Compacted soil shear strength characterization on inclined plane test

Caractérisation de la résistance au cisaillement d'un sol compacté avec l'essai au plan incliné

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ABSTRACT

Many landfill cover system slidings have occurred and small shear strength in the interface between cover components has been evoked as the main reason of the observed failures. However, the poor compaction characteristics and low associated strength of some cover soils suggest that in some instances their can influence the cover sliding instead of interface between components. In this paper, the inclined plane test is used to determine geosynthetics interfaces and compacted soil shear strength in cases where the initial normal stress is small. It will be shown that the poorly compacted cover soil can determine the potential failure surface under some conditions and that an appropriate compaction can increase the angle of shearing resistence and to ensure the soil stability in the cover system.

RÉSUMÉ

Des glissements ont été vus aux couches de couverture des centres de stockage de déchets et la faible résistance au cisaillement au niveau des interfaces des différents matériaux en contact est souvent identifiée comme la cause principale des ruptures observées. Si on considère que le compactage du sol de couverture sur site n'est en général pas optimum, surtout dans les pentes des centres de stockage des déchets, il est possible que le glissement d'une couche de couverture ait lieu dans le sol compacté, et non sur les interfaces. Dans cet article, l'essai plan incliné est utilisé pour déterminer la résistance au cisaillement des interfaces sol compacté-géosynthétique et du sol compacté sous de faibles contraintes de confinement et sous de différents degrés de compactage. Il sera montré que la faible condition de compactage du sol peut déterminer la surface potentielle de rupture sous quelques conditions particulières. Un processus de compactage approprié peut augmanter la résistance au cisaillement du sol et assurer son stabilité dans la couche de couverture.

Keywords: landfill cover system, compacted soil, interface shear strength, inclined plane test

1 INTRODUCTION

The design goal for landfills is to maximize containment space and minimize land area, and this demands relatively steep slopes. For the cover soil stability analysis, this situation is exacerbated and it is obviously matter of concern. Many landfill cover system slidings have occurred and small shear strength in the interface between cover components, such as geosynthetic-geosynthetic or geosynthetic–soil interfaces, has been evoked as the main reason of the observed failures.

The materials of concern are in general the geomembrane-soil, geosynthetic clay liner-soil and compacted clay liner-soil interfaces. However, in some instances, the soil cover slope is poorly compacted resulting in low shear strength. This suggests that under some conditions the low shear strength of the soil influence the potential failure surface instead of the interfaces between components.

The direct shear apparatus used for soil testing is often employed to measure interface shear resistance; however, the test is not easy to be carried out at the low normal stress levels applicable to field conditions such as side slopes in landfill covers.

The inclined plane test (IPT) is an option to measure either soil-geosynthetic or geosynthetic-geosynthetic interface shear strength, especially in cases where the normal stress on the interface is small. In this paper, the IPT is used to study the shear strength behavior of compacted soil-geosynyhetic interfaces and compacted soil (soil-soil interfaces) under different compaction conditions.

2 METHODS

The general layout of IPT is shown in Figure 1. Under standard test conditions (see Fig. 1a), a geosynthetic layer is installed bonded to the base plane (0.80 m x 1.30 m). An upper box filled with soil is positioned over the geosynthetic and the plane is inclined at a constant speed ($d\beta/dt = 3°/min$).

The measurement of the relative box displacement allows to calculate the interface shear strength. The apparatus can also be used for tests on soil-soil interfaces. For soil-soil tests the rigid support is replaced by a tank filled with soil (see Fig. 1b).

The initial normal stress is σ_o. It's induced by the weight of the soil filling the upper box. The thickness of soil is limited to 50 mm and the complementary weight required to reach the desired value of σ_o is obtained by adding dead loads over the soil layer. The box is fitted with front and rear side walls inclined (inclination θ) with respect to a line perpendicular to the plane in order to limit uneven shear stress along the interface tested (Lalarakotoson, 1998).

The following parameters can be assessed during testing (see Fig. 2):

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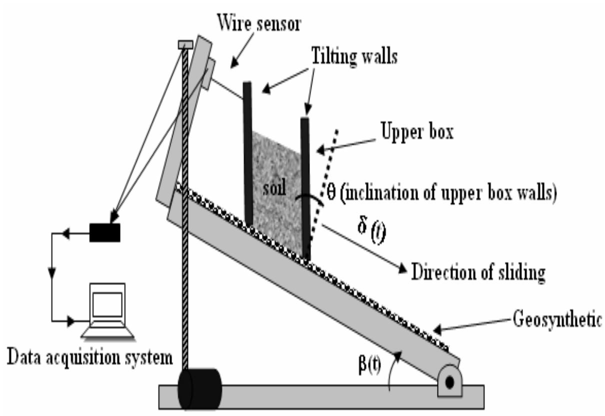
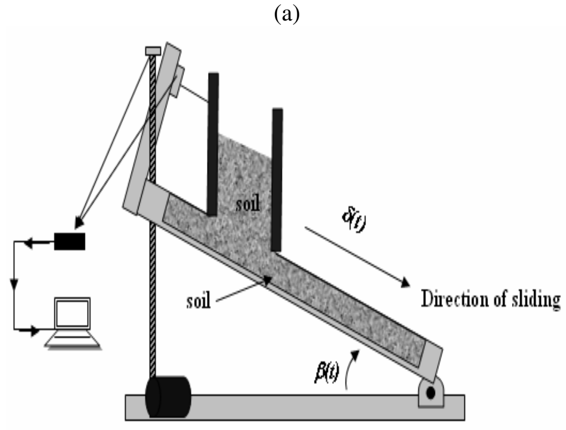
- ✓ β_0: plane inclination corresponding to the beginning of the upper box movement;
- ✓ β_{50}: plane inclination corresponding to a standard displacement δ = 50mm;
- ✓ β_{lim}: plane inclination corresponding to the non-stabilized sliding.



(a)



(b)

Figure 1. IPT for tests on soil-geosynthetic (a) and soil-soil (b) interfaces.

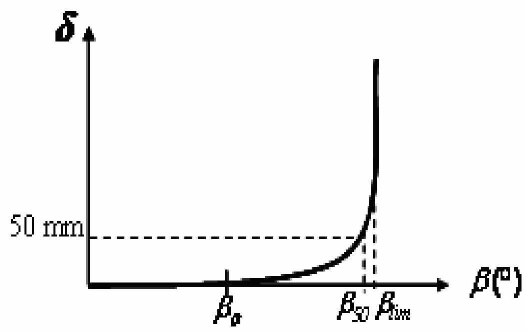


Figure 2. Parameters assessed during testing in the IPT.

According to the Standard EN ISO 12957-2 (2005), the friction angle is conventionally determined for an inclination β_{50} corresponding to a sliding displacement δ= 50mm, assuming static equilibrium even if dynamic conditions are generally the conditions for this displacement value. This gives rise to ϕ_{50}^{stat}, the standard static friction angle, (Briançon, 2002; Briançon et al., 2002; Lalarakotoson et al., 1999; Purwanto, 1996).

According to Equation 1 (Gourc and Reyes-Ramirez 2004):

$$\tan\phi_{50}^{stat} = \frac{(m_b + m_s)\cdot g \sin\beta_{50} - T_{guide}}{m_s \cdot g \cdot \cos\beta_{50}} \quad (1)$$

where m_b corresponds to the upper box mass, m_s to the soil-metallic plates mass in the upper box, g is the acceleration of gravity and T_{guide} is the parasitic tangential friction resistance of the guidance system of the upper box.

3 MATERIALS

The present paper considers the interface between geosynthetics materials and compacted soil. The geosynthetics comprised geotextile (GT needlepunched) and smooth geomembrane (GM hdpe). Table 1 shows some characteristics of the geosynthetics used in the tests.

Table 1. Main characteristics of the geosynthetics used.

Type of Geosynthetic	Material	Thickness (mm)
Geomembrane	High Density Polyethylene (hdpe)	1.5
Geotextile	Nonwoven needlepunched geotextile	1

The soil is silty sand whose optimum conditions from Standard Proctor test are w_{opt}= 7.3% and $\gamma_{d_{\max}}$ = 16.2 kN/m^3. This soil could be considered as a soil representative of the material used as veneer layer for the cap cover of landfill even if different soils are employed in this kind of application.

The soil is placed within upper box and compacted at unit weight γ = 14.2 kN/m^3 and w = 6.5%, in accordance with the usual poor compaction characteristics of cover slopes in the field (in this case, Compaction Degree CD = 82%). A somewhat higher compaction degree (γ = 15.1 kN/m^3, w=6.5%, Compaction Degree CD = 87%) was used to evaluate the soil improvement and demonstrates the difference in strength behavior between two soil samples at different densities.

The value of the weight of the upper box is m_b.g = 282.24 N. The values of the weights of the material (soil and metallic plates) filling the upper box are m_s.g = 352.80 N, 743.40 N and 1310.40 N for σ_o = 2.8 kPa, 5.9 kPa and 10.4 kPa, respectively. The result of calibrating the apparatus (tests on an empty box with different surchages) for the static condition was that the sliding resistance offered by the rail guidance system is independent of the normal load. For the static condition, T_{guide} = 5.4 N.

4 RESULTS AND DISCUSSION

The typical displacement δ versus inclination β relationships for each test is shown in Figure 3. The tests were repeated on two or three different samples for each value of the initial normal stress σ_0. For the compacted soil at CD = 87%, the higher applied normal stress was σ_0 = 9.4 kPa. The observed sliding phenomenon is usually of the "gradual sliding" type, except for the geomembrane that presents a typical behavior of "sudden sliding".

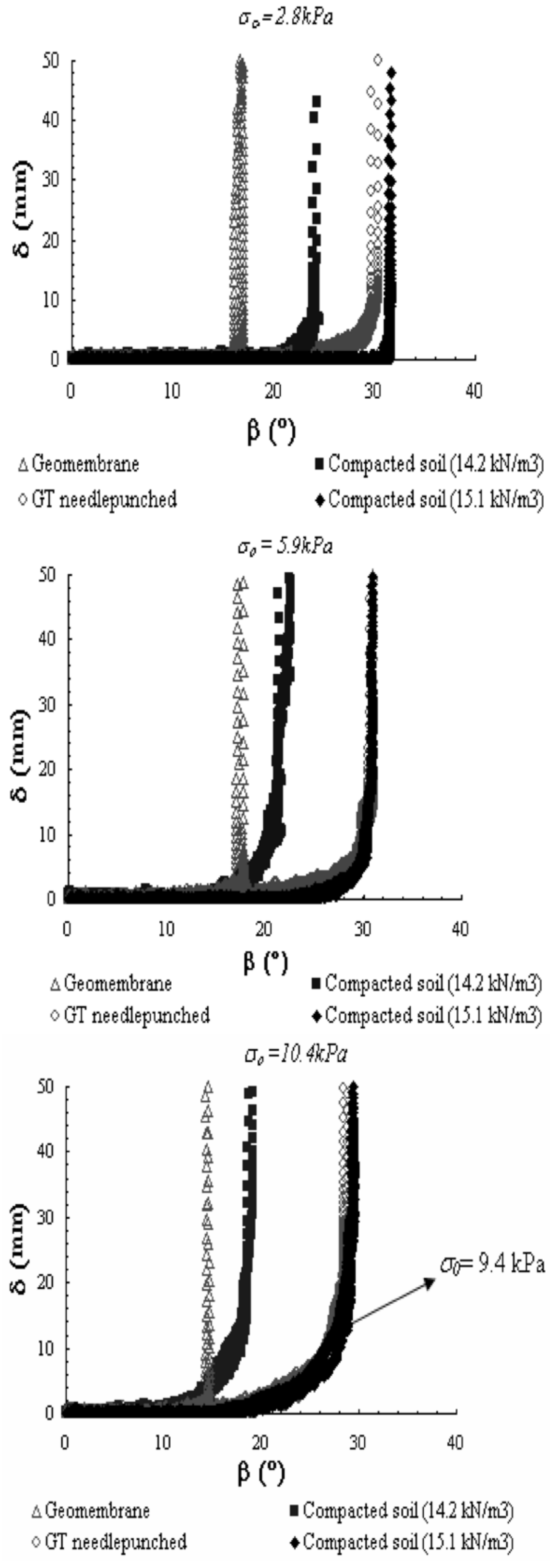


Figure 3. Displacement δ versus inclination β relationships.

Sudden sliding behavior corresponds to the condition of abrupt displacement (δ) of the upper box under non-stabilized sliding with an almost negligible transient phase ($\beta_o = \beta_{lim}$). Under this sliding condition, there is a sudden reduction of the interface friction angle after $\beta = \beta_o$.

Gradual sliding behavior corresponds to the condition of displacement (δ) progressively increasing with inclination (β). In this case, after β_o has been reached, a transient phase arises and the interface friction angle increases ($\beta_o<\beta<\beta_{lim}$) until non-stabilized sliding takes place.

Figure 4 presents the values of ϕ_{50}^{stat} for each test, using the specific interpretation of the test presented in the Equation 1. The friction angles tend to decrease significantly with normal stress. The special tests performed to elucidate the shear strength of soil in the inclined plane tests have shown that the standard static friction angles, ϕ_{50}^{stat}, for soil at a loose density and under low normal stress, were lower than that of geotextile-soil interface tested.

This suggests that in the case of a soil cover sliding in the presence of a geotextile interface, the sliding would take place within the soil layer and not at the interface (see Fig. 5a). However, if a geomembrane interface is considered, the sliding should occur in the interface as the friction is smaller than the shear strength of soil (see Fig. 5b).

Figure 5c shows observed failure surface, where one can appreciate the complex behavior of the compacted soil at γ =14.2 kN/m^3. In general, for the normal stress range tested, the sliding is occurring into the superficial part of soil (a thickness around 20 mm).

It is possible to observe on the picture the indented surface of soil (saw teeth) after sliding of the upper box, feature that should characterize the soil slip surface in this type of test. The observed sliding behavior is different of the sliding occurring at the compacted soil–geosynthetic interfaces since generally the shear zone enters in the layer of the soil support and is not limited to the interface.

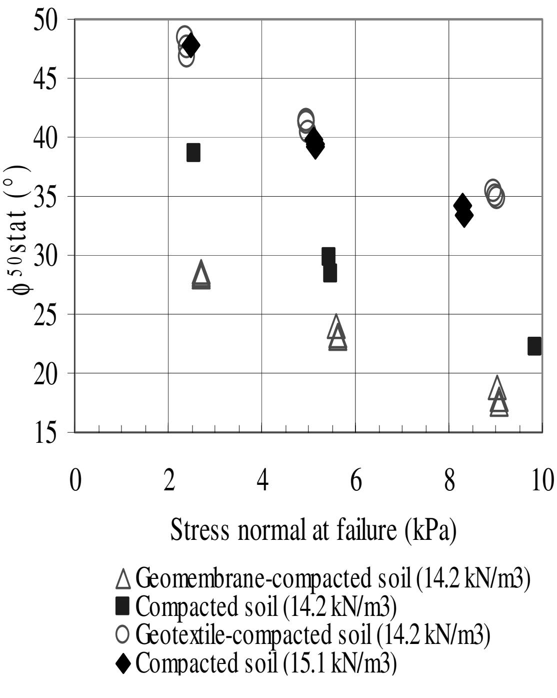
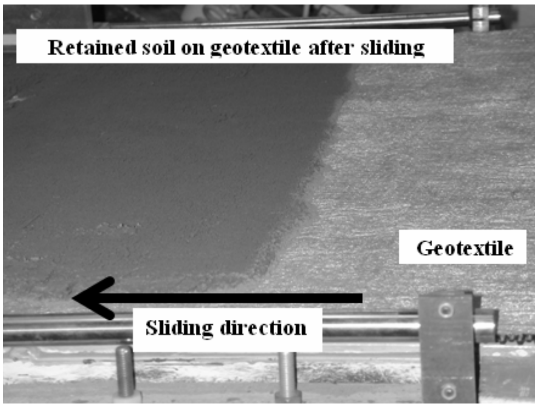
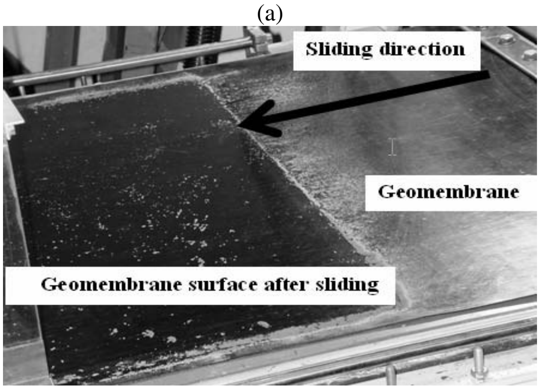


Figure 4. Values of ϕ_{50}^{stat}.



(a)



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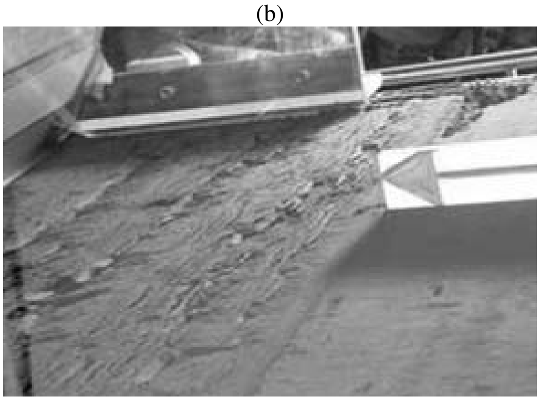

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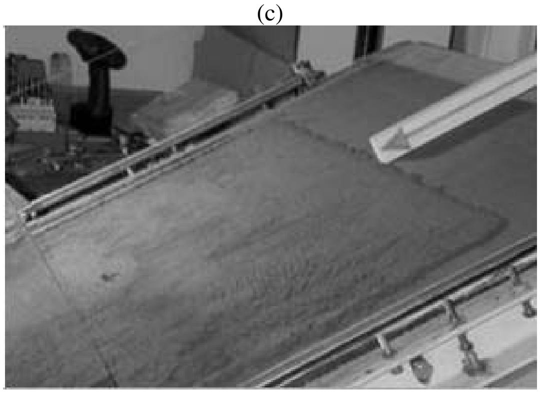

(d)

Figure 5. Observed failure surfaces: (a) geotextile - compacted soil (CD=82%); (b) geomembrane - compacted soil (CD=82%); (c) compacted soil (CD=82%); (d) compacted soil (CD=87%).

The increasing of the Compaction Degree is able to increase the friction angle of the soil, as illustrated in Figure 4. Friction angles of the same order of magnitude of the geotextile-compacted soil at 14.2 kN/m^3 can be attained for the soil compacted at 15.1 kN/m^3.

The rupture surface of the soil compacted to 15.1 kN/m^3 is different from the soil compacted at low density as can be appreciated in Figure 5d. This emphasizes the importance of an adequate compaction in landfills cover.

5 CONCLUSIONS

Intuitively, common sense suggests that the surface of geosynthetic in contact with the veneer soil layer constitutes a poor frictional interface and manufacturers are looking for a way to improve the roughness of the geotextile in contact with the soil cover.

In this study, it was possible to distinguish the friction behavior of different compacted soil – geosynthetic interfaces and compacted soil under low values of normal stress using the inclined plane test.

It was shown that the compacted poorly cover soil can determine the potential failure surface under some conditions. However, if a geomembrane interface is considered, the sliding should occur in the interface as the friction is smaller than the shear strength of soil.

It was shown that an appropriate compaction can increase the angle of shearing resistence and to ensure the soil stability in the cover system. These results must be seen with caution and confirmed by additional tests and analysis as is still doubtful whether testing such a soil rupture duplicates field behavior.

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