Discussion of "Passive Earth Pressure with Critical State Concept" by Yung-Show Fang, Ying-Chieh Ho, and Tsang-Jiang Chen

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The authors should be congratulated for synthesizing the results of an interesting experimental investigation on the passive earth pressure on retaining walls. The authors have used these experimental results as a guide in developing design procedures.

In their tests, the authors have adopted a sand placing technique, namely by placing the sand in thin layers and then applying surface compaction on each layer to achieve the required unit weight and angle of shearing resistance of the sand. It is evident that the application of vibratory densification produces an overconsolidation effect in the sand mass. Depending on the level of energy transferred to a given layer, the in situ stress level in the said layer will increase. The energy transferred depends on the thickness of the placed sand layer, compaction effort applied on the surface of each layer, compaction duration, and the order of the respective layer (the energy input for a lower layer is the total of the energy applied to the said layer plus the energy transferred to this layer during compaction of the subsequent layers). The energy transferred to a given layer reflects on the in situ stress level and accordingly, the value of the overconsolidated ratio (OCR) of the said layer (Hanna and Ghaly 1990; Hanna and Soliman-Saad 2001). The OCR has a direct and significant effect on the coefficients of earth pressure (Hanna and Soliman-Saad 2001; Bellotti 1976; Feda 1984; Brooker and Ireland 1965). The authors have treated the produced sand beds in the laboratory as if they were normally consolidated; i.e., the case of $\mathrm{OCR}=1$.

The authors have reported that the experimental test results agreed well with both Coulomb and Terzaghi theories for the case of loose sand, while a wide disagreement was noted for the case of dense sand (using the peak angle of shearing resistance of the sand). In treating this problem, the authors have proposed to adopt the residual-internal angle of friction of the sand, instead of the peak angle of shearing resistance. As expected, by reducing the angle of shearing resistance to the residual value (which is equivalent to the ultimate value for the case of loose sand) agreement will be achieved with Coulomb and Terzaghi theories. In implementing such assumption the authors have allowed a wall movement of about 10–20% of the height of the wall for medium to dense sand respectively. Such high wall displacement may not be allowed in statically indetermined structures such as bridges. Furthermore, beyond the peak angle of shearing resistance, ϕ, the inter-particles stresses will be the dismantled due the dilation (Hanna 2001). Thus, the in situ stress level will be reduced and accordingly the OCR will reach the value for normally consolidated sand $(\mathrm{OCR}=1)$. This procedure suggests that sands at all states will be treated as if it is loose sand, and accordingly Coulomb and Terzaghi theories may be used to predict the passive earth pressures on retaining walls. It should be made here however, that this assumption is a conservative one, as passive earth pressure is a resistive force.

The authors have limited the results of their investigation to the case of low values of wall friction angles. $(9.8–14°)$. This assumption or rather the condition of the tests may be regarded for the cases closer to Rankine condition (smooth walls) than Coulomb condition (rough walls).

The discusser has recently conducted similar tests, where the in situ stresses in the sand mass were measured and accordingly the OCR values were calculated. Similar conclusions were drawn with respect to Rankine, Coulomb, and Terzaghi theories. It is reasonable to assume that both Terzaghi and Coulomb theories were developed, guided or validated by test results on overconsolidated dense sands. This work is currently under review.

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The experimental data presented by the authors are useful for a better understanding of the passive earth pressure problems. These data have confirmed that a peak passive resistance may develop when a rigid wall is pushed towards a mass of dense sand and the resistance will subsequently reduce to an ultimate value at

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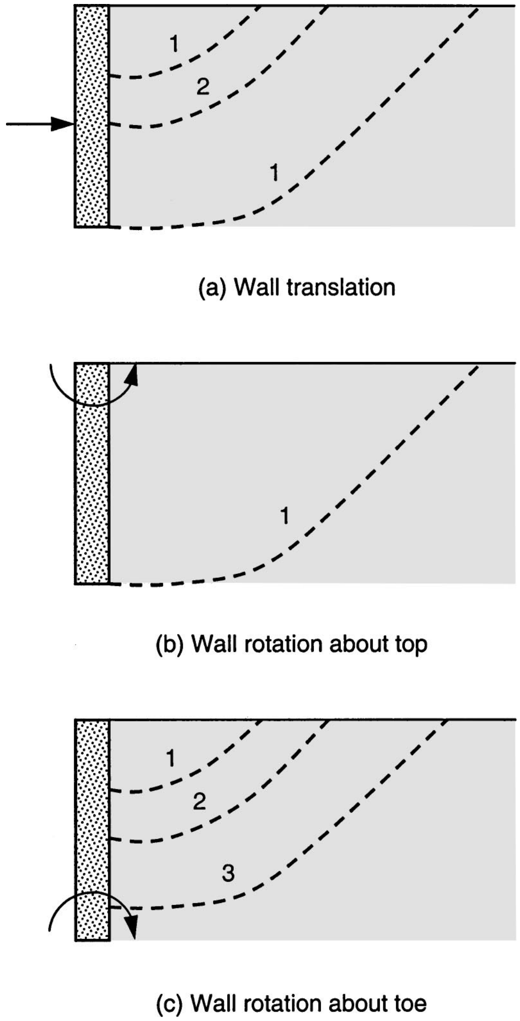

Fig. 1. Formation of rupture surfaces in dense sand for three modes of wall movement (after James and Bransby 1970)

a large wall movement. The observations are in consistence with the stress–strain–dilatancy behavior of sands subjected to a shear.

It would be of more interest, however, if the authors presented their experimental observations on the formation of sliding surfaces in both the cases of dense and loose backfill. The information can be used to further justify the available theories and assumptions. It is known that the widely employed Coulomb theory assumes that the passive failure mechanism involves the sliding along a planar surface, whereas the log spiral theory proposed by Terzaghi uses a curved failure surface that is considered to represent a more probable failure mechanism. Previous experimental investigations, however, seem to indicate that no theory can correctly predict the shape and location of the sliding path. For example, the observations by Rowe and Peaker (1965) on the rupture surfaces in a mass of loose sand subjected to the translation of a wall indicate that, in the region near the sand surface where Rankine passive states are normally assumed to occur, the slope of the sliding path could be much steeper than that predicted by theory. More complicated patterns of rupture surfaces were reported by James and Bransby (1970) for three typical modes of wall movement. As shown in Fig. 1, when the wall is pushed towards a mass of dense sand, the sliding surface will first extend from the toe of the wall, followed by a zone of radial shear growing out from the top of the wall; when the wall rotates about its top, there will be a single sliding surface extending from the toe of the wall; and when the wall rotates about its toe, a zone of radial shear will extend from the top of the wall and will grow in size when the rotation continues.

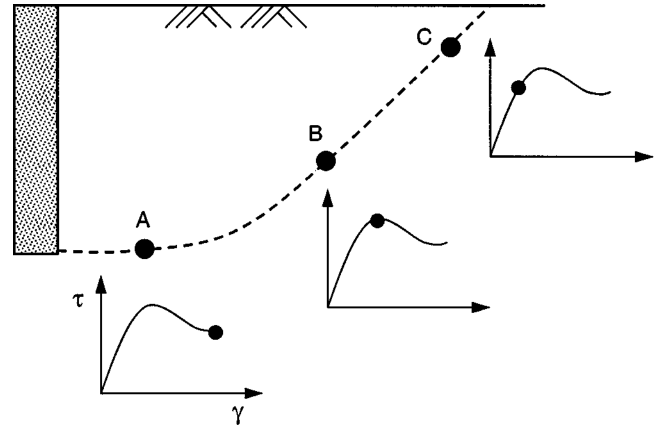

Fig. 2. Schematic diagram of progressive failure mechanism

The shape of the sliding path is believed to be largely dependent on the mobilized, rather than the maximum, angle of wall friction. In addition, the direction of sliding surface may also be influenced by the mobilization of shearing resistance along the surface. The mobilization of wall friction is more influential on the lower, curved part of the sliding path while the effect of shearing resistance mobilization in the sand mass could be more prominent for the upper, straight part of the sliding path near the sand surface. In actual cases the formation of sliding surface is more likely to involve a progressive failure mechanism (Fig. 2), and it may become especially complex when multiple sliding surfaces appear. Experimental measurements of quality are to be accumulated in order to improve current understanding of passive failure mechanisms and the formation of sliding surfaces. It would thus be helpful to see the authors' observations in their tests reported.

In their paper, the authors also made a recommendation on the design of passive earth pressure problems. The design using the residual, critical state strength, as recommended by the authors, certainly will put the design on the safe side. Though, it should be mentioned that this design concept suffers from several limitations that do not allow it to be, as stated, a reasonable one. First,

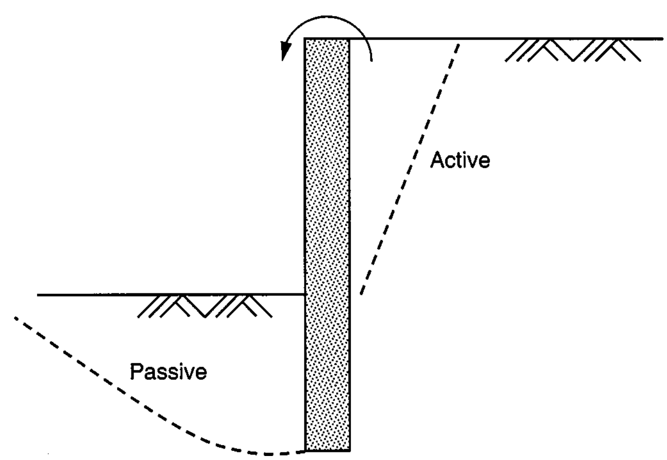

Fig. 3. Typical retaining wall problem in geotechnical applications

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since the critical sate strength of a sand is independent of its density, this design concept treats the case of loose backfill and the case of dense backfill as the same. In other words, it ignores the advantage of compaction of backfill, which however is generally required in engineering practice. Second, in a typical retaining wall problem in geotechnical applications (Fig. 3), the active and passive states are simultaneously involved such that a large movement of the wall would not be allowed. It has been confirmed experimentally that the active state is reached with very little movement; a small displacement of the order of 5–10 times the mean diameter (D_{50}) of sand particles in the sliding direction is enough to reduce the mobilized shearing resistance from peak to residual value (Bolton and Steedman 1985; Yoshida and Tasuoka 1997). Therefore, if the wall is proportioned so that this is equilibrium with the active earth pressures, then the movement of the wall will be small. With this small movement the mobilized shearing resistance along the sliding surface in the passive zone is not possible to reach the ultimate, critical state strength.

How to properly account for the dilation and strain softening effects in earth pressure problems remains to be a subject of interest. It is the discusser's opinion that the single use of either the residual or the peak shear strength in design is not a satisfactory way; rather, the design concept that involves both peak and residual strength and meanwhile is displacement or performance based would be more rational.

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The authors wish to express their appreciation to Jun Yang and Adel Hanna for their interest in the paper and helpful comments.

The authors agree with Yang's comment that experimental observations regarding the formation of rupture surfaces would improve current understanding of passive failure mechanism. Fang et al. (1994) studied the passive earth pressure acting against a vertical rigid wall, which moved into a mass of dry sand under different wall-movement modes. Displacement fields in the backfill are reported in Fig. 17 of their paper. Fig. 17(a) shows the displacement vectors of backfill induced by the translational wall movements. Large displacements occurred in front of the wall. Unfortunately, rupture surfaces determined with both Rankine and Terzaghi theories are not able to define the backfill movement properly. Fang et al. (1994) concluded that backfill displacement is strongly influenced by the movement mode of the wall as reported by James and Bransby (1970).

It should be noted that to observe the displacement in the backfill is not an easy task. To simulate a plane-strain testing condition, the shearing stresses on the intermediate-principal plane should be equal to zero. In this study, a lubrication layer was furnished for the earth-pressure experiments to minimize the friction between the sidewall and backfill. However, unless the sidewall friction can be totally eliminated, a quantitative soil displacement observation cannot be achieved at the backfill–sidewall interface.

The authors agree with Yang's and Hanna's comment that the design of passive earth pressure problem using the critical state strength would be a conservative one. It should be emphasized that Fig. 15 provides the upper and lower bounds for estimating the passive earth pressure. An overestimation of passive resistance based on the peak soil strength would put the design on the unsafe side, and an underestimation of passive resistance based on the residual soil strength may be too conservative. Fig. 14 shows the complete variation of lateral earth resistance as a function of wall movement. When calculating the passive earth pressure, it is recommended that one consider the dilation and subsequent strength reduction of a dense backfill. The authors agree with Yang's comment that a more rational design concept should be displacement-based and involve both peak and residual strength.

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The writers presented the results of research which is interesting from an academic standpoint and certainly from a practical point

Table 1. Dry Unconfined Compressive Strength of Specimens (Mpa)

Type of mix	Delay time (min)	Curing time (days) 1	7	28
Control	0	4.3	9.3	9.8
(50% sand+50% fly ash	30	Could not be molded		
Mixture				
(50% sand+50%	0	5.6	17.4	19.1
fly ash+1%				
Aluminum	30	5.1	12.6	17.5
ammonium sulfate)				
	30	3.0	7.15	10.2
	120	1.5	5.2	6.7

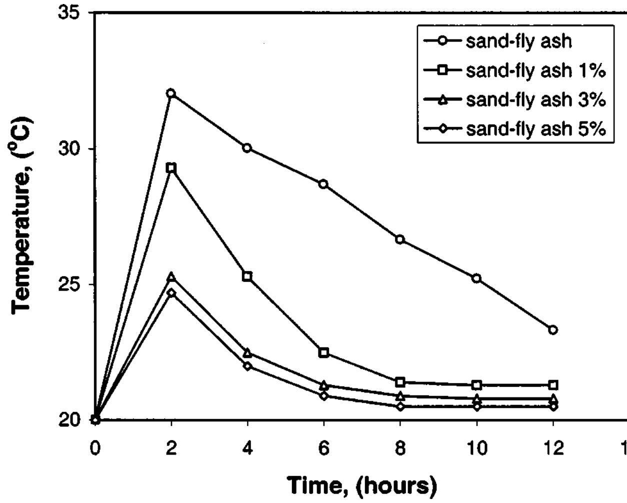

Fig. 2. Heat of hydration of sand/fly ash/aluminum ammonium sulfate mixtures (results with aluminum sulfate were similar)

of view. It is very timely; it can find application in the area that gains significance because of the increasing need of grouting. The following remarks constitute the discusser's experience and are not to be construed as criticism but rather an enriching information.

1. In working with class C fly ash (CaO 27%) in Oklahoma (actually, it was Wyoming coal), the discusser experimented with retarders using standard Ottawa sand and fly ash at 50–50 ratio (Tohidian 1986). During the exploratory phase of this effort, a number of retarders, including sodium hydroxide were tested. As a result, the selection list was narrowed down to aluminum sulfates, sodium fluoride and two other chemicals CA-3 and R-007. For the last two the producers did not release any information.

Of these, the most efficient (optimum conditions) were the aluminum sulfate and the aluminum ammonium sulfate (Tohidian and Laguros 1987). This final selection was based on weighing a number of parameters such as density, effect of delayed compaction and curing time on unconfined compressive strength, heat of hydration. Additionally x-ray diffraction (XRD) and scanning electron microscopy (SEM) were employed for mineral identification and crystalline formation. That sodium hydroxide did not give the most desirable results may be due to the fact that the Oklahoma fly ash differs from the Ptolemaida fly ash at least in terms of the CaO content (Oklahoma 27%, Ptolemaida 32%) and granulometry (for Oklahoma retained on U.S. std. Sieve No. 325 or plus 44 μm 13.1%, for Ptolemaida-based on graphical extrapolation from Fig. 1 of the paper less than 5%) Unfortunately, chemical analysis of the Ptolemaida fly ash is not provided and, therefore, no further comparison can be made.

2. The case of the aluminum ammonium sulfate is presented in Table 1 and is depicted in Fig. 1. Delayed compaction made the control mixes uncompactable. By adding the sulfate based retarders to the sand/fly ash mixtures, at the rate of 1% per dry weight of the mix, the unconfined compressive strength increased for specimens cured for various curing times but compacted with no delay. Also, as the delayed time was prolonged, it tended to minimize the strength increase. At extended delay time, 60 min or longer, it tended to decrease the compressive strength to the point where it attained levels lower than that of the control mix (Tohidian and Laguros 1987). It should be born in mind that compaction immediately after mixing is possible in the laboratory but delayed compaction is inevitable in the field.

3. The heat of hydration curves (Tohidian and Laguros 1987) depicting temperature time relationships (Fig. 2) showed conclusively that the retarders reduced the generation of heat and that

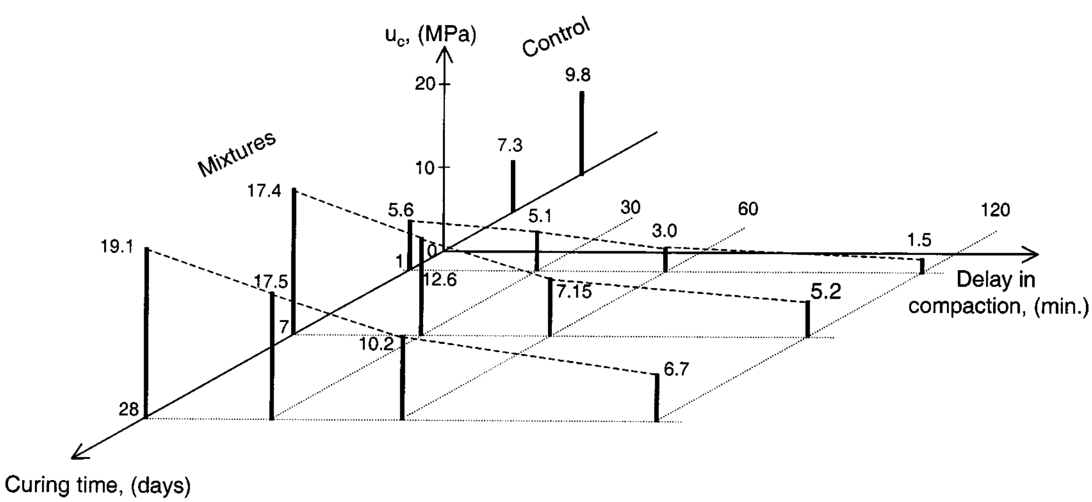

Fig. 1. Effect of delay in compaction on unconfined compressive strength, u_c, of sand/fly ash/1% aluminum ammonium sulfate retarder mixtures

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the drop of temperature with time was far less when retarders were used and that the lowest peak temperature 28.2°C (83°F) compared to the no retarder mix 31°C (88°F) was associated with the highest compressive strength.

4. The XRD indicated (Tohidian and Laguros 1987) the formation of tricalcium aluminate ($3CaO-Al_2O_3$) that augmented with time as evidenced when comparing the 1 day and 28 day cured specimens. The formation of ettringite $3CaO \cdot Al_2O_3 \cdot 3CaSO_4 \cdot 32H_2O$ was noticeable for the 1 day cure as was its reduction for the 28 day cure. The depletion of the sulfate ions or decomposition of ettringite gives rise to a lower hydrate monosulfoaluminate ($3CaO \cdot Al_2O_3 \cdot CaSO_4 \cdot 12H_2O$). The selection of the cation (NH_4^+, Al^{3+}, or Ca^{2+}) does not appear to be as critical as the availability of the sulfate ions having an influence on the crystalline compounds formed (Tohidian 1986). Also, present in all cases was the formation of stratlingite ($Ca_2Al_2SiO_7 8H_2O$).

5. The authors present Fig. 8 from which it is difficult to discern the morphological features such as crystalline formations. In the discusser's findings (Tohidian and Laguros 1987) crystalline formations of the needle type structures as well as hexagonal plate types are abundant but less so when there has occurred prolonged delayed compaction. Delayed compaction with no retarder causes the skeletal framework formed (fast chemical reactions leading to crystal formations) to break down into smaller fragments.

6. It would have been interesting to determine the void domain matrix of the mixes through the use of SEM photomicrographs. The technique developed (Zenieris and Laguros 1989) renders possible the calculation of the degree of propensity in void reduction. It has been applied successfully in assessing soil stabilization (Laguros and Hayes 1990). It appears reasonable to apply it, too, to the solidification of the grouted fabric. These would have provided additional information on the status of ''macropores'' and help better explain the influence of grouting on the hydraulic conductivity of the mixes, the development of pore water pressures and, in general, the ''behavior of sands grouted with PFA suspensions.''

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The authors thank the discusser for his interest in their paper and for his comments. The discusser provided valuable information on the utilization of fly ash for soil stabilization and results of experimental investigations on the performance of retarders, including sodium hydroxide, when compacting Ottawa sand–fly ash mixtures. It should be pointed out that sodium hydroxide was used as accelerator and not as retarder in the authors' research. Bearing in mind the variability in chemical composition of fly ashes and the effect that several factors have on their behavior, the authors believe that comparable behavior should be expected only when similar materials are tested. For comparison purposes, the chemical composition of Ptolemaida fly ash is provided in Table 1. The authors agree with the discusser that it is not possible to make any comments on crystalline formations from the photograph presented in Fig. 8 of the paper. Moreover, the magnification capacity of the stereoscopic zoom microscope used, was insufficient for such an investigation. However, the purpose of the observations of grouted sand specimens through this microscope was to evaluate qualitatively the degree of filling of the sand voids with suspension solids and the adhesion capacity of the grout solids on the sand grains. These points are clearly depicted in the photograph (Fig. 8). The authors have made no attempts to quantify the void domain matrix using scanning electron microscopy photomicrographs or other techniques. The concept of ''macropores'' and ''micropores'' is referred to by the authors in conjunction with stable suspensions which have been documented to yield minimum development of macropores.

Table 1. Chemical Composition of Ptolemaida Fly Ash

Oxide	Content (%)
SiO_2	35
Al_2O_3	14
Fe_2O_3	5
CaO	32
MgO	3
SO_3	5
Na_2O+K_2O	0.5
Loss on ignition	3