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Comparison of Pseudo-Static and Pseudo-Dynamic Methods for Seismic Earth Pressure on Retaining Wall

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ABSTRACT
Design of retaining wall needs the complete knowledge of earth pressures for both active and passive conditions. Under earthquake condition, the design requires special attention to reduce the devastating effect of this natural hazard. But under seismic condition, the available literatures mostly give the pseudo-static analytical value of the earth pressures as an approximate solution to the real dynamic nature of the complex problem. In the present work, a recently developed pseudo-dynamic method, which incorporates time dependent effect of applied earthquake load and effect of shear and primary waves, is applied to study effect of variation of parameters like soil friction angle, wall friction angle, time period of earthquake ground motion, seismic shear and primary wave velocities of backfill soil and seismic peak horizontal and vertical ground accelerations on the seismic earth pressures. Again a complete analysis between these two design methodologies shows that the time dependent non-linear behaviour of the pressure distribution obtained in the pseudo-dynamic method results more realistic design values of earth pressures under earthquake condition.

INTRODUCTION

Estimation of the seismic earth pressure is an important topic of research for the safe design of retaining wall in the seismic zone. It is a common practice to consider the seismic accelerations in both horizontal and vertical directions in terms of equivalent static forces, called pseudo-static accelerations. Using the pseudo-static approach, several researchers have developed different methods to determine the seismic earth pressure on a rigid retaining wall due to earthquake loading starting from the pioneering works by Okabe (1926) and Mononobe & Matsuo (1929), commonly known as Mononobe-Okabe method (see Kramer, 1996) based on the pseudo-static approach, which gives the linear earth pressure distribution in a very approximate way irrespective of static and seismic conditions. Kumar (2001) had determined the seismic passive earth pressure coefficients for sands using limit equilibrium method. Dewaikar & Halkude (2002) have proposed a pseudo- static numerical analysis of seismic active and passive thrust on retaining wall, using Kotter's equation. Kumar & Chitikela (2002) obtained the seismic passive earth pressure coefficients using method of characteristics. Madhav & Kameswara Rao (1969); Choudhury & Nimbalkar (2002); Choudhury (2004); Subba Rao & Choudhury (2005) have adopted limit equilibrium for determining individually the seismic passive earth pressure coefficients corresponding to unit weight, surcharge and cohesion components. Choudhury & Singh (2006) have determined active earth pressure coefficients under static and seismic conditions using modified Culmann method. However, all the above methods are based on pseudo-static method of analysis, which does not consider the time effect of the applied earthquake load and the effect of shear and primary waves passing through the soil media. To overcome these drawbacks, the analytical method based on pseudo-dynamic approach as given by Steedman & Zeng (1990) and modified by Choudhury & Nimbalkar (2005, 2006) is used for the present analysis for calculation of seismic passive and active earth pressure.

Steedman & Zeng (1990) considered in their analysis a vertical rigid retaining wall supporting a particular value of soil friction angle (ϕ) and a particular value of seismic horizontal acceleration ($k_h g$, where g is the acceleration due to gravity) only. Again they have considered effect of horizontal seismic acceleration due to vertically propagating shear waves through the backfill behind retaining wall. In an improvement over this method, Choudhury & Nimbalkar (2006) have incorporated effect of vertical seismic acceleration due to vertically propagating primary waves through the backfill soil. Again, they have studied the effect of various parameters such as wall friction angle (δ), soil friction angle (ϕ), shear wave velocity (V_s), primary wave velocity (V_p), both the horizontal and vertical seismic accelerations ($k_h g$ and $k_v g$) on the seismic active earth pressure behind a rigid retaining wall by the pseudo-dynamic. Choudhury & Nimbalkar (2005) have extended this modified work for estimation of seismic passive earth pressure.

In pseudo-dynamic method, vertically propagating shear and primary waves through the backfill generate vibrations in horizontal and vertical directions respectively. These horizontal and vertical vibrations correspond to horizontal and vertical time dependent seismic inertia forces respectively. Time dependent nature of these seismic inertia forces is considered in the present analysis.

In this paper, a complete analytical study describes the behaviour of seismic earth pressure distribution for different soil friction angle, wall friction angle, shear wave velocity, primary wave velocity and horizontal and vertical seismic accelerations for both active and passive conditions of earth pressures.

MATHEMATICAL MODEL

The pseudo-dynamic method of analysis considers finite shear and constrained modulus of the backfill soil leading to finite shear and primary wave velocity. A fixed base vertical cantilever rigid retaining wall of height H, supporting a cohesionless backfill material with horizontal ground is considered in the analysis as shown in Figs. 1 and 2. The shear wave velocity, V_s and primary wave velocity, V_p are assumed to act within the soil media due to earthquake loading. The period of lateral shaking, $T = 2\pi/\omega$, where ω is the angular frequency is considered in the analysis.

Let the base of the wall is subjected to harmonic horizontal seismic acceleration, a_h (= $k_h g$) and harmonic vertical seismic acceleration a_v (= $k_v g$), the accelerations at any depth z and time t, below the top of the wall can be expressed as follows,

$$a_h(z,\ t) = a_h \sin\omega\left[t - \frac{H-z}{V_s}\right] \quad (1)$$

$$a_v(z,\ t) = a_v \sin\omega\left[t - \frac{H-z}{V_p}\right] \quad (2)$$

The horizontal and vertical seismic accelerations acting on the soil wedge as described in Eqs. (1) and (2) are not constants but dependent on effect of both, time and phase difference in shear and primary waves propagating vertically through the backfill as proposed in the pseudo-dynamic method of analysis.

Whereas in pseudo-static method, horizontal and vertical accelerations are considered constant acting on the soil wedge with the neglect of time effect as shown below,

$$a_h = k_h . g \quad (3)$$

$$a_v = k_v . g \quad (4)$$

In the pseudo-dynamic method, as a special case, if the soil wedge is assumed to behave as rigid wedge having infinite shear and primary waves, then the pseudo-dynamic method of analysis reduces to pseudo-static method of analysis as shown below.

$$\lim_{v_s \to \infty} (Q_h)_{\max} = \frac{\gamma H^2 a_h}{2g\tan\alpha} = \frac{a_h}{g} W = k_h W \quad (5)$$

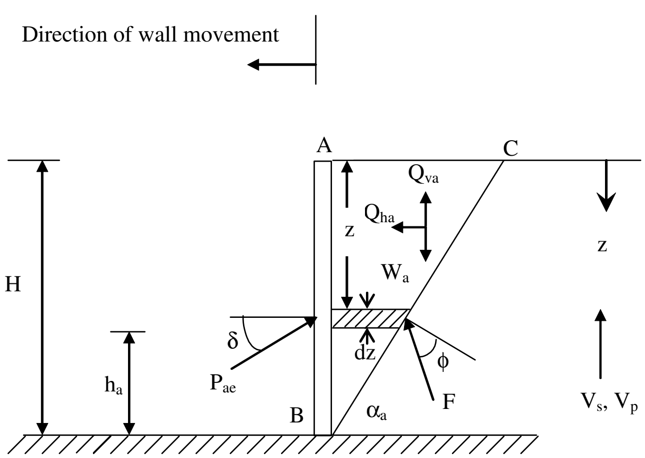

Figure 1. Model retaining wall considered for computation of pseudo-dynamic active earth pressure (Choudhury & Nimbalkar 2006).

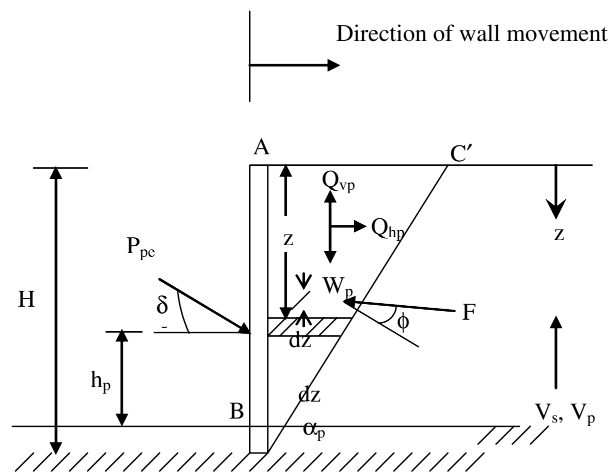

Figure 2. Model retaining wall considered for computation of pseudo-dynamic passive earth pressure (Choudhury & Nimbalkar 2005).

$$\lim_{v_P \to \infty} (Q_v)_{\max} = \frac{\gamma \, \mathrm{H}^2 \, \mathrm{a_v}}{2\mathrm{g}\tan\alpha} = \frac{\mathrm{a_v}}{\mathrm{g}} W = \mathrm{k_v} W \qquad (6)$$

where, Q_h and Q_v are horizontal and vertical seismic inertia forces respectively.

SEISMIC ACTIVE EARTH PRESSURE

Fig. 1 shows the active state of earth pressure acting on the rigid retaining wall. A planar failure surface BC at an inclination of a_a with respect to horizontal is considered in the analysis. In Fig. 1, W_a is the weight of the failure wedge, Q_{ha} and Q_{va} are the horizontal and vertical seismic inertia force components, F is the soil reaction acting at an angle of ϕ (soil friction angle) to the normal to the inclined failure wedge, P_{ae} is the total active thrust acting at height h_a from the base of the wall at an inclination of δ (wall friction angle) to the normal to the wall.

The mass of a thin element of wedge at depth z is

$$m_a(z) = \frac{\gamma}{\mathrm{g}} \frac{H - z}{\tan\alpha_a} dz \qquad (7)$$

where, γ is the unit weight of the backfill.

The weight of the whole wedge is,

$$W_a = \frac{1}{2} \frac{\gamma H^2}{\tan\alpha_a} \qquad (8)$$

The total horizontal inertia force acting on the wall can be expressed as

$$Q_{ha}(t) = \int_0^{\mathrm{H}} \mathrm{m_a(z).a_h(z,\,t)dz} = \frac{\lambda \, \gamma \, a_h}{4\pi^2 g \tan\alpha_a} [2\pi \mathrm{H}\cos\omega\zeta + \lambda(\sin\omega\zeta - \sin\omega t)] \qquad (9)$$

Again total vertical inertia force acting on the wall can be expressed as

$$Q_{va}(t) = \int_0^{\mathrm{H}} \mathrm{m_a(z).a_v(z,\,t)dz} = \frac{\eta \, \gamma \, a_v}{4\pi^2 g \tan\alpha_a} [2\pi \mathrm{H}\cos\omega\psi + \lambda(\sin\omega\psi - \sin\omega t)] \qquad (10)$$

Where, $\lambda = TV_s$ is the wavelength of the vertically propagating shear wave and $\eta = TV_p$, is the wavelength of the vertically propagating primary wave. And $\zeta = t - H/V_s$ and $\psi = t - H/V_p$. As the horizontal acceleration is acting from left to right and vice-versa and the vertical acceleration is acting from top to bottom and vice-versa, only the critical directions of $Q_{hs}(t)$ and $Q_{vs}(t)$ are considered to result the maximum seismic active earth pressure.

The total (static + seismic) active thrust, P_{ae} can be obtained by resolving the forces on the wedge and considering the equilibrium of the forces and hence P_{ae} can be expressed as follows,

$$P_{ae} = \frac{W_a \sin(\alpha_a - \phi) + Q_{ha}\cos(\alpha_a - \phi) + Q_{va}\sin(\alpha_a - \phi)}{\cos(\delta + \phi - \alpha_a)} \qquad (11)$$

where, W_a = Weight of the failure wedge in active case
α_a = Angle of inclination of the failure surface with the horizontal in active case
Q_{ha} = horizontal inertia force due to seismic accelerations respectively in active case
Q_{va} = vertical inertia force due to seismic accelerations respectively in active case

P_{ae} is maximized with respect to trial inclination angle of failure surface, α_a and then the seismic active earth pressure distribution, p_{ae} can be obtained by differentiating P_{ae} with respect to depth, z and can be expressed as follows,

$$p_{ae} = \frac{\gamma z}{\tan\alpha_a} \frac{\sin(\alpha_a - \phi)}{\cos(\delta + \phi - \alpha_a)} + \frac{k_h \gamma z}{\tan\alpha_a} \frac{\cos(\alpha_a - \phi)}{\cos(\delta + \phi - \alpha_a)} \sin\left[w\left(t - \frac{z}{V_s}\right)\right] + \frac{k_v \gamma z}{\tan\alpha_a} \frac{\sin(\alpha_a - \phi)}{\cos(\delta + \phi - \alpha_a)} \sin\left[w\left(t - \frac{z}{V_p}\right)\right] \qquad (12)$$

SEISMIC PASSIVE EARTH PRESSURE

Fig.2 shows the passive state of earth pressure/ resistance on the rigid retaining wall. Again, a planar failure surface BC¢ at an inclination of α_p with respect to horizontal is considered in the analysis. In Fig. 2, W_p is the weight of the failure wedge, Q_{hp} and Q_{vp} are the horizontal and vertical seismic inertia force components, F is the soil reaction acting at an angle of ϕ (soil friction angle) to the normal to the inclined failure wedge, P_{pe} is the total passive resistance acting at height h_p from the base of the wall at an inclination of δ (wall friction angle) to the normal to the wall.

For the thin element of thickness dz at depth z as shown in Fig. 2, mass is given by,

$$m_p(z) = \frac{\gamma}{\mathrm{g}} \frac{H - z}{\tan\alpha_p} dz \qquad (13)$$

The weight of the whole wedge is,

$$W_p = \frac{1}{2} \frac{\gamma H^2}{\tan\alpha_p} \qquad (14)$$

The total horizontal inertia force acting on the wall can be expressed as

$$Q_{hp}(t) = \int_0^{\mathrm{H}} \mathrm{m_p(z).a_h(z,\,t)dz} = \frac{\lambda \, \gamma \, a_h}{4\pi^2 g \tan\alpha_p} [2\pi \mathrm{H}\cos\omega\zeta + \lambda(\sin\omega\zeta - \sin\omega t)] \qquad (15)$$

Again total vertical inertia force acting on the wall can be expressed as

$$Q_{vp}(t) = \int_0^{\mathrm{H}} \mathrm{m_p(z).a_v(z,\,t)dz} = \frac{\eta \, \gamma \, a_v}{4\pi^2 g \tan\alpha_p} [2\pi \mathrm{H}\cos\omega\psi + \lambda(\sin\omega\psi - \sin\omega t)] \qquad (16)$$

As the horizontal acceleration is acting from left to right and vice-versa and the vertical acceleration is acting from top to bottom and vice-versa, only the critical directions of $Q_{hs}(t)$ and $Q_{vs}(t)$ are considered to result the minimum seismic passive earth pressure. The total (static + seismic) passive resistance, P_{pe} can be obtained by resolving the forces on the wedge and considering the equilibrium of the forces and hence P_{pe} can be expressed as follows,

$$P_{pe} = \frac{W_p \sin(\alpha_p + \phi) - Q_{hp}\cos(\alpha_p + \phi) - Q_{vp}\sin(\alpha_p + \phi)}{\cos(\delta + \phi + \alpha_p)} \quad (17)$$

where, W_p = Weight of the failure wedge in passive case

α_p = Angle of inclination of the failure surface with the horizontal in passive case

Q_{hp} = horizontal inertia force due to seismic accelerations respectively in passive case

Q_{vp} = vertical inertia force due to seismic accelerations respectively in passive case

P_{pe} is minimized with respect to trial inclination angle of failure surface, a_p and then the seismic passive earth pressure distribution, p_{pe} can be obtained by differentiating P_{pe} with respect to depth, z and can be expressed as follows,

$$p_{pe} = \frac{\gamma z}{\tan\alpha_p}\frac{\sin(\alpha_p+\phi)}{\cos(\delta+\phi+\alpha_p)} - \frac{k_h \gamma z}{\tan\alpha_p}\frac{\cos(\alpha_p+\phi)}{\cos(\delta+\phi+\alpha_p)}\sin\left[w\left(t-\frac{z}{V_s}\right)\right] - \frac{k_v \gamma z}{\tan\alpha_p}\frac{\sin(\alpha_p+\phi)}{\cos(\delta+\phi+\alpha_p)}\sin\left[w\left(t-\frac{z}{V_p}\right)\right] \quad (18)$$

RESULTS AND DISCUSSION

Results are presented in graphical form for normalized seismic active and passive earth pressures along the normalized depth of the wall (z/H). Variations of parameters considered are as follows: $\phi = 25^0, 35^0$; $\delta/\phi = 0.0, 0.5$; $k_h = 0.0, 0.1, 0.2, 0.3$; $k_v = 0.0k_h$ and $0.5k_h$.

SEISMIC ACTIVE EARTH PRESSURE

Fig. 3 shows the normalized active earth pressure distribution with depth for different values of soil friction angle, ϕ and wall friction angle, δ under static condition (i.e. $k_h = k_v = 0$). It is clear that under static condition, the active earth pressure distribution is exactly linear as expected (see Kramer 1996).

Results of the normalized seismic active earth pressure distribution with depth for different values of soil friction angle, ϕ and wall friction angle, δ under seismic condition of $k_h = 0.2$ and $k_v = 0$, $H/\lambda = 0.3$, $H/\eta = 0.16$ are shown in Fig. 4. Again in Fig. 5, the results are given for $k_h = 0.2$, $k_v = 0.1$, $H/\lambda = 0.3$, $H/\eta = 0.16$. All these results show the non-linear active earth pressure distribution under seismic conditions.

Comparing Figs.3 and 4, it is seen that, as k_h increases, seismic active earth pressure also increases, for example, with $\delta = 0.5\phi$ and $\phi = 35^0$, as k_h increases from 0.0 to 0.2, keeping all other parameters same, p_{ae} increases maximum at the base of the wall by 31.12%. Again from Figs.4 and 5, it is seen that as k_v increases, seismic active earth pressure also increases, for example, with $\delta = 0.5\phi$ and $\phi = 35^0$, as k_v increases from 0.0 to $0.5k_h$, keeping all other parameters same, p_{ae} increases maximum at the base of the wall by 35.64%. Also it is evident from Figs. 4 and 5 that, the

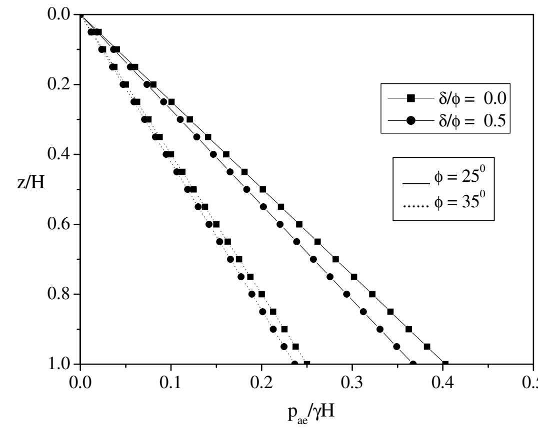

Figure 3. Normalized static active earth pressure distribution with depth for different values of soil friction angle, ϕ and wall friction angle, δ with $k_h = 0.0$, $k_v = 0.0$.

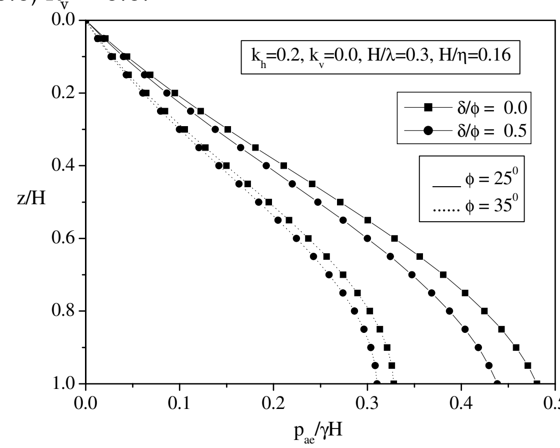

Figure 4. Normalized seismic active earth pressure distribution with depth for different values of soil friction angle, ϕ and wall friction angle, δ with $k_h = 0.2$, $k_v = 0.0$, $H/\lambda = 0.3$, $H/\eta = 0.16$.

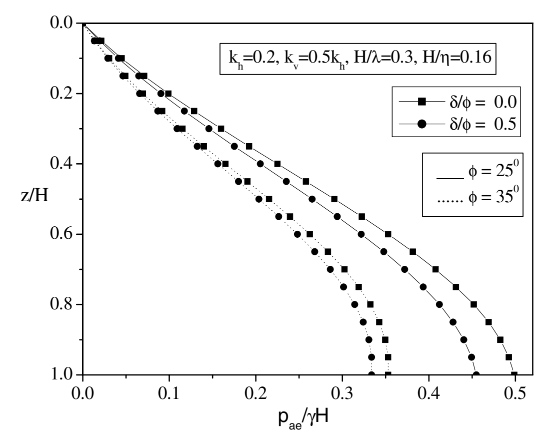

Figure 5. Normalized seismic active earth pressure distribution with depth for different values of soil friction angle, ϕ and wall friction angle, δ with $\phi = 35^0$, $k_h = 0.2$, $k_v = 0.1$, $H/\lambda = 0.3$, $H/\eta = 0.16$.

seismic active earth pressure show significant decrease with increase in soil internal friction angle f and that the seismic active earth pressure show marginal decrease with increase in wall friction angle d. For example, with $\delta = 0.5\phi$, $k_h = 0.2$, $k_v = 0.5k_h$, as ϕ increases from 25^0 to 35^0, p_{ae} decreases maximum at the base of the wall by 41.76% and with $\phi=35^0$, $k_h = 0.2$, $k_v = 0.5k_h$, as δ increases from 0 to 0.5ϕ, p_{ae} shows marginal decrease of about 5.73% at the base of the wall.

Under static and seismic conditions the active earth pressure reduces with increase in both soil friction angle, ϕ and wall friction angle, δ. And the effect of wall friction angle is less significant than that of soil friction angle. Degree of non-linearity of the curves also increases with the seismic effect leading to the rise of the point of application of total seismic active thrust (h_a) from the static value of 1/3rd from the base of the wall, which is commonly used in the design of retaining wall.

SEISMIC PASSIVE EARTH PRESSURE

Fig. 6 shows the normalized passive earth pressure distribution with depth for different values of soil friction angle, ϕ and wall friction angle, δ under static condition (i.e. $k_h = k_v = 0$). It is clear that the passive earth pressure distribution is exactly linear as expected in static case.

Results of the normalized seismic passive earth pressure distribution with depth for different values of soil friction angle, ϕ and wall friction angle, δ under seismic condition of $k_h = 0.2$ and $k_v = 0$, $H/\lambda = 0.3$, $H/\eta = 0.16$ are shown in Fig. 7. Again in Fig. 8, the results are given for $k_h = 0.2$ and $k_v = 0.1$, $H/\lambda = 0.3$, $H/\eta = 0.16$. All these results show the non-linear passive earth pressure distribution under seismic conditions.

Comparing Figs. 6 and 7, it is seen that as k_h increases, seismic passive earth pressure decreases, for example, with $\delta = 0.5\phi$ and $\phi = 35^0$, as k_h increases from 0.0 to 0.2, keeping all other parameters same, p_{pe} decreases maximum at the base of the wall by 10 %. Again from Figs. 7 and 8 it is seen that as k_v increases, seismic passive earth pressure also decreases, for example, with $\delta = 0.5\phi$, as k_v increases from 0.0 to $0.5k_h$, keeping all other parameters same, p_{pe} decreases maximum at the base of the wall by 5.75 %. Also it is evident from Figs. 7 and 8 that, the seismic passive earth pressure show significant increase with increase in both, soil internal friction angle f and wall friction angle d. For example, with $\delta = 0.5\phi$, $k_h = 0.2$, $k_v = 0.5k_h$, as f increases from 25^0 to 35^0, p_{pe} increases maximum at the base of the wall

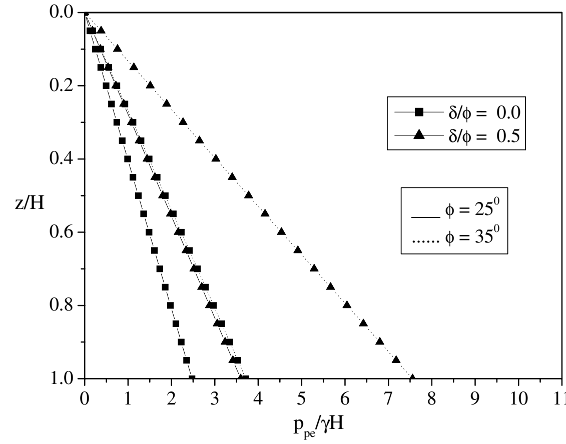

Figure 6. Normalized static passive earth pressure distribution with depth for different values of soil friction angle, ϕ and wall friction angle, δ with $k_h = 0.0$, $k_v = 0.0$.

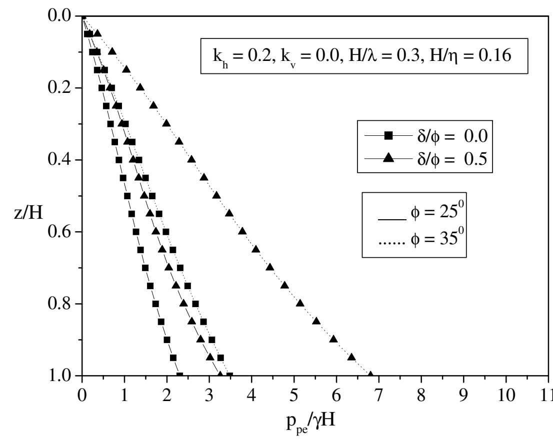

Figure 7. Normalized seismic passive earth pressure distribution with depth for different values of soil friction angle, ϕ and wall friction angle, δ with $k_h = 0.2$, $k_v = 0.0$, $H/\lambda = 0.3$, $H/\eta = 0.16$.

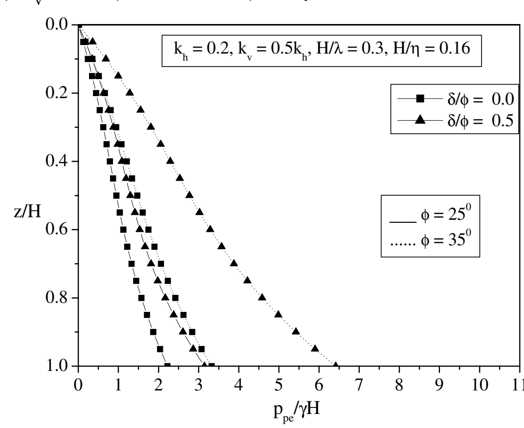

Figure 8. Normalized seismic passive earth pressure distribution with depth for different values of soil friction angle, ϕ and wall friction angle, δ with $k_h = 0.2$, $k_v = 0.1$, $H/\lambda = 0.3$, $H/\eta = 0.16$.

by 50.95% and with $\phi = 35^0$, $k_h = 0.2$, $k_v = 0.5k_h$, as δ increases from 0 to 0.5ϕ, p_{pe} increases maximum at the base of the wall by 48.17%.

Under static and seismic conditions the passive earth pressure increases with increase in both soil friction angle, ϕ and wall friction angle, δ. And as compared to seismic active case, the effect of wall friction angle is more pronounced for the seismic passive condition. Degree of non-linearity of the curves also increases with the seismic effect leading to the reduction in height of the point of application of total seismic passive resistance (h_p) from the static value of 1/3rd from the base of the wall.

COMPARISON OF RESULTS

Fig. 9 shows the comparison of seismic active earth pressure coefficient, K_{ae} (where $P_{ae} = 0.5\gamma K_{ae}H^2$) for different values of horizontal seismic coefficient (k_h) with $k_v = 0.5k_h$, $\phi = 35^0$, $\delta = \phi/2$, $H/\lambda = 0.3$, $H/\eta = 0.16$ calculated by Mononobe-Okabe method and present study respectively. It is evident from Fig. 9 that, seismic active earth pressure coefficient by present study is maximum as compared to Mononobe-Okabe which is desirable for the design purpose. Again Fig. 10 shows the comparison of seismic passive earth pressure coefficient, K_{pe} (where $P_{pe} = 0.5\gamma K_{pe}H^2$) for different values of horizontal seismic coefficient (k_h) with $k_v = 0.5k_h$, $\phi = 35^0$, $\delta = \phi/2$, $H/\lambda = 0.3$, $H/\eta = 0.16$ calculated by Mononobe-Okabe method, pseudo-static approach by Choudhury (2004) and present study respectively. It is clear from Fig. 10 that, seismic passive earth pressure coefficient by present study is minimum and thus proves to be safer as per the design criteria as compared to other methods which is again desirable for the design purpose.

For the case of $k_h = 0.2$, $k_v = 0.5k_h$, $\phi = 35^0$, $\delta = \phi/2$, $H/\lambda = 0.3$, $H/\eta = 0.16$, comparison of present results with conventional Mononobe-Okabe method for non-dimensional seismic active earth pressure distribution is shown in Fig. 11. Under the seismic condition, the non-linear active earth pressure distribution and hence the change of point of application from the static case is clearly shown in Fig. 11 compared to the results obtained by pseudo-static method. This non-linearity of seismic active earth pressure distribution was also monitored by Steedman and Zeng (1991) in centrifuge tests. Again, for the case of $k_h = 0.2$, $k_v = 0.5k_h$, $\phi = 35^0$, $\delta = \phi/2$, $H/\lambda = 0.3$, $H/\eta = 0.16$, comparison of present results with conventional Mononobe-Okabe method for non-dimensional seismic passive earth pressure distribution is shown in Fig. 12. Under the seismic condition, the non-linear passive earth pressure distribution and hence the change of point of application from the static case is clearly shown in Fig. 12 compared to the results obtained by pseudo-static method.

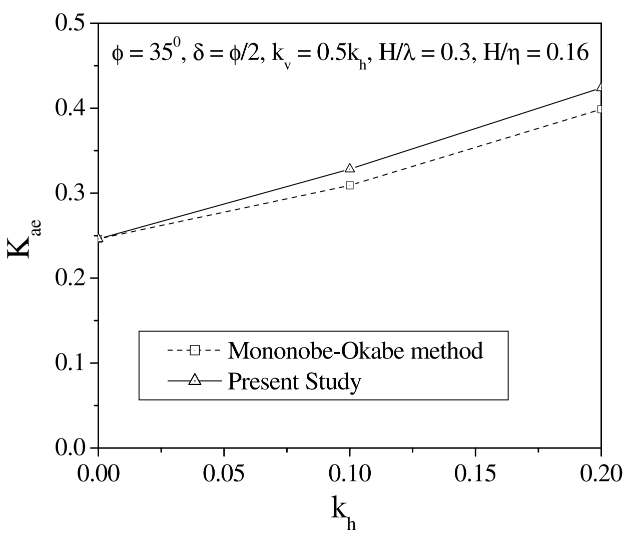

Figure 9. Comparison of typical results of seismic active earth pressure coefficient, K_{ae} for different values of horizontal seismic coefficient (k_h) with $k_v = 0.5k_h$, $\phi = 35^0$, $\delta = \phi/2$, $H/\lambda = 0.3$, $H/\eta = 0.16$.

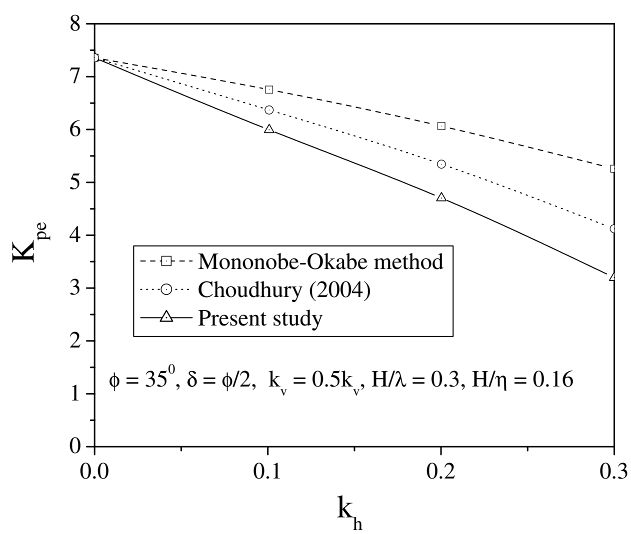

Figure 10. Comparison of typical results of seismic passive earth pressure coefficient, K_{pe} for different values of horizontal seismic coefficient (k_h) with $k_v = 0.5k_h$, $\phi = 35^0$, $\delta = \phi/2$, $H/\lambda = 0.3$, $H/\eta = 0.16$.

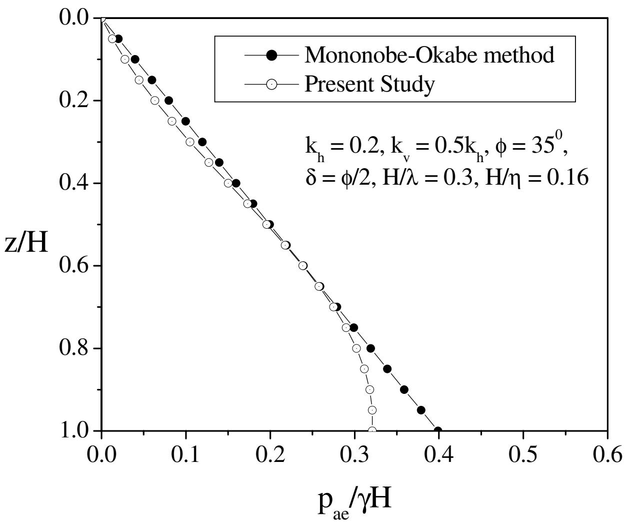

Figure 11. Comparison of typical results of non-dimensional seismic active earth pressure distribution for $k_h = 0.2$, $k_v = 0.5k_h$, $\phi = 35^0$, $\delta = \phi/2$, $H/\lambda = 0.3$, $H/\eta = 0.16$.

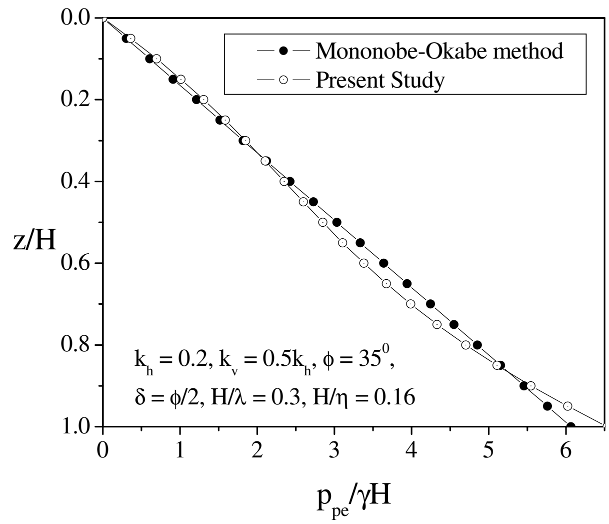

Figure 12. Comparison of typical results of non-dimensional seismic passive earth pressure distribution for $k_h = 0.2$, $k_v = 0.5k_h$, $\phi = 35^0$, $\delta = \phi/2$, $H/\lambda = 0.3$, $H/\eta = 0.16$.

CONCLUSIONS

The pseudo-dynamic method of analysis, presented in this paper, highlights the effect of time and phase change in shear and primary waves propagating in the backfill behind the rigid retaining wall on the seismic earth pressures. It gives more realistic non-linear seismic active earth pressure distribution behind the retaining wall as compared to the Mononobe-Okabe method using pseudo-static approach. But the conventional pseudo-static approach gives only linear earth pressure distribution irrespective of static and seismic condition leading to a major drawback in the design criteria.

The seismic passive earth pressure is more sensitive to wall friction angle as compared to the seismic active earth pressure. By applying the pseudo-dynamic method presented in this paper, the seismic active earth pressures are more and seismic passive earth pressures are less as compared to those calculated by using conventional pseudo-static method of analysis. Thus the present method gives the desirable design values of seismic active and passive earth pressure coefficients compared to the existing values by pseudo-static method as it leads to safe approach of design of retaining wall against devastating effect of earthquake.

APPENDIX : LIST OF NOTATIONS

a_h, a_v = amplitude of horizontal and vertical seismic acceleration respectively
g = acceleration due to gravity
H = height of the retaining wall
K_{ae}, K_{pe} = seismic active and passive earth pressure coefficient respectively
k_h, k_v = seismic acceleration coefficient in the horizontal and vertical direction respectively
P_{ae}, P_{pe} = pseudo-dynamic active thrust and passive resistance respectively
Q_{ha}, Q_{va} = horizontal and vertical inertia force due to seismic accelerations respectively in active case
Q_{hp}, Q_{vp} = horizontal and vertical inertia force due to seismic accelerations respectively in passive case
t = time
T = period of lateral shaking
V_s, V_p = shear and primary wave velocity respectively
α_a = angle of inclination of the failure surface with the horizontal in active case
α_p = angle of inclination of the failure surface with the horizontal in passive case
γ = unit weight of the soil
ϕ = soil friction angle
δ = wall friction angle
ω = angular frequency of base shaking
$\zeta = t - H/V_s$
$\psi = t - H/V_p$
$\lambda = TV_s$
$\eta = TV_p$

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