A Modified Procedure to Evaluate Seismic Active Earth Pressure Considering Effects of Strain Localization in Backfill Soil

by

J. Koseki[1], F. Tatsuoka [2], Y. Munaf [3], M. Tateyama [4] and K. Kojima [5]

ABSTRACT

A modified pseudo-static and limit-equilibrium approach to evaluate active earth pressure at high seismic load levels is proposed. Although it is similar to the Mononobe-Okabe method, the proposed method considers the effects of strain localization in the backfill soil and associated post-peak reduction in the shear resistance from peak to residual values along a previously formed failure plane. The proposed method can reflect differences in the peak shear resistance of the backfill soil with different degrees of compaction; yields a realistic active earth pressure coefficient; can be adapted to analyses with a large horizontal seismic coefficient; and renders a reduced and more realistic size of active failure zone in the backfill soil at high seismic load levels compared to that predicted by the Mononobe-Okabe method.

INTRODUCTION

The well-known Mononobe-Okabe method, based on a pseudo-static and limit-equilibrium approach, was proposed to calculate the seismic earth pressure (Okabe, 1924 and Mononobe and Matsuo, 1929). Since then, a variety of shaking model tests have been conducted to study on the seismic earth pressure (e,g., Ohara et al., 1970, Ichihara and Matsuzawa, 1973 and Ishibashi and Fang, 1987). It was suggested by these previous investigations that, in general, the Mononobe-Okabe method can reasonably predict the total active earth pressure during earthquake, although its point of application is located higher than that derived from the assumption of hydrostatic distribution, which may lead to an underestimation of overturning moment of the wall due to the earth pressure. It is to be noted that seismic loads examined in the experimental studies were limited to relatively low levels; the amplitude of input acceleration was smaller than 500 cm/sec^2.

For retaining walls with relatively less importance, the aseismic design is usually omitted by assuming that a wall that is designed against static loads has a safety margin and the margin would cover the additional resistance required against seismic loads (e.g., JRA, 1987). In many of the design specifications or guidelines in Japan for relatively important retaining walls, the Mononobe-Okabe method has been adopted together with the assumption of hydrostatic distribution to evaluate the seismic active earth pressure. Alternatively, the trial wedge method, which is in principle equivalent to the Mononobe-Okabe method when the surface of the backfill soil is flat, has been employed for backfill soil having an irregular cross-section.

[1] Junichi Koseki, Associate Professor, Institute of Industrial Science, University of Tokyo

[2] Fumio Tatsuoka, Professor, Department of Civil Engineering, University of Tokyo

[3] Yulman Munaf, Graduate Student, Institute of Industrial Science, University of Tokyo

[4] Masaru Tateyama, Chief Engineer, Railway Technical Research Institute

[5] Kenichi Kojima, Engineer, Railway Technical Research Institute, ditto

In these aseismic design procedures, relatively small values are assigned for the shear resistance angle of the backfill soil. These shear resistance angles are apparently lower than the peak angles of the backfill that is compacted to a dense state as specified by the design specifications; rather these values are similar to the residual angles. The use of such low friction angles may lead to a conservative aseismic design or may be balanced with use of relatively low design seismic loads. On the other hand, the assumption of the hydrostatic distribution may be less conservative by itself, as mentioned before.

The 1995 Hyogoken-Nanbu earthquake caused severe damage to a number of conventional type retaining walls, particularly gravity-type retaining walls, for railway and road embankments (Tatsuoka et al., 1995, 1996a and 1996b). The peak ground acceleration (PGA) at the sites of these severely damaged retaining walls were estimated to be very high, up to 800 cm/sec^2 (Koseki et al., 1996a and 1996b). On the other hand, even when subjected to such a high seismic load, some modern reinforced-concrete (RC) retaining walls and geosynthetic-reinforced soil retaining walls located at Tanata and other locations performed satisfactorily (Tatsuoka et al, 1996a). To understand such behavior of damaged and undamaged retaining walls which experienced very high seismic loads, stability analysis of these retaining walls using a high seismic coefficient that exceeds, for example, 0.5 is required (Koseki et. al., 1996b).

After the earthquake, a two stage aseismic design procedure based on two different levels for the combination of seismic load and expected structural performance has been proposed to be applied to several types of civil engineering structures (JSCE, 1996), and retaining walls are not an exception. For example, the standard horizontal design seismic coefficient employed to evaluate the earth pressure during earthquake is assigned 0.2 (level 1) and 0.4 (level 2) for the aseismic design of retaining walls for railways on serviceability limit and ultimate limit conditions, respectively (RTRI, 1997).It is, therefore, necessary to develop a new procedure, which can rationally evaluate the active earth pressure for relatively high seismic loads.

In this paper, a modified procedure to evaluate active earth pressure is proposed based on the pseudo-static and limit-equilibrium approach. Differently from the Mononobe-Okabe method, it considers the effects of strain localization in the backfill soil and associated post-peak reduction in the shear resistance along a previously formed failure plane. Possible advantages of the proposed procedure over the Mononobe-Okabe method are also discussed.

MONONOBE-OKABE METHOD

The Mononobe-Okabe method (denoted herein as "M-O method") considers effects of the inertia force acting uniformly in the backfill soil having a Coulomb type soil wedge with its horizontal and vertical components $k_h \cdot W$ and $k_v \cdot W$, respectively (see Fig. 1, where W is the self weight of the soil wedge, k_h and k_v are the horizontal and vertical seismic coefficients. The total active earth pressure P_a can be evaluated as;

$$P_a = 1/2 \cdot \gamma H^2 \cdot (1 - k_v) \cdot K_a \qquad (1)$$

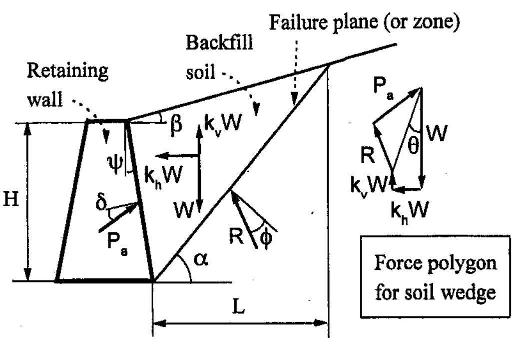


Fig. 1. Schematic force diagram by the Mononobe-Okabe method

where γ is the unit weight of the backfill soil, H is the total height of the retaining wall, and K_a is the active earth pressure coefficient calculated as;

$$K_a = \frac{\cos^2(\phi - \psi - \theta)}{\cos\theta \cdot \cos^2\psi \cdot \cos(\delta + \psi + \theta)\left[1 + \sqrt{\dfrac{\sin(\phi + \delta)\cdot \sin(\phi - \beta - \theta)}{\cos(\delta + \psi + \theta)\cdot \cos(\psi - \beta)}}\right]^2} \qquad (2)$$

where ϕ is the soil shear resistance angle which is uniform and isotropic in the backfill, δ is the frictional angle at the interface between the back face of the retaining wall and the backfill soil, ψ is the inclination of the back face of the retaining wall measured from the vertical direction, β is the angle of the surface slope of the backfill soil measured from the horizontal direction, and θ denotes the direction of the total of the inertia force and the self weight of the soil wedge measured from the vertical, which is given by;

$$\theta = \tan^{-1}(k_h/(1-k_v)) \qquad (3)$$

The angle α, measured from the horizontal direction, defines the direction of the failure plane, which is the bottom plane of the critical soil wedge mobilized at active failure condition. The angel α can be calculated from;

$$\cot(\alpha - \beta) = -\tan(\phi + \delta + \psi - \beta) + \sec(\phi + \delta + \psi - \beta)\cdot\sqrt{\frac{\cos(\psi + \delta + \theta)\cdot \sin(\phi + \delta)}{\cos(\psi - \beta)\cdot \sin(\phi - \beta - \theta)}} \qquad (4)$$

Then the ratio of the failure zone length L in the backfill soil, which is defined in Fig. 1, to the wall height H is given by;

$$L/H = (1 + \tan\psi \cdot \tan\beta)/(\tan\alpha - \tan\beta) \qquad (5)$$

As an example, the active earth pressure coefficient K_a and the ratio L/H are plotted versus the horizontal seismic coefficient k_h, respectively, in Figs. 2 and 3. For this case, k_v, ψ, β and δ are all zero and the value of ϕ is set equal to 30 and 50 degrees. With a constant ϕ, the value of K_a gradually increases with an increase in k_h. There are applicable upper limits of k_h in the M-O method, beyond which K_a cannot be evaluated

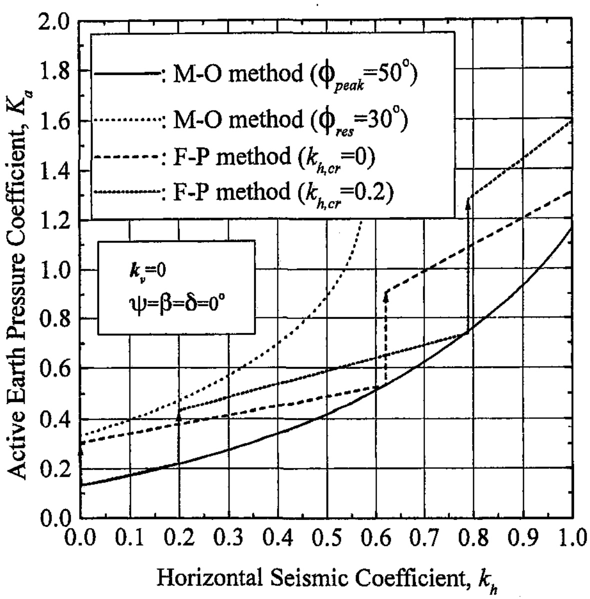


Fig. 2. Active earth pressure coefficient by the Mononobe-Okabe and the proposed methods

where the term of " ϕ - β - θ " in the square-root term in Eq. (2) becomes negative.

As seen from Fig. 3, the ratio L/H also increases as k_h increases. The rate of increase in L/H is accelerated, and the value of L/H becomes an unrealistically large value when k_h approaches and reaches the applicable limit explained above.

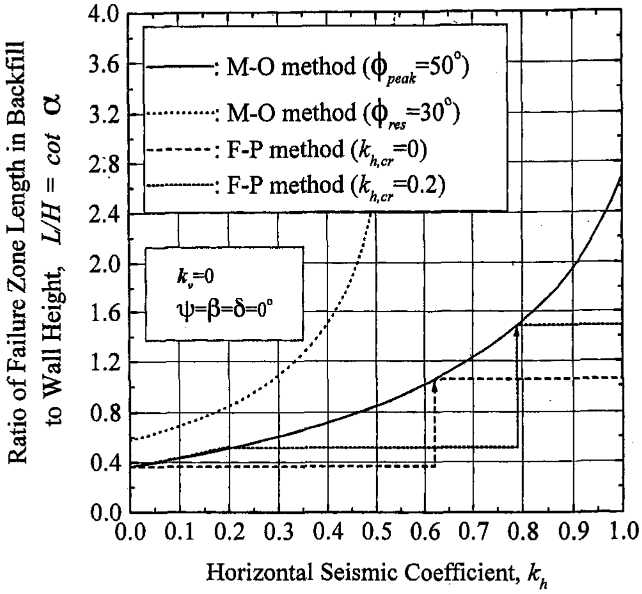


Fig. 3. Ratio of failure zone length in backfill soil to wall height by the Mononobe-Okabe and the proposed methods

EFFECTS OF STRAIN LOCALIZATION

In the M-O method, the shear resistance of the backfill soil is assumed to be uniform, isotropic and constant. It has been shown, however, that the behavior of a soil mass is affected by such factors as strength anisotropy, progressive failure and strain localization. Among them, effects of strain localization into a shear band (or a failure plane) and associated strain-softening in the shear band will be considered in the proposed procedure. Effects of essential difference between the actual dynamic behavior of the wall-soil system and its modeling based on the pseudo-static and limit-equilibrium approach is beyond the scope of the present paper.

It has been shown by Yoshida et al. (1994) and Yoshida and Tatsuoka (1997), based on results from a series of plane strain compression tests on dense sands and gravels, that the relative displacement in the direction parallel to the shear band which is enough to reduce the mobilized shear resistance angle from the peak value to the residual value is rather proportional to the particle size and about 5 to 10 times of its mean diameter D_{50}. Based on results from dynamic centrifuge tests on retaining wall models, Bolton and Steedman (1985) also showed that the shear resistance angle mobilized along a failure plane, which was formed in the backfill sand by shaking, dropped from 50 degrees to 33 degrees by a relative displacement of the order of 10 times the mean particle diameter. These results indicate that in full-scale field cases, the drop of soil shear strength from the peak to residual values is very fast.

The amount of outward wall displacement to trigger the active failure of the backfill associated with the mobilization of the peak friction angle in the shear band (or failure plane) is known to be very small; for a wall rotating about its base, the outward displacement at the wall top is about 0.1 % of the wall height from the at rest condition (Terzaghi, 1920). For actual retaining walls, considering their finite bending stiffness and their relatively low resistance against external instability (sliding, overturning and loss of bearing capacity) compared to that against internal instability (physical damage to the wall body), it is likely that only slight deformation or displacement of the wall is enough to trigger active failure in the backfill soil. Therefore, the active failure in the backfill may occur at a seismic level which is far below the level where the ultimate external failure of the wall takes place. It would be also the case with very stiff and statically very stable walls, although the difference between the seismic load level of the active failure in the backfill and the level of the ultimate external wall failure would be smaller. Note that the active failure in the backfill and the ultimate failure of wall by, for example, complete overturning or unstable large outward sliding at the wall base, are different processes; in most cases, the latter type of failure occurs after the former type of failure occurred as schematically shown in Fig. 4. The solid lines in this figure indicate the relationships between the outward displacement of the wall top δ and the active earth pressure at different seismic load levels, and the dashed line indicates the relationship between δ and the wall resistance such as the friction at the wall base against the earth pressure. Intersection of these lines at a certain seismic load was denoted as an equilibrium point at that condition, which moves from right to left in the figure when the seismic load increases, finally reaching the ultimate external wall failure condition. It should be noted that the post-peak reduction of soil shear resistance in the shear band or along the failure plane, which has been formed by the previous active failure (denoted as "initial active failure") prior to the ultimate external wall failure, may affect the consecutive mobilization of earth pressure at higher seismic loads, as illustrated below.

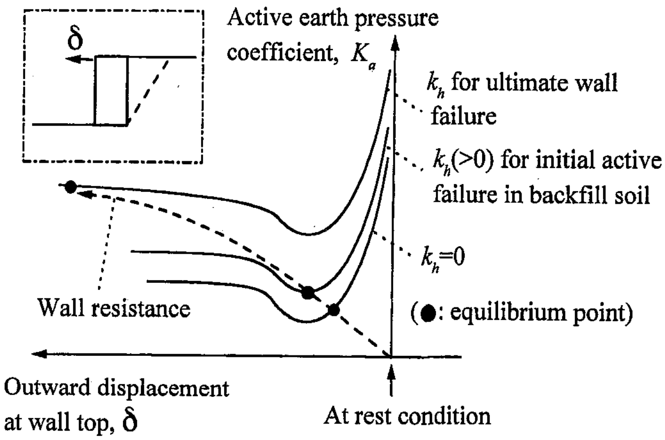
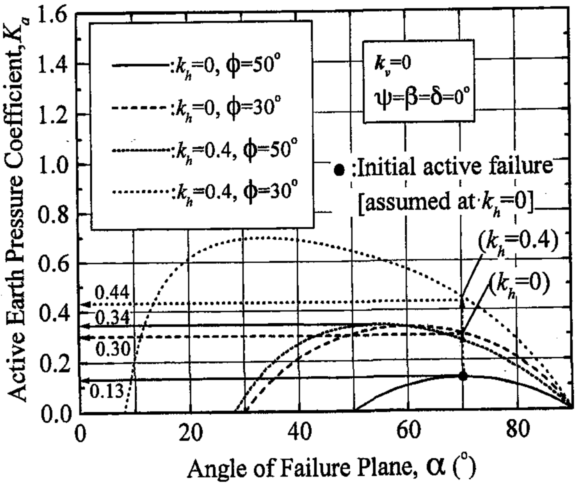


Fig. 4. Schematic relationship between displacement at wall top and active earth pressure coefficient

As an example, the active earth pressure coefficient K_a was calculated by changing the failure plane angle α based on the force equilibrium as shown in Fig. 1. The results are plotted versus α in Figs. 5 (a) and (b), where k_v, ψ, β and δ are all set zero for simplicity and ϕ is assigned to be either 30 or 50 degrees as peak (ϕ_{peak}) and residual (ϕ_{res}) values for typical dense sands. For each value of ϕ, k_h is set 0 and 0.4 in Fig. 5 (a) and 0.62 and 0.8 in Fig. 5 (b). The maximum values of K_a as indicated by solid horizontal arrows for cases of ϕ=50 degrees with k_h = 0 to 0.8 are, in principle, equal to those obtained by the M-O method with the same ϕ, while those values cannot be evaluated for cases of ϕ=30 degrees with k_h =0.62 and 0.8, because these cases are out of the applicable limit of the M-O method as mentioned previously.

Here, an assumption that the initial active failure occurs at k_h =0 is employed. Then a failure plane (or a shear band) should be formed at an angle of α=70 degrees, which in this case is equal to "45 degrees + (ϕ_{peak}) /2" as derived from the Coulomb's active earth pressure theory with zero values of k_h, k_v, ψ, β and δ. Along this failure plane, the shear resistance angle decreases to ϕ_{res}=30 degrees by a slight movement of the wall, while along other potential failure planes, the maximum possible shear resistance angle is still kept at ϕ_{peak}=50 degrees. This change results in an increase in the earth pressure at a constant k_h after the initial active failure as indicated by the lower vertical dotted arrow shown in Fig. 5 (a); i.e., the earth pressure coefficient increases from 0.13 to 0.3.

When seismic load equivalent to k_h =0.4 is applied, the mobilized earth pressure coefficient should be the larger one of "the value of K_a obtained for $\phi = \phi_{res}$ with k_h =0.4 and α=70 degrees" and "the maximum value of K_a obtained for $\phi = \phi_{peak}$ with k_h =0.4." For the latter case, α is equal to the value α_{cr} which gives the maximum value of K_a, and the α_{cr} value becomes smaller than 70 degrees. Since the former and the latter values are 0.44 and 0.34 as seen from Fig. 5 (a), the earth pressure coefficient mobilized at k_h =0.4 becomes 0.44, which is controlled by the failure plane previously formed at the initial active failure. This failure mode will be predominant until k_h reaches 0.62, at which both "the value of K_a for $\phi = \phi_{res}$ with k_h =0.62 and α=70 degrees" and "the maximum value of K_a for $\phi = \phi_{peak}$ with k_h =0.62 and $\alpha = \alpha_{cr}$" are equal to be 0.53, as indicated in Fig. 5 (b). At this condition of k_h =0.62, secondary active failure will be triggered by forming another failure plane with an angle of α =44 degrees. Then the earth pressure coefficient will increase to 0.91 because of the reduction of the shear resistance angle along this secondary failure plane from ϕ_{peak} to ϕ_{res}.



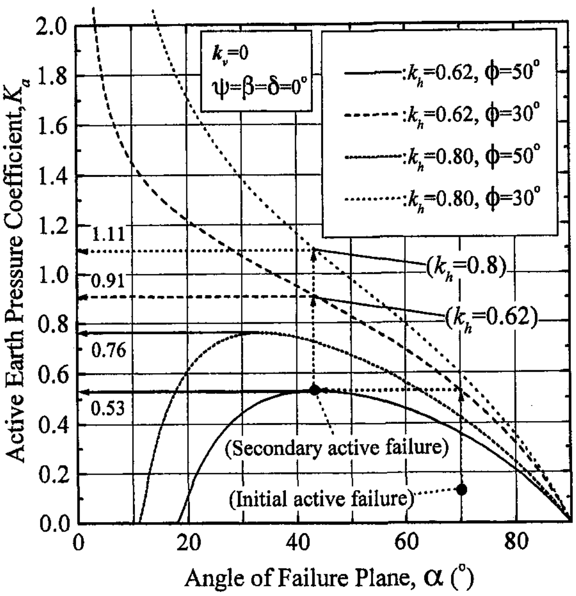


Fig. 5. Active earth pressure coefficient calculated from force equilibrium for ϕ=30°, 50° and δ=0°; (a) k_h =0, 0.4; (b) k_h =0.6, 0.8

This secondary failure plane will control the consecutive behavior in the same manner as above; for example, as shown in Fig. 5 (b), the earth pressure coefficient at k_h =0.8 will become 1.11 when this secondary failure plane is mobilized for $\phi = \phi_{res}$, which is larger than 0.76 as the maximum value for $\phi = \phi_{peak}$ with $\alpha = \alpha_{cr}$, which is not equal to 44 nor 70 degrees.

In summary, when the effects of strain localization are considered in the pseudo-static and limit-equilibrium based approach, the failure plane formed by the initial active failure in the backfill soil will control the consecutive mobilization of earth pressure at higher seismic loads until the secondary active failure occurs along another failure plane, which is deeper than the initial one, as schematically illustrated in Fig. 6.

PROPOSED PROCEDURE

For the proposed procedure, first, the values of ϕ_{peak} and ϕ_{res} of the backfill soil should be evaluated by a proper method, either empirically or experimentally, reflecting the degree of compaction of the backfill. Second, the condition of the initial active failure in the backfill should be evaluated by using the M-O method for $\phi = \phi_{peak}$ to obtain the angle α of the initial failure plane. The relative stiffness of walls and their resistance against external instability should be taken into account when evaluating the initial active failure. Third, the active earth pressure coefficient K_a mobilized by the initial failure plane is calculated as;

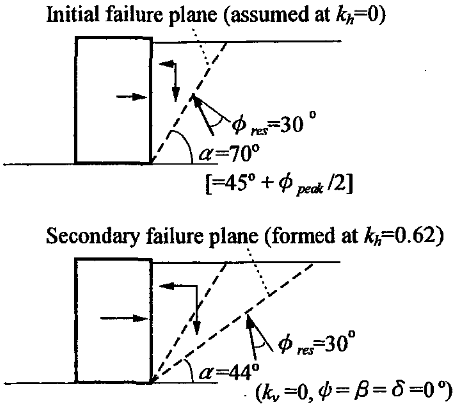


Fig. 6. Initial and secondary active failures considered in the proposed method for ϕ_{peak}=50°, ϕ_{res}=30° and δ=0°

$$K_a = \frac{\cos(\alpha - \phi)\cdot(1+\tan\psi\cdot\tan\alpha)\cdot(1+\tan\psi\cdot\tan\beta)\cdot(\tan(\alpha-\phi)+\tan\theta)}{\cos(\alpha-\phi-\psi-\delta)\cdot(\tan\alpha-\tan\beta)} \tag{6}$$

where a reduced shear resistance angle ϕ equal to ϕ_{res} is used, and the failure plane angle α is fixed to the critical value α_{cr} for the initial active failure. This coefficient K_a is compared with the one evaluated by the M-O method with $\phi = \phi_{peak}$. If the former value is smaller than the latter, the secondary active failure is judged to have already occurred, for which the critical angle α_{cr} for the secondary failure plane should be re-evaluated in order to calculate the coefficient K_a mobilized by this newer failure plane. Otherwise, the coefficient K_a calculated for the initial failure plane is considered to be still mobilized.

Values of K_a and $L/H = \cot\alpha$ evaluated by the proposed procedure based on the fixed failure plane angle (denoted as "F-P method") are plotted versus k_h in Figs. 2 and 3, respectively, where ϕ_{peak}=50 degrees, ϕ_{res}=30 degrees, and k_v, ψ, β and δ are all set zero similarly to the results by the M-O method as also shown in these figures. The parameter $k_{h,cr}$ denotes the horizontal seismic coefficient to trigger the initial active failure, which was arbitrarily assigned either 0 or 0.2. Corresponding values evaluated by the M-O methods with $\phi = \phi_{peak}$ and $\phi = \phi_{res}$ are also shown for reference. The values of K_a when k_h =0, 0.4, 0.62 and 0.8 for the case of $k_{h,cr}$ =0 are equal to those indicated in Figs. 5 (a) and (b) because all the assumptions are identical. It is also seen that the results for the case of $k_{h,cr}$ =0.2 are different from those for $k_{h,cr}$ =0 and that their difference becomes larger as k_h increases.

Based on the results shown in Figs. 2 and 3, the following advantages of the proposed method over the M-O method can be expected;

(1) It evaluates an active earth pressure coefficient K_a which is larger than that predicted by the M-O method with $\phi = \phi_{peak}$; the latter method underestimates the actual one because the post-peak reduction of the shear resistance in the backfill soil is not considered. On the other hand, it evaluates a K_a value which is smaller than that predicted by the M-O method

with $\phi = \phi_{res}$; the latter value is too conservative and cannot rationally reflect differences in the ϕ_{peak} value for different compaction levels of the backfill.

(2) It can evaluate the active earth pressure coefficient at large k_h values where the M-O method with $\phi = \phi_{res}$ (as employed widely in current practice) is not applicable.

(3) The failure zone length L is reduced to be considerably smaller than that predicted by the M-O method with $\phi = \phi_{res}$, and even smaller than that predicted by the M-O method with $\phi = \phi_{peak}$.

On the other hand, the largest disadvantage of the proposed method may be that the results are affected by estimated initial active failure conditions (i.e., the value of $k_{h,cr}$), while at this moment, the method to evaluate the $k_{h,cr}$ value has not been established. Further study on this point will be required. Investigations to validate the proposed procedure are in progress by means of model tests (Munaf, et al., 1997) with respect to the active earth pressure coefficient K_a and the failure plane angle α, which will be reported elsewhere in the future.

CONCLUSIONS

Based on the pseudo-static and limit-equilibrium approach, seismic active earth pressure was calculated by considering the effects of strain localization in the backfill soil and associated post-peak reduction in the shear resistance angle from ϕ_{peak} to ϕ_{res} along a previously formed failure plane. It was shown that the failure plane formed by the initial active failure in the backfill soil will control the consecutive mobilization of earth pressure at higher seismic load levels until the secondary active failure occurs along another new failure plane, which is deeper than the initial one.

Incorporating the above-mentioned effects, a modified procedure was proposed to evaluate the active earth pressure. Compared to the Mononobe-Okabe method, the proposed method can rationally reflect the differences in ϕ_{peak} for different backfill conditions, while it yields reasonable seismic active earth pressure, which is smaller than that predicted by the Mononobe-Okabe method with $\phi = \phi_{res}$. The proposed method can also provide a realistic and reduced size of active failure zone in the backfill soil compared to that predicted by the Mononobe-Okabe method.

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