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Experimantal Examination of Active and Passive Wedge in Backfill Soil of Model Cantilever Retaining Wall

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***Abstract*— In the scope of this study, failure tests were performed in plane strain conditions with a model cantilever retaining wall supporting two different sands placed in layers so as to have same internal friction angle. The passive and active cases were created by moving forward and backward the model wall very slowly. Failure surfaces and geometry of active and passive wedges behind the wall were observed. As a result of model tests it was determined that in passive case shape of the wedge is similar with the other studies in the literature. However, observed failure surfaces and shape of active wedge was quite different from generally accepted Coulomb's wedge theory. However, Coulomb's wedge theory and Rankine's theory can be applied to find active and passive earth pressure with suitable failure surface assumption of soil wedge.**

***Keywords*— Cantilever retaining wall, Coulomb wedge theory, Passive case, Active Case**

I. Introduction

Slope stability problem was one of the earliest problems of geotechnical engineers. Lateral forces such as surcharge, water or earth pressures are commonly encountered problems in civil engineering. The stability problems of the retaining structures which appears to be negligible or unobtrusive, can cause much more damage and injuries than their current values [1]. Therefore, it is important to verify safety of retaining structures.

The forces acting on the retaining walls, soil movements and failure surfaces attract many researcher's attentions. Two limit states were observed to occur in soil medium depending on lateral movement. In active case lateral expansion takes place, whereas in passive case there is lateral compression. Some researchers investigated active earth pressures occured behind rigid retaining walls in state of different displacement conditions ([2], [3], [4], [5]). On the other hand, numerous studies related with the passive earth pressure were performed ([6], [7], [8], [9], [10]).

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Although many important examinations and experimental works have been performed about the lateral earth pressure ([11], [12], [13], [14]), Coulomb's (1776) and Rankine's (1857) theory still remains up to date for determination of lateral earth pressure. In Coulomb's theory, it is regarded that there is friction between the wall and soil and this is taken into account by using a soil-wall friction angle of δ. Additionally, in minimum active and maximum passive states, force equilibrium is used to determine the lateral pressure on wall. On the other hand, Rankine (1857) proposed that a condition of limit equilibrium exists in the soil mass retained behind a vertical wall and the retained soil mass will slip along a planar surface. Limit equilibrium describes the state of a soil mass that is on the verge of failure, i.e. the applied stresses are equal to the available strength along the slip plane.

Unlike the approaches above, lateral forces have been calculated with finite element method in recent years. In this studies, shape of the failure surfaces are neglected and deformations of backfill and wall can be determined ([15], [16], [17]).

Shape of the failure surfaces that estimated by using conventional calculation methods is not realistic especially for the passive failure state. Due to this reason, model tests are useful in determination of real scale soil behaviour. So that experimental observation, which performed to determine the correct failure surface, is the most appropriate approach.

In the scope of this study, failure tests were performed in plane strain conditions with a model cantilever retaining wall supporting two different sands placed in layers to have same internal friction angle. The passive and active cases were created by moving forward and backward the model wall very slowly. Failure surfaces and geometry of active and passive wedges behind the wall were observed. As a result of model tests it was determined that in passive case, failure surface and passive wedge geometry of backfill soil of cantilever retaining wall are similar to theoretical studies (Coulomb, 1776; Rankine, 1857). However, in active case failure surfaces and shape of active wedge are quite different from generally accepted Rankine's and Coulomb's theory. Therefore, a new approach in accordance with the observed geometry of active wedge was proposed for calculation of active force acting on cantilever walls.

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II. Experimental Study

An Experimental setup consisting of a tank, a model cantilever wall, a pulling - pushing system, strain gauge, proving ring, sands and a digital camera was used to perform active and passive cases tests. The tests and other soil tests were carried out in performed in the Geotechnical Engineering Laboratory of Karadeniz Technical University. The components of the test setup are explained in detail below.

A. *Test Tank*

The tank was designed as a rectangular prism and internal dimensions of the tank are 0.90 m length, 0.1 m width, and 0.65 m height (Figure 1). Bottom and lateral parts of the tank were made of hard wood and L shaped canals were carved in order to place glass plates. Wooden triangular chocks were mounted on both sides of the tank to provide lateral rigidity. Thus, lateral deformations of wooden frame were prevented. Glass plates of 20 mm thickness were placed front and back sides of the tank to observe and take photos of the failure surfaces. This glass plates were beneficial to create rigid surfaces which is perpendicular to the axis of model retaining wall. A square mesh was drawn on the front surfaces of the glass plates to observe compaction of the sand layers and determine failure surfaces.

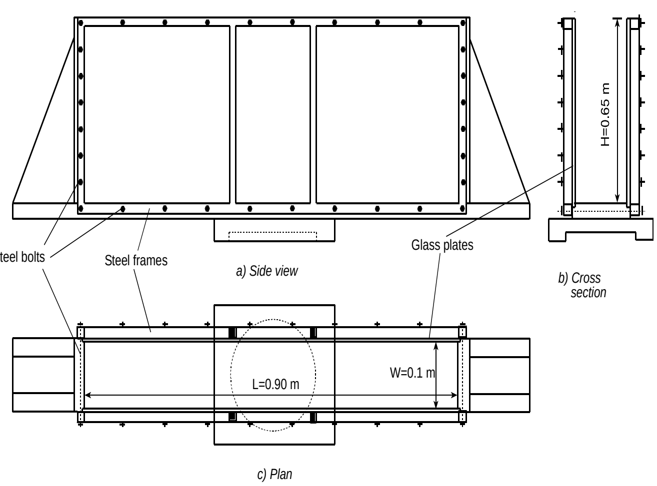

Figure 1. Dimensions of the test tank

B. *Model Cantilever Wall*

Model retaining wall was made of wood and the joint between the stem and footing of the model wall was strengthened by using steel plates. The dimensions of the model wall were determined in accordance with predimensioning ratios [1]. The model cantilever wall were shown in Figure 2.

Figure 2. Model retaining wall used model tests

C. *Sand*

Two types of sand, which have different colours, were used in the experimental study. The sands were obtained from east and west coasts of Trabzon City in Turkey. The sands were sieved from No. 4 sieve (4.76 mm) to remove foreign matters and coarse particles and dried in an oven. The sands, namely yellow sand and black sand, were classified as poorly graded sand (SP) according to USCS (Unified Soil Classification System) and particle size distribution curves of the sands are shown in Figure 3.

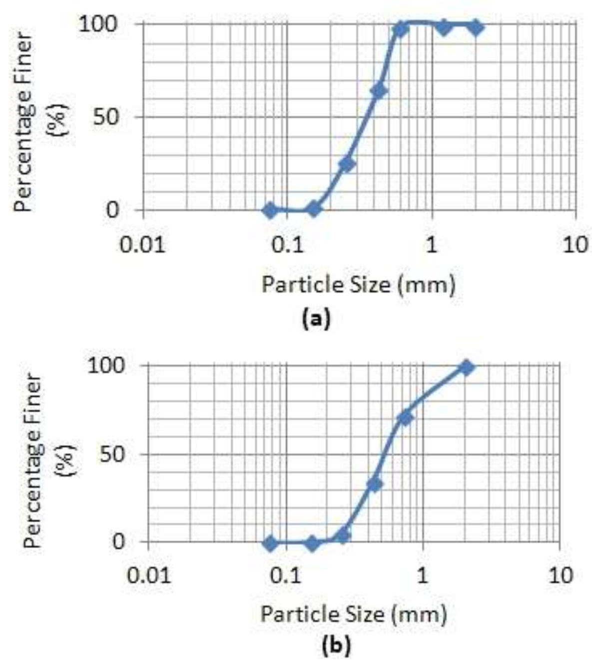

Figure 3. Particle size curves of the sands a) Yellow Sand b) Black Sand

After the sieve analysis (dry method) D_{10}, D_{30}, D_{60}, C_u, C_r values were determined by using the particle size curves. Specific gravity, modified proctor, minimum density, and shear box tests were performed on the sands and the results are summarized in Table 1.

TABLE 1. PROPERTIES OF THE SANDS USED IN THE TESTS

Property	Quantity	
	Yellow Sand	*Black Sand*
Specific gravity, G_s	2,69	3,07
Maximum dry unit weight, γ_{dmax} (kN/m^3)	17,8	20,08
Minimum dry unit weight, γ_{dmin} (kN/m^3)	12,1	15,1
Effective size, D_{10} (mm)	0,18	0,28
D_{30} (mm)	0,25	0,4
D_{60} (mm)	0,4	0,6
Coefficient of uniformity, C_u	2,22	2,14
Coefficient of curvature, C_r	0,87	0,95
Angle of internal friction, $\phi_{direct\ shear}$ (degrees) D_r=25	38,76	28,75
Angle of internal friction , $\phi_{direct\ shear}$ (degrees) D_r=50	42,95	45,14

D. *Pulling-Pushing System*

The experimental setup can be seen in Figure 4. In order to create pulling and pushing effect loading device of the shear box test was used. The loading capacity of machine is 3 kN and its lateral constant displacement rate was chosen as 0.15 mm/min. Active and passive states were created by the loading apparatus of the shear box test which is fixed to the model retaining wall.

Figure 4. General view of test set up

E. *Typical Test*

The different colored sands were placed in layers in the tank sequentially to provide contrast between sand layers and to observe failure surfaces and shape of soil wedge behind the model wall clearly (Figure 5). The sand layers were placed with same internal friction angle of 40º to gain similar failure behavior in yellow and black sand layers. Internal friction angles of the sands were determined with shear box test at different relative densities (Table 1). The relative densities corresponding to internal friction angle of 40º were found for each sand by interpolation. Dry densities of these relative densities were found with the equation below.

$$D_r = \frac{\gamma_{d\max}}{\gamma_d}\left(\frac{\gamma_d - \gamma_{d\min}}{\gamma_{d\max} - \gamma_{d\min}}\right) \quad (1)$$

The quantities (7380 g for black sand and 6075 g for yellow sand) for a 50 mm thick layer were deposited in the tank loosely as a uniform thick layer. To confirm 50 mm thickness, horizontal lines at 50 mm intervals were drawn on the internal face of the glass plate. These loose sand layers were lightly compacted with a wooden hammer to 50 mm thickness to achieve 200 mm height sand mass below the base level of model wall. The model cantilever wall was fixed and the masses of the black and yellow sand layers above base level were calculated as 4920 g and 4050 g, respectively. The sand mass of 300 mm height was created behind the model wall using same procedure.

Figure 5. Beginning of the test

III. Results and Discussions

A. *Active State*

In order to create active state, model retaining wall was slipped with aid of pulling by operating in reverse direction. Vertical constant displacement rate of the apparatus was chosen as 0.15 mm/min and the model retaining wall was moved laterally via loading piston of the shear box test machine. Therefore, active state was occurred in the backfill soil and formation of active wedge was observed by taking photographs periodically. A soil wedge seen on Figure 6 occurred behind the model wall as a result of active state created by means of pulling.

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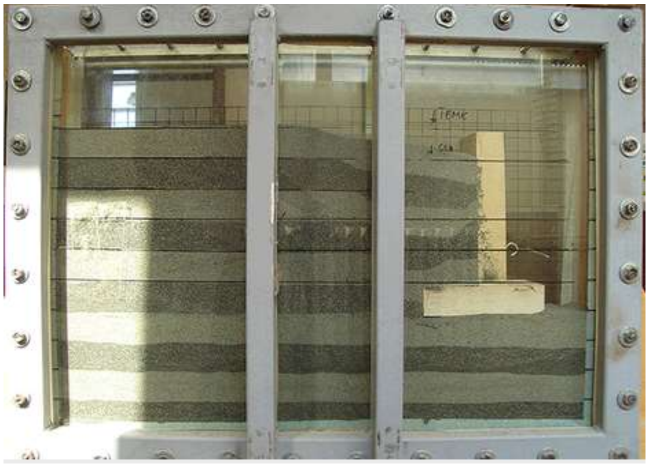

Figure 6. End of the test (active state)

Soil wedge formed behind the wall is shown in Fig. 7. This observed wedge has some differences with the proposed wedges of some researchers ([18], [19], [21]). Active wedge is a concave polygon and two edges of the polygon are line segments which are 45+ϕ/2 from horizontal. Also, a triangular soil mass moving along with the cantilever wall is available. Active earth pressure acting on the wall can be found by Coulomb's wedge theory [21] in accordance with Teng's [18] recommendation about trial wedge. In this case, internal friction angle of backfill soil is used in well-known Coulomb's equation of active earth pressure instead of wall-soil friction angle.

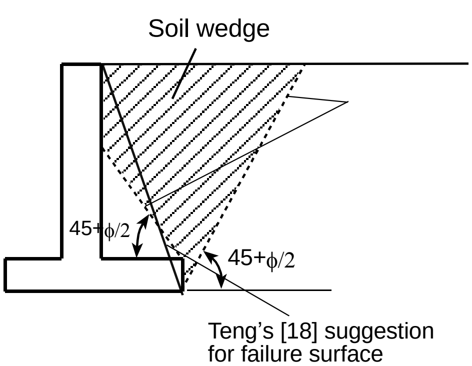

Figure 7. Failure surface shapes of backfill in active case

Passive State

With the aim of creating passive state loading piston of the shear box test machine was moved forward. Similar to the active state, vertical constant displacement rate of the apparatus was chosen as 0.15 mm/min and the model retaining wall was moved laterally via loading piston of the shear box test machine. Consequently, passive state was created by moving the model wall. Along the moving process formation of passive wedge was obtained by taking photographs periodically (Fig. 8).

Figure 8. End of the test (pasive state)

Soil wedge occuring in backfill soil in passive case is shown in Fig. 9. Failure surfaces are two broken linear lines.

This situation can be result of different amount of compression developed in backfill soil at footing and stem level. Because, footing is much more rigid than the soil mass above footing and passive case develops at footing level before stem level. The other reason can be width of the tank. After the observation of the failure surface shapes it was determined that the wedges has some similarities with the proposed wedges of other researches. Coulomb's Wedge Theory [21] and Rankine's Theory [20] can be applied to find passive earth pressure.

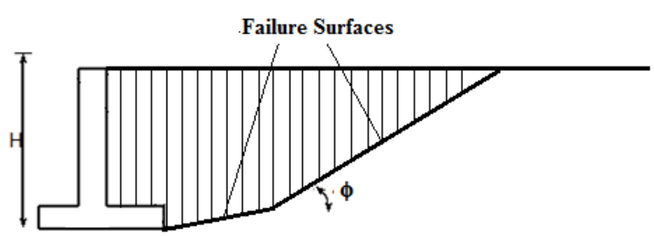

Figure 9. Failure surface shapes of backfill in passive case

IV. Conclusions

In this study, failure surfaces and soil wedges that occurred behind a model retaining wall were examined in active and passive cases with model tests. In model tests, different colored sands were placed in layers to observe failure surfaces and soil wedges. Active and passive states were created by moving the model cantilever retaining wall in plain strain conditions. The following conclusions are drawn from the study:

- Observed failure surfaces in active case were highly different than assumption of Coulomb's theory. However, active earth pressure acting on the wall can be found by Coulomb's Wedge Theory in accordance with Teng's [18] recommendation about trial wedge.

- Failure surface is assumed to be linear in Coulomb's Wedge theory. Whereas, in the experiments it is seen that this failure surface consists of multi-pieced lines in passive case. However, Coulomb's Wedge Theory and Rankine's Theory can be applied to find passive earth pressure safely.
- In passive case, the soil mass above the footing of the wall moves along with the wall.

These results revealed the requirement of measurement of the forces acting on the wall by making the model and real scaled experiments about the topic. Thus, the applicability of Coulomb theory and just improved method for console walls will become more clear.

References

[1] Whitlow, R. (2001). "Basic Soil Mechnanics". Prentice Hall. London.

[2] Hazarika,H. and Matsuzawa,H. (1997), "Coupled Shear Band Method and it's Application as Compressible Inclusion" Research Report of Maizuru National College of Technology, Kyoto, Japan.

[3] Huang, C.C., Luo, W.M., (2009). "Behaviour of Soil Retaining Walls on Deformable Foundations" Engineering Geology, 105(1): 1-10.

[4] Pietrzak M. and Leśniewska D. (2012), "Failure Evolution in Granular Material Retained by Rigid Wall in Active Mode", Studia Geotechnica et Mechanica, Vol. 34, No. 4. 365

[5] Şadoğlu, E., Kamiloğlu, H.A., Kuvat, A. and Uzuner, B.A. "The Experimantal And Analytical Examination of Active Wedge Occuring In Cantilever Walls" 5th Geotechnical Symposium Çukurova University Adana.

[6] Terzaghi, K. (1920), "Old Earth Pressure Theories & New Test Results", English News-Recordings, Vol. 85, No. 14: 14–32.

[7] Brinch-Hansen, J. (1953), "Earth Pressure Calculation. Copenhagen", Danish Technical Press

[8] Hanna, A. ve Al-Khoury, I. (2005), "Passive Earth Pressure of Overconsolidated Cohesionless Backfill", Journal of Geotechnical & Geoenvironmental Engineering, ASCE, Vol.131, No. 8: 978-86.

[9] Fang, Y.S.,C.C. Lee, C.C. ve Chen, T.J. (2006), "Passive Earth Pressure with Various Backfill Densities", Physical Modelling in Geotechnics-6th ICMPG'6-Ng. London.

[10] Tejchman, J., Bauer, E. ve Tantono, S.F. (2007), "Influence of Initial Density of Cohesionless Soil on Evolution of Passive Earth Pressure", Acta Geotechnica, Vol. 2, No. 1: 53-63.

[11] Terzaghi, K. (1932). "Record earth pressure testing machine." Engrg. News Record 109 (Sept. 29), 365-369.

[12] Schofield, A. N. (1961). "The development of lateral force of sand against the vertical face of a rotating model foundation." Proc., 5th Int. Conf. Soil Mech. and Found. Engrg., Paris, Vol. 2, 479.

[13] Duncan, J. M., and Seed, R. B. (1986). "Compaction-induced earth pressures under Ko condition." J. Geotech. Engrg., ASCE, 112(1), 1-22.

[14] Tejchman, J., Bauer, E. ve Tantono, S.F. (2007), "Influence of Initial Density of Cohesionless Soil on Evolution of Passive Earth Pressure", Acta Geotechnica, Vol. 2, No. 1: 53-63.

[15] Nakai, T (1985), "Finite element computations for active and passive earth pressure problems on retaining wall", Soils & Found., 25(3), 98-112.

[16] Bakeer, R.M. and Bhatia, S.K. (1989), "Earth pressure behind a gravity retaining wall", Numer. Anal. Methods in Geomech., 13, 665-673.

[17] Goh, A.T.C. (1993), "Behavior of cantilever retaining walls", Journal of Geotechnical Engineering Division (ASCE), 199(11), 1751-1770.

[18] Teng, W.C. (1962), "Foundation Design" Prentice Hall. Eaglewood cliffs. New jersey.

[19] Uzuner, B.A. (2012) "Temel Zemin Mekaniği" Derya Kitapevi Trabzon.

[20] Rankine , W.J.M. (1857). " On the stability of Loose Earth." Philosophical trans. Of The Royal Society , 147(2),9-27.

[21] Coulomb, C.A. (1776). " Essai Sur Une Application des Regles des Maximums Et Minimums a Qualques Problemes de Statique Relatifs a I'architecture." Memoris Academie Royal des Sciences, 3,38.

[22] Abzal, M., Altunbaş, A., Gezgin, A.T.,Çinicioğlu, Ö. (2013). " Pasif Göçme Durumlarında Dolgu Kayma Yüzeyleri ve Göçme Mesafeleri" 5. Geoteknik Sempozyumu, Çukurova Üniversitesi, Adana.