THE INFLUENCE OF STRAIN LOCALIZATION AND PHASE ON THE ESTIMATION OF EARTH PRESSURES BY PSEUDO-DYNAMIC METHOD

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ABSTRACT: The paper focuses on the determination of seismic active earth pressure coefficients when subjected to high earthquake loading ($a_h = 0.4-1.0g$). Planar failure surface has been used in conjunction with the modified pseudo-static approach to compute the seismic active earth pressures on the retaining wall. The proposed modified Mononobe-Okabe 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, phase difference in shear waves and soil amplification along with the horizontal seismic accelerations. The influence of various design parameters on the magnitude of seismic active earth pressure is presented.

1. INTRODUCTION

A retaining wall should be designed so as to withstand the damage from the lateral earth pressure and earthquake loading. In the following sections the modified pseudo-static method for the estimation of seismic active earth pressure coefficients is discussed.

1.1 Modified Pseudo-Static Procedure Considering the Influence of Phase Change

Pseudo-static method, known as the Mononobe-Okabe (M-O) method (Kramer, 2003) is a popular method and assumes that the soil block is rigid and therefore acceleration is constant throughout the block. However, it does not represent the actual oscillatory nature of earthquake loading. Additionally, this method does not account for certain aspects such as earthquake duration, frequency of motion, effects of phase difference in the waveform between the base and crest of the abutment and the amplification of accelerations within the backfill. To addresses the above aspects, Steedman & Zeng (1990) modified the pseudo-static method. Choudhury & Nimbalkar (2006) computed the active earth pressures considering both shear and primary waves propagating through the backfill with variation in time by considering sinusoidal horizontal and vertical seismic accelerations for rigid retaining walls. Basha & Babu (2008) and Basha (2009) proposed an approach for computing seismic passive earth pressure coefficients using composite failure surface (log-spiral and planar) based on the modified pseudo-static method which takes into account the influence of phase change in the shear and primary waves.

Further, Basha & Babu (2009a,b) presented a formulation for the calculation of sliding component of response of bridge abutments and seismic reliability assessment of reinforced soil structures respectively using modified pseudo-static method considering the amplification of vibrations. However the above studies did not take into account the effect of strain localization in the shear zone and the effect of the resultant strain softening as discussed in the following section.

1.2 Modified Pseudo-Static Procedure Considering the Effects of Strain Localization

The M-O method further assumes that the shear strength of backfill is isotropic and constant. In reality, the internal friction angle of soil along a failure surface decreases from the peak strength to residual strength. For example, Bolton & Steedman (1985) conducted shake table tests on cantilever retaining wall and observed that the shear resistance angle mobilized along a failure plane reduces from 50° to 33°. Koseki et al. (1998) proposed a modified M-O procedure that takes into consideration the effect of strain localization in the shear zone and the effect of the resultant strain softening as illustrated below:

1. For the calculation of the angle of failure plane, the peak internal friction angle of backfill ($\phi_{peak}$) should be employed. In addition, it assumes that the first failure plane forms at $k_h = 0.0$ and the same failure plane continues to be activated up to certain level of $k_h$ value. However, as the seismic coefficient, $k_h$ is increased to a certain level (i.e. 0.6 – 0.8), another failure plane will be activated.

2. For the computation of active earth pressure, the residual internal friction angle ($\phi_{res}$) should be used, since the post peak reduction of internal friction angle evolves after the shear band formation.
As pointed out by Koseki et al. (1998), the above proposed hypothesis has a few advantages: 1) It provides active earth pressure coefficients even at high seismic loads (0.6 to 0.8 g are considered in the seismic design against the level 2 earthquakes, and 2) It provides more realistic values of seismic active earth pressure coefficients and size of active failure zone than the conventional methods. For the practical considerations, Shirato et al. (2002, 2006) further modified the above stated procedure which was introduced in Japanese Specifications for Highway Bridges (JRA, 2002) as follows. It is suggested to evaluate the angles of the initial failure plane at $k_h = 0.0$ and second failure plane at $k_h = 0.42$, but consider only the second failure plane for the computation of seismic active earth pressure coefficients for all values of $k_h$. Accordingly, JRA (2002) has adopted the following two equations to evaluate the seismic active earth pressure coefficients ($K_{ae}$) in the revised seismic design specifications of highway bridges:

\[ K_{ae} = 0.22 + 0.81k_h \quad \text{for dense sand and gravel} \quad (1) \]
\[ K_{ae} = 0.26 + 0.97k_h \quad \text{for dense sandy soils} \quad (2) \]

**1.3 Seismic Stability of Retaining Walls**

The review of the above literature indicates that the seismic stability of gravity retaining walls should consider the revisions suggested by Steedman & Zeng (1990) in M-O method, like the influence of phase change in the horizontal acceleration in the backfill behind a retaining wall as shear waves propagate from the base of wall towards ground surface, and amplification of soil vibration. In addition, it should consider the revised seismic design specifications suggested in JRA (2002) like strain localization in the backfill soil and associated post-peak reduction in the internal friction angle from peak to residual values along a previously formed failure plane. Hence, a proper design procedure is presented in this paper to address these issues.

**2. SEISMIC ACTIVE EARTH PRESSURE**

During earthquakes, the peak values of horizontal inertial force, vertical inertial force and active earth pressure may not be activated simultaneously and the phase differences among these three components should be largely different among different earthquakes. In addition, Seed & Whitman (1970) reported that for most earthquakes the horizontal acceleration components are considerably greater than the vertical components and concluded that vertical acceleration can be neglected for practical purposes. Accordingly, following the suggestions of Seed & Whitman (1970), Koseki et al. (1998), Shirato et al. (2002, 2006) and Japan Road Association JRA (2002), the vertical ground acceleration is neglected in the present study. In the following sections, the seismic active earth pressure due to weight and inertia of the wedge ‘CGD’ (Fig. 1) behind the retaining wall is presented using modified pseudo-static procedure as explained in the above sections.

**2.1 Phase Angle and Amplification Effect on Sinusoidal Accelerations**

The present method considers finite shear wave and primary velocity within the backfill behind the abutment. Similar to Steedman & Zeng (1990), Choudhury & Nimbalkar (2006), Basha (2009) and Basha & Babu (2009a,b), it is assumed that the horizontal seismic acceleration in soil vary linearly from the input seismic acceleration at the base to the higher value at the top of the wall (Fig. 2).

Period of lateral shaking can be defined as

\[ T = \frac{2\pi}{\omega} \quad (3) \]

where $\omega$ = angular frequency of the base shaking. The horizontal sinusoidal acceleration at any depth, $z$ below the top of the wall and time ($t$) with soil amplification factor ($f$) can be expressed as follows:

\[ a_s(z,t) = \left[ 1 + \frac{H-z}{H} (f-1) \right] k_g \sin \left( \omega t - \frac{H-z}{v_s} \right) \quad (4) \]

where $v_s$ is the shear wave velocity propagating through the backfill. In the study, it is assumed that the base of the wall is subjected to horizontal sinusoidal acceleration with amplitude of accelerations, $k_g$, where ‘$g$’ is the acceleration due to gravity. The details of the calculation of horizontal inertial force due to triangular wedge $CGD$ ($Q_{CGD}$) are reported in Choudhury & Nimbalkar (2006). The forces acting on the triangular wedge $CGD$ are shown in Figure 2. The estimated values of $Q_{CGD}$ are useful in the derivation of seismic active earth pressure ($P_{ae}(t)$) as explained below. The weight of the triangular wedge $CGD$ can be written as,

\[ W_{CGD} = \frac{1}{2} \gamma H^2 \cot \alpha \quad (5) \]

The seismic active earth pressure ($P_{ae}(t)$) can be obtained by resolving the forces on the wedge, $CGD$ horizontally and vertically as follows: By considering the horizontal equilibrium condition ($\sum H = 0$), we get:

\[ Q_{ae,CGD} + R \sin(\alpha - \phi) = P_{ae}(t) \cos \delta \quad (6) \]
\[ R \sin(\alpha - \phi) = P_{ae}(t) \cos \delta - Q_{ae,CGD} \quad (7) \]
where \( \phi \) = friction angle of the backfill, \( \delta = \) soil-abutment interface friction angle, \( \alpha = \) angle of the failure plane with horizontal, and \( R = \) resultant force acting on the triangular wedge ‘CGD’. By considering the vertical equilibrium condition \((\Sigma V = 0)\) for the wedge CGD, we have

\[
R \cos(\alpha - \phi) + P_a(t) \sin \delta = W_{CGD}
\]

(8)

\[
R \cos(\alpha - \phi) = W_{CGD} - P_a(t) \sin \delta
\]

(9)

solving the Eqs. (7) and (9), \( P_{aw}(t) \) is given by the following equation:

\[
P_{aw}(t) = \frac{\tan(\alpha - \phi)W_{CGD} + Q_{a,CGD}}{\cos \delta + \sin \delta \tan(\alpha - \phi)}
\]

(10)

2.2 Location of the Critical Failure Surface by Optimization

In this section, the state at which \( P_{aw}(t) \) achieves a maximum value and the failure surface corresponding to this state i.e. critical failure surface are determined. The geometry of the planar failure surface \((GD)\) is governed by the angle \((\alpha)\) as shown in Figure 2. Following the suggestions of JRA (2002) and Shirato et al. (2006), the condition of the initial active failure plane for \( k_h = 0.0 \) and second active failure plane for \( f \times k_h = 0.42 \) (note here that the influence of amplification factor is also taken into consideration) in the backfill should be evaluated for \( \phi = \phi_{peak}, \delta = \phi_{peak}/2 \) to obtain the critical angles \((\alpha_{cr}),\) and \( t/T \) ratios of the initial and second failure planes subjected to bound constraints such as \( 0^\circ < \alpha < 90^\circ \) and \( 0 < t/T < 1 \). However as suggested by Shirato et al. (2006), the critical values, \( \alpha_{cr}, \) and \( t/T \) ratio corresponding to second failure plane are taken into consideration in estimating the values of \( K_{aw} \) using \( \phi = \phi_{peak}, \delta = \phi_{peak}/2 \) and for all values of \( k_h \) with the help of the following equation:

\[
K_{aw} = \frac{\tan(\alpha_{cr} - \phi_{cr}) \cot \alpha_{cr} + Q_{a,CGD}}{0.5 H^2 \cos \delta + \sin \delta \tan(\alpha_{cr} - \phi_{cr})}
\]

(11)

Refer the appendix for the derivation of \( Q_{a,CGD} \) and \( K_{aw}. \)

3. RESULTS AND DISCUSSION

This study includes the influence of the strength parameters of backfill and characteristics of earthquake ground motions. The parameters values involved in the study are the unit weight \((\gamma) = 18 \text{kN/m}^3, \phi_{peak} = 50^\circ, \phi_{res} = 35^\circ, \delta/\phi = 0.5, \ k_h = 0.0-1.0, \ f = 1.0, \ v_s = 65 \text{m/s}, \ v_w = 1500 \text{m/s}, \ (\phi), \ H = 6 \text{m}, \ T = 0.3, \ H/\lambda = H/(Tv_s) = 0.2 - 0.5 \) and \( H/\phi = H/(Tv_w) = 0.3. \)

3.1 Comparison of M-O Method, Modified M-O Method by Shirato et al. (2006) and Present Method

The seismic active earth pressure coefficients \((K_{aw})\) calculated using M-O method (refer the appendix), modified M-O method by Shirato et al. (2006) and present method are plotted in Figure 3 for the horizontal seismic coefficient \((k_h) = 0.0 \) to 1.0. It is assumed that the internal friction angles of dense sand and gravel are \( \phi_{peak} = 50^\circ \) and \( \phi_{res} = 35^\circ. \) It can be observed that for constant value of \( \phi = \phi_{res}, \) the magnitudes of \( K_{aw} \) increases gradually with increase in \( k_h \) value until \( \phi_{res} > \tan^{-1}(k_h), \) beyond which \( K_{aw} \) value cannot be evaluated when the term \((\phi_{res} > \tan^{-1}(k_h)) \) in the square root becomes negative. It can be noted that Shirato et al. (2006) approach predicts the critical angle \((\alpha_{cr}),\) for second active failure plane as \( 49.7^\circ \) for \( \phi_{peak} = 50^\circ, \delta = \phi_{peak}/2 \) and \( k_h = 0.42, \) (Fig. 3) and the following linear function using \( \phi_{res} = 35^\circ, \delta = \phi_{res}/2 \) was proposed to estimate the magnitude of \( K_{aw}:\)

\[
K_{aw} = 0.22 + 0.81 k_h
\]

(12)

However the proposed Eq. (13) (JRA, 2002) does not take into consideration the influence of phase change in horizontal acceleration in the backfill as shear waves propagate from the base of wall towards ground surface, and amplification of soil vibration. Hence, the present study incorporates the effect of phase change in Shirato et al. (2006) approach and the following differences have been noted. It can be observed from Figure 3 that the present study predicts the critical values for second active failure plane as
\( \alpha_{ci} = 52^\circ \) and \( t/T = 0.45 \) for \( \phi_{\text{peak}} = 50^\circ \), \( \delta = \phi_{\text{peak}} / 2 \), \( k_h = 0.42 \), \( H/\lambda = 0.3 \) and \( f = 1.0 \). Substituting the values of \( \alpha_{ci} = 52^\circ \) and \( t/T = 0.45 \) for \( \phi_{\text{peak}} = 35^\circ \), \( \delta = \phi_{\text{peak}} / 2 \), \( H/\lambda = 0.3 \) and \( f = 1.0 \) in Eq. (12), the following linear function has been proposed to estimate the value of \( K_{ae} \):

\[
K_{ae} = 0.22 + 0.68k_h
\]  

(13)

Fig. 3: Comparison of \( K_{ae} \) values using Mononobe-Okabe (M-O) method, Modified M-O method by Shirato et al. (2006) and present method for \( \phi_{\text{peak}} = 50^\circ \), \( \phi_{\text{res}} = 35^\circ \) and \( \delta/\phi = 0.5 \)

It can also be noted from Figure 3 that the values of \( K_{ae} \) predicted by the present method are lower than the modified M-O method by Shirato et al. (2006) approach (which was adopted in JRA, 2002). As for an illustration from Figure 3, for the case of \( k_h = 0.6 \), the estimated values of \( K_{ae} \) are 1.05, 0.70 and 0.63 with M-O method, Modified M-O method of Shirato et al. (2006) and present method respectively. This difference occurs due to the limitations of pseudo-static analysis such as non-consideration of the effect of time and phase difference due to finite shear wave velocity.

4. CONCLUSIONS

A formulation is outlined in this paper for the computation of seismic active earth pressure considering the influence of oscillatory nature of earthquake loading (like time, soil amplification and phase difference in the shear wave propagating through the backfill behind the abutment), strain localization in the shear zone and the effect of the resultant strain softening in the framework of modified pseudo-static method. Further, the procedure for estimating the point of application of total thrust is presented. The methodology presented in the paper provides a rational and systematic procedure for the determination of active earth pressure acting on retaining walls during earthquake conditions. The main findings of the present investigation are as follows: It is shown that the seismic active earth pressure coefficients (\( K_{ae} \)) predicted by the present method are lower than the modified Mononobe-Okabe earth pressure theory by Shirato et al. (2006) which does not take into account the effect of phase difference in body wave propagating through the backfill soil. Hence, addressing these issues, a new linear function has been proposed to evaluate more realistic values of \( K_{ae} \) with respect to the horizontal seismic acceleration coefficient.

REFERENCES


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