

Seismic Behaviour of Retaining Structures, Design Issues and Requalification Techniques

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Abstract The realistic estimation of seismic earth pressure is very crucial for the design of retaining structures in seismic-prone areas. Several researchers have developed analytical and numerical methods for the estimation of seismic earth pressure. Some experimental studies are also reported to clearly present the seismic behaviour of retaining structures. Pseudo-static and pseudo-dynamic methods are the ones which are popularly used for the calculation of seismic earth pressure. Pseudo-dynamic method is a modification of the conventional pseudo-static method by eliminating most of the limitations. Recently,

the researchers have shown that the new dynamic method considering Rayleigh wave, which plays a major role in the calculation of seismic earth pressures to maintain compatible dynamic stress boundary conditions, is better than pseudo-dynamic method as validated through the available dynamic centrifuge test results. This state-of-the-art paper presents a critical review of the literature on the available procedures for the seismic analysis, design and requalification of retaining structures. The methods which are currently used in routine practice for the seismic design of retaining structures are also explained briefly. Indian and some other international design codes for the seismic design of retaining structures are explained. For new design and requalification of existing retaining structures in seismic-prone areas, a worked out example is provided with recommendations for techniques of requalification.

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Introduction

Many researchers have developed several methods for analysis and design of retaining structures in the seismic-prone areas. Pseudo-static and pseudo-dynamic methods are most popularly used analytical methods in Geotechnical Engineering practice. Various researchers worked to estimate the static earth pressures on retaining walls and to observe their behaviour [28]. Mononobe and Matsuo [47] and Okabe [54] explained for the first time the application of pseudo-static approach for the estimation of dynamic/seismic active and passive earth pressures on rigid retaining walls. Terzaghi [78] (see Kramer [43])

explained the concept of pseudo-static approach for seismic stability analysis of soil slopes. In recent years, analytical expressions have been presented for the dynamic active and passive earth pressures, considering different practical aspects, by several researchers. This paper explains all these developments along with their advantages and limitations. Also it includes the requalification aspects of retaining wall under seismic conditions.

The Pseudo-Static Method

The pseudo-static approach is the earliest approach used to analyse the seismic stability of retaining structures against earthquake loads/forces. In this approach, the effects of earthquake forces are represented by constant horizontal and vertical accelerations. Mononobe and Okabe in 1926 and 1929 [43] first showed the application of the pseudo-static approach to the analysis and design of retaining walls. Various researchers like Saran and Prakash [59], and Madhav and Kameswara Rao [44] estimated the seismic earth pressures on retaining walls using the pseudo-static approach. The details of pseudo-static approach for slope stability analysis were described by Terzaghi [78] (see Kramer [43]). The pseudo-static method applies a static force to a designed facility as kW , where k is the seismic acceleration coefficient and W is the weight of the facility. This is the first approach used to design the retaining structure against additional destabilizing earthquake forces by applying force-based analysis.

Theoretical background of seismic acceleration coefficient lies in the application of D'Alembert's principle of mechanics. The major problems associated with the value of seismic coefficient (k) are:

1. The value of k depends on the region, seismic activity, importance of facilities, local geology and soil conditions.
2. Different countries recommend different values of k . For example, the value of k in Japan is 0.15–0.2 or greater [79].

Richards and Elms [57] proposed a displacement-based approach for seismic design of gravity retaining walls. The effect of wall inertia was observed to be of the same order as that of the dynamic earth pressure computed by the Mononobe–Okabe (M–O) (see Kramer [43]) analysis. Richards and Elms [57] suggested this new design approach based on the initial choice of a specified limiting wall displacement, and presented the following expressions:

Considering the equilibrium of the wall,

$$W = \left[\frac{1}{2} \gamma H^2 (1 - k_v) K_{ae} \right] C_{IE}, \quad (1)$$

where

$$C_{IE} = \frac{\cos(\beta + \delta) - \sin(\beta + \delta) \tan \phi_b}{(1 - k_v)(\tan \phi_b - \tan \theta)} \quad (2)$$

with, W is the self-weight of the retaining wall; ϕ_b the friction angle between the wall and the soil on which it is resting; δ the wall friction angle; β is ground slope angle; k_h and k_v are the seismic acceleration coefficients in horizontal and vertical directions, respectively; K_{ae} the seismic active earth pressure coefficient; γ the unit weight of soil; and H is the height of retaining wall.

Richards and Elms [57] have given expression for estimating permanent block displacement (d_{perm}) as shown below:

$$d_{pem} = 0.087 \frac{v_{max}^2 a_{max}^3}{a_y^4}, \quad (3)$$

where v_{max} is the peak ground velocity, a_{max} is the peak ground acceleration (PGA), a_y is the yield acceleration for the wall-backfill system.

Choudhury and Subba Rao [23] presented a detailed study which explains the procedure for the estimation of passive earth resistance under seismic conditions by adopting the pseudo-static approach. This approach was given for negative wall friction case, and it can be used while determining the uplift capacity for ground anchor as shown by Choudhury and Subba Rao [24]. The failure surface was considered as an arc of a log spiral. Limit equilibrium approach was used in this force-based analysis. Choudhury and Subba Rao [23] proposed the following expression to estimate the total seismic passive resistance (P_{pd}) acting on the wall.

$$P_{pd} = \left(2cHK_{pcd} + qHK_{pqd} + \frac{1}{2} \gamma H^2 K_{pyd} \right) \frac{1}{\cos \delta}, \quad (4)$$

where P_{pd} is seismic passive resistance, and K_{pyd} , K_{pqd} , and K_{pcd} are the seismic passive earth pressure coefficients corresponding to the unit weight (γ), surcharge (q) and cohesion (c) of backfill soil respectively. Result given by Choudhury and Subba Rao [23] showed that the seismic passive earth pressure coefficients decrease with an increase in the vertical seismic acceleration. However, the horizontal seismic acceleration can result in either an increase or a decrease in the earth pressure coefficients. Saran and Gupta [58] also adopted pseudo-static approach to obtain the seismic active earth pressure behind rigid retaining wall.

Choudhury et al. [26] proposed an analytical model using method of horizontal slices for the determination of seismic passive resistance and its point of application. Steedman and Zeng [75] mentioned an expression for the

point of application of seismic active thrust above the base of the wall and expressed by Eq. (5).

$$h_d = H - \frac{2\pi^2 H^2 \cos \omega \zeta + 2\pi \lambda H \sin \omega \zeta - \lambda^2 (\cos \omega \zeta - \cos \omega t)}{2\pi H \cos \omega \zeta + \pi \lambda (\sin \omega \zeta - \sin \omega t)}, \quad (5)$$

where

$$\lambda = \frac{2\pi V_s}{\omega} \quad \text{and} \quad \zeta = \left(t - \frac{H}{V_s}\right). \quad (6)$$

ω is excitation frequency, V_s and V_p are shear and primary wave velocities respectively, H is height of the retaining wall. It was also mentioned that the point of application of the total passive resistance will not always act at $0.33H$ from the base of the wall with vertical height H . Choudhury et al. [26] also highlighted that the distribution of seismic passive earth pressure along the depth is nonlinear in most of the cases. They explained advantages of displacement-based analysis over the force-based analysis by considering a numerical example for the retaining wall, and suggested a modification of the Indian seismic design code IS 1893 for seismic design of retaining wall.

Another study based on the pseudo-static limit equilibrium analysis was performed by Subba Rao and Choudhury [76] for determination of seismic passive resistance on rigid retaining wall. They explained the computation of the seismic passive earth resistance coefficients by using a composite (planar + log spiral) failure surface for positive wall friction angle. While most of the previous researchers deal with sand as backfill material, a generalized solution was proposed by this study in which consideration of the cohesive backfill was also made. From the results reported by Subba Rao and Choudhury [76], seismic passive earth pressure coefficient $K_{p,d}$ was observed to decrease with an increase in seismic horizontal acceleration coefficient (k_h) value for a given seismic vertical acceleration coefficient (k_v) value. Effects of surcharge and self weight of the backfill soil were also considered. Subba Rao and Choudhury [76] presented the design charts for direct computation of seismic passive earth resistance coefficients using the limit equilibrium method of force-based analysis. A pseudo-static approach for the calculation of seismic forces was adopted. This study has applications for the estimation of seismic bearing capacity of foundations as shown by Choudhury and Subba Rao [25].

Choudhury and Ahmad [13] used pseudo-static method to analyze the stability of waterfront retaining wall under active state of earth pressure. Authors presented a methodology in which the combined effect of earthquake forces was considered along with the hydrodynamic pressure including inertial forces acting on a retaining wall. Similarly for the passive case of earth pressure, Choudhury and Ahmad [14] had proposed factor of safety against sliding

and overturning modes of failure for waterfront retaining wall by using pseudo-static approach under the combined action of earthquake and tsunami.

Shukla et al. [70] presented an analytical expression for the total seismic active pressure from the c - ϕ soil backfills on the retaining wall with a vertical smooth back face by considering both horizontal and vertical seismic acceleration coefficients, k_h and k_v , respectively using the pseudo-static method. This study provided a closed-form expression for the total seismic active earth pressure in terms of seismic active earth pressure coefficient along with an explicit analytical expression for the critical inclination of the failure plane to the horizontal. How to incorporate the effect of tension cracks was also reported. Additionally several design charts were presented for practical applications [31]. For similar simplified field situation, Shukla and Habibi [68] presented a closed-form expression for the total seismic passive pressure in terms of seismic passive earth pressure coefficients along with an explicit analytical expression for the critical inclination of the failure plane to the horizontal.

In order to incorporate more and more field parameters, the attempts were made to extend the fundamental expressions developed by Shukla et al. [70] for the total seismic active earth pressure [62, 64, 67, 69] and the fundamental expressions developed by Shukla and Habibi [68] for the total seismic passive earth pressure [63, 65, 66, 71].

Shukla and Zahid [69] presented an analytical expression for the total dynamic active pressure from the c - ϕ soil backfill with surcharge under seismic loading conditions. They also presented graphical presentations to recognize the importance of angle of shearing resistance and cohesion of the soil backfill, surcharge and direction of vertical seismic loadings. As observed by previous researchers, they also showed that the total dynamic active force decreases with an increase in the angle of shearing resistance of the backfill soil. They have concluded that for any value of angle of shearing resistance, the presence of cohesion in the backfill decreases the active force. However, the surcharge load has a significant effect on active force irrespective of the presence of seismic load.

Shukla and Habibi [68] presented an analytical expression for the total seismic passive pressure on a retaining wall from the c - ϕ soil backfill subjected to both horizontal and vertical seismic inertial forces. Shukla et al. [71] have presented the derivation of an analytical expression for the total passive earth pressure on a retaining wall from the c - ϕ soil backfill subjected to surcharge and seismic loads. They concluded that the design value of the total dynamic passive force should be obtained with consideration of vertically upward seismic inertial force along with the horizontal seismic inertial force towards the soil backfill. Shukla [63] gave analytical expression for the total seismic

passive earth pressure from the c - ϕ soil backfills on rigid retaining walls subjected to surcharge and seismic loads. The expression was derived by considering wall friction and adhesion. The closed-form solution also presents an explicit expression for the critical inclination to the horizontal of the failure plane within the backfill.

Shukla and Bathurst [67] presented the derivation of an analytical expression for the dynamic active thrust for a rigid retaining wall supporting from c - ϕ backfill with wall friction and adhesion. As this work is for active earth pressure, they have considered the tension cracks in the backfill along with a uniform surcharge on the backfill, and horizontal and vertical seismic loadings.

More recently Shukla [66] presented the generalised analytical expression for the total dynamic/seismic passive earth pressure from the c - ϕ soil backfills. This expression is generalised because the derivation considered most practical parameters related to the wall geometry, soil backfill and loadings, such as wall height, wall-backfill face inclination, backfill slope angle, wall friction, wall-backfill adhesion, cohesion and angle of shearing resistance of backfills, surcharge, and both horizontal and vertical seismic loadings. The development of an explicit expression for the critical inclination of the failure plane within the soil backfill was also presented for the generalized case. It was shown that the generalized analytical expressions result in simpler cases reported previously in the literature for several static and dynamic field conditions.

Critical Remarks on Pseudo-Static Method

The pseudo-static method has been popular among engineers for routine design works because the mathematical expressions are simple and in closed-form. No advanced or complicated analysis is necessary for the calculation of factor of safety. It made a great contribution to the improvement of seismic design procedure of many geotechnical structures. However, it has some serious limitations. Seismic force is cyclic, changes direction and magnitude with time, and exhibits for a limited duration. Whereas, the pseudo-static method applies a seismic force as a constant, unidirectional static force. This seismic force seriously overestimates the risk of earthquake failure, making mostly the design over safe with few exceptions of unsafe design in some specific cases.

Towhata [79] reported a case study of 1994 Northridge earthquake. The maximum horizontal acceleration of 1.8g or possibly 1.9g was recorded at Tarzana site. Within ten meters from the accelerometer here, a small hut did not suffer damage. So, was this structure well designed against a horizontal static force as intense as 1.8 times its weight? Towhata [79] highlighted with this example about the uncertainties involved in the determination or predefined

values of seismic coefficient. Also, a relation between design value of k and the maximum ground acceleration is not clear. For example, 1.9g acceleration does not mean k_h or $k_v = 1.9$ [79].

Besides these limitations, the pseudo-static method is widely used till date because of its simplicity. It is widely used because it uses the limit state equilibrium analysis which is routinely conducted by the geotechnical engineers. The computations are easy to understand and perform. However, the accuracy of the pseudo-static approach is governed by the accuracy with which the simple pseudo-static inertial forces are computed or estimated or assumed.

The Pseudo-Dynamic Method

As mentioned in the previous section, the pseudo-static method has inherent limitations. To overcome these limitations, an attempt was made by Steedman and Zeng [75]. They considered a phase difference due to finite shear wave propagation behind a retaining wall. The method given by Steedman and Zeng [75] considered the effects of horizontal acceleration as time and frequency dependent parameters for seismic design of retaining structures. A vertical rigid retaining wall supporting a cohesionless backfill with a definite 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) was considered by Steedman and Zeng [75].

Choudhury and Nimbalkar [18] developed the pseudo-dynamic method by modifying the method given by Steedman and Zeng [75] by also considering also the effect of vertical seismic acceleration (k_v) including primary wave, along with horizontal seismic acceleration for passive case of earth pressure. Choudhury and Nimbalkar [18] studied the effects of a wide range of parameters like wall friction angle (δ), soil friction angle (ϕ), shear wave velocity (V_s), primary wave velocity (V_p) and horizontal and vertical seismic accelerations (k_h and k_v , respectively) on seismic passive earth pressure. Authors considered that the shear modulus (G) is constant with depth of a retaining wall throughout the backfill and only the phase and not the magnitude of accelerations are varying along the depth of the wall.

Further, Choudhury and Nimbalkar [19] had highlighted that the pseudo-dynamic method gives more realistic non-linear seismic active earth pressure distribution behind the retaining wall when compared to the Mononobe–Okabe method using pseudo-static approach. Choudhury and Nimbalkar [19] extended the work of Choudhury and Nimbalkar [18] for the calculation of seismic active earth pressure. The model considered by Choudhury and Nimbalkar [19] for active case of earth pressure under seismic condition can be seen from Fig. 1a with a special condition

of $\theta = 0^\circ$ and $f_a = 1.0$, where θ is the wall batter angle with vertical face and f_a is the soil amplification factor. Figure 1b shows typical results in terms of non-dimensional seismic active earth pressure distribution as obtained by Choudhury and Nimbalkar [19]. It can be observed from Fig. 1b that the seismic active earth pressure distribution varies non-linearly with the dimensionless depth parameter (z/H), which cannot be obtained by the pseudo-static approach. Choudhury and Nimbalkar [20] applied the pseudo-dynamic method to compute the rotational displacements of rigid retaining wall supporting cohesionless backfill under seismic conditions for passive state. They proposed a methodology which considers time, phase difference and effect of amplification in both shear and primary waves propagating through both the backfill and wall. The influence of ground motion characteristics on rotational displacement of the wall is evaluated in this method. It has been observed that, the rotational displacement of the wall increases substantially with increase in amplification of both shear and primary waves, time of input motion, period of lateral shaking. However, rotational displacement of the wall decreases with an increase in soil friction angle (ϕ), wall friction angle (δ). Choudhury and Nimbalkar [20] considered vertical rigid gravity wall of height H and width b_w , supporting horizontal cohesionless backfill. The study showed the variation of rotational displacement (θ) with horizontal seismic acceleration coefficient (k_h) for different values of time history of input motion (t). It was also observed that the rotational displacement (θ) increases with an increase in time of input motion (t). Choudhury and Nimbalkar [21] had given solutions for seismic rotational stability against active state of earth pressure using pseudo-dynamic approach.

Ghosh [34] gave a methodology to calculate the seismic active earth pressure behind a non-vertical cantilever retaining wall using pseudo-dynamic analysis (Fig. 1a). The effects of soil friction angle (ϕ), wall inclination, wall friction angle (δ), amplification of vibration, and horizontal and vertical earthquake acceleration (k_h and k_v) on the active earth pressure were studied by Ghosh [34]. This study also predicted non-linear variation of seismic active earth pressure along the wall. Figure 1a shows a rigid non-vertical retaining wall of height H supporting cohesionless, horizontal backfill as considered in the analysis. It is observed that the values predicted by the Mononobe–Okabe method are slightly higher than the values obtained by the method proposed by Ghosh [34]. The observed difference is due to the fact that the pseudo-static method does not consider the effect of time and phase but only a constant value of seismic acceleration. Figure 2 shows that the magnitude of seismic active earth pressure coefficient increases continuously with an increase in the magnitude of horizontal seismic acceleration coefficient. Sreevalsa and Ghosh [73] presented a study on the seismic active earth pressure behind a bilinear rigid cantilever retaining wall using pseudo-dynamic analysis. The effect of non-uniform shear modulus distribution with depth was also considered as it affects the amplification of acceleration and eventually the magnitude of earth pressure. The study of Sreevalsa and Ghosh [73] explored the effects of soil friction angle, angle of inclination of the upper part of wall backface, angle of inclination of the lower part of wall backface, interface friction angle between wall backface and soil medium, horizontal earthquake acceleration coefficient, vertical earthquake acceleration coefficient, amplification factor, depth exponent causing shear modulus variation, shear wave velocity and primary wave velocity on the

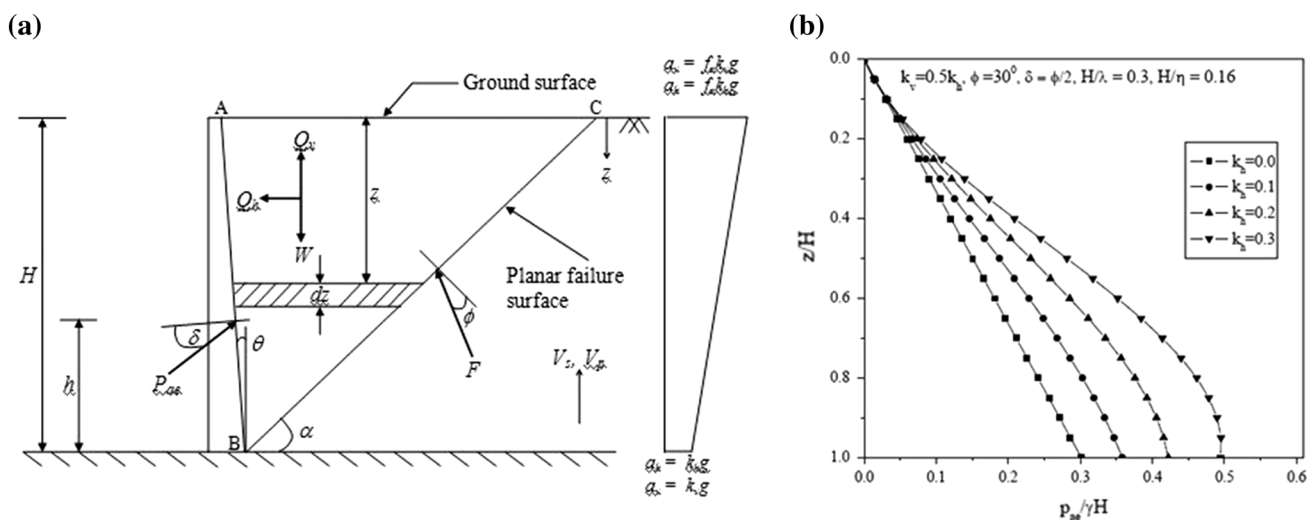


Fig. 1 a Model retaining wall considered for computation of pseudo-dynamic active earth pressure on retaining wall (modified after Ghosh [32]). b Typical variation for seismic active earth pressure distribution

with $k_v = 0.5k_h$, $\phi = 30^\circ$, $\delta = \phi/2$, $H/\lambda = 0.3$, $H/\zeta = 0.16$, $f_a = 1.0$, $\theta = 0^\circ$ (modified after Choudhury and Nimbalkar [19])

seismic active earth pressure using pseudo-dynamic approach. The limit equilibrium method, with a planar failure surface behind the retaining wall, was considered to compute the active resistance of the wall with bilinear backface. It was observed that the magnitude of active earth pressure on both upper and lower parts increases continuously with an increase in magnitude of horizontal seismic acceleration coefficient.

Nimbalkar and Choudhury [52] developed a theory similar to the one developed by Choudhury and Nimbalkar [18, 19] to estimate the seismic passive earth pressure by using the pseudo-dynamic approach by considering the soil amplification. The seismic accelerations were modified to consider amplification as given by Eqs. (7) and (8).

$$a_h(z, t) = \left\{ 1 + \frac{H - z}{H} (f_a - 1) \right\} k_h g \sin \omega \left(t - \frac{H - z}{V_s} \right), \tag{7}$$

$$a_v(z, t) = \left\{ 1 + \frac{H - z}{H} (f_a - 1) \right\} k_v g \sin \omega \left(t - \frac{H - z}{V_p} \right), \tag{8}$$

where V_s shear wave velocity, V_p is primary wave velocity, f_a is soil amplification factor, H is the height of retaining wall in meter, t is time in second and z is the depth from ground surface. The horizontal seismic inertia force can be calculated by using Eq. (9).

$$Q_{ha}(t) = \int_0^H m_a(z) a_h(z, t) dz. \tag{9}$$

Nimbalkar and Choudhury [52] observed that effects of the soil parameters are more pronounced on the passive

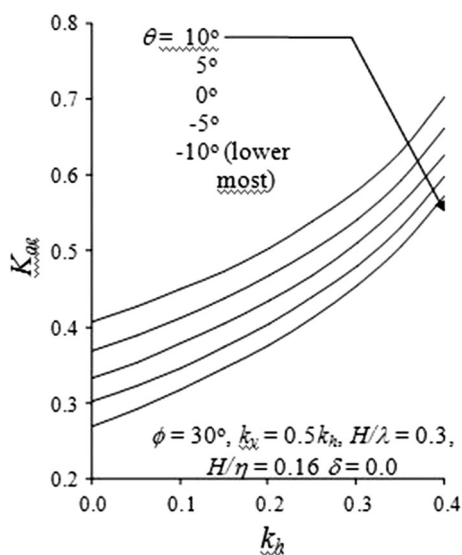


Fig. 2 Variation of seismic active pressure coefficient K_{ae} with α_h for $\phi = 30^\circ$, $H/\lambda = 0.3$, $H/\eta = 0.16$, $f_a = 1$ (modified after Ghosh [34])

state of seismic earth pressure than for earth pressure at active state. Authors evaluated stability of a retaining wall by pseudo-dynamic seismic forces acting on the soil wedge and the wall for active case. Nimbalkar and Choudhury [53] also determined design weight of the wall under seismic conditions under active earth pressure condition.

Figure 3a shows a rigid vertical gravity wall, of height H and width b_w which supports cohesionless backfill as considered by Nimbalkar and Choudhury [53]. The shear wave velocity through the backfill soil is given by $V_{ss} = (G_s/\rho_s)^{1/2}$, where, ρ_s is the density of the backfill material and primary wave velocity, $V_{ps} = (G_s(2 - 2\nu_s)/\rho_s(1 - 2\nu_s))^{1/2}$, where ν_s is the Poisson's ratio of the backfill. Similarly, the shear wave velocity through the wall is expressed as $V_{sw} = (G_w/\rho_w)^{1/2}$ where, ρ_w is the density of the wall material and primary wave velocity $V_{pw} = (G_w(2 - 2\nu_w)/\rho_w(1 - 2\nu_w))^{1/2}$ where, ν_w is the Poisson's ratio of the wall material, are assumed to act within the retaining wall due to earthquake loading.

Figure 3b presents a typical variation of design factors viz. soil thrust factor (F_T), wall inertia factor (F_I) and combined dynamic factor (F_w). From Fig. 3b, it can be seen that the presence of seismic forces, in any direction (i.e. horizontal or vertical), induces reduction in seismic stability of retaining wall. This leads to the higher values of the design factors which ultimately lead to requirement of higher weight of the wall to maintain equilibrium against sliding under seismic conditions. Similarly for the seismic stability of waterfront retaining wall, Ahmad and Choudhury [1, 3] proposed design factors against sliding mode of failure of the wall. And the rotational stability of waterfront retaining wall was considered for seismic design by Ahmad and Choudhury [4] using pseudo-dynamic approach. Very recently Chakraborty and Choudhury [12] had obtained the design charts for the seismic stability of non-vertical waterfront retaining wall subjected to both earthquake and tsunami.

Ahmad and Choudhury [2] analyzed internal stability of reinforced waterfront retaining wall by pseudo-dynamic approach when subjected to earthquake forces and tsunami. Both linear and poly-linear failure surfaces are considered in analysis. Again Choudhury and Ahmad [16] gave seismic design solutions for the external stability of reinforced waterfront retaining wall. Ghosh [33] extended the methodology of Choudhury and Nimbalkar [18] to calculate the seismic passive earth pressure behind a non-vertical cantilever retaining wall using pseudo-dynamic analysis. The effects of soil friction angle (ϕ), wall inclination, wall friction angle (δ), amplification of vibration, and horizontal and vertical earthquake acceleration (k_h and k_v) on the passive earth pressure were studied by Ghosh [33]. Sreevalsa and Ghosh [74] reported a study on the seismic passive earth pressure behind a bilinear rigid cantilever

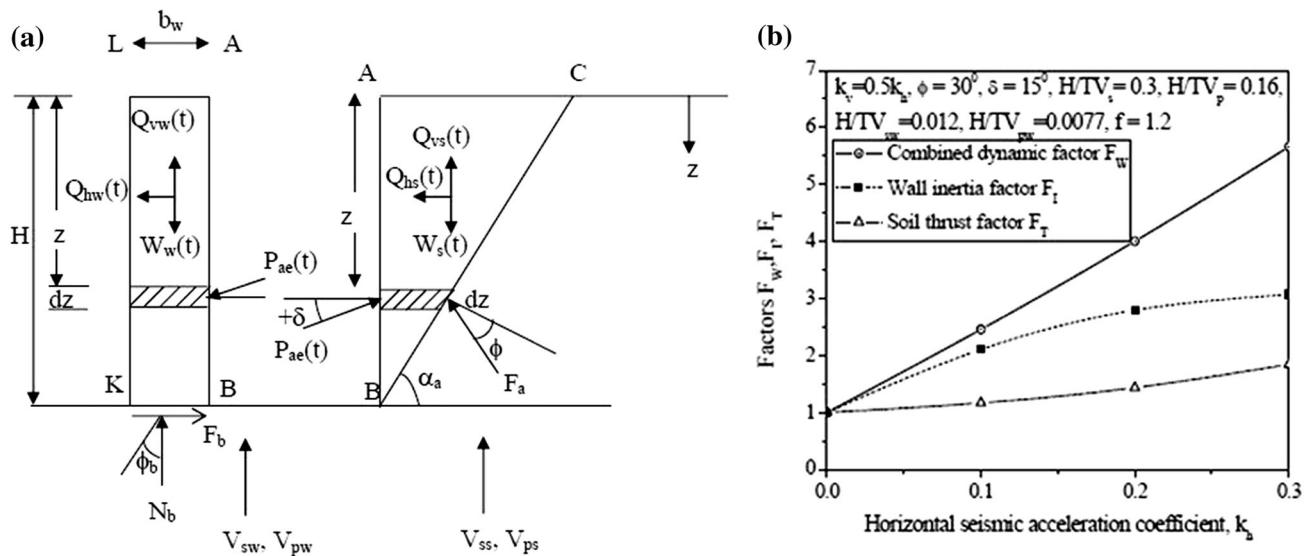


Fig. 3 a Details of forces acting on the soil wedge and the wall for active case under sliding stability (modified after Nimbalkar and Choudhury [53]). b Typical variation of soil thrust factor F_T , wall

inertia factor F_I and combined dynamic factor F_W with $\delta = \phi/2$ for sliding stability in active case (modified after Nimbalkar and Choudhury [53])

retaining wall using pseudo-dynamic analysis. Like Sreevalsa and Ghosh [73], here also authors considered the effect of non-uniform shear modulus distribution on the magnitude of passive earth pressure. It was noted that the magnitude of passive earth pressure on both upper and lower parts decreases continuously with an increase in magnitude of horizontal seismic acceleration coefficient.

Choudhury and Ahmad [15] presented a study of a waterfront retaining wall retaining a partially submerged backfill, subjected to seismic forces which are calculated using the pseudo-dynamic approach. It was observed that as the value of soil friction angle increases, there is an increase in the factor of safety in the sliding mode. This is because of the fact that an increase in soil friction angle indicates the increase in soil strength, hence an increase in factor of safety has been observed.

Basha and Babu [7], and Basha [6] proposed an approach for computing seismic passive earth pressure coefficients using composite curved rupture surface (combination of the arc of a logarithmic spiral and straight line) based on the pseudo-dynamic method. Using the proposed theory, Basha and Babu [8] presented a formulation for the calculation of sliding displacements of gravity retaining walls. Basha and Babu [9] used the pseudo dynamic method to compute the rotational displacements of gravity retaining walls under the passive condition when subjected to seismic loads. Figure 4a shows the model considered by Basha and Babu [9].

For the comparison of rotational displacements using planar and composite failure mechanism as shown in Fig. 4a, the expression for passive earth pressure coefficient reported in Nimbalkar and Choudhury [52] is used

and the results are plotted in Fig. 4b. It can be noted from Fig. 4b that the rotational displacements computed using planar failure mechanism are underestimated. It can also be noticed that the critical seismic acceleration coefficients for rotation (k_{cr}) computed using planar failure mechanism are overestimated. This is due to the fact that, pseudo-dynamic method considering composite failure mechanism gives lower seismic passive resistance than the value obtained using assumption of planar failure mechanism. Basha and Babu [9] highlighted that the assumption of a planar failure mechanism for the rough soil–wall interfaces significantly overestimates the threshold seismic accelerations, thus underestimating the rotational displacements.

Ghosh and Sharma [36] presented the pseudo-dynamic analysis for calculating seismic active earth pressure for non-vertical retaining wall supporting $c-\phi$ backfill. This work is an extension of methodology presented by Choudhury and Nimbalkar [19]. Considering a planar rupture surface, the effect of wide range of parameters like inclination of retaining wall, wall friction angle (δ) and soil friction angle (ϕ), shear wave velocity (V_s) and primary wave velocity (V_p), horizontal and vertical seismic coefficients (k_h and k_v) were taken into account to evaluate the seismic active force.

Ghosh and Sharma [36] also compared the seismic active earth pressure values supporting $c-\phi$ backfill obtained by the pseudo-dynamic analysis with those obtained by the pseudo-static analysis proposed by Shukla et al. [70]. The comparison shows that the pseudo-static analysis gives slightly higher values of seismic active earth pressure than those given by the pseudo-dynamic analysis. Also Choudhury and Nimbalkar [18] showed that the

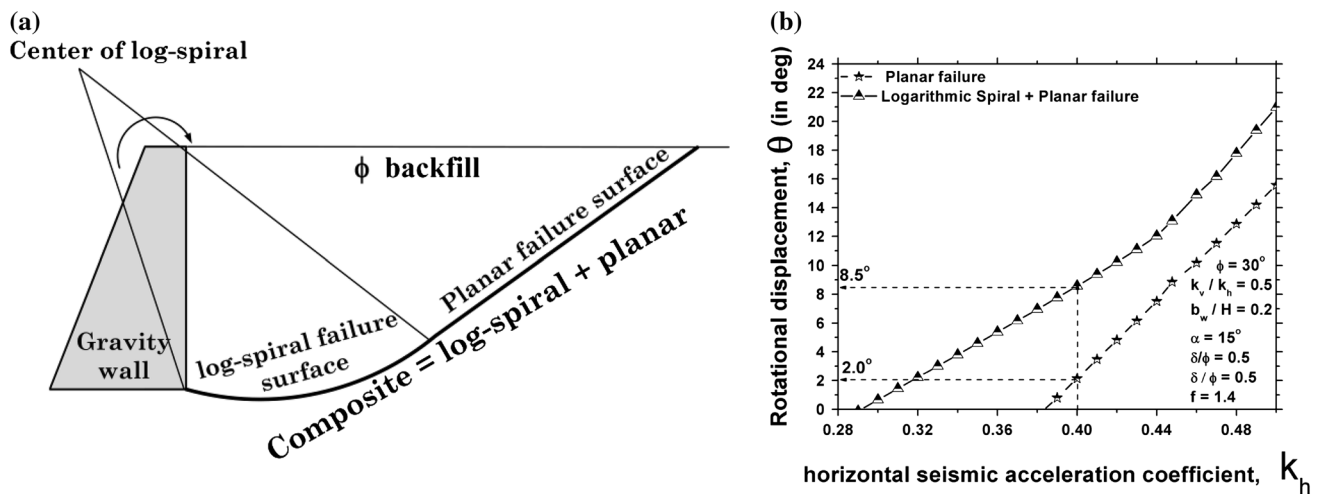


Fig. 4 a Gravity wall with composite failure mechanism (modified after Basha and Babu [9]). b Influence of k_h on rotational displacement (θ) of gravity retaining wall (modified after Basha and Babu [9])

seismic passive earth pressure coefficient obtained by pseudo-static analysis is higher than that obtained by using pseudo-dynamic analysis. These comparisons reveal that the pseudo-dynamic analysis estimates the lower values of seismic earth pressures in comparison to the values obtained by pseudo-static analysis.

Bellezza et al. [11] claimed that a more rational pseudo-dynamic approach has been developed for fully submerged soil with an assumption that a restrained or free water condition exists within the backfill. The results obtained by them demonstrate that the analysis proposed is consistent with the widely used pseudo-static approach. They have also extended their study to consider the effect of amplification phenomena, and highlighted that the acting point of the seismic active thrust is very close to a height of $H/3$ from the base of the wall for input parameters for range of practical interest.

Other Methods

Karpurapu and Bathurst [41] presented the finite element models that were used to simulate the behaviour of two retaining walls. The models were carefully constructed to monitor the large-scale geosynthetic reinforced soil. The walls were constructed using a dense sand fill and layers of extensible polymeric geosynthetic reinforcements which were attached to two very different facing treatments. A uniform load was applied till the model walls collapsed. GEOFOAM, a lightweight geosynthetic block, was used as a fill to simulate this test experiment numerically.

Ho and Rowe [38] assessed the effect of different geometric parameters like length of geosynthetic reinforcement, its distribution along the height of the wall and the number of layers on the overall stability of a typical

reinforced soil retaining wall using finite element study. They adopted a rigid base beneath the structure, and concluded that the most critical parameter affecting the results in terms of stability of the wall was the ratio of geosynthetic reinforcement length and the height of the wall. Whereas, the number of geosynthetic reinforcement layers did not seem to affect the results to significant extent. Reddy et al. [56] proposed pseudo-dynamic solutions for reinforced soil–wall by considering oblique displacement.

Zeng [80] presented the behaviour of gravity quay walls under earthquake loading using data from three centrifuge tests. Zeng [80] studied the wedge angle wall, and ground settlement in the backfill, influence of pore pressure on the wedge angle using the pseudo-static approach. In the centrifuge model test for quay wall under seismic active earth pressure condition, as performed by Zeng [80], it is observed to settle 0.16 m vertically and moved out by 0.54 m horizontally without noticeable tilting. It was concluded that the results of limiting pore water pressure plays a very important role in the design method adopted.

Various other researchers developed methods for analysis and design of retaining structures in seismic areas. Shahgholi et al. [61] presented a horizontal slice method for investigating the seismic stability of reinforced soil walls. Shahgholi et al. [61] considered simplified formulation of the horizontal slice method by considering the vertical equilibrium for individual slices together with the overall horizontal equilibrium for the whole wedge. They assumed a polylinear failure surface, and showed that the results obtained using the polylinear failure surface were in fair agreement with the results produced using a log-spiral failure surface. Basha and Basudhar [10] proposed a closed-form solution for the calculation of seismic active earth pressure acting on reinforced soil structures ensuring both internal and external stability.

Atik and Sitar [5] conducted experimental and analytical programs to evaluate the magnitude and distribution of seismically induced lateral earth pressures on cantilever retaining structures with dry medium dense sand backfill. They concluded that designing the cantilever retaining walls for maximum dynamic earth pressure increment and maximum wall inertia is the current practice. However, they stated that it is overly conservative and does not reflect the true seismic response of the wall-backfill system. The results were presented in terms of relationship between the seismic earth pressure increment coefficient at the time of maximum overall wall moment and the PGA obtained from their experiments and suggested that seismic active earth pressures on the cantilever retaining walls can be neglected at accelerations below 0.4g.

In addition to the above researchers, Murali Krishna and Madhavi Latha [48–50] carried out extensive experimental analysis for reinforced soil–wall in 1-g shake table for various seismic input motions applied to the reinforced soil–wall to understand the behaviour of such wall under seismic loading conditions. Madhavi Latha and Murali Krishna [45, 46] also commented on the suitability of shake table tests on reinforced soil–wall under seismic conditions and the suitability of relative density of backfill material for better performance of such wall under seismic condition. However, it may be noted that 1-g shake table tests suffer from the inherent limitations due to scale effects related to the geotechnical engineering problems. Hence dynamic centrifuge test results are considered as more reliable experimental data which removes the major limitations of 1-g shake table tests to estimate the seismic earth pressures on retaining walls.

The limitations of the pseudo-static method are well understood and are well explained in the previous section. The pseudo-dynamic method considers the effect of shear and primary waves. However, it is well known that the Rayleigh wave carries a major portion of seismic energy. The Rayleigh wave is a surface wave, hence it exists near the surface, and it carries about 67 % of the total seismic energy (see Kramer [43]). Also, the existing pseudo-static and pseudo-dynamic methods do not satisfy the boundary conditions at the ground surface i.e. the shear and normal stresses do have a finite value at the ground surface (i.e. at $z = 0$) as reported by Katdare and Choudhury [42]. To overcome this limitation Choudhury and Katdare [17] developed a methodology to calculate seismic passive earth pressure by considering shear, primary and Rayleigh waves all together. Figure 5 shows the model considered by Choudhury and Katdare [17].

Figure 6 shows the variation of seismic passive earth pressure distribution using new dynamic approach as proposed by Choudhury and Katdare [17]. It was observed that the seismic earth pressure reduced significantly as the

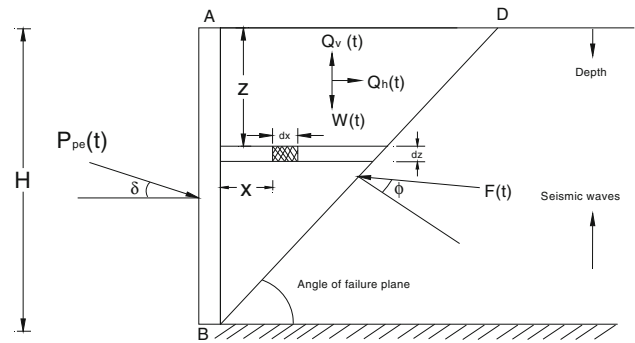


Fig. 5 Forces acting on rigid retaining wall for estimation of seismic passive earth pressure using new dynamic approach considering all seismic waves (modified after Choudhury and Katdare [17])

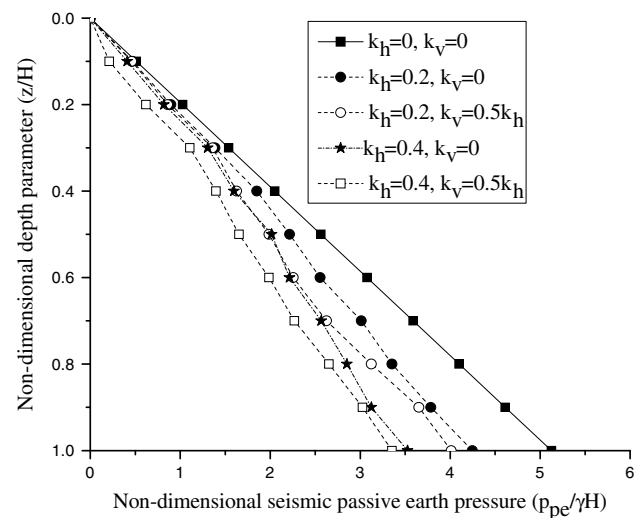


Fig. 6 Effect of k_h and k_v on seismic passive earth pressure distribution for $\phi = 30^\circ$, $\delta = \phi/2$ using new dynamic approach (modified after Choudhury and Katdare [17])

seismic acceleration coefficient increased. Table 1 shows that the design value obtained by using new dynamic method as given by Choudhury and Katdare [17] is more critical than that obtained by previous researchers.

The study by Choudhury and Katdare [17] showed a significant effect of the Rayleigh wave also in addition to shear and primary waves on the seismic passive earth pressure. Similar observations were also found and reported by Katdare and Choudhury [42] and Choudhury et al. [27] for the case of seismic active earth pressure.

Table 2 shows the comparative results of seismic active earth pressure coefficient obtained by various researchers. Katdare and Choudhury [42] highlighted that the Rayleigh wave should be considered along with shear and primary waves while calculating the seismic earth pressure for shallow depth, i.e., for structures like retaining wall, shallow anchors, and shallow footings. As can be seen from Fig. 7, Choudhury et al. [27] reported that the consideration of Rayleigh wave for computation

Table 1 Comparison of results of seismic passive earth pressure coefficient (K_{pe}) for $\phi = 30^\circ$, $\delta = (\phi/2)$, $k_h = 0.2$, $k_v = 0.1$ (modified after Choudhury and Katdare [17])

Value of seismic passive earth pressure coefficient (K_{pe})		
By pseudo-static method ^a	By pseudo-dynamic method ^b	By new dynamic method ^c
4.120	3.220	3.112

^a Pseudo-static method by Mononobe–Okabe method (see Kramer [43])

^b Pseudo-dynamic method by Choudhury and Nimbalkar [18]

^c New dynamic method by Choudhury and Katdare [17]

Table 2 Seismic active earth pressure coefficient (K_{ae}) obtained by various researchers

Method	Value of K_{ae}
Mononobe–Okabe (see Kramer [43])	0.39
Richards and Elms [57]	0.39
Das and Puri [30]	0.44
Greco [37]	0.41
Choudhury and Singh [22]	0.45
Shukla et al. [66]	0.32
Ghosh and Sharma [33]	0.39
Atik and Sitar [5]	0.44
Ghosh [35]	0.41
Shukla [62]	0.36
Katdare and Choudhury [42]	0.49

For input data: $k_h = 0.2$, $k_v = 0.5k_h$, $\phi = 30^\circ$, $\delta = \phi/2$

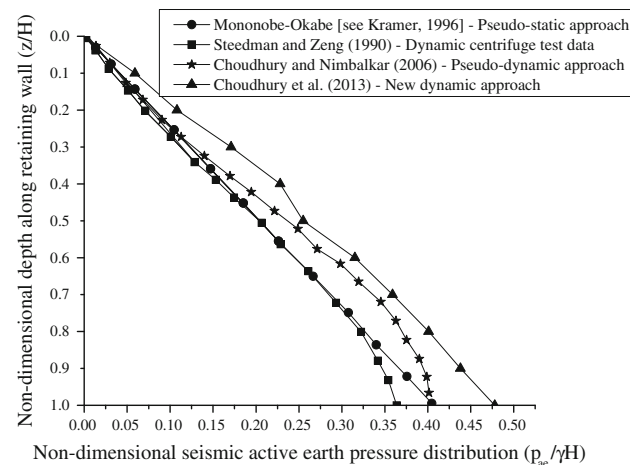


Fig. 7 Comparison of results for non-dimensional seismic active earth pressure ($p_{ae}/\gamma H$) versus non-dimensional depth (z/H) as obtained by various researchers for $k_h = 0.3$, $k_v = 0.15$, $\phi = 33^\circ$, $\delta = 16^\circ$ (modified after Choudhury et al. [27])

of seismic earth pressure is not only giving the compatibility of stress boundary conditions under dynamic loadings but also shows best comparison with the

dynamic centrifuge test results compared with other existing pseudo-static and pseudo-dynamic approaches for the analytical estimation of seismic earth pressures on retaining walls.

Codal Provisions

Indian Design Code

IS 1893—Part 3 [40] provides the information regarding earthquake resistant design of retaining walls for active and passive cases. Currently the pseudo-static method is used which is based on the Mononobe–Okabe method of analysis (see Kramer [43]) and excludes the deformation criteria. Being a force-based method it does not specify the permissible displacements also. It is known that the static component of the total pressure shall be applied at an elevation ($H/3$) above the base of the wall. IS 1893 arbitrarily mentions that the point of application of the dynamic increment shall be assumed to be at mid-height of the wall, for the active case. However, for the passive case, the static component of the total pressure can only be applied at an elevation ($H/3$) above the base of the wall. The point of application of the dynamic decrement shall be assumed to be at an elevation $0.5H$ above the base of the wall. This standard also mentions the effect of saturation on lateral earth pressure.

The seismic active earth pressure exerted against wall is estimated by,

$$P_{ae} = (1/2)\gamma H^2 C_a, \tag{10}$$

where

$$C_a = \frac{(1 \pm k_v) \cos^2(\phi - \lambda - \alpha)}{\cos \lambda \cos^2 \alpha \cos(\delta + \lambda + \alpha)} \times \left[\frac{1}{1 + \left\{ \frac{\sin(\phi + \delta) \sin(\phi - i - \lambda)}{\cos(\alpha - i) \cos(\delta + \alpha + \lambda)} \right\}^2} \right]^2, \tag{11}$$

where k_h is the horizontal seismic coefficient, $k_v (= 2k_h/3)$ is the vertical seismic coefficient, and ϕ is the angle of internal friction of soil, α is the angle which the earth face of the wall makes with vertical, i is the slope of earth fill, δ is the angle of friction between the wall and the earth fill, and,

$$\lambda = \tan^{-1}[k_h/1 \pm k_v]. \tag{12}$$

Similarly, the seismic passive earth pressure is computed by Eq. (13) as,

$$P_{pe} = (1/2)\gamma H^2 C_p, \tag{13}$$

where

$$C_p = \frac{(1 \pm k_v) \cos^2(\phi + \alpha - \delta)}{\cos \lambda \cos^2 \alpha \cos(\delta - \alpha + \lambda)} \times \left[\frac{1}{1 - \left\{ \frac{\sin(\phi + \delta) \sin(\phi + i - \lambda)}{\cos(\alpha - i) \cos(\delta - \alpha + \lambda)} \right\}^{\frac{1}{2}}} \right]^2 \tag{14}$$

European Design Code

Eurocode 8 [32] explains the design of earthquake resistant structures. The seismic calculations explained in Eurocode 8 [32] are based on the pseudo-static method that follows the displacement-based approach given by Richards and Elms [57]. Hence, the permissible displacements for the translation and rocking modes are considered in the analysis. It mentions that the, design seismic inertia forces, F_H and F_V acting on the ground mass, for the horizontal and vertical directions, respectively, in pseudo-static analyses shall be taken as (Clause 4.1.3.3),

$$\left. \begin{aligned} F_H &= 0.5jSW \\ F_V &= \pm 0.5F_H, \text{ if the ratio } a_{vg}/a_g \text{ is greater than } 0.6 \\ F_V &= \pm 0.33F_H, \text{ if the ratio } a_{vg}/a_g \text{ is not greater than } 0.6 \end{aligned} \right\} \tag{15}$$

where j is the ratio of the design ground acceleration on type A ground, a_g , to the acceleration of gravity g ; a_{vg} is the design ground acceleration in the vertical direction, a_g is the design ground acceleration for type A ground (also, topographic amplification factor for a_g shall be taken into account according to clause 4.1.3.2 (2), S is the soil parameter of EN 1998-1:2004, 3.2.2.2, W is the weight of the sliding mass.

Eurocode 8 [32] highlights the guidelines for selecting the values of k_h and k_v in absence of any study. Clause 7.3.2.2 of the Eurocode explains that in the absence of specific studies, the horizontal (k_h) and vertical (k_v) seismic coefficients affecting all the masses shall be taken as:

$$\left. \begin{aligned} k_h &= j \frac{S}{r}, \\ k_v &= \pm 0.5k_h, \text{ if } a_{vg}/a_g \text{ is larger than } 0.6 \\ k_v &= \pm 0.33k_h, \text{ otherwise} \end{aligned} \right\} \tag{16}$$

It assumes the point of application of dynamic increment at mid height of the wall and mentions that hydro-dynamic force for saturated backfill is assumed to act at $0.4H$ from the base of the wall.

International Building Code (IBC)

Earthquake loads are categorized into various categories by IBC [39]. Based on these categories, the seismic design can be carried out for retaining walls in earthquake prone areas.

IBC [39] mentions that the retaining walls shall be designed to ensure stability against overturning, sliding, excessive foundation pressure and water uplift. It mentions a factor of safety of 1.5 for designing retaining wall against lateral sliding and overturning.

Field Observations

Tatsuoka et al. [77] explained various case histories of reinforced wall performances which were constructed in Japan. It was named as ‘Tanata wall’, which was observed to perform well in the event of seismic activity. Even though the intensity was severe at the site, this wall was observed to perform better when compared with other type of unreinforced. The total length of the wall was 305 m, and the greatest height was 6.2 m. It was highlighted that the geotextile reinforced soil-retaining walls performed better when they were exposed to severe seismic loadings.

Sitar et al. [72] have studied the data from several case histories and the experimental work to show that the current methods are based on the conservative design procedure in region where PGA exceeds 0.4g. Based on the experimental data, they concluded that the seismic pressure distribution for the moderate size retaining structures, on the order of 6–7 m high, is triangular increasing with depth. They commented that there is no significant increase in seismic earth pressure between unbraced and braced structures with fixed base, while the loads on free standing cantilever structures are substantially lower owing to their ability to translate and rotate. A well documented case history of 1971 San Fernando earthquake was also presented. Clough and Frigaszy [29] showed that the concrete cantilever structures which was well designed and detailed for static loading, performed without any sign of distress at accelerations up to 0.4g. This was also observed by Seed and Whitman [60].

Requalification of Retaining Structures

Prakash [55] identified three questions that need to be answered when designing a retaining wall for seismic loads.

1. What is the magnitude of total earth pressure on the wall?
2. Where is the point of application of the resultant force?
3. How much is the displacement of the structure?

To answer these questions, various researchers have performed numerous analytical and experimental studies in the past and their results are very much in connection with their experimental design. The shake table and the

dynamic geotechnical centrifuge are the two most cost-effective approaches to assess the problem experimentally.

General guidelines given in Eurocode [32] for retaining wall design are given below:

1. The structure choice should be pertaining general principles of EN 1997-1:2004, Section 9.
2. Attempt should be made for careful grading and compaction of backfill material in situ.
3. Continuity with the existing soil mass is achieved, as much as possible.
4. Drainage systems behind the structure should be properly designed for absorbing transient and permanent movements and care should be taken that for their proper functioning.
5. In case of cohesionless soils, the draining shall be effective to well below the potential failure surface behind the structures.

Eurocode [32] mentions that in absence of a more detailed study, the point of application of the dynamic earth force shall be taken at mid-height of the wall. Choudhury et al. [26] mentioned the following points, as a requalification measure for a retaining wall against seismic activity:

1. For the design of retaining wall under seismic condition, maximum active earth pressure and minimum passive earth pressure should always be considered, as they are critical.
2. Special care must be given while assuming the point of application of seismic earth pressure based on some logical analysis, rather than some arbitrary selection of values.
3. In displacement-based analysis, wall dimensions should be determined for the following factor of safety values. For sliding = 1.5, for overturning = 1.5, and for bearing capacity = 2.5. The eccentricity should be (1/6)th of the base size/width.
4. Cumulative displacements and rotations of the wall then must be calculated and checked for different loadings based on the magnitude of earthquake.
5. In displacement-based analysis, if computed displacements are more than permissible displacements, the wall section should be redesigned in which the computed displacements are less than permissible displacements.

Worked Out Example of Requalification of Typical Rigid Retaining Wall Under Seismic Loading Condition

Problem statement: For re-qualification of a rigid vertical retaining wall, the following input data are provided: unit

weight of backfill soil (γ_s) = 16 kN/m³, unit weight of wall (γ_w) = 24 kN/m³, soil friction angle (ϕ) = 30°, wall friction angle (δ) = 15°, vertical height of the wall (H) = 10 m, wall base friction angle (ϕ_b) = 30°, amplitude of seismic horizontal acceleration (a_h) = 0.2g (hence $k_h = 0.2$), amplitude of seismic vertical acceleration (a_v) = 0.1g (hence $k_v = 0.1$), soil amplification factor (f) = 1.2, shear wave velocity in soil (V_{ss}) = 100 m/s, primary wave velocity in soil (V_{ps}) = 187 m/s, shear wave velocity in wall (V_{sw}) = 2,500 m/s, primary wave velocity in wall (V_{pw}) = 3,900 m/s, and frequency of input ground motion = 3 Hz.

Determine:

- (a) Design value of seismic active earth pressure coefficient (K_{ae}) by using various available methods viz., pseudo-static approach, pseudo-dynamic approach, new dynamic approach considering all seismic waves, IS code method, and EURO code method.
- (b) Design value of seismic passive earth pressure coefficient (K_{pe}) by using various available methods viz., pseudo-static approach, pseudo-dynamic approach, new dynamic approach, IS code method, and EURO code method.
- (c) The sliding stability of retaining wall to assess the wall against sliding mode of movement.
- (d) The rotational displacement of retaining wall for a critical case of $k_h = k_v$.
- (e) The rotational displacement of the wall by using pseudo-dynamic approach together with composite curved rupture surface.
- (f) Comment on the seismic stability for sliding and rotational modes of failure for the same wall section if it is used as a waterfront retaining wall (active case) which needs to be designed for supporting a free standing water height of 50 % of wall height in one side and the water table on backfill side is of 75 % of wall height with pore pressure ratio = 0.2. Consider the saturated unit weight of soil = 19 kN/m³.

Solution:

I. Pseudo-static approach

- (a) Seismic active pressure coefficient by using Mononobe–Okabe method (see Kramer [43]),

$$K_{ae} = \frac{\cos^2(\phi - \theta - \psi)}{\cos\psi \cos^2\theta \cos(\delta + \theta + \psi) \left[1 + \sqrt{\frac{\sin(\delta + \phi) \sin(\phi - \beta - \psi)}{\cos(\delta + \theta + \psi) \cos(\beta - \theta)}} \right]^2},$$

where $\psi = \tan^{-1}[k_h/1 - k_v]$.

Substituting the input values into the above expression, one gets, $K_{ae} = 0.39$.

- (b) Seismic passive pressure coefficient by using Mononobe–Okabe method (see Kramer [43]),

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

Substituting the input values in the above expression, one can get, $K_{pe} = 4.120$.

II. Pseudo-dynamic approach

Given, $k_h = 0.2$ and $k_v = 0.1$ and $f = 1.2$

- (a) (i) Calculation of seismic active earth pressure coefficient *without amplification*:
Using the formulation given by Choudhury and Nimbalkar [19], substituting into equation and optimizing for (t/T) and α , one gets, $K_{ae} = 0.51$.
- (ii) Calculation of seismic active earth pressure coefficient *with amplification*:
Nimbalkar and Choudhury [52] presented a closed-form solution for calculating K_{ae} . For amplification factor, $f = 1.2$, substituting input values into the above equation and optimizing (getting maximum value) in terms of (t/T) and α_a , one can get, $K_{ae} = 0.57$.
- (b) (i) Calculation of seismic passive earth pressure coefficient *without amplification*:
Substituting the given input values in equation given by Choudhury and Nimbalkar [18] and optimizing with respect to (t/T) and α , one can get, $K_{pe} = 3.220$
- (ii) Calculation of seismic passive earth pressure coefficient *with amplification*:
For $f = 1.2$, by adopting the procedure given by Nimbalkar and Choudhury [52], and optimizing with respect to (t/T) and α_p , one can get, $K_{pe} = 2.71$.

III. New dynamic approach considering all seismic waves

- (a) Seismic active earth pressure coefficient:
Calculations have been done in accordance with the new dynamic approach developed by Katdare and Choudhury [42] by considering effect of Shear, Primary and Rayleigh waves all together. Substituting the input values one gets, $K_{ae} = 0.49$.
- (b) Seismic passive earth pressure coefficient:
New dynamic approach developed by Choudhury and Katdare [17] by considering all seismic waves is used to estimate the seismic passive

earth pressure coefficient using the input values. One gets, $K_{pe} = 3.112$

IV. IS code method: (IS 1893—Part 3 [40])

- (a) Seismic active earth pressure coefficient:
Substituting the input values into the equation given in IS 1893—Part 3 [40], one can get, $C_a = 0.38$.
- (b) Similarly, seismic passive earth pressure coefficient can be obtained by using the equation given in IS 1893—Part 3 [40], and $C_p = 4.269$.

V. EURO Code method: [Eurocode 8 [32]]

- (a) Seismic active earth pressure coefficient:
For group A, $S = 1.0$, $\alpha = 0.1$, assume $r = 2$. Given, $k_h = 0.2$, $k_v = 0.1$. Substituting above values and other input values, one can get, $K_{ae} = 0.39$.
- (b) Seismic passive earth pressure coefficient:
Here also, for group A, $S = 1.0$, $\alpha = 0.1$, assume $r = 2$. Using other given input values, one gets, $K_{pe} = 4.120$.
Recommendations: As obtained in Table 3, for the safe design of rigid retaining wall under seismic loading condition for re-qualification of retaining structures, it is recommended to use the most critical values for seismic active and passive earth pressure coefficients. Hence the maximum value of seismic active earth pressure coefficient and the minimum value of seismic passive earth pressure coefficient will satisfy the re-qualification of the retaining walls.

VI. Sliding stability of retaining wall

The values of factor of safety against sliding mode of failure for rigid retaining wall are calculated by adopting the pseudo-dynamic approach given by Nimbalkar and Choudhury [53], Active case: for $f = 1.2$, Combined safety factor, $F_w = 4.00$; Inertia safety factor, $F_I = 2.79$; Thrust safety factor, $F_T = 1.43$.

The weight of the wall required (W_w) for sliding stability of wall is calculated for active case, by using the formulations given by Nimbalkar and Choudhury [53], $W_w = 890$ kN.

Again, the values of factor of safety against sliding mode of failure for rigid retaining wall are also calculated by adopting the pseudo-static approach given by Richards and Elms [57], Active case: for $f = 1.2$, $F_w = 4.15$, $F_I = 2.84$, $F_T = 1.51$.

The weight of the wall is calculated for active case, $W_w = 898$ kN.

Table 3 Values of seismic active (K_{ae}) and passive (K_{pe}) earth pressure coefficients obtained by various methods

S. No.	Method	K_{ae}	K_{pe}
1	M–O method (pseudo-static method)	0.39	4.120
2	Pseudo-dynamic method	0.51	3.220
3	New dynamic method	0.49	3.112
4	IS 1893 Part 3	0.39	4.269
5	EURO code 8	0.39	4.120

For passive case, the values of factor of safety against sliding mode are calculated by adopting the pseudo-dynamic procedure given by Nimbalkar and Choudhury [51], $F_w = 0.39$, $FI = 0.71$, $FT = 0.54$. Hence, the minimum weight of the wall (W_w) can be computed as, $W_w = 125$ kN.

Eurocode 8 recommends that the permissible lateral/sliding displacement for rigid retaining wall should be, $300 \alpha S$ (mm) = 30 mm.

Richards and Elms [57] recommended that the permissible displacement should be calculated using Eq. (3) as given in this paper. Hence one can obtain the sliding displacement if the v_{max} and a_y for a particular seismic zone is known/estimated using the wall-backfill system as suggested by Richards and Elms [57].

IS code 1893 gives only force-based approach and hence it does not comment on estimation and recommendation of permissible displacement of retaining wall.

VII. Rotational displacement of rigid retaining wall

Rotational displacement can be computed by pseudo-static approach by using the formulation given by Richards and Elms [57].

The value of rotational displacement for active case works out to be 2.1° .

The value of rotational displacement for passive case comes to be 3.2° .

However, using the pseudo-dynamic approach as proposed by Nimbalkar and Choudhury [53], for active case: The total angle of displacement = 2° .

For passive case, the pseudo-dynamic approach adopted is given by Choudhury and Nimbalkar [20], and one can get, The total angle of displacement = 3° .

The above calculations are based on the theories where only planar rupture surface were used. But it is noted by the researchers that for passive case of earth pressure, composite/curved rupture surface is the actual failure surface compared to planar one. Hence the rotational displacements of the wall by pseudo-dynamic approach with composite curved

rupture surface by using the approach given by Basha and Babu [9] are worked out as follows:

The value of critical seismic acceleration coefficient for rotation for passive case = 0.41 (refer to Fig. 4b). The value of rotational displacement for passive case = 0° as the input value of $k_h = 0.20$ is less than 0.41 (critical seismic acceleration coefficient for rotation).

VIII. Sliding and rotational stability of rigid waterfront retaining wall

For sliding stability with active case and no amplification:

As per the given theory and design charts of Ahmad and Choudhury [3], for the given dataset, $F_{w(wet)} = 5.319$, compared to the retaining wall in dry case having $F_w = 3.217$ as per design charts given by Nimbalkar and Choudhury [53]. It clearly shows that for sliding stability of the waterfront retaining wall under given seismic loading conditions, extra weight of wall (W_w) required = 2.102 times the weight of the wall required for sliding stability under static loading condition.

For rotational stability with active case and no amplification:

And the theory and design charts of Ahmad and Choudhury [4] shows that for the given dataset, rotational displacement of wall = 0° , because the critical (threshold) seismic acceleration coefficient (i.e. beyond which the wall rotates) $k_{tr} = 0.351$, as against the actual seismic acceleration coefficient $k_h = 0.2$. Hence no rotation of the waterfront wall is expected under the given loading conditions.

Recommendation: For seismic re-qualification of rigid retaining wall, both sliding and overturning/rotational displacements need to be computed and checked with the permissible values along with the estimation of factor of safety, if applicable.

Conclusions

Determination of the realistic values of seismic active and passive earth pressures is important for the cost-effective design of retaining structures in the earthquake-prone areas. The pseudo-static method is still the first approach for analyzing retaining structures in seismic areas because of its simplicity, though it has some serious limitations. The pseudo-dynamic method considers the time-dependant nature of the earthquake force. Hence it gives more accurate values of dynamic earth pressures compared with those determined by the pseudo-static method. Recent researches show that the Rayleigh wave significantly affects the

seismic earth pressures; and it got validated by dynamic centrifuge test results. Hence, the Rayleigh wave should be considered along with shear and primary waves while calculating the seismic earth pressure at shallow depths, that is, for structures like retaining walls, shallow anchors, and shallow footings. It is better to use the displacement-based analysis rather than using the force-based analysis for designing retaining walls. Other methods which use the tools like the finite element method, and the method of horizontal slices can also be considered for analysis of retaining structures in seismic-prone areas. Seismic re-qualification techniques as explained in different codes and shown here through an example should be studied for the safe seismic design of retaining structures.

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