Manuscript title: Sliding stability analysis of a retaining wall constructed by soilbags

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Abstract

Model tests were conducted to analyse the sliding stability of a retaining wall constructed by soilbags. The aim was to obtain an equation calculating the active resultant earth pressure of sand acting on the wall in the ultimate state. Additionally, shear tests on multi-layers of vertically-stacked soilbags were designed to investigate how the interlayer friction resistance varied with the height of the wall. The results show that the active earth pressure acting on the soilbag-constructed retaining wall in the ultimate state is non-linear, but it can be calculated from force equilibrium of a differential element. The interlayer friction resistance of soilbags is found to be related to the shape of the sliding surface. Based on the obtained equation and the unique shear tests results, the sliding stability of the retaining wall constructed by soilbags could be appropriately analysed.

Keywords: Retaining walls; Soilbag; Sands; Sliding stability; Earth pressure; Friction resistance

List of notations

dy	is the thickness of differential flat element
dW	is the weight of the differential element
H_{crit}	is the height of the wall above the slip surface
K	is the active lateral pressure coefficient
p_x	is the horizontal reaction on the wall
p_y	is the vertical reaction on the wall

31	q	is the uniformly distributed stress on wall top surface
32	r	is the normal reaction of the soil at rest
33	y	is the depth from surface of backfill
34	δ	is the frictional angle between the back of the wall and the backfill
35	γ	is the unit weight of backfill
36	θ	is the angle of failure line to horizontal $\theta=\arctan(\tan\phi+\sqrt{\tan^2\phi+\tan\phi/\tan(\phi+\delta)})$
37	ϕ	is the internal friction angle of the backfill
38	τ_1	is the shear between the backfill and the back of the retaining wall
39	τ_2	is the shear between the sliding backfill and the remaining backfill at rest

1 Introduction

Soilbags or more exactly geotextile bags filled with soils or soil-like materials are commonly used to build embankments during floods, and to construct temporary structures after disasters (Kim et al., 2004). Early research was concentrated in investigating the mechanical behaviour of individual soilbags. Matsuoka and Liu (2003) found that soilbags have a very high compressive strength from experimental and theoretical studies. The high compressive strength of soilbags can be theoretically explained by the increased apparent cohesion that develops due to the tensile force of the wrapped bag under external loading; this theory was further verified by numerous researchers (Tantono and Bauer, 2008, Xu et al., 2008, Cheng et al., 2016, Liu et al., 2018). Ansari et al. (2011) numerically analysed the mechanical behaviour of a soilbag subject to compression and lateral cyclic shear loading; they reported that the stiffness and compressive load capacity of a soilbag are considerably higher than those of an unwrapped granular material. Since then soilbags have been widely used to reinforce foundations (Liu et al., 2014, Ding et al., 2017, Ding et al., 2018), and to construct retaining walls (Portelinha et al., 2014, Wang et al., 2015, Liu et al., 2019), slopes (Huang et al., 2008, Liu et al., 2012, Liu et al., 2015, Wen et al., 2016) and small dams (Aqil et al., 2006, Li et al., 2017). Soilbags can be filled at site using the in-situ soil (e.g. a 5m high wall reported by Liu at al. (2019)) or prepared remotely in advance and transported to the site (e.g. the more than a 20m high slope of the south-to-North Water transfer project in China reported by Liu at al., 2015).

Retaining walls constructed by soilbags generally have the advantages of low cost, light weight, good adaptation to foundation deformation, and good seismic performance similar to geosynthetic-reinforced earth retaining wall (Matsuoka and Liu, 2014). At present the soils that have been used as filling material in soilbags include natural river sand (Liu at al., 2016; Matsushima at al., 2008), clayey soils (Liu at al., 2019), small-size stones, expansive soil (Liu at al., 2015; Wang at al., 2015), loam soils (Liu at al., 2016), and dry ash (Li, 2017), etc. The common features of these soils are that the particle sizes are relatively small such that they can be filled into the bags easily, and do not have obvious sharp edges or corners so they cannot easily cut through the bags. These fill materials for soilbags were found to not significantly affect the overall performance (Matsuoka and Liu, 2014). Due to these advantages, soilbags have been widely used in many projects with retaining walls (Liu, 2017). However, when compared to concrete gravity retaining walls, the retaining walls constructed by soilbags are thicker. Moreover, protective measures such as thick concrete facing or masonry facing should be considered to prevent bags from being directly exposed to ultraviolet radiation. Additionally, there is still no appropriately documented design guideline. Matsushima et al. (2008) showcased many examples of soilbag-constructed retaining walls failure, and found that one of the major drawbacks of this type of wall is the relatively low stability caused by slippage along the horizontal interface in between the adjacent soilbags, which results in a catastrophic failure. Hence, sliding stability should be the most important issue in the design of a retaining wall constructed with soilbags. It was also stated in the reference (Matsushima at al., 2008) that the shear strength of multi-layered soilbags is highly anisotropic when they are stacked horizontally and inclined, but only the sliding stability of the retaining wall constructed with horizontally-stacked soilbags, usually used in practical engineering, was studied in this paper.

To analyse the sliding stability of these walls, this paper presents model tests of soilbag-constructed retaining wall and simple shear tests on five-layers of vertically-stacked soilbags. In the model test, the displacement, sliding surface and lateral earth pressure of the wall were monitored. An equation for calculating the active earth pressure on the retaining wall of soilbags in the ultimate state was derived from the force equilibrium of a differential element. The interlayer friction resistance of soilbags was then obtained from the shear tests to analyse sliding stability.

2 Active earth pressure at failure

2.1 Model test

The model tests were performed in a cuboid box with a length of 180cm, width of 80cm and height of 140cm, as shown in Fig.1. Two sheets of 2cm thick glass, which were rigid enough against deformation, were placed on the inner faces of the box to reduce side friction and for observation. A soilbag-constructed wall with a height of 125cm was set up in the box. Soilbags of two sizes (20cm×20cm×5cm and 20cm×10cm×5cm) were staggered as shown in Fig. 1 to construct the model wall. Behind the wall a dry river sand (see Table 1 for its physical and mechanical properties) was placed in layers and compacted by tamping to a desired relative density of 70% (ρ=1.76g/cm^3) using a hand operated vibrator. On the top surface of the backfill vertical uniform loads were applied to a loading plate with a size of 70cm×60cm using an oil jack. The width of the loading plate is smaller than that of the sand box as it is easier to put the loading plate into the box, and it prevents friction between the loading plate and the box. The applied vertical load was increased until a sliding surface appeared in the wall.

To evaluate the behaviour of the model testing retaining wall, a number of monitoring instruments were installed as shown in Fig.1. Twelve earth pressure cells were buried during the construction of the wall to measure the lateral earth pressures on the backfill soil and in the soilbags. Five flexible displacement meters were installed to measure the displacement of the wall face. A number of marker lines were drawn on both the inside and outside of the glass. The inside lines moved with the movement of the backfill, while the outside lines remained stationary. The displacement of the backfills could be obtained by measuring the relative displacement of corresponding marker lines both inside and outside. A camera was positioned in front of model test to monitor the movement of markers at regular intervals.

The soilbags used in the model tests were filled with natural the river sand (see Table 1) as backfill. The woven bags were made of polypropylene with a weight of 150g per-square meters. The tensile strengths of the bags are 37.1kN/m and 28.0kN/m in warp and weft directions, respectively. The warp and weft elongations are both less than 25%

at failure and the friction coefficient of the bags is 0.54.

2.2 Test results

In this model test the retaining wall failed when the vertical load applied on the loading plate reached 8.7kPa, and a slip surface appeared in the backfill soil. The angle between the slip surface and the horizontal line is about 60°, which is close to the value of the coulomb sliding friction angle (θ=59°). As shown in Fig.2 a ladder-like sliding surface appeared in the soilbag-constructed retaining wall, which runs through three layers of soilbags. The top of the ladder-like surface appeared to be connected to the bottom of the slip surface. The wall above the ladder-like surface undergoes a rigid body translation, and the height H_{crit} of it is 0.95m.

Fig. 3 shows the stacked soilbags which are arranged in a staggered manner. Due to the flexibility of soilbags, the soilbag in the upper layer can deform into the gaps between soilbags in the lower layer, with embedded contacts when subjected to vertical load; this is defined as the interlayer insertion in this paper. We believe that the formation of this ladder-like sliding surface is a result of the interlayer insertion of soilbags. Fig.4 shows the experimental distribution of earth pressure (measured) under ultimate load. It can be seen that the earth pressure acting on the wall is non-linear, which is different from the linear prediction under the assumption of Coulomb theory.

2.3 Active resultant earth pressure calculation

A correct estimation of the magnitude and distribution of the active earth pressure acting on retaining structures is important for safety, economical design and construction. Coulomb's theory assumes a linear distribution of the active earth pressure and has been widely used for that purpose. However, many experimental and field data (Tsagareli, 1965, Sherif et al., 1984, O'Neal and Hagerty, 2011, Khosravi et al., 2013, Vo et al., 2016) showed that the distributions of the active earth pressure behind a wall is nonlinear, indicating that the Coulomb's theory is not appropriate. Many investigations have been conducted to study the non-linear active earth pressures associating the mode of wall movement to the force equilibrium of a differential element, and preferable results have been achieved (Wang, 2000, Paik and Salgado, 2003, Goel and Patra, 2008). Following this approach in this model test, the non-linear active earth pressures acting on the soilbag-constructed retaining wall is calculated by a differential element method.

Based on the model test results, it is first assumed that the earth pressure against the back of the wall is due to the thrust exerted by the sliding wedge when the wall moves forward. Taking the sliding wedge as an isolated unit, as shown in Fig.5(a), a differential flat element of thickness, dy is taken from the wedge at a depth, y below the ground surface. From Fig.5(b), the forces acting on this element include the vertical pressure, p_y on the top of the element, the vertical reaction, $p_y + dp_y$ on the bottom of the element, the horizontal reaction, p_x of the retaining wall, the shear, τ_1 between the backfill and the back of the retaining wall, the normal reaction, r of the soil at rest, the shear, τ_2

between the sliding backfill and the remaining backfill at rest, and the weight, dW of the element.

By analysing the stress of the differential element, (Wang, 2000) derived the following expression of the horizontal unit earth pressure:

$$p_x = K\left[(q - \frac{\gamma H_{\text{crit}}}{\alpha K - 2})(\frac{H_{crit} - y}{H_{crit}})^{\alpha K - 1} + \frac{\gamma}{\alpha K - 2} H_{crit} - y \right]$$

1.

where q is the uniformly distributed stress on wall top surface and q is obtained by dividing the force loaded in the loading plate by the corresponding area of the backfill (80cm×60cm). K is the active lateral pressure coefficient, $K = \frac{p_x}{p_y}$. $\alpha = \frac{\cos(\theta - \phi - \delta)}{\sin(\theta - \phi)} \frac{\tan\theta}{\cos\delta}$, in which ϕ is internal friction angle of backfill, and δ is frictional angle between the back of the wall and the backfill. The detailed derivation is shown in the appendix.

The resultant earth pressure was given by

$$P_x = \int_0^h p_x dy = KqH_{\text{crit}} + \frac{1}{2\alpha}\gamma H_{crit}{}^2$$

2.

However, Wang (2000) did not give an expression for the active lateral pressure coefficient. Pick (2003) proposed an equation to calculate the active lateral pressure coefficient under the assumptions that the trajectory of the minor principal stress takes the shape of a circular arc, giving:

$$K = \frac{3(m\cos^2\omega + \sin^2\omega)}{3m - (m-1)\cos^2\omega}$$

3.

in which

$$\omega = \arctan\left[\frac{m - 1 + \sqrt{(m-1)^2 - 4m\tan\delta}}{2\tan\delta}\right], \quad m = \tan^2(45°+\phi/2)$$

In the model test, natural river sand was used as the backfill, with a unit weight γ=17.6 kN/m^3 and internal friction angle $\mathit{\Phi}$=35.4°. Separate shear tests were done by vertically loading a single soilbag that was placed on a large box filled with sand to obtain the relationship between the shear force acting on the soilbag and the applied normal stress. This was used to calculate the frictional angle between the back of the wall and the backfill (δ = 28.1°). The earth pressures calculated using the equations presented above are shown in Fig.4, it can be seen that this provides a better agreement with the experiment data than that obtained by Coulomb theory.

3 Interlayer friction

Liu et al (2016) found that the interlayer friction of soilbags is the major factor for maintaining the sliding stability of a retaining wall constructed with soilbags. Here, a special simple-shear apparatus as shown in Fig.6 was designed to obtain the correct interlayer friction of the soilbags with increasing height of wall. Since the interlayer friction of the soilbags acted along a ladder-like failure surface in the retaining wall, simple direct shear tests using only two-layers of vertically-stacked soilbags were inappropriate. Instead, simple shear tests using five-layers of vertically-stacked soilbags were carried out. The different vertical loads, *N* imposed by the iron plates correspond to different additional heights of the soilbag-constructed retaining wall. The measured shear force corresponds to the interlayer sliding force, *F*. In fact, as the lateral earth pressure on the retaining wall constructed with soilbags generate moment that increases with an increase of the wall height, the distribution of the applied vertical pressure on the soilbags is not uniform but eccentric. However, due to the limitations of the test equipment the moment generated by applied the lateral load was not applied in the tests.

Fig.7 shows the relationship between shear force and shear displacement measured during the simple shear tests. It can be seen that the shear force increased with an increase in the lateral shear displacement under different vertical loads, and the peak shear strength increased with an increase of the vertical loads. Shear tests on two-layers of vertically-stacked soilbags (Fig.8) were also performed; the peak shear strength result is given in Fig.7(b). As shown in the figure, the peak shear strength of the five-layers of vertically-stacked soilbags is larger than that of the two vertically-stacked soilbags. As we know, the friction F can be expressed as:

$$F = \mu \cdot N$$

where μ is the friction coefficient, and N is the vertical load.

As the same soilbags as those used in the shear tests on five-layers of soilbags and were used in that of two-layers soilbags tests, the μ should be the same. However, Fig.8(b) shows there is a difference between the two curves: curve A is a straight line, but curve B is not, the difference is explained in Fig.9. The sliding surface in the shear tests using two-layers of soilbags is purely the interface between the soilbags, but the sliding surface in the tests using five-layers of soilbags is not. When H_{crit} is no more than 5cm, the sliding surface (red line) of the five-layers test is almost horizontal, as shown in Fig.9(a), while the sliding surface is ladder-like, as shown in Fig.9(b), when H_{crit} is larger than 25cm. This is the same ladder-like sliding surface as seen previously in the model test (Fig.2). The reason why the shape of the sliding surface changes from a straight line to ladder-like is that the insertions of soilbags increases with the vertical load. Hence, the horizontal force applied at the upper layer soilbags was partially distributed to the soilbags at a lower layer. More in-depth study of this mechanism will be explored in a separate paper (Fan at al., 2019).

4 Sliding stability analysis

After obtaining the active resultant earth pressure and the interlayer friction resistance, the sliding stability of the retaining wall constructed with soilbags can be analysed. Fig.10 shows the resultant earth pressure calculated using equation (2) and the interlayer friction resistance (Fig.7). When F(h)=P(h) it is found that Hcrit is 0.915m, whereas the experiment result is 0.95m, the difference is smaller than the height of one soilbag (0.05m). However, Hcrit would be 1.04m if using the resultant earth pressure calculated by Coulomb theory. The overestimation is 0.09m, which is approximately two layers of soilbags. It should be noted that regardless of it being simple and easy to operate there is a limitation in this study. Using a loading plate to exert vertical load on the backfills such that the retaining wall fails cannot completely restore the general loading condition but may produce a slightly different ratios between the lateral to vertical load compared to actual retaining walls.

5 Conclusions

Model tests on soilbag-constructed retaining walls and simple-shear tests on vertically-stacked soilbags were carried out to analyse the sliding stability of the wall. Based on the tests results the following conclusions can be obtained:

1) The sliding surface developed within the soilbags wall is not a straight line, but ladder-like due to the insertion characteristic of the soilbags. The wall above the ladder-like sliding surface was found to undergo a rigid body translation.

2) Horizontal sliding failure of the wall creates a non-linear active earth pressure distribution at failure. Calculations using force equilibrium of differential elements produces a better match to the experimental data than Coulomb’s theory.

3) The sliding friction resistance of the wall is found to be related to the shape of the interlayer sliding surface of the soilbags. When the wall height is small, the sliding surface is horizontal; when the wall height is large, the sliding surface is ladder-like. This was obtained from a specially designed shear apparatus for stacked soilbags. The chosen number of the soilbags used in the shear tests should depend on both the actual thickness of the wall and the potential height of the sliding surface.

4) The sliding stability of the retaining wall constructed with soilbags could be

appropriately obtained using the intersection of failure earth pressure calculated by differential elements and the sliding friction resistance obtained from the shear tests. This proposed method can be adopted for the design of a soilbag-constructed retaining wall.

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Appendix 1

It can be shown(Fig.5) from the equilibrium condition of the horizontal forces on the element, that

$$p_x dy + \tau_2 \frac{dy}{\sin\theta}\cos\theta - r \times \frac{dy}{\sin\theta}\cos(90^\circ - \theta) = 0 \tag{1.1}$$

Equation (1.1) can be written as

$$p_x + \tau_2 \cot\theta - r = 0 \tag{1.2}$$

The following equation can be obtained from the equilibrium condition of the vertical forces on the element:

$$\begin{aligned} &p_y(H_{crit} - y)\cot\theta + dW - (p_y + dp_y)(H_{crit} - y - dy)\times\cot\theta - \tau_1 dy \\ &-\tau_2 \frac{dy}{\sin\theta}\sin\theta - r \times \frac{dy}{\cos\theta}\sin\theta = 0 \end{aligned} \tag{1.3}$$

where $dW = \frac{\left[(H_{crit} - y)\cot\theta + (H_{crit} - y - dy)\cot\theta\right]dy}{2}\gamma$

Substitute dW into equation (1.3), and omit the second order differential terms, equation (1.3) can be simplified to

$$\frac{dp_y}{dy} = \gamma + \frac{1}{H_{crit} - y}[p_y - r - (\tau_1 + \tau_2)\tan\theta] \tag{1.4}$$

Let

$$\begin{aligned} p_x &= Kp_y \\ \tau_1 &= p_x \tan\delta \\ \tau_2 &= r\tan\varphi \end{aligned} \tag{1.5}$$

where K is the active lateral pressure coefficient at failure, δ is the frictional angle between the back of the wall and the backfill and Φ is the internal friction angle of the backfill.

Substituting equation (1.5) into equation (1.2), it can be shown that

$$r = K\frac{\sin\theta\cos\varphi}{\sin(\theta - \varphi)}p_y \tag{1.6}$$

Substitute equations (1.5) and (1.6) into equation (1.4), the following equation can be obtained

$$\frac{dp_y}{dy} = [1 - \frac{\cos(\theta - \varphi - \delta)}{\sin(\theta - \varphi)}\frac{\tan\theta}{\cos\delta}K]\frac{p_y}{H_{crit} - y} + \gamma \tag{1.7}$$

Let

$$\alpha = \frac{\cos(\theta - \phi - \delta)}{\sin(\theta - \phi)}\frac{\tan\theta}{\cos\delta} \tag{1.8}$$

equation (1.7) can be written as

$$\frac{dp_y}{dy} = \text{-}(aK - 1)\frac{p_y}{H_{crit} - y} + \gamma \tag{1.9}$$

By differentiation, the general solution of equation (1.9) is

$$p_y = A\frac{1}{K}(H_{crit} - y)^{aK-1} + \frac{\gamma}{aK-2}(H_{crit} - y) \tag{1.10}$$

in which A is a constant, which can be determined by the boundary condition. Suppose that a surcharge q is exerted on the backfill surface, i.e. $p_y = q$ when y=0. Substitute equation (1.10) into the boundary condition, the constant A can be determined as

$$A=(q-\frac{\gamma H_{crit}}{aK-2})\frac{K}{{H_{crit}}^{aK}-1} \tag{1.11}$$

Substitute above equation (1.11) into equation (1.10), this leads to

$$p_y = (q-\frac{\gamma H_{\mathrm{crit}}}{\alpha K-2})(\frac{H_{crit}-y}{H_{crit}})^{aK-1} + \frac{\gamma}{\alpha K-2}H_{crit} - y \tag{1.12}$$

According to equation (1.5), $p_x = Kp_y$, so that the horizontal unit earth pressure can be obtained

$$p_x = K\left[(q-\frac{\gamma H_{\mathrm{crit}}}{\alpha K-2})(\frac{H_{crit}-y}{H_{crit}})^{aK-1} + \frac{\gamma}{\alpha K-2}H_{crit} - y\right] \tag{1.13}$$

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Table 1 Physical and mechanical parameters of a natural river sand

Figure 1. Photo and schematic view of the model test (unit: cm) (a), Photo (b), schematic diagram

Figure 2. Deformation of the retaining wall and backfills (a), Photo (b), Schematic diagram

Figure 3. Schematic view of the insertion between two layers of solibags

Figure 4. Distribution of the lateral earth pressures on backfills and within the soilbags under ultimate load

Figure 5. Analytic model (a), Deformation mode of the backfill soil behind soilbags' retaining wall (b), Analysis of the forces acting on the thin layer element

Figure 6. Schematic view the simple shear test on stacked soilbags

Figure 7. Results of the simple shear tests on vertically-stacked soilbags (a), Shear force F versus horizontal shear displacement (b), Peak shear force F_p versus the critical wall height H_{crit} above the sliding surface.

Figure 8. Slip surface in the shear tests on two-layers soilbags

Figure 9. Different sliding surfaces in the shear tests on five-layers soilbags

Figure 10. Resultant earth pressure and interlayer friction of soilbags with the height of the retaining wall