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SEISMIC PASSIVE RESISTANCE AT SOIL-WALL INTERFACE

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SUMMARY

The limit equilibrium method is adopted in the present work for determining individually the seismic passive earth pressure coefficients corresponding to unit weight, surcharge and cohesion components. In the determination of each of these components, curved failure surfaces have been considered for different cases of positive and negative wall friction angle in passive case. Pseudo-static seismic forces are found to affect passive earth pressures at soil-wall interface significantly. Effects of a wide range of parameters like wall friction angle, soil friction angle, wall-soil adhesion to soil cohesion ratio, horizontal and vertical seismic accelerations on seismic passive earth pressure coefficients have been studied. Results are provided in the tabular form. Comparisons of present results with those of the available results in literatures are carried out.

INTRODUCTION

Estimation of passive earth resistance is an important topic of research and more so under seismic conditions. This estimation is required for the design of retaining walls, anchors, foundations etc. In static condition, different theories to compute passive resistance at a soil-wall interface are available. But the same under seismic condition are still scarce. It is a common practice to consider the seismic accelerations in both horizontal and vertical directions in terms of equivalent static forces, called pseudo-static accelerations. Using the pseudo-static approach, researchers like Okabe (1926), Mononobe and Matsuo (1929), Davies *et al.* (1986), Morrison and Ebeling (1995), Soubra (2000), Kumar (2001), Choudhury (2003) to name a few, had analyzed seismic passive earth pressure problems for rigid retaining wall. Positive wall friction angle or positive delta cases are those where soil moves up relative to the wall in passive condition. In this case, the shearing resistance along the soil-wall interface will act in the downward direction. All the above mentioned works in the seismic case deal with only this positive wall friction angle case. And the reverse condition of wall moving up relative to the backfill soil is termed as negative wall friction case was hardly received any attention except a very few research in this direction e.g. Choudhury and Subba Rao (2002).

ANALYTICAL METHOD

Problem Statement

Rigid retaining wall supporting dry, homogeneous backfill with surcharge is assumed in the analysis.

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Uniform seismic accelerations are assumed in the domain under consideration. Terzaghi (1943) has shown that for smooth walls, the rupture surface is planar and for values of wall friction angle $\delta > \phi/3$, where ϕ is the soil friction angle, only curved rupture surfaces should be assumed in the analysis for passive condition.

Kumar and Subba Rao (1997a) had also shown that the composite failure surface becomes critical for the case of positive wall friction angle under passive condition. In the present analysis, composite (logarithmic spiral + planar) failure surface is considered for positive wall friction case. Whereas, for the negative wall friction case, it was observed that the logarithmic spiral failure surface gives the minimum value of the passive resistance (Choudhury and Subba Rao, 2002). Hence the same is also considered in the present analysis.

In Fig. 1, for the case of positive wall friction, the vertical rigid retaining wall AB, supporting a horizontal c-ϕ backfill with uniform ground surcharge is shown. The composite (logarithmic spiral+planar) is assumed in the analysis as considered by Choudhury (2003) for the case of positive wall friction angle case. In Fig. 1, the portion BD is a logarithmic spiral, with portion DE as passive planar failure surface. F is the focus of logarithmic spiral and is at a distance of L from the point A. The initial radius r_o and the final radius r_f of the logarithmic spiral are given by distances FB and FD respectively.

The seismic passive resistance P_{pd} is split into three components as,

(i) Unit weight component $P_{p\gamma d}$ ($\gamma \neq 0$, $c = q = 0$)

(ii) Surcharge component P_{pqd} ($q \neq 0$, $\gamma = c = 0$)

(iii) Cohesion component P_{pcd} ($c \neq 0$, $\gamma = q = 0$)

where γ , c and q are the unit weight of the soil, unit cohesion and surcharge pressure respectively. The principle of superposition is assumed as valid and the minimum of each component is added to get the total minimum seismic passive resistance. Hence,

$$P_{pd} = P_{p\gamma d} + P_{pqd} + P_{pcd} \qquad (1)$$

The forces shown in Fig. 1 are the seismic passive resistance P_{pd} (acting on AB), which is divided into three components as mentioned in Eq. (1). Point of application of $P_{p\gamma d}$ is assumed at a height of H/3 from the base of the wall whereas P_{pqd} and P_{pcd} are assumed to act at a height of H/2. Uniform surcharge pressure of q is acting on AE (surcharge load Q = q.AE).

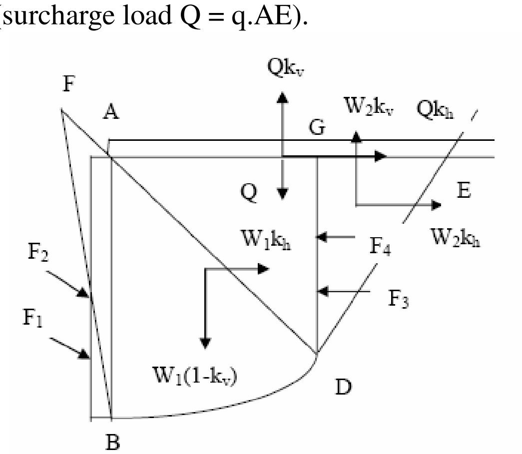

Fig. 1. Failure surface and forces considered for positive wall friction case (Choudhury, 2003)

In Fig. 1, $F_1 = P_{p\gamma d}$, $F_2 = P_{p\gamma d} + P_{p\gamma d}$, $F_3 = P_{p\gamma R}$, $F_4 = P_{p\gamma R} + P_{p\gamma R}$, W_1 = Weight of the portion of soil mass ABDGA and W_2 = Weight of the portion of soil mass DGE.

Weight of the soil mass ABDGA is W_1. Cohesive force C is acting on the failure surface BD along with normal force N and frictional force Ntanϕ. Adhesive force C_a is acting on the retaining wall-soil interface AB. Rankine passive resistances P_{pcR}, P_{pqR} and $P_{p\gamma R}$ are acting on the surface DG. Pseudo-static forces due to seismic weight component for zone DGE are W_2k_h and W_2k_v in horizontal and vertical directions respectively. Pseudo-static forces due to seismic weight component for zone ABDGA are W_1k_h and W_1k_v in horizontal and vertical directions respectively. And pseudo-static forces due to Qk_h and Qk_v in horizontal and vertical directions respectively are acting on AE. Generally both the horizontal and vertical seismic accelerations k_hg and k_vg, where g is the acceleration due to gravity can act in either of the directions. The critical directions of seismic acceleration coefficients producing minimum passive resistance are shown in Fig. 1.

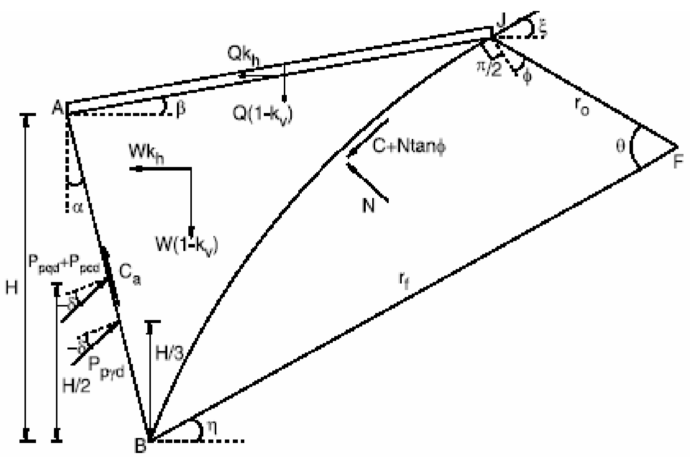

Fig. 2. Failure surface and forces considered for negative wall friction case (Choudhury and Subba Rao, 2002)

For the case of negative wall friction, a retaining wall AB of vertical height H, wall batter angle α, ground slope β, wall friction angle δ, soil friction angle ϕ, coefficient of seismic horizontal acceleration k_h and coefficient of seismic vertical acceleration k_v are considered in the analysis as shown in Fig. 2 (Choudhury and Subba Rao, 2002). Considering Fig. 2, the portion BJ is a logarithmic spiral with F as its focus. This focus point is a variable and it is found out by varying η value so as to result in the minimum seismic passive resistance. The initial radius r_o and the final radius r_f of the logarithmic spiral are given by distances FJ and FB respectively.

In Fig. 2, the forces considered are the seismic passive force P_{pd} (acting on AB), which is divided into three components as mentioned in Eq. (1), uniform surcharge pressure of q acting on AJ (surcharge load Q = q.AJ), weight W of the soil mass ABJ, cohesive force C on the failure surface BJ, normal force N on the failure surface BJ, frictional force Ntanϕ on the failure surface BJ, adhesive force C_a on the retaining wall-soil interface AB, pseudo-static forces due to seismic weight component for zone ABJ as Wk_h and Wk_v in the horizontal and vertical directions respectively and pseudo-static forces Qk_h and Qk_v in the horizontal and vertical directions respectively on AJ. The critical directions of the seismic acceleration coefficients to produce minimum passive earth resistance are shown in Fig. 2. In the present paper, analysis is carried out for vertical wall with horizontal ground i.e. $\alpha = 0^0$, $\beta = 0^0$, case only.

Determination of $P_{P\gamma d}$ ($\gamma \neq 0$, $c = q = 0$)

To determine the unit weight component of the seismic passive earth pressure $P_{p\gamma d}$, only the forces related to unit weight of the soil are considered and all other forces related to surcharge and cohesion are considered as zero. Hence in this analysis for positive wall friction, the forces considered are W_1k_h, W_1(1-

k_v), W_2k_h, W_2k_v, N, Ntanϕ, $P_{p\gamma R}$ and $P_{p\gamma d}$ (Fig. 1). Considering the moment equilibrium of all these forces about the focus F of the logarithmic spiral (Fig. 1),

$$M_{Pp\gamma d} = M_{W1\,d} + M_{Pp\gamma R} + M_{W2\,kv} - M_{W2\,kh} \quad (2)$$

where $M_{Pp\gamma d}$ = moment of $P_{p\gamma d}$.

$M_{W1\,d}$ = moment of soil mass ABDGA, together with seismic components.

$M_{Pp\gamma R}$ = moment of Rankine passive resistance $P_{p\gamma R}$.

$M_{W2\,kv}$, $M_{W2\,kh}$ = moments of seismic components of weight DGE.

The minimum value of $P_{p\gamma d}$ is obtained by considering different logspirals i.e. by varying the distance L.

For negative wall friction case, the forces considered are Wk_h, W(1-k_v), N, Ntanϕ and $P_{p\gamma d}$ (Fig. 2). Considering the moment equilibrium of all these forces about the focus F of the logspiral,

$$M_{Pp\gamma d} = M_{Wd} \quad (3)$$

where $M_{Pp\gamma d}$ = moment of $P_{p\gamma d}$.

M_{Wd} = moment of soil mass ABJ, together with seismic components.

Minimum value of $P_{p\gamma d}$ is obtained by considering different logspirals i.e. by varying η.

After determining the minimum $P_{p\gamma d}$, the seismic passive earth pressure coefficient for unit weight component, $K_{p\gamma d}$ is obtained using Eq. (4).

$$P_{p\gamma d} = \left(\frac{1}{2}\gamma H^2 K_{p\gamma d}\right) / \cos\delta \quad (4)$$

In the similar way, the other two components of passive resistance, viz. P_{pqd} and P_{pcd} are determined and hence the corresponding passive earth pressure coefficients.

RESULTS AND DISCUSSIONS

For this analysis, the values of ϕ used is given by,

$$\phi > \tan^{-1}\left(\frac{k_h}{1-k_v}\right) \quad (5)$$

because, certain combinations of k_h and k_v will cause the phenomenon of shear fluidization (i.e. the plastic flow of the material at a finite effective stress) even in the case of dry soil and will result in almost negligible passive earth resistance (Richards *et al.* 1990).

Results are presented in tabular form for seismic passive earth pressure coefficients with respect to cohesion, surcharge and unit weight components. Variations of parameters considered are as follows: δ/ϕ = -1.0, -0.5, 0.0, 0.5, 1.0; $\phi = 20^0, 30^0, 40^0$; c_a/c = 0.0 to (tan δ/tan ϕ); k_h = 0.0, 0.1, 0.2, 0.3, 0.4; $k_v = 0.0k_h$, $0.5k_h$ and $1.0k_h$.

The values of K_{pcd}, K_{pqd} and $K_{p\gamma d}$ increase with increase in positive δ/ϕ values. From Tables 1, 2 and 3, it is clear. For example, under the static condition ($k_h = k_v = 0.0$), for $c_a/c = 0.0$, $\phi = 30^0$, for a change in δ/ϕ value from 0.0 to 1.0, the increases in earth pressure coefficients K_{pcd}, $K_{p\gamma d}$ and K_{pqd} are 97.7%, 92.8% and 67.6% respectively. But under seismic condition say for $k_h = k_v = 0.3$, for the same sets of values of c_a/c and ϕ, for the same change in δ/ϕ value from 0.0 to 1.0, the corresponding increases are 97.7%, 104.7% and 133.7% respectively.

Table 1. Values of Seismic Passive Earth Pressure Coefficient $K_{p\gamma d}$

ϕ	δ/ϕ	k_h												
		0.0	0.1			0.2			0.3			0.4		
		k_v	k_v			k_v			k_v			k_v		
		0.0	0.0	0.05	0.1	0.0	0.1	0.2	0.0	0.15	0.3	0.0	0.2	0.4
20^0	-1.0	0.95	0.98	0.93	0.89	1.01	0.92	0.82	1.04	0.89	-	-	-	-
	-0.5	1.53	1.68	1.61	1.53	1.82	1.66	1.51	1.94	1.70	-	-	-	-
	0.0	2.04	1.89	1.78	1.68	1.71	1.50	1.29	1.48	1.08	-	-	-	-
	0.5	2.52	2.34	2.21	2.08	2.06	1.79	1.52	1.70	1.19	-	-	-	-
	1.0	2.89	2.75	2.59	2.43	2.50	2.15	1.80	1.98	1.33	-	-	-	-
30^0	-1.0	0.83	0.83	0.78	0.74	0.82	0.73	0.65	0.80	0.67	0.54	0.79	0.61	-
	-0.5	1.80	1.93	1.83	1.74	2.03	1.85	1.66	2.12	1.84	1.56	2.21	1.83	-
	0.0	3.00	2.82	2.67	2.52	2.63	2.32	2.02	2.42	1.94	1.47	2.17	1.49	-
	0.5	4.46	4.25	4.07	3.89	3.90	3.46	3.02	3.45	2.51	2.03	3.00	1.98	-
	1.0	5.78	5.55	5.22	4.89	5.30	4.66	3.97	4.95	4.08	3.00	4.40	2.92	-
40^0	-1.0	0.67	0.64	0.61	0.57	0.60	0.54	0.47	0.56	0.46	0.35	0.51	-	-
	-0.5	2.07	2.14	2.04	1.93	2.20	1.99	1.79	2.26	1.94	1.63	2.31	1.88	1.45
	0.0	4.60	4.38	4.15	3.92	4.15	3.69	3.23	3.91	3.21	2.51	3.66	2.70	1.70
	0.5	9.09	8.82	8.37	7.93	8.55	7.65	6.75	8.28	6.76	5.25	7.75	5.49	3.15
	1.0	14.45	14.0	13.1	12.2	13.5	11.8	10.0	13.0	10.4	7.73	12.6	9.09	5.39

Table 2. Values of Seismic Passive Earth Pressure Coefficient K_{pqd}

ϕ	δ/ϕ	k_h												
		0.0	0.1			0.2			0.3			0.4		
		k_v	k_v			k_v			k_v			k_v		
		0.0	0.0	0.05	0.1	0.0	0.1	0.2	0.0	0.15	0.3	0.0	0.2	0.4
20^0	-1.0	0.99	0.93	0.88	0.83	0.88	0.78	0.68	0.83	0.68	-	-	-	-
	-0.5	1.51	1.42	1.35	1.27	1.33	1.18	1.03	1.24	1.02	-	-	-	-
	0.0	2.04	1.89	1.77	1.65	1.71	1.45	1.19	1.48	1.08	-	-	-	-
	0.5	2.36	2.34	2.25	2.16	2.30	2.03	1.76	2.22	1.72	-	-	-	-
	1.0	2.70	2.68	2.55	2.21	2.63	2.35	2.08	2.55	2.13	-	-	-	-
30^0	-1.0	0.88	0.84	0.80	0.76	0.81	0.72	0.63	0.77	0.64	0.48	0.74	0.56	-
	-0.5	1.76	1.68	1.60	1.51	1.60	1.43	1.25	1.52	1.26	0.99	1.44	1.09	-
	0.0	3.00	2.82	2.56	2.31	2.63	2.25	1.87	2.42	1.86	1.31	2.17	1.49	-
	0.5	4.17	4.16	3.91	3.67	4.14	3.66	3.19	4.12	3.40	2.68	3.73	2.83	-
	1.0	5.03	5.02	4.77	4.52	5.00	4.49	3.99	4.96	4.20	3.43	4.91	3.43	-
40^0	-1.0	0.72	0.69	0.66	0.62	0.67	0.60	0.53	0.65	0.55	0.39	0.63	-	-
	-0.5	1.98	1.92	1.82	1.72	1.85	1.65	1.45	1.78	1.48	1.19	1.71	1.32	0.92
	0.0	4.60	4.38	3.99	3.60	4.15	3.37	2.59	3.91	2.79	1.68	3.66	2.70	1.70
	0.5	8.71	8.70	8.20	7.71	8.69	7.66	6.63	8.68	7.26	5.84	7.94	6.17	4.39
	1.0	11.03	11.0	10.5	9.92	11.0	9.91	8.81	11.0	9.34	7.68	10.9	8.37	6.54

"-" indicates non-existence of solution.

Table 3. Values of Seismic Passive Earth Pressure Coefficient K_{pcd}

ϕ	δ/ϕ									
	For $c_a/c = 0.0$					For $c_a/c = \lvert\tan\delta/\tan\phi\rvert$				
	-1.0	-0.5	0.0	0.5	1.0	-1.0	-0.5	0.0	0.5	1.0
20^0	0.85	1.11	1.43	1.81	2.18	-	0.69	1.43	2.07	2.33
30^0	0.70	1.09	1.73	2.66	3.42	-	0.66	1.73	2.94	3.49
40^0	0.52	1.03	2.14	4.33	5.95	-	0.58	2.14	4.65	5.97

"-" indicates non-existence of solution.

Again, for example, under the static condition ($k_h = k_v = 0.0$), for $c_a/c = 0.0$, $\phi = 30^0$, for a change in δ/ϕ value from -1.0 to -0.5, the increases in earth pressure coefficients K_{pcd}, $K_{p\gamma d}$ and K_{pqd} are 55.7%, 116.4% and 100.5% respectively. But under seismic condition say for $k_h = k_v = 0.3$, for the same sets of values of c_a/c and ϕ, for the same change in δ/ϕ value from -1.0 to -0.5, the corresponding increases are 55.7%, 187.5% and 107.3% respectively.

COMPARISON OF RESULTS

Typical comparisons are shown for the static and seismic passive earth pressure coefficients. Table 4 shows the comparison of $K_{p\gamma d}$ values for $\delta/\phi = 1.0$ in the static case ($k_h = k_v = 0$) with those from other available methods and analyses. Results are found to be identical with those obtained by Kumar and Subba Rao (1997a, 1997b). Limit analysis by Chen and Rosenfarb (1973) and the method of slices by Janbu (1957) estimate higher values of passive earth pressure coefficients than from the present study. The friction circle method by Kerisel and Absi (1990) and the method of characteristics by Sokolovski (1965) estimate lower values of passive earth pressure coefficients than from the present study.

Table 4. Comparison of $K_{p\gamma d}$ values obtained by present study with available theories in static case for $\delta/\phi = 1.0$

Methods	Values of $K_{p\gamma d}$ for ϕ		
	20^0	30^0	40^0
Present study	2.89	5.78	14.45
Zhu and Qian (2000)	3.06	6.59	18.24
Kumar and Subba Rao (1997a)	2.89	5.78	14.45
Kumar and Subba Rao (1997b)	2.89	5.78	14.45
Kerisel and Absi (1990)	2.91	5.63	13.79
Chen and Rosenfarb (1973)	2.98	6.15	16.01
Sokolovski (1965)	2.86	5.67	13.94
Janbu (1957)	3.08	5.96	14.29

Table 5 shows a comparison of K_{pcd} values obtained from present study with those obtained by Clayton and Militsky (1986). Table 6 compares $K_{p\gamma d}$ values for a specific case of $\delta/\phi = -0.5$ with those of Kerisel and Absi (1990), Kumar and Subba Rao (1997a, 1997b).

Table 5. Comparison of K_{pcd} values obtained by present study with available theories in static case for $\phi = 20^o$

Case for	Values by	
	Clayton and Militisky (1986)	Present study
$\delta/\phi = 0.0$, $c_a/c = 0.0$	1.40	1.43
$\delta/\phi = 1.0$, $c_a/c = \tan\delta/\tan\phi$	2.35	2.33

Table 6. Comparison of $K_{p\gamma d}$ values obtained by present study with available theories in static case for $\delta/\phi = -0.5$

ϕ	Values of $K_{p\gamma d}$ by			
	Kerisel and Absi (1990)	Kumar and Subba Rao (1997a)	Kumar and Subba Rao (1997b)	Present study
20^0	1.50	1.53	1.53	1.53
30^0	1.74	1.81	1.80	1.80
40^0	1.97	2.09	2.07	2.07

Table 7 shows the comparison of $K_{p\gamma d}$ values in the seismic case for positive wall friction case with $\phi = 30^0$. For $\delta/\phi = 0.5$, it is clearly seen that the present study mostly results in the least values of the coefficients. However for $\delta/\phi = 1.0$, these coefficients are not necessarily the least values but are very near to the available least values.

Table 7. Comparison of $K_{p\gamma d}$ values obtained by present study with available theories in seismic case for $\phi = 30^o$

δ/ϕ	k_h	k_v	Mononobe-Okabe (1929)	Morrison and Ebeling (1995)	Soubra (2000)	Kumar (2001)	Present study
0.5	0.0	0.0	4.807	4.463	4.530	-	4.458
	0.1	0.0	4.406	4.240	4.202	-	4.240
		0.1	4.350	4.160	-	-	3.890
	0.2	0.0	3.988	3.870	3.854	-	3.860
		0.2	3.770	3.600	-	-	3.020
	0.3	0.0	3.545	3.460	3.470	-	3.450
		0.3	2.823	2.750	-	-	2.034
	0.4	0.0	3.058	3.010	-	-	3.000
		0.2	2.400	2.400	-	-	1.981
1.0	0.0	0.0	8.743	6.150	5.941	5.785	5.783
	0.1	0.0	7.812	5.733	5.500	5.361	5.400
	0.2	0.0	6.860	5.280	5.020	4.902	5.100
	0.3	0.0	5.875	4.940	4.500	4.400	4.750
	0.4	0.0	4.830	4.300	-	3.900	4.100

CONCLUSIONS

Using limit equilibrium method and pseudo-static approach for seismic accelerations, passive earth pressure coefficients on a rigid vertical retaining wall supporting a generalized c-ϕ horizontal backfill are obtained. For the passive case of earth pressure, both positive and negative wall friction angle cases are considered depending on the soil-wall interface movement under seismic conditions. It has been found that the seismic accelerations both in the horizontal and vertical directions significantly affect the passive earth pressure coefficients. The results obtained from the present analysis can be used to design the rigid retaining walls and also as extended passive earth pressure problems for foundation design and anchor uplift capacity determination for seismic case using positive and negative wall friction cases respectively. Present results compare well with the available results in the literature for static case and with a very few available results in the seismic case.

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