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Seismic analysis and design of embedded cantilever retaining wall considering non-linear earth pressure distribution effect

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ABSTRACT: In the present study, a simple limit equilibrium approach has been presented for the design of embedded cantilever retaining walls (ECRW) subjected to earthquake loads. Effect of non-linear seismic earth pressure distribution has been taken into consideration by taking different points of application for the static and dynamic components of seismic active earth thrusts. Dry cohesionless soils have been considered in the analyses. Parametric studies have been performed by varying internal friction angle of soil, wall friction angle, excavated to embedment depth ratio. It is seen that the values of critical seismic acceleration and maximum bending moment on the wall increase with increase in strength of the soil and higher height to embedment ratio. Moreover, the present approach gives a safer estimation of bending moment and critical seismic acceleration when compared with the existing pseudo-static methods and match fairly well with the published numerical results.

1 INTRODUCTION

Embedded retaining walls are extensively used in excavations, cofferdams, slope stabilization, flood protection wall and quay walls. Failures of these walls often occur during major earthquakes (e.g 2001 Bhuj earthquake). Therefore, accurate analysis and safe optimum design of the wall are highly desirable in earthquake-prone regions. If stability analysis of the soil-wall system and the design of the wall are done by considering maximum seismic acceleration which is expected at the site, the soil wall system will never reach up to the limiting state of collapse. Thereby, theoretically, no permanent displacement of the wall is going to take place at the end of an earthquake. However, considering the maximum possible seismic acceleration in the seismic design will cause unnecessary conservative and exorbitant design. Hence, a reduced equivalent seismic acceleration, i.e. critical acceleration (a_c), can be adopted for the design, due to that during an earthquake there will be time gaps in which the soil-wall system will reach the limiting condition and the wall will undergo permanent displacement. If the permanent displacement remains within the permissible limit that a wall can sustain, the critical acceleration a_c can be used for the stability analysis and structural design of embedded retaining walls. Magnitude of the permanent displacement can be determined by the Newmark (1965) sliding-block method which is originally proposed for rigid retaining wall. In the sliding-block method, permanent displacement of the wall is estimated by integrating the relative acceleration $\{a(t) - a_c\}$ over the duration of the earthquake. Where $a(t)$ is the seismic acceleration at any time t. Studies carried out by Neelakantan et al. (1992), Richards & Elms (1992) and Callisto & Soccodato (2010), showing that for singly propped embedded retaining wall, the Newmark's sliding block method predicts permanent displacement of the wall caused by an earthquake reasonably well.

Though there are several studies available for the static and seismic analysis and design of ECRWs carried out by Powrie (1996), Choudhury et al. (2006), Diakoumi & Powrie (2013).

Also, experimental studies carried out by Zeng & Steedman (1993), Conti et al. (2012), and numerical studies by Callisto et al. (2008), Cilingir et al. (2011), deal with the problem of seismic analysis and design of ECRWs and propped wall. Yet, none of these studies gives a general formulation to calculate a_c and the corresponding design values of internal forces. Traditionally, the pseudo-static analytical approach with limit equilibrium method is used for the estimation of a_c and design of ECRWs subjected to earthquake load (Callisto & Soccodato 2010, Conti & Viggiani 2013, Conti et al. 2014, Conte et al. 2017). In which, the dynamic nature of the seismic force is disregarded and the seismic force is represented by an equivalent static force which is proportional to a maximum seismic acceleration anticipated at the ground surface. Okabe (1926) and Mononobe & Matsuo (1929) first proposed the pseudo-static approach which is familiar as Mononobe-Okabe method (see Kramer, 1996) that predicts the magnitude of seismic active and passive earth thrust reasonably correct. However, the method does not provide any hint about the actual point of application of seismic earth thrusts. Generally, the position of the point of application of seismic earth thrust is considered at one-third height of the wall above the base and the distribution is considered as linear irrespective of wall movement mode and magnitude. However, many analytical, experimental and numerical studies such as Ichihara & Matsuzawa (1973), Sherif & Fang (1984), Ishibashi & Fang (1987), Steedman & Zeng (1990), Choudhury & Rao (2002) and Choudhury & Nimbalkar (2006) show that the seismic earth pressure distribution is nonlinear and the point of application of the seismic earth thrust is located above one third height of the wall. Still, in the existing analytical studies, linear triangular distribution of earth pressure and the point of application of seismic thrust at the one-third height of the wall are considered which severely underestimates the wall stability and internal forces.

In this paper, existing pseudo-static limit equilibrium methods for the analysis and design of ECRWs have been discussed. The main shortcomings of the existing methods are pointed out and a modified pseudo-static limit equilibrium method has been proposed which is able to calculate critical seismic acceleration and internal forces of ECRWs more accurately compared to the available pseudo-static methods.

2 SEISMIC BEHAVIOUR OF EMBEDDED CANTILEVER RETAINING WALLS

2.1 *Existing pseudo-static approaches*

Callisto & Soccodato (2010) extended the Blum (1931) method in order to calculate the critical acceleration a_c and internal forces for ECRWs and compared it with numerical studies. In this method, it is considered that the seismic active earth pressure K_{ae} is fully mobilised in the backfill/retained side of the wall to a depth (h+d_0) from the ground level and a factored seismic passive pressure, K_{pe}/F is mobilised below the excavation level down to the depth d_0 when the magnitude of horizontal seismic acceleration a_h (=$k_h g$) is less than the value of critical acceleration a_c (=$k_c g$) (Figure 1a). However, when the horizontal seismic acceleration attains the critical value a_c, the factor of safety becomes unity (F=1) which signifies full mobilisation of the seismic passive earth pressure at the excavated side. But when the results above are compared with the numerical and experimental studies, the Blum's method gives an over-conservative estimation of a_c and underestimate the maximum bending moment (M_{max}) an ECRW can experience during an earthquake. These differences arise because in the Blum's method, distribution of seismic active earth pressure is considered as hydrostatic and a fixed position of the point of rotation is considered at a depth of *0.8d* from the excavation level. Conti & Viggiani (2013) and Conti et al. (2014) modified the Blum's method and assumed a horizontal seismic earth pressure distribution as shown in Figure 1b. However, when the results obtained from the above studies are compared with numerical and experimental results (Conti & Viggiani 2013), comparison clearly shows that these studies underestimate the value of a_c and M_{max} both. This disparity is due to consideration of static passive earth pressure along the excavated depth and linear, triangular distribution of seismic active earth pressure over the depth down to h+d_0 at the retained side of the wall thus ignoring the higher point of application of seismic active thrust (P_{ae}).

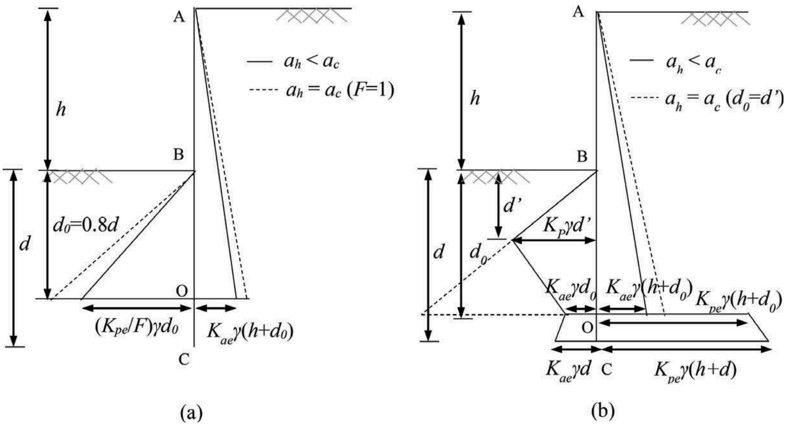

Figure 1. Seismic earth pressure distributions for flexible retaining walls considered by (a) Callisto & Soccodato (2010) (Blum's method 1931) (b) Conti & Viggiani (2013) and Conti et al. (2014).

2.2 *Proposed method*

The pseudo-static limit equilibrium method is employed to determine the a_c and the M_{max} for ECRWs subjected to earthquake loading. The distribution of horizontal seismic earth pressures considered in this study is shown in Figure 2. Here, it is assumed that the wall is rotating about a pivot point 'O' located near the tip of the wall and the seismic active earth pressure is fully mobilized at the retained side of the wall down to the depth (h+d_0). The effect of the nonlinear distribution of seismic active earth pressure and a higher point of application of seismic active earth thrust given by Seed & Whitman (1970) have been considered (described in section 2.2.1). On the dredged side, the seismic passive earth pressure is fully mobilised down to a depth d'. Below d' depth, it is partially mobilised and decreases linearly down to a depth d_0. Moreover, when the horizontal seismic acceleration coefficient k_h reaches the value of critical acceleration coefficient k_c, full seismic passive earth pressure mobilised up to the depth d_0. Below the pivot point, seismic active state and passive state exist at the excavated and retained side of the wall respectively. Value of the seismic active (K_{ae}) and passive (K_{pe}) earth pressure coefficient have been computed by using the Mononobe-Okabe pseudo-static approach. Vertical seismic acceleration has been ignored in this particular study.

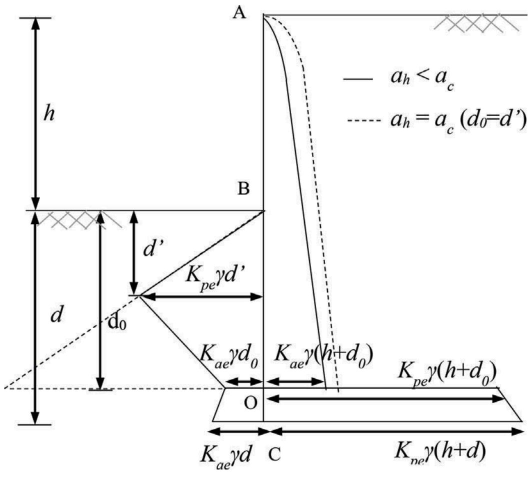

Figure 2. Seismic earth pressures distribution on flexible retaining walls as considered in the present method.

2.2.1 *Seismic active earth pressure distribution and its point of application*

As mentioned in the introduction, the distribution of seismic active earth pressure is non-linear and the point of application of P_{ae} is located above one-third height of the wall from the base. Seed & Whitman (1970) suggested that the component of static active thrust (P_a) act at a height $H/3$ and the dynamic component of the seismic active thrust (ΔP_{ae}) act at a height of $0.6H$ from the base, where H is the height of the wall, essentially ($h+d_0$) in this case. Therefore, the point of application of P_{ae} at the retained side of the wall (see Figure 2) can be determined by the given expression

$$\bar{y} = \left(\frac{\frac{P_a}{3} + 0.6\Delta P_{ae}}{P_{ae}}\right)(h + d_0) = \left(0.6 - 0.266\frac{K_a}{K_{ae}}\right)(h + d_0) \tag{1}$$

where $\bar{y}$ is the height of the point of application of P_{ae} from the point of rotation 'O', K_a is the static active earth pressure coefficient, h is the excavation depth or retained height, d_0 is the depth of pivot point from the dredged level.

By using Mononobe and Okabe method, P_{ae} can be calculated by

$$P_{ae} = \frac{1}{2}\gamma K_{ae}(h + d_0)^2 \tag{2}$$

And the moment of P_{ae} about the ground surface is given by

$$M_{ae} = P_{ae}\left[1 - \left(0.6 - 0.266\frac{K_a}{K_{ae}}\right)\right](h + d_0) \tag{3}$$

Now, it is assumed that seismic active earth pressure at any depth y from the ground surface can be expressed as

$$p_{ae_y} = Ky^m \tag{4}$$

Where K and m are constant.

From equation (4), the total seismic active earth thrust P_{ae} can be computed by

$$P_{ae} = \int_0^{h+d_0} Ky^m dz = \frac{K(h + d_0)^{m+1}}{m + 1} \tag{5}$$

Similarly, the moment of P_{ae} about the ground surface can be computed by

$$M_{ae} = \int_0^{h+d_0} Ky^m y dz = \frac{K(h + d_0)^{m+2}}{m + 2} \tag{6}$$

By solving equation 2, 3, 5 and 6, one can obtain the value of K and m. Accordingly, seismic active earth pressure distribution at the retained side of the wall can be calculated.

2.2.2 *Stability analysis of the embedded cantilever retaining wall*

In order to guarantee the stability of ECRWs, both force balance and moment equations must be satisfied. In the force equilibrium, it is a general practice for embedded retaining walls to assumed vertical equilibrium is automatically satisfied as the weight of the wall is negligible compared to the horizontal forces acting on it.

The balance equation in the horizontal direction is obtained by equating the forces acting on the left-hand side (P_{LHS}) and on the right-hand side (P_{RHS}) of the wall:

$$P_{LHS} = \frac{1}{2}\gamma K_{pe} d'^2 \cos\delta + \frac{1}{2}\gamma\left(K_{pe}d' + K_{ae}d_0\right)(d_0 - d\prime)\cos\delta + \frac{1}{2}\gamma K_{ae}\left(d^2 - d_0^2\right)\cos\delta \qquad (7)$$

$$P_{RHS} = P_{ae}\cos\delta + \frac{1}{2}\gamma K_{pe}(2h + d + d_0)(d - d_0)\cos\delta \qquad (8)$$

Also, the moment equilibrium is satisfied by taking moment about the pivot point 'O':

$$M_o^{anticlockwise} = \left[P_{ae}\bar{h} + \frac{1}{6}\gamma K_{ae}(d - d_0)^2(2d + d_0)\right]\cos\delta \qquad (9)$$

$$M_o^{clockwise} = \frac{1}{2}\gamma\cos\delta\left[\begin{array}{c} K_{pe}d'^2\left(d_0 - \frac{2}{3}d'\right) + \frac{1}{3}\left(K_{pe}d' + K_{ae}d_0\right)(d_0 - d')^2\left(\frac{K_{ae}d_0 + 2K_{pe}d'}{K_{ae}d_0 + K_{pe}d'}\right) \\ + \frac{1}{3}K_{pe}(3h + 2d + d_0)(d - d_0)^2 \end{array}\right] \qquad (10)$$

By solving equations 7, 8, 9 and 10, one can obtain the value of *d'* and d_0. Also, the bending moment distribution, the maximum bending moment and the shear force acting on the wall can be derived once the distribution of seismic active and passive earth pressures are known on both sides of the wall.

3 RESULTS AND DISCUSSION

Variation of critical seismic acceleration coefficient k_c against the *h*/*d* ratio has been plotted in Figure 3 for different values of internal friction angle of soil ϕ (25°, 30° and 35°) which represent loose to medium dense sand. Also, the study has been carried out for different values of soil-wall friction angle δ (δ =0, 0.25ϕ and 0.5ϕ). It can be noticed that with the increasing soil friction angle and soil-wall friction angle, the value of critical acceleration increases substantially. Also, with the increment of *h*/*d* ratio a drastic decline in k_c value can be observed.

Figure 4a exhibits the variation of *d'*/*d* and d_0/d with the ratio of k_h/k_c. Four different values of *h*/*d* have been considered and it can be seen that under static conditions (k_h =0), the ratio of *d'*/*d* strongly depends on the *h*/*d* ratio and soil strength. When the seismic acceleration reaches the critical acceleration (k_h=k_c), which is refers as critical condition, the passive earth resistance gets fully mobilised below the dredged level down to the point of rotation. Also, it is interesting to observe that the position of the pivot point locates nearly at the bottom tip of the wall under no earthquake condition and gradually moves upward as the applied seismic acceleration increases. It has been observed that at the critical acceleration, the point of rotation is located nearly at a depth of 0.9*d* irrespective of the *h*/*d* ratio and soil strength. It occurs,

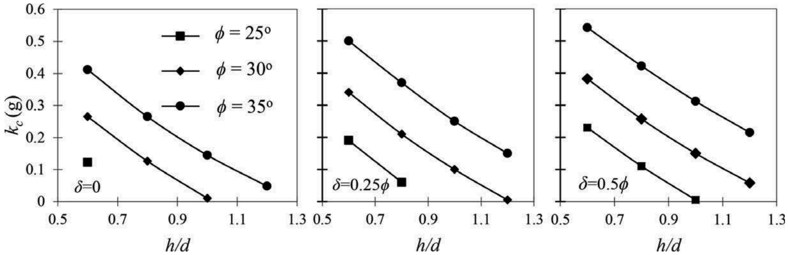

Figure 3. Variation of critical acceleration coefficient k_c with *h*/*d* for various soil and wall friction angles.

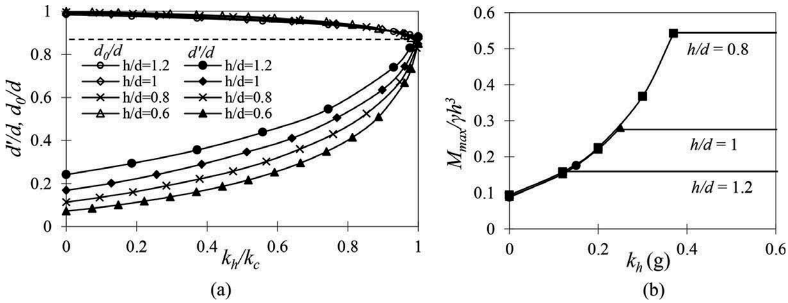

Figure 4. Variation of (a) d'/d and d_0/d with k_h/k_c and (b) maximum normalized bending moment with k_h (for ϕ=35°; δ = 0.5ϕ; γ = 18kN/m^3; h = 4m).

because, nature of the earth pressure distribution is almost identical for all soil-wall systems when they attain their respective critical seismic acceleration regardless of the h/d ratio and soil shear strength.

Figure 4b shows variation of the normalised maximum bending moment $M_{max}/\gamma h^3$ as a function of k_h for different values of h/d and soil parameter ϕ =35°; δ =0.5ϕ; γ =18kN/m^3. The value of $M_{max}/\gamma h^3$ increases with the increment of h/d and k_h and reaches the peak at k_h=k_c. Beyond k_c, even if the k_h value increases, there is no further increase in the value of M_{max} and remains constant thereafter, as soil mass at the retained side of wall reaches the passive state of plastic equilibrium and does not offer any further resistance against the wall movement or increasing k_h.

Distributions of the horizontal earth pressure and the bending moment calculated with the present method have been presented in Figures 5a, b for different values of k_h. As the applied seismic acceleration k_h increases, the seismic active earth pressure and the bending moment increases and reach to their respective maximum values at k_h=k_c. The nonlinear distribution of seismic active earth pressure at the retained side of the wall taken in the present study can be seen in Figure 5a. The degree of nonlinearity increases with the increase of k_h. On the excavated side of the wall, fully mobilised passive earth pressure zone gradually propagate downward with the increase of k_h and it reaches down to the pivot point (d'=d_0) when the seismic acceleration attains its critical value (k_h=k_c).

4 COMPARISON OF RESULTS

Figure 6a depicts a typical comparison of bending moment distributions behind an ECRW obtained by the proposed method with existing pseudo-static (Conti & Viggiani 2013), numerical (Conti et al. 2014) studies and with the Blum method for the parameters of k_h =0.2, ϕ =35°, δ =20°, γ =20kN/m^3 and h = 4m, d = 4m. Since numerical model is capable of imitating actual field condition in the problem solution and furnishes result close to field values, the numerical results proposed by Conti et al. (2014) are taken as the reference solution in this comparison. The comparison shows that bending moment distribution calculated by Conti & Viggiani (2013) and the Blum method are not agreeing well with the numerical study and clearly underestimate at any seismic acceleration below the critical acceleration. Whereas bending moment distribution calculated by the present study is matching very well. Because in the Blum method, a factored, triangular passive earth pressure distribution is considered at the excavated side in lieu of partially mobilised earth pressure for a_h<a_c. Also, a constant position of the point of application of seismic active earth thrust is considered in this method, instead of taking the actual position which gets shifted upward depending on the severity of an earthquake. However, for Conti & Viggiani (2013), even though it consider partial mobilisation of earth pressure along the excavated depth, yet it does not consider the actual point of application of seismic active earth thrust. For these same reasons, the Blum method

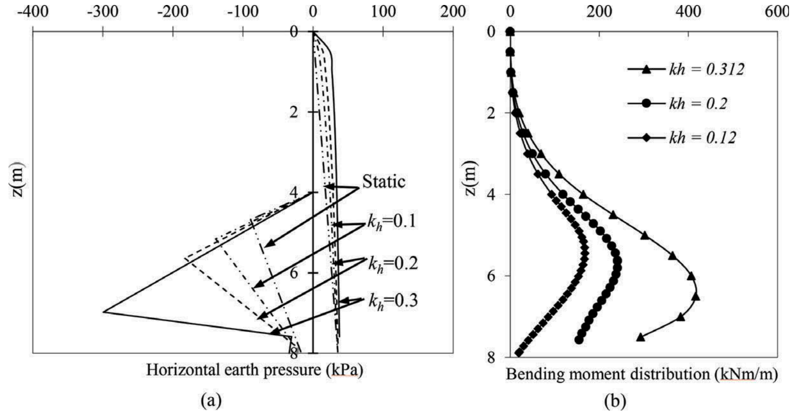

Figure 5. Distribution of (a) horizontal earth pressure and (b) bending moment for various values of k_h (ϕ=35°; $\delta = 0.5\phi$; γ = 18kN/m^3; h = 4m).

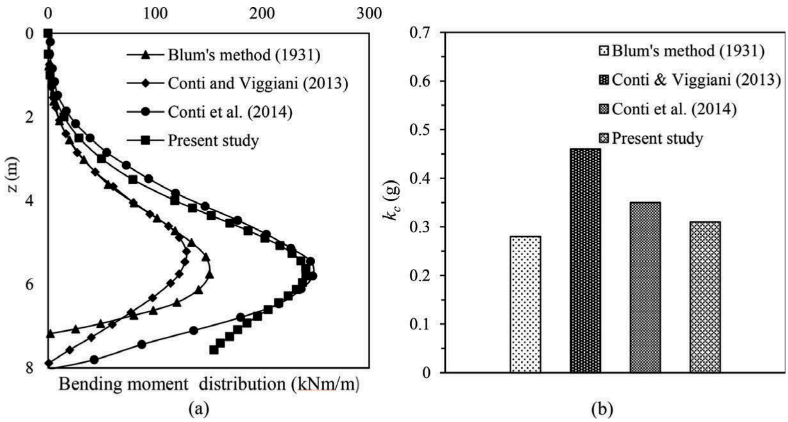

Figure 6. Comparison of present results with the existing pseudo-static and numerical results.

underestimates and Conti & Viggiani (2013) highly overestimates the value of k_c, which can be seen in Figure 6b. But the proposed method predicts the value of k_c fairly well when compared with the numerical result obtained by Conti et al. (2014) since the above mentioned shortcomings are addressed in this study. In Figure 6b, results are calculated for a single set of parameter, i.e. h/d =1, ϕ =35°, δ =20°, γ =20kN/m^3 for comparison.

5 CONCLUSIONS

In the present study, a modified pseudo-static limit equilibrium method has been proposed for the design of embedded cantilever retaining walls. Instead of taking a linear triangular earth pressure distribution at the retained side of the wall, a nonlinear distribution and a higher point of application of seismic active earth thrust are considered. A parametric study has been carried out by varying geotechnical parameters such as internal friction angle of soil ϕ, wall-soil friction angle δ and geometrical parameter like the height of the excavation h and depth of embedment d. Results show that the critical seismic acceleration and the maximum bending moment acting on the wall become higher with the increase of soil shear strength and embedment depth of the wall. Also, the results from the present study have been compared with the existing limit equilibrium method and numerical studies and it is evident that the proposed analytical method gives a better estimation of critical seismic acceleration and maximum bending moment than the existing pseudo-static methods when compared with numerical results. Hence, this approach can be effectively used for practical design purposes.

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