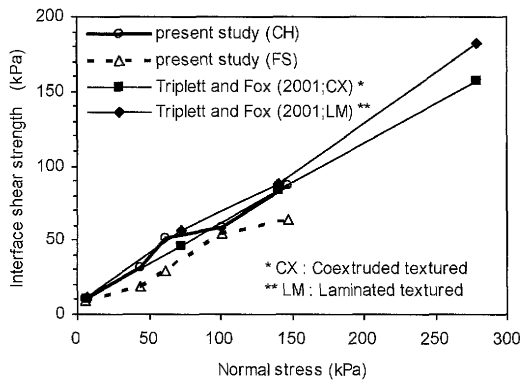
GCL(A)/Geomembrane

As the reinforced type GCL, i.e. GCL(A), can transmit more shear stress across the bentonite layer than the unreinforced type GCL, i.e. GCL(B), the former type is widely used as an alternative to CCL (Compacted Clay Liner) in landfills for higher shear strength applications. Triplett and Fox (2001) employed a different hydration method for interface shear testing, where GCL specimens were hydrated under a normal stress applied during shearing, using the four-day and two-stage procedure described by Fox et al. (1998). The results of Triplett and Fox (2001) were compared with those of the present study to examine the influence of different hydration methods on the interface shear strength characteristics. The comparison results are illustrated for S-GM and T-GM in Fig. 9.

The data shown in Fig. 9 confirm that the results in the CH condition were in good agreement with those of Triplett and Fox (2001), whereas the results in the FS condition were lower than the published data. This comparison supports that the modified hydration method applied in this experiment to shorten hydration time can give relevant test results.



(a) GCL(A)/Smooth GM(S-GM)



(b) GCL(A)/Textured GM(T-GM)

Fig. 9. Comparison with published data for the interface of GCL(A)/GM

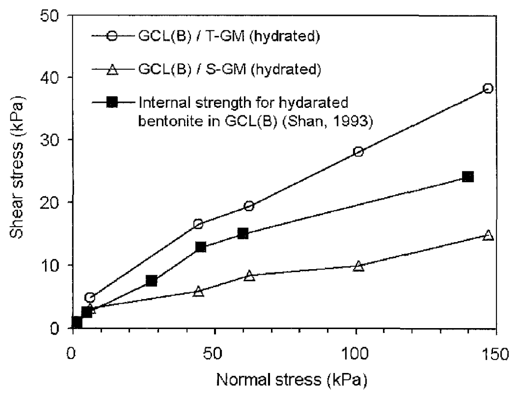


Fig. 10. Comparison of interface shear strength with internal strength of GCL(B)

GCL(B)/Geomembrane

Daniel et al. (1998) constructed 14 full-scale test plots and examined the stability of the final cover system containing various GCLs. In their research, they chose the internal shear strength of unreinforced GCL as a design parameter to evaluate the slope stability, where the bentonite portion of the geomembrane-supported GCL, i.e. GCL(B), faced downward. The internal shear strength of GCL used in their research is provided in Fig. 10, together with the interface shear strength of GCL(B) with geomembranes obtained in this experiment.

The comparison results shown in Fig. 10 demonstrated that the internal shear strength of GCL(B) is located between the interface shear strength of GCL(B)/T-GM and GCL(B)/S-GM. This figure indicated that when this type of GCL is in contact with S-GM, the interface surface shows relative high weakness of sliding failure in comparison with the internal shear strength of unreinforced GCL. Hence, this kind of combination should be avoided for final steep covers of landfills or other steep structures.

Daniel et al. (1993) used the internal shear strength, instead of GCL(B)/T-GM interface strength, to evaluate the factor of safety on failure sites where T-GM was the interface with the bentonite portion of unreinforced GCL. The conservative use of internal shear strength for the stability evaluation of GCL(B)/T-GM interface can be supported by the fact that failure occurred within the GCL(B) instead of GCL(B)/T-GM interface.

Applicability of Direct Shear Test Results

Until now, the result of direct shear tests has been widely used in design of most landfills. Daniel et al. (1998) used results of the interface direct shear test to analyze a failure case in pilot scale test plots. Furthermore, Villard et al. (1999) suggested satisfactory results in numerical analysis for evaluating stability of geosynthetic liner system by using direct shear test results. In addition, when direct shear test results are used in designing the landfill liner system or estimating the behavior of geosynthetic-involved interface, following technical suggestions related with this direct shear tests can be considered.

When external loading after geosynthetic installation is forced to geosynthetic interfaces, the weakest interface starts to slide downward. Significant displacements of the interface can be followed by severe failure condition. In Table 5, peak secant friction angle strength means the starting point on a geosynthetic interface in case the interface has the lowest shear strength among geosynthetic interfaces. In most geosynthetic interfaces considered here, strength reduction was seen due to effect of water, which can move up the starting point of sliding in a geosynthetic interface. Failure cases in the test plot (Daniel et al., 1998) gave an example of reduction in GCL shear strength due to GCL hydration, resulting in sliding failure.

The direct shear test results obtained from modified testing method were compared with those of previous research and good agreements were obtained as can be seen in previous section. Also, in this research program, testing time needed for hydration and shearing could be reduced in comparison with hydrated time in Gilbert et al. (1996). Therefore, the testing method used in this testing program can be applicable to geosynthetic interface tests on dry and wet (or hydrated) conditions.

From the interface direct shear tests, reasonable test results applicable to landfill design in various site conditions were obtained. In contrast to the former research which generally focused on a few kinds of geosynthetic interface combinations, this testing program tested comprehensive kinds of interface combinations easily found in the landfill site. Also, characteristics of interface shear strength were evaluated and compared quantitatively on dry and wet (or hydrated) conditions through detailed analysis of test results.

Generally, tests are carried out with the same geosynthetic installed in a landfill site to get a design parameter of geosynthetic-involved site. Therefore, the design parameters suggested here should not be used when different geosynthetics are installed in a landfill site. If so, shear tests should be additionally performed with the new geosynthetic. However, the design parameters presented in this paper can be reference values in design of landfill liner system because other geosynthetic interface with the same interface type shows qualitatively similar behavior.

Design Implications

Generally, a factor of safety is calculated to evaluate the overall stability of landfill sites in which various geosynthetics are installed. Then, the effect of water or water-flow on interface shear strength is also taken into account (Koerner and Hwu, 1991; Giroud et al., 1995). On calculating the safety factor of slope stability, Briancon et al. (2002) suggested from the inclined plane test results that water has different effects on friction angle, cover soil weight and friction force due to the water pressure at the interface. It was also observed that the friction angle changes considerably with respect to water and the normal stress applied.

On the other hand, peak shear strength may not be an appropriate parameter to ensure safe stability because the weak interfaces between geosynthetics show strain-softening behavior. Deformations in the waste material are sufficient in many cases to limit the available shear resistance along an interface to less than the peak strength. Based on comparisons of data at the Kettleman Hills landfill failure with FEM analysis considering progressive failure which reflects the strain-softening characteristic of geosynthetic interfaces, Filz et al. (2001) reported that the mobilized strengths were about 7% higher than the residual strengths. Gilbert and Byrne (1996) also suggested an analytical model to provide useful insight into the available shear resistance along interfaces in containment systems. They emphasized that the potential for reductions in mobilized resistance is considerably affected by waste stiffness, rate of strain-softening and length of the slip surface. Therefore, the determination between peak and residual shear strength parameters should be assessed carefully so that the design is neither made too conservatively nor too unstably.

In addition, it is particularly recommended that the tests be carried out at the anticipated normal stress level as can be identified from the changes in friction angle with normal stresses (Fig. 8; Wasti and Özdüzgün, 2001). Moreover, the water condition of the interface, hydration of GCL (Bonaparte et al., 1997) and loading condition when hydrated should also be taken into account in the design process of landfill sites, including various geosynthetics.

For the unreinforced GCL, i.e. GCL(B), the bentonite part of GCL(B) is known to be difficult to hydrate when the geomembrane is placed upon GCLs because two impermeable geomembranes surround the bentonite. Therefore, the use of interface shear strength in the dry condition will be acceptable for most landfills only if good QA/QC (Quality Assurance/Quality Control) results for the landfill site and the installed geosynthetic condition are assured. However, as reported by 14 full-scale field test plots (Daniel et al., 1998), the bentonite often gets hydrated with a resultant decrease in shear strength. Therefore, the use of value in the dry condition should be carefully considered in the design of GCL(B)-involved interface.

CONCLUSIONS

A series of large-scale direct shear tests were performed on GT/GM and GCL/GM interfaces to examine the effect of water presence or hydration on interface shear strengths. The interface shear strengths were measured between the following geosynthetics, i.e. GT, S-GM, T-GM and GCLs and the following conclusions were drawn:

(1) The conventional large direct shear tests were performed to estimate the interface shear behaviors between geosynthetics. To investigate the effect of wet condition in the interface, dry and tap-water induced wet (or hydrated for GCL) conditions were applied for all the interfaces. Especially, a modified hydration method was applied for the GCL(A) to shorten the hydration time. Then, the hydration time for two GCLs (GCL(A) and GCL(B)) was determined based on both preliminary test results and published data.

(2) Shear stress and displacement relationship curves showed that interface shear behaviors were clearly influenced by the water presence at the interface or the hydration of the bentonite portion of GCL. However, the effects of moisture on the interface shear behavior were not consistent in all interfaces, and varied depending on the type of interface and the materials involved. In addition, a significant decline in the shear stress was observed for GCL(B)/GM interface with hydration.

(3) Peak and residual shear strength failure envelopes were approximated to have a linear relationship with the normal stress. Changes of the interface shear strength with wetting or hydration were clearly identified from the linear approximations. For GT/S-GM interface, the shear strength was reduced with interface wetting. In the case of GCL(A)/GM, the strength in the FS (Free Swelling) condition was the lowest among all conditions due to the most intruded bentonite and water into the shear interface. The shear strength of the GCL(B)-involved interface decreased significantly with hydration of bentonite portion.

(4) Test results showed significant post-peak strength reduction from the peak to residual state and this decrease was the largest in the T-GM interface. The calculated strength ratio shows how much geosynthetic interface shear strength can decrease due to sliding on steep slopes or significant settlements in the waste landfill site.

(5) The plot of peak secant friction angle against normal stress shows the decrease of friction angle due to increasing normal stress. The influence of normal stress is more pronounced for T-GM. Friction angles at the peak state also decrease due to the effect of moisture for all interfaces except GT/T-GM interfaces under the high normal stress.

(6) Test results were compared with some published data for the GT/S-GM interface, with generally good consistency being found for all interfaces. The friction angle corresponding to the wet condition was consistently smaller by 1° to 2° than that corresponding to the dry condition. However, the cohesion decreased while the friction angle increased on interface wetting of the GT/T-GM interface. Therefore, the interface shear strength decreased on low normal stress due to the reduction of cohesion, which coincides very well with the published data.

(7) The shear strength of GCL(A) in CH condition, with hydration load of 6 kPa applied before shearing, showed good agreement with published data obtained through different hydration methods. The findings from the comparison support the usefulness of the modified hydration method applied in this study to shorten the hydration time. Furthermore, comparisons with published test results showed that the internal shear strength of GCL(B) was located between the interface shear strengths of GCL(B)/S-GM and GCL(B)/T-GM, which means that the GCL(B)/S-GM interface has more weakness of sliding than GCL(B) itself.

(8) For the design of GCL(B)-installed sites, the value of interface shear strength in the dry condition will be acceptable for landfill sites only if good QA/QC results are assured. However, as reported by 14 full-scale field test plots, it should be noted that the bentonite often gets hydrated, resulting in a decrease in shear strength. Therefore, the use of value in the dry condition should be careful in design of GCL(B)-involved interface.

ACKNOWLEDGMENT

This paper was funded by the Korea Institute of Construction and Transportation Technology Evaluation and Planning under the Ministry of Construction and Transportation in Korea (Grant No. 04-C01).

REFERENCES

1) ASTM (1998): Standard test method for determining the coefficient of soil and geosynthetic or geosynthetic and geosynthetic friction by the direct shear method, *ASTM D 5321-92 (reapproved 1998)*, ASTM, West Conshohocken.
2) ASTM (1999): Standard test method for determining the internal and interface shear resistance of geosynthetic clay liner by the direct shear method, *ASTM D 6243*, ASTM, West Conshohocken.
3) Bonaparte, R., Othman, M. A., Rad, N. S., Swan, R. H. and Vander Linde, D. L. (1997): Evaluation of various aspects of GCL performance, *1995 Workshop on Geosynthetic Clay Liners*, Rep. No. EPA/600/R-96/149, U.S. EPA, Washington, D.C., F1-F34.
4) Bouazza, A. (2002): Geosynthetic clay liners, *Geotextiles and Geomembranes*, **20**, 3-17.
5) Briançon, L., Girard, H. and Poulain, D. (2002): Slope stability of lining systems-experimental modeling of friction at geosynthetic interfaces, *Geotextiles and Geomembranes*, **20**, 147-172.
6) Daniel, D. E., Shan, H. Y. and Anderson, J. D. (1993): Effects of partial wetting on the performance of the bentonite component of a geosynthetic clay liner, *Proc. Geosynthetics '93*, IFA, St. Paul, Minnesota, USA, 1483-1496.
7) Daniel, D. E., Koerner, R. M., Bonaparte, R., Landreth, R. E., Carson, D. A. and Scranton, H. B. (1998): Slope stability of geosynthetic clay liner test plots, *J. of Geotech. and Geoenvir. Engrg.*, ASCE, **124**(7), 628-637.
8) Eid, H. T., Stark, T. D. and Doerfler, C. K. (1999): Effect of shear displacement rate on internal shear strength of a reinforced geosynthetic clay liner, *Geosynthetics International*, **6**(3), 219-239.
9) Ellithy, G. and Gabr, M. A. (2001): A model for interface shear strength of nonwoven geotextile, *Proc. Geosynthetics Conference*, USA, 121-133.
10) Filz, G. M., Esterhuizen, J. J. B. and Duncan, J. M. (2001): Progressive failure of lined waste impoundments, *J. of Geotech. and Geoenvir. Engrg.*, ASCE, **127**(10), 841-848.
11) Fox, P. J., Rowland, M. G., Scheithe, J. R., Davis, K. L., Supple, M. R. and Crow, C. C. (1997): Design and evaluation of a large direct shear machine for geosynthetic clay liners, *Geotech. Testing J.*, **20**(3), 279-288.
12) Fox, P. J., Rowland, M. G. and Scheithe, J. R. (1998): Internal shear strength of three geosynthetic clay liners, *J. of Geotech. and Geoenvir. Engrg.*, ASCE, **124**(10), 933-944.
13) Gabr, M. A. and Valero, S. N. (1995): Geotechnical properties of municipal solid waste, *Geotech. Testing J.*, **18**(2), 241-251.
14) Gilbert, R. B. and Byrne, R. J. (1996): Strain-softening behavior of waste containment system interfaces, *Geosynthetics International*, **3**(2), 181-203.
15) Gilbert, R. B., Fernandez, F. and Horsfield, D. W. (1996): Shear strength of reinforced geosynthetic clay liner, *J. of Geotech. Engrg.*, ASCE, **122**(4), 259-266.
16) Girard, H., Fisher, S. and Alonso, E. (1990): Problems of friction posed by the use of geomembranes on dam slopes-examples and measurements, *Geotextiles and Geomembranes*, **9**, 129-143.
17) Giroud, J. P., Bachus, R. C. and Bonaparte, R. (1995): Influence of water flow on the stability of geosynthetic-soil layered systems on slopes, *Geosynthetics International*, **2**(6), 1149-1180.
18) Goodhue, M. J., Edil, T. B. and Benson, C. H. (2001): Interaction of foundry sands with geosynthetics, *J. of Geotech. and Geoenvir. Engrg.*, ASCE, **127**(4), 353-362.
19) Jones, D. R. V. and Dixon, N. (1998): Shear strength properties of geomembrane/geotextile interfaces, *Geotextiles and Geomembranes*, **16**, 45-71.
20) Kavazanjian, E. (2001): Mechanical properties of municipal solid waste, *Proc. Sardinia 2001, 8th International Waste Management and Landfill Symposium*, Cagliari, Italy, 1-5 October 2001, 415-424.
21) Koerner, R. M. and Hwu, B. L. (1991): Stability and tension considerations regarding cover soils on geomembrane lined slope, *Geotextiles and Geomembranes*, **10**, 335-355.
22) Koerner, R. M. (1998): *Designing with Geosynthetics*, 4th Ed., Prentice Hall, Upper Saddle River, New Jersey.
23) Koutsourais, M. M., Sprague, C. J. and Pucetas, R. C. (1991): Interfacial friction study of cap and liner components for landfill design, *Geotextiles and Geomembranes*, **10**(6), 149-166.
24) Martin, J. P., Koerner, R. M. and Whitty, J. E. (1984): Experimental friction evaluation of slippage between geomembranes, geotextiles, and soils, *International Conference on Geomembranes*, Denver, IFAI, St. Paul, Minnesota, 191-196.
25) Mitchell, J. K. (1975): *Fundamentals of Soil Behavior*, 2nd Ed. John Wiley & Sons, Inc., 31.
26) Mitchell, J. K., Seed, R. B. and Seed, H. B. (1990): Kettleman Hills waste landfill slope failure. I: Liner-system properties, *J. of Geotech. Engrg.*, ASCE, **116**(4), 647-668.
27) Seed, R. B., Mitchell, J. K. and Seed, H. B. (1990): Kettleman Hills waste landfill slope failure. II: Stability analyses, *J. of Geotech. Engrg.*, ASCE, **116**(4), 669-690.
28) Shan, H. Y. (1993): Stability of final covers placed on slopes containing geosynthetic clay liners, *Ph.D. Dissertation*, University of Texas at Austin, Texas, 296.
29) Stark, T. D., Williamson, T. A. and Eid, H. T. (1996): HDPE geomembrane/geotextile interface shear strength, *J. of Geotech. Engrg.*, ASCE, **122**(3), 197-203.
30) Triplett, E. J. and Fox, P. J. (2001): Shear strength of HDPE geomembrane/geosynthetic clay liner interfaces, *J. of Geotech. and Geoenvir. Engrg.*, ASCE, **127**(6), 543-552.
31) Villard, P., Gourc, J. P. and Feki, N. (1999): Analysis of geosynthetic lining system (GLS) undergoing large deformations, *Geotextiles and Geomembranes*, **17**, 17-32.
32) Wasti, Y. and Özdüzgün, Z. N. (2001): Geomembrane-geotextile interface shear properties as determined by inclined board and direct shear box tests, *Geotextiles and Geomembranes*, **19**, 45-57.
33) Yegian, M. K. and Lahlaf, A. M. (1992): Dynamic interface shear strength properties of geomembranes and geotextiles, *J. of Geotech. Engrg.*, ASCE, **118**(5), 760-779.