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Seismic Earth Pressure Analysis of Retaining Wall

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Abstract **The determination of seismic earth pressure of retaining wall plays a crucial role in appropriate design of retaining structures. However, a current analytical method consists of various assumptions, complicated calculations and required lot of time to get results. This paper, compared different methods of analysis such as conjugate stress method, pseudo dynamic method, limit equilibrium method, kinematic limit analysis and displacement based approach of determining seismic earth pressure. The results show that seismic earth pressure coefficient increases with increasing parameters viz. angle of wall friction, slope of backfill and decreases with increase of angle of internal friction. Also, Pseudo Dynamic method gave the optimum value of seismic earth pressure coefficient while Displacement Based Approach gave higher value of seismic earth pressure coefficient. A program on MATLAB was also developed for calculation of seismic earth pressure.**

Keywords — seismic earth pressure, conjugate stress method, pseudo dynamic method, limit equilibrium method, kinematic limit analysis and displacement based approach

INTRODUCTION

The dynamic response of even simplest type of retaining wall is quite complex. The determination of seismic active earth pressure is essential for safe design of retaining wall in the seismic zone. Earthquake-resistant design of earth retaining structures like retaining walls, earth dams and foundations are very important problems to minimize the devastating effect of earthquake hazards[1]. Many researchers have developed several methods to determine the seismic active earth pressure on a rigid retaining wall due to earthquake loading. The pioneering work on earthquake induced earth pressure under active and passive condition acting on retaining wall had been carried out since 1926. Furthermore, the work is being carried out using forced based and displacement based approach. In this paper five different methods of seismic earth pressure analysis were studied and compared. For the calculation of seismic earth pressure based on Mononobe - Okabe method and Conjugate Stress method a program on MATLAB software was developed.

LITERATURE REVIEW

The most commonly adopted method for determining the dynamic lateral pressure on retaining structures was developed by Mononobe and Okabe. The method was developed for dry cohesion less materials. The active pressure during the earthquake was computed by the Coulomb theory except that the additional forces kh W and kv W were included in the computation

Based on pseudo-static analysis and the concept of intermediate soil wedge with curved surface, a new methodology was developed to evaluate seismic earth pressures under any condition between the active and passive states. An equivalent seismic coefficient was introduced to take into account non-uniform seismic acceleration distribution with depth. This new earth pressure equation was only suited for normally consolidated soils [2].

The pseudo-dynamic method which was used to compute the distribution of seismic active earth pressure on a rigid retaining wall supporting cohesion less backfill in more realistic manner by considering time and phase difference within the backfill. Planar rupture surface was considered in the analysis. Effects of a wide range of parameters like wall friction angle, soil friction angle, shear wave velocity, primary wave velocity and horizontal and vertical seismic accelerations on seismic active earth pressure had been studied [3].

A simple analytical method for analysis of reinforced retaining walls with cohesive-frictional backfill under pseudo-dynamic loadings was proposed for inclined retaining walls. Based on Horizontal Slice Method a new formulation was obtained for determining the characteristics of inclined walls in granular and or frictional cohesive soils. The suggested formulation for n layers (slices), including 5n equations and 5n unknowns for analysis of each retaining wall, is presented based on limit equilibrium principles and the horizontal slices method. It shows that active earth

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pressure (Ka) and seismic active pressure coefficient (Kae) both increase linearly with increase in slope of retaining wall [4].

The Rankine's classic earth pressure solution had been expanded for the calculation of seismic earth pressure on rigid retaining walls supporting cohesion less backfill. The expanded solution was based on the conjugate stress concept, without employing any additional assumptions. Seismic lateral pressure increases with depth, and the point of application of the resultant was calculated based on stratigraphy [5].

The extensive theoretical solutions in the form of charts which had been generated based on a kinematically admissible translational mechanism for computing the active resistance of cohesive frictional soil retained by a rigid wall due to seismic body forces with the application of the upper bound limit theorem of plasticity. The reduction in passive earth-pressure coefficients was largely influenced by roughness, orientation of wall, slope angle of backfill soil, and the position of surcharge from the wall [6].

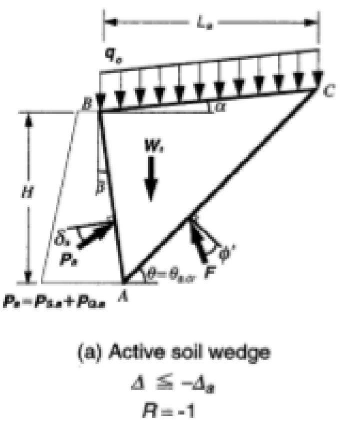
The present paper presents the comparative study of different methods used for computation of seismic active earth pressure.

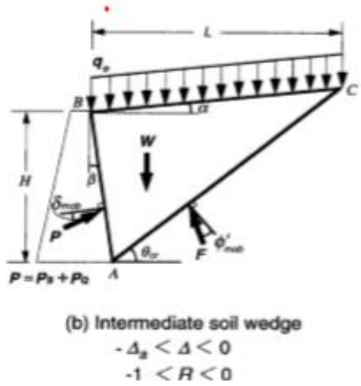
METHODOLOGY

The displacement based approach; pseudo dynamic method, conjugate stress method, kinematic limit analysis, and horizontal slice method were used in this work to determine seismic active earth pressure. These methods are shortly described as below.

Displacement Based Approach [2]

The backfill soil may extend or compress with the wall displacement, which makes the soil under different strain constraints. Lateral strain parameter *R* is used to represent the lateral deformation of the soil, which is caused by the wall displacement (Fig.1). The relation between the soil lateral strain parameter *R* and the wall displacement △ can be estimated by the formulas. The expression for seismic active earth pressure is





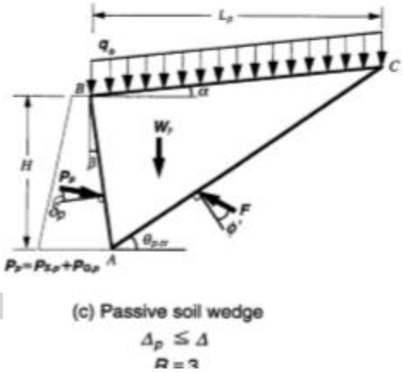


Figure 1: Variation in the Dimensions and Weight of Static Intermediate Soil Wedge with Lateral Wall Displacement

$$P_E = P_{ES} + P_{ED} + P_{EQ}$$

$$= \left(\frac{1}{2}\gamma H^2 + \frac{\cos\beta}{\cos(\beta-\alpha)} q_o H\right)(1-k_v)K_E$$

$$K_E = \frac{2\cos^2(\phi'-\beta-i)}{\cos^2(\phi'-\beta-i)(1+R) + \cos i \cos^2\beta \cos(\delta_{mob}+\beta+i)(1-R)}$$

$$\text{x} \quad \frac{1}{\left[1+\sqrt{\frac{\sin(\phi'+\delta_{mob})\sin(\phi'-\alpha-i)}{\cos(\delta_{mob}+\beta+i)\cos(\beta-\alpha)}}\right]^2}$$

$$(-1.0 \le R \le 1.0)$$

Pseudo Dynamic Approach [3]

The analysis considers the effect of various parameters like friction angle (ϕ), shear wave velocity (V_s), primary wave velocity (V_p), both horizontal and vertical seismic coefficient (k_h and k_v). For analysis the vertical rigid retaining wall was considered as shown in Fig. 2.

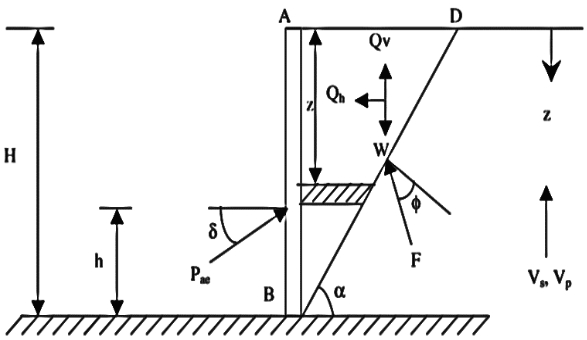


Figure 2: Model of Retaining Wall Considered for Computation of Pseudo Dynamic Active Earth Pressure.

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The expression for seismic active earth pressure was given by,

$$P_{ae}(t) = \frac{W\sin(\alpha-\phi) + Q_h(t)\cos(\alpha-\phi) - Q_v(t)\sin(\alpha-\phi)}{\cos(\delta+\phi-\alpha)}$$

$$K_{ae} = \frac{2P_{ae}}{\gamma H^2}$$

γ = unit weight of backfill material, Q_h and Q_v are horizontal and vertical inertial forces.

Conjugate Stress Method [5]

The combined action of gravitational acceleration, g, and the horizontal and vertical pseudo static accelerations, a_h and a_v, can be represented by a single acceleration field acting at an angle ω to the vertical, as denoted in Fig.3 The seismic earth thrust on the wall, P_{AE} is given by the following expression.

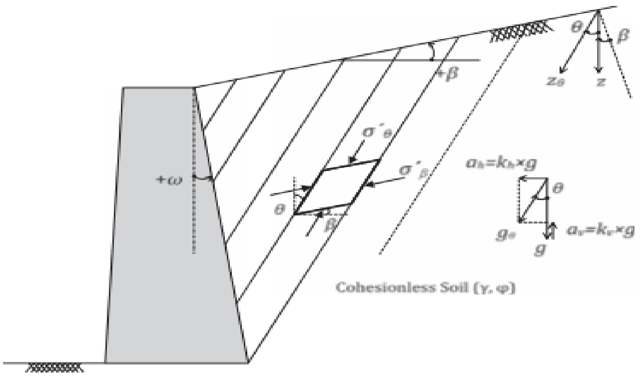

Figure 3: Problem geometry and conjugate stress state in a soil element of the backfill

$$P_{AE} = \frac{1}{2}(1-k_v)\,\gamma\, H^2 K_{AE}$$

where, K_{AE} = coefficient of seismic lateral earth pressure and is obtained as follows:

$$K_{AE} = \frac{\cos^3(\beta-\omega)m_a + \cos(\beta-\omega)\sin^2(\theta+\omega)}{\cos\alpha\cos^2\omega\cos(\beta+\theta)\cos\theta}$$

$$m_a = \frac{\cos(\beta+\theta) - \left[\cos^2(\beta+\theta)-\cos^2\phi\right]^{1/2}}{\cos(\beta+\theta) + \left[\cos^2(\beta+\theta)-\cos^2\phi\right]^{1/2}}$$

$$\alpha = \tan^{-1}\left\{\frac{\sin 2(\theta+\omega) + m_a\sin 2(\beta-\omega)}{2[\sin^2(\theta+\omega) + m_a\cos^2(\beta-\omega)]}\right\}$$

Kinematic Limit Analysis [6]

An improved composite collapse mechanism, as illustrated in Fig. 4, had been used to perform the analysis, in which a radial shearing zone OBC was chosen in between two triangular blocks OAB and OCD. The zone OBC was a continuous deforming shear zone in which the energy dissipation occurs within the region OBC and along the shearing boundary BC.

The resultant seismic active force Pa can be calculated by expression given as

$$P_a(\theta,\mu,\varepsilon) = \frac{\gamma h^2}{2}K_{a\gamma s} + qhK_{aqs} - chK_{acs}$$

where, Ka γ s, Kaqs, and Kacs = individual seismic active earth thrust coefficients due to the components of soil unit weight, surcharge, and soil cohesion, respectively.

Horizontal Slice Method [7]

The stability of retaining wall subjected to seismic loads is analyzed using a new limit equilibrium method identified as the Horizontal Slice Method. In this approach, the sliding wedge was divided into a number of horizontal slices as seen in Fig. 5.

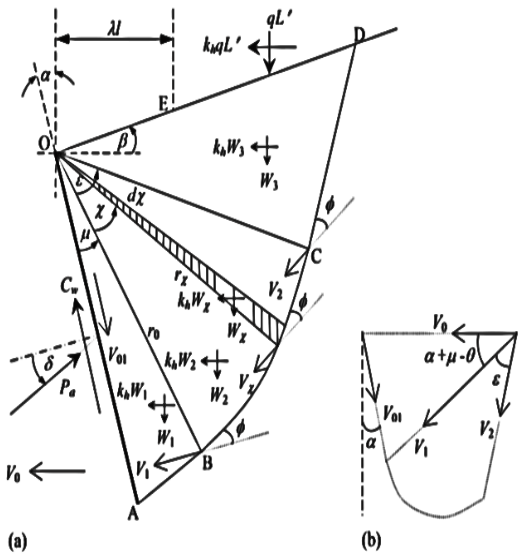

Figure 4: (a) Proposed Composite Collapse Mechanism; (b) Velocity Hodograph

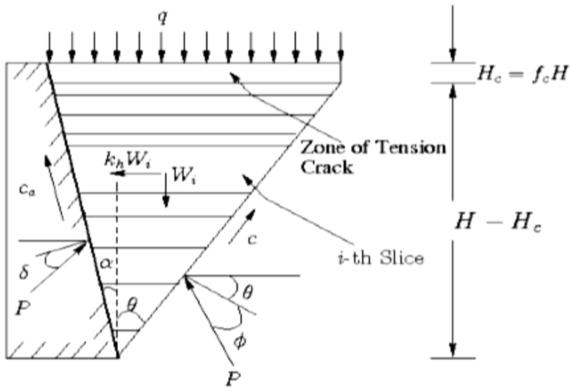

Figure 5: Horizontal Slices in Planar Failure Wedge and Forces Acting on the Wedge

The resultant Active earth pressure can be calculated using expression as

$$P = \gamma H^2 N_{a\gamma} + qHN_{aq} - cHN_{ac}$$

where, $N_{a\gamma}$, N_{aq}, N_{ac} are the Earth Pressure Factor (EPF)s and are given by

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$$N_{\gamma i} = \frac{[\cos(\theta_{ave} + \varphi) + k_h \sin(\theta_{ave} + \varphi)]}{\sin(\beta + \delta)}$$

$$N_{qi} = \frac{[\cos(\theta_{ave} + \varphi) + k_h \sin(\theta_{ave} + \varphi)]}{\sin(\beta + \delta)}$$

$$N_{aci} = \frac{\sin(\theta_i^* + \theta_{ave} + \varphi)}{\sin(\beta + \delta)} - \frac{t_i}{L_i} \cdot \frac{[\cos(\theta_{ave} + \varphi) - \tan\alpha \sin(\theta_{ave} + \varphi)]}{\sin(\beta + \delta)}$$

$$\beta = \varphi + \theta + \alpha$$

where, θ = the inclination angle of the failure plane, φ= internal angle of friction of the soil, crack depth factor f_c = H_c/H, angle of wall friction is δ, α = batter angle.

MATLAB PROGRAM

The program had been developed using MATLAB software for computation of seismic active earth pressure using Conjugate Stress method and Mononobe Okabe method. The flowchart for this program is shown in Fig 6.

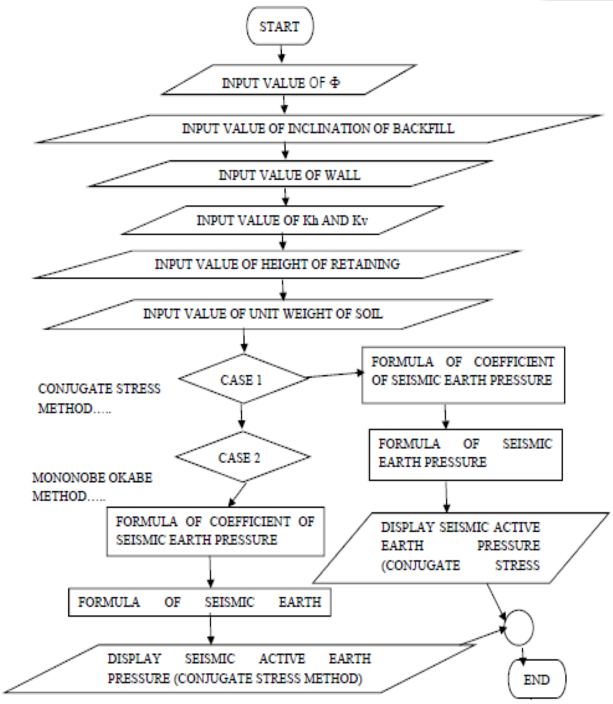


Figure 6: Flowchart of MATLAB Program

The retaining wall was analyzed with the properties of soil and wall as γ= 18, ϕ= 35, H= 5 m, δ= 23.33, k_h= 0.112, k_v= 0.027, β= 0, α=0 for all different mehtods and results were compared.

RESULTS AND DISCUSSION

A retaining wall of 5m height was considered for analysis. For different theories a same data of wall, soil and earthquake was used and the results were compared for each theory. Table 1 shows results obtained by different methods and the variation with respect to Mononobe Okabe method.

Table 1: Percent Increase/Decrease in Seismic Earth Pressure

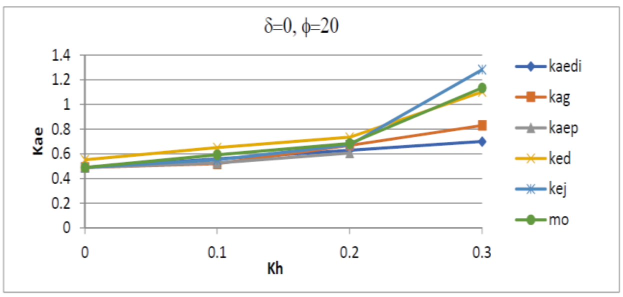
Sr. No.	Method	Seismic Earth Pressure (KN/m)	Increase/ Decrease (%)
1.	Mononobe Okabe Method	69.43	Nil
2.	Conjugate Stress Approach	69.10	-0.47
3.	Pseudo Dynamic Approach	68.85	-0.83
4.	Kinematic Equilibrium Method	73.67	+6.1
5.	Horizontal Slice Method	70.10	+0.96
6.	Displacement Based Approach	73.55	+5.93

The seismic active earth pressure coefficient was determined by various methods for different parameters and shown in Table 2. These studies show the effective influence of different parameters on seismic coefficient of active earth pressure for each method.

Table 2: Coefficient of Seismic Active Earth Pressure by Various Methods

Sr. No.	α	γ	H	ϕ	δ	kh	Horizontal Slice Method	Kinematic Limit Analysis	Pseudo Dynamic Approach	Conjugate Stress Method	Displacement Based Approach	Mononobe-Okabe Method
1	0	18	6	20	0	0	0.4154	0.49	0.4903	0.4902434	0.5524841	0.49024
2	0	18	6	20	0	0.1	0.47476785	0.52	0.5261	0.5516116	0.6507878	0.59259
3	0	18	6	20	0	0.2	0.5341122	0.67	0.6075	0.6723553	0.734976	0.68441
4	0	18	6	20	10	0	0.41542351	0.45	0.4467	0.4902434	0.5162051	0.4467
5	0	18	6	20	10	0.1	0.47476785	0.52	0.4937	0.5516116	0.620197	0.55458
6	0	18	6	20	10	0.2	0.5341122	0.62	0.5996	0.6723553	0.7190257	0.66523
7	0	18	6	20	20	0	0.41542351	0.44	0.4269	0.4902434	0.4952916	0.42683
8	0	18	6	20	20	0.1	0.47476785	0.52	0.4847	0.5516116	0.6057384	0.54297
9	0	18	6	20	20	0.2	0.5341122	0.615	0.6172	0.6723553	0.7194882	0.6749
10	0	18	6	30	0	0	0.29676705	0.33	0.3333	0.3332811	0.3908744	0.33328
11	0	18	6	30	0	0.1	0.3604334	0.4	0.3677	0.3832358	0.4745921	0.41349
12	0	18	6	30	0	0.2	0.42409975	0.47	0.4243	0.4780708	0.5425909	0.48123
13	0	18	6	30	15	0	0.29676705	0.305	0.3014	0.3332811	0.3610996	0.30137
14	0	18	6	30	15	0.1	0.3604334	0.365	0.3426	0.3832358	0.4475832	0.38417
15	0	18	6	30	15	0.2	0.42409975	0.45	0.4132	0.4780708	0.5246661	0.46289
16	0	18	6	30	30	0	0.29676705	0.32	0.2927	0.3332811	0.3505871	0.29713
17	0	18	6	30	30	0.1	0.3604334	0.37	0.348	0.3832358	0.4430068	0.38913
18	0	18	6	30	30	0.2	0.42409975	0.47	0.4398	0.4780708	0.5321772	0.48625
20	0	18	6	40	0	0	0.19972782	0.22	0.2174	0.2173914	0.2628537	0.21739
21	0	18	6	40	0	0.1	0.27431789	0.265	0.2499	0.2657914	0.3326735	0.28011
22	0	18	6	40	0	0.2	0.34890796	0.33	0.2973	0.3612884	0.3921752	0.33535
23	0	18	6	40	20	0	0.19972782	0.2	0.1994	0.2173914	0.2437874	0.19936
24	0	18	6	40	20	0.1	0.27431789	0.25	0.2365	0.2657914	0.3153292	0.26428
25	0	18	6	40	20	0.2	0.34890796	0.325	0.2941	0.3612884	0.3814204	0.32773
26	0	18	6	40	40	0	0.19972782	0.22	0.2102	0.2173914	0.245043	0.21016
27	0	18	6	40	40	0.1	0.27431789	0.28	0.2583	0.2657914	0.3239225	0.28775
28	0	18	6	40	40	0.2	0.34890796	0.37	0.3396	0.3612884	0.4027211	0.3726

Fig. 7 shows the variation of K_{ae} with respect to K_h keeping δ and ϕ constant. It can be observed that as k_h increases from 0 to 0.2, K_{ae} slightly increases, but after k_h = 0.2, sudden increase in K_{ae} for Mononobe Okabe Method, Conjugate Stress Method, and Displacement Based Approach observed.



[kaedi- Horizontal Slice Method, kag- Kinematic Limit Equilibrium, kaep- Pseudo Dynamic Method, ked- Displacement Based Approach, kej- Conjugate Stress Method, mo- Mononobe Okabe Method]

Figure 7: Variation of K_{ae} against k_h (δ=0, ϕ=20)

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The variation for K_{ae} with respect to ϕ is shown in Fig.8. As ϕ increase, K_{ae} decreases for all methods. Kae is inversely proportional to angle of internal friction ϕ.

For same ϕ and K_h, the variation for K_{ae} with respect to β is shown in Fig.9.It can be observed that slight increase in the value of Kae observed with increase of β.

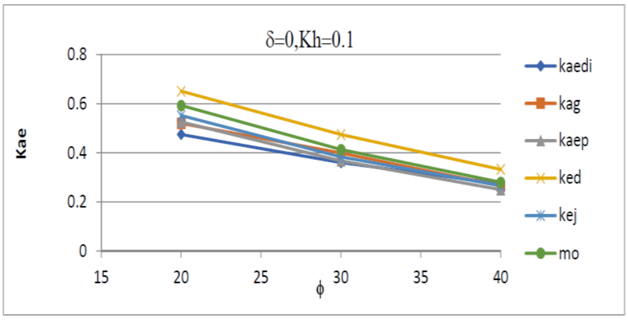

Figure 8: Variation of K_{ae} against ϕ (δ=0,K_h=0.1)

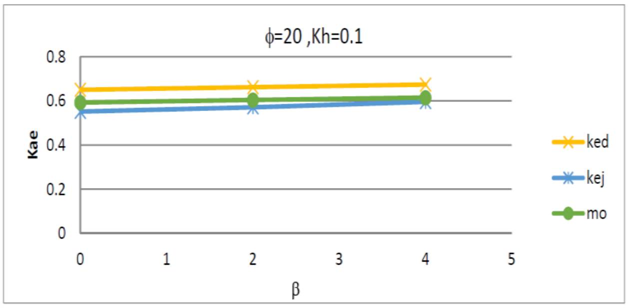

Figure 9: Variation of K_{ae} against β (ϕ=20,K_h=0.1)

The variation for K_{ae} with respect to ω is shown in Fig.10. As ω, wall inclination angle increases value of K_{ae} also increases.

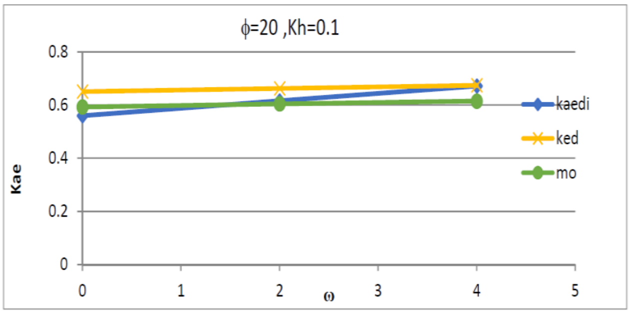

Figure 10: Variation of K_{ae} against ω (ϕ=20,Kh=0.1)

The variation for K_{ae} with respect to δ is shown Fig.11. It can be observed that as δ increases K_{ae} decreases slightly in each method.

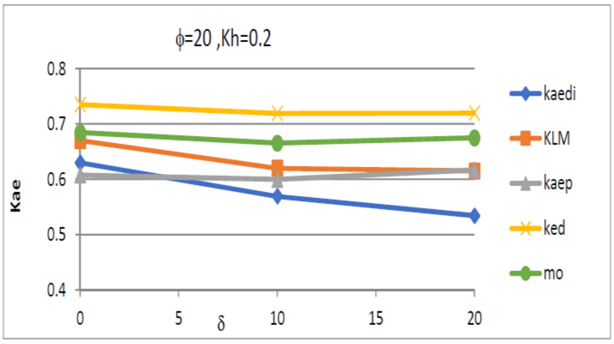

Figure 11: Variation of K_{ae} against δ (ϕ=20,Kh=0.2)

CONCLUSIONS

The study of seismic active earth pressure by different method for various parameters was carried out. From the study it can be concluded that;

1. The seismic earth pressure coefficient varies with method of approach. The displacement based approach kinematic limit analysis and horizontal slice method gave higher seismic active earth pressure than Mononobe Okabe method.
2. The seismic active earth coefficient affected by of parameters viz. k_h, β, ω ϕ and δ.
3. The coefficient of seismic active earth pressure (K_{ae}) increases with increase in the values of parameters (k_h, β, ω) and decreases with increase in the internal parameters (ϕ, δ).

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