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DESIGN OF ANCHORED SHEET-PILE WALLS FOR SEISMIC LOADS

CONCEPTION DE MURS DE PARPLANCHES ANCRES COMME PROTECTION ANTI-SEISMIQUE

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SYNOPSIS : Seismic design procedure of anchored sheet-pile walls particularly used as permanent retaining structures is investigated based on the developed method of predicting the seismic earth pressures in c-ϕ soils. The proposed design procedure considers a change in magnitudes of effective seismic earth pressures due to seepage flow between the two water levels. Also, seismically induced hydraulic pressures and excess pore water pressures are taken into account. Using the proposed solution procedure, the effects of various design parameters are examined and the results of each case considered are discussed. Further, the effects of different definitions and values of safety factors applicable to the seismic design of anchored sheet-pile walls are analyzed.

INTRODUCTION

Anchored sheet-pile walls whose design is governed by the passive and active earth pressures imposed by the soils are frequently used in many types of civil engineering construction, particularly in port and harbor facilities along rivers and seacoasts. Many researchers, including the well-known Tschebotarioff and Rowe, have continuously studied the static stability and design of the sheet-pile walls. In the case of seismic loads, however, an accepted design procedure has not yet been established and is still under developing. It is quite important that a standard static design of anchored sheet-pile walls may prove to be inadequate even for moderate seismic loads since the passive resistance at the toe decreases while the active thrust increases, thus, greatly enhancing the likelihood that the toe of the wall kicks out in a rotational mode of collapse.

In the present study an analytical solution procedure, including a pseudostatic method of predicting the seismic earth pressures in c−ϕ soils, is proposed for the seismic design of anchored sheet-pile walls with free earth support. The proposed solution procedure considers a change in magnitudes of effective seismic earth pressures due to seepage flow between the two water levels. Also, seismically induced hydraulic pressures and excess pore water pressures are taken into account.

Using the proposed solution procedure, the effects of various design parameters(differential in water levels, anchor position, wall friction, dredge line slope, and wall adherence) on the depth of penetration, anchor force, and maximum bending moment are examined and the results of each case considered are discussed. Also, the effects of different definitions and values of safety factors applicable to the seismic design of anchored sheet-pile walls are analyzed.

SOLUTION PROCEDURE FOR A SEISMIC DESIGN

Before proceeding to the detailed seismic design of anchored sheet-pile walls, it is necessary to derive a diagram of the seismic active pressures and passive resistances acting on both sides of the wall. These are determined essentially by the height of wall, the soil properties, water levels, surcharge loads and earthquakes.

The seismic active thrust P_{A1} exerted by suitable cohesionless filling behind a vertical sheet-pile wall shown in Fig. 1 is determined from the well-known Mononobe-Okabe method of analysis(Richards and Elms 1979).

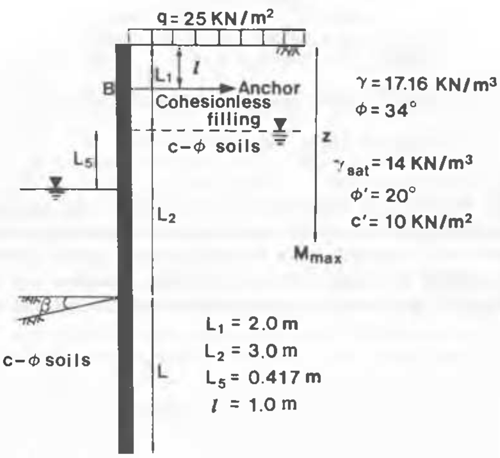

Fig. 1. Representation of an anchored sheet-pile wall

In order to further estimate the seismic effective active thrust P_{A2} imposed by saturated c−ϕ soils, the method proposed for the seismic design of a retaining wall with known height (Kim and Kang 1991) is modified. The resulting expression derived in the present study is as follows.

$$P_{A2} = \gamma' \cdot (L_2+L)^2 \cdot A_1 + Q \cdot (L_2-L) \cdot A_2 - c' \cdot (L_2+L) \cdot A_3 \quad \cdots\cdots (1)$$

where, $Q = \gamma \cdot L_1 + q$

$$A_1 = \frac{1}{2} \cdot (\sqrt{1+k_h^2}) \cdot \cos\delta \cdot \frac{\sin(\alpha_1-\phi'+\theta)}{\cos(\delta-\alpha_1+\phi')} \cdot \frac{(1-n^2)}{\tan\alpha_1}$$

$$A_2 = \frac{\sin(\alpha_2-\phi'+\theta)}{\cos(\delta-\alpha_2+\phi')} \cdot (\sqrt{1+k_h^2}) \cdot \frac{\cos\alpha_2}{\sin\alpha_2} \cdot (1-n) \cdot \cos\delta$$

$$A_3 = (1-n)\cdot\mu\cdot\left\{\frac{\sin(\alpha_3-\phi'+\theta)}{\cos(\delta-\alpha_3+\phi')}\cdot\frac{\cos\delta}{\cos\theta}\cdot\left(\frac{1}{2}\cdot\tan\theta\cdot\tan\delta+1\right)-\frac{1}{2}\cdot\tan\theta\right\}-\frac{(1-n)}{\sin\alpha_3}\cdot\left\{\frac{\sin(\alpha_3-\phi'+\theta)}{\cos(\delta-\alpha_3+\phi')}\cdot\frac{\cos\delta}{\cos\theta}\cdot(\cos\alpha_3\cdot\tan\delta-\sin\alpha_3\cdot\tan\theta\cdot\tan\delta-\sin\alpha_3)-(\cos\alpha_3-\sin\alpha_3\cdot\tan\theta)\right\}$$

where γ = unit weight of cohesionless filling; γ' = submerged unit weight of c−ϕ soils; ϕ' = effective internal friction angle; c' = effective cohesion; $a = \mu\cdot c'$ = wall adhesion; $n\cdot(L_2 + L)$ = depth of tension crack; δ = friction angle at wall-backfill interface; q = uniform surcharge on the ground surface behind the wall; $\theta = \tan^{-1}(k_h)$; and k_h = horizontal coefficient of acceleration. Note that only the horizontal inertia force is considered in the present method of analysis.

The values of the coefficients A_1, A_2 and A_3 in Eq. 1 are determined by optimizing each coefficient. In other words, the angles α_1 and α_2 corresponding to the maximum values of A_1 and A_2 are determined, and also the angle α_3 corresponding to the minimum value of A_3 is determined.

The expression for a computation of the seismic passive thrust P_P in c−ϕ soils below the dredge line with a negative slope angle β is similarly derived and is given below. Note that the effects of surcharge and tension crack are not included.

$$P_P = \gamma'\cdot L^2\cdot B_1 + c'\cdot L\cdot B_3 \qquad \cdots\cdots\cdots\cdots(2)$$

where, $$B_1 = (\sqrt{1+k_h^2})\cdot\cos\delta\cdot\frac{\sin(\alpha_1'+\phi'-\theta)}{\cos(\alpha_1'-\delta-\phi')}\cdot\frac{1}{\tan\alpha_1'\cdot\tan\beta}$$

$$B_3 = \mu\cdot\left\{\frac{\sin(\alpha_3'+\phi'-\theta)}{\cos(\alpha_3'-\delta-\phi')}\cdot\frac{\cos\delta}{\cos\theta}\cdot\left(\frac{1}{2}\cdot\tan\theta\cdot\tan\delta+1\right)+\frac{1}{2}\cdot\tan\theta\right\}+\frac{1}{\tan\alpha_3'\cdot\tan\beta}\cdot\frac{1}{\cos\alpha_3'}\cdot\left\{\frac{\sin(\alpha_3'+\phi'-\theta)}{\cos(\alpha_3'-\delta-\phi')}\cdot\frac{\cos\delta}{\cos\theta}\cdot(\cos\alpha_3'\cdot\tan\delta+\sin\alpha_3'\cdot\tan\theta\cdot\tan\delta+\sin\alpha_3')+(\cos\alpha_3'+\sin\alpha_3'\cdot\tan\theta)\right\}$$

In reality, the errors in magnitudes of the seismic active and passive thrusts computed by the preceding procedure are somewhat expected since the angles $\alpha_1(\alpha_1')$, α_2 and $\alpha_3(\alpha_3')$ do not match exactly. However, the errors analyzed for a typical case of retaining wall shown in Fig. 2 are less than 3 % and also the results described in Table 1 are on the safe side.

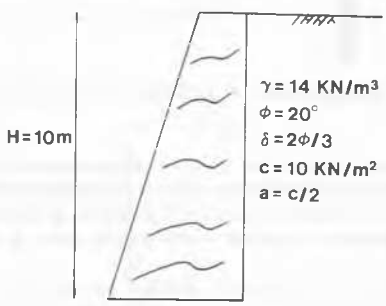

Fig. 2. Typical retaining wall with soil properties

Table 1-a. Percentage errors for the active case

		Static Load	k_h 0.05	k_h 0.10	k_h 0.15
Kim and Kang	P_A^*(KN/m)	166.46	191.69	219.75	251.30
	Angle α	49.85	47.09	44.05	40.66
Present study	P_A(KN/m)	166.74	192.91	223.03	258.39
	Angle α_1	50.01	46.65	42.83	38.46
	Angle α_2	52.23	52.00	51.78	51.55
$(P_A - P_A^*) / P_A^* \times 100(\%)$		+0.17	+0.64	+1.49	+2.82

Table 1-b. Percentage errors for the passive case

		Static Load	k_h 0.05	k_h 0.10	k_h 0.15
Kim and Kang	P_P^*	2342.35	2342.44	2235.51	2142.96
	Angle α'	25.80	25.22	24.56	23.80
Present study	P_P	2444.82	2339.33	2229.42	2113.73
	Angle α_1'	24.90	23.87	22.67	21.23
	Angle α_3'	25.57	25.50	25.43	25.36
$(P_p - P_p^*) / P_p^* \times 100(\%)$		-0.06	-0.13	-0.27	-0.53

Effects of Seepage

For drainage conditions it is necessary to take account of pore water pressures acting in the soil. Where different water levels exist on either side of the wall, as for a quay wall in tidal waters, and the wall does not penetrate to an impermeable stratum, seepage flow will take place beneath the toe of the wall.

To allow for the effect of seepage around the wall some assumption as to the manner in which the pressure varies around the wall is required. In the present study the assumption that the pressure head decreases linearly down the back and up the front of the wall between the two water tables is employed. Considering the situation depicted in Fig. 3 and taking the datum coincident with the water table behind the wall, the pressure head h_p at any point around the wall is given by

$$h_p = \frac{-x\cdot L_5}{(L_2+2L+w)} - h_e \qquad \cdots\cdots\cdots\cdots\cdots\cdots(3)$$

where x is the distance around the wall from the datum level, h_e is an elevation head and w is a width of the sheet-pile section. Seepage pressures without a discontinuity at the toe of the wall may be then evaluated from the pressure heads in Eq. 3 and the unit weight of water γ_w.

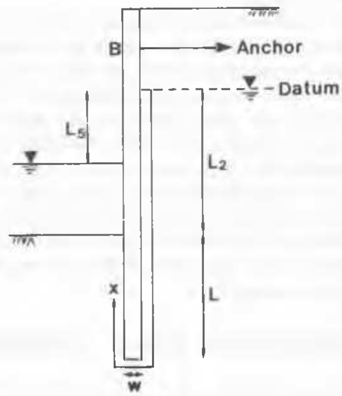

Fig. 3. Seepage pressure variation

Additionally, the seepage flow would possibly to increase the effective unit weight of soil on the active side and thus slightly increase seismic active pressures, and will decrease the effective unit weight of soil on the passive side, possibly giving rise to a significant decrease in seismic passive pressures. The resulting changes in magnitudes of the seismic effective active and passive thrusts due to a seepage flow could be estimated, based on the seismic earth pressure coefficients determined from Eqs. 1 and 2 as well as the differences between hydrostatic pressures corresponding to a no flow situation and seepage pressures.

Hydrodynamic Effects of the Water

During an earthquake, the hydrodynamic effects of the water also have to be taken into consideration. Seismically induced hydraulic pressures and excess pore water pressures determined based on the Westergaard theory as well as the experimental results reported by Matsuo-O'hara are included in the present solution procedure. Matsuo and O'hara suggested that the increase of the pore water pressure in saturated soil deposits under seismic load was approximately 70% of that given by Westergaard theory. The necessary expressions derived to compute the hydrodynamic pressures associated with sheet-pile walls are described in detail in a reference paper(Kim et al. 1991). Note that the total dynamic water force on the seaward side acts outward, resulting in a clockwise bending moment about an anchor point.

Seismic Design Procedure

For stability of an anchored sheet-pile wall under seismic loads, a minimum penetration depth of the wall required to satisfy moment equilibrium about an anchor point is first determined. Subsequently, the required anchor force is determined based on the horizontal force equilibrium. Finally a maximum bending moment to which the sheet-pile would be subjected is determined, and hence the required section modulus of the sheet pile is obtained by dividing M_{max} by the allowable working stress for the particular steel to be used. It is noted here that the procedure of moment reduction expected due to a redistribution of soil pressures as proposed by Rowe for static loads is not dealt with due to a lack of adequate experimental data for seismic loads.

ANALYTICAL PARAMETRIC STUDY

The large number of calculations performed using the free earth support solution procedure described in the preceding sections, makes it possible to discuss the results of various cases considered for the seismic design of anchored sheet-pile walls. Instead, the author has reviewed all of the calculations and only the main points of the results are presented here. The geometry, soil conditions, surcharge and water table levels adopted in the present analyses are given in Fig. 1. The choice of suitable values for safety factors is also a difficult problem because much depends on current practice. The minimum values for safety factors adopted in the present analyses are FS_{static} = 2.0(static case 1) or 1.5(static case 2) and $FS_{seismic}$ = 1.2. For seismic loads the horizontal design coefficient of acceleration k_h is limited up to the value of 0.15 by assuming the condition of mild earthquakes which is supposed to be typical in Korea. The value of n which defines a depth of tension crack in c-ϕ soils on the active side is assumed to be 0.2.

Effects of Safety Factor

The definition of factor of safety currently used in Code of Practice on Earth Retaining Structures(CP2) is compared with a factor of safety applied to the strength parameters ϕ' and c' by diving these parameters by a factor of safety F_r. Current CP2 definition entails applying a factor of safety F_p to the effective passive soil resistances below the dredge line. The comparisons between different definitions and values of safety factors are made and the results are given in Table 2.

For values of the input parameters adopted in the present analyses, the magnitudes of penetration depth L, anchor force F and maximum bending moment M_{max} obtained by the strength method definition are in general greater than those by CP2 method definition, reflecting that use of F_r is likely to lead conservative designs. It is further observed that the use of F_r gives higher values of z(points for zero shear force, Fig. 1).

It is also noted that for relatively small design values of k_h, that is $k_h \leq 0.05$, the required magnitudes of L, F and M_{max} computed based on both the definitions of factor of safety seem to be smaller than those obtained in static case 1. As pointed out previously, this is because the seismic passive resistances in front of the wall decrease even for moderate seismic loads. However, in the case of k_h = 0.10, only the magnitude of M_{max} is slightly smaller than that required in static case 1, indicating that the selection of a sheet-pile section shouid be careful. In static case 2, all the magnitudes of L, F and M_{max} for seismic loads are greater than those for static loads except for values of F and M_{max} at k_h = 0.05.

Table 2. Effects of safety factor

		Static Load		k_h		
		case 1	case 2	0.05	0.10	0.15
CP2 definition	L(m)	2.96	1.83	2.51	4.18	10.06
	F(kN/m)	69.37	60.10	59.61	72.70	88.24
	z(m)	4.80	4.31	3.84	3.99	4.04
	M_{max}(kN·m/m)	99.51	72.21	65.47	90.04	119.02
Strength method definition	L(m)	3.02	1.99	2.66	4.40	9.92
	F(kN/m)	69.81	61.41	60.20	73.41	88.28
	z(m)	4.83	4.38	3.87	4.01	4.04
	M_{max}(kN·m/m)	100.88	75.82	67.08	92.04	119.18

In the following subsections the results are computed based on current CP2 definition probably the most widely used in practice.

Effects of Differential Water Levels(L_5)

The effects of a variation in L_5 are illustrated in Table 3. These results show that the magnitudes of L, F and M_{max} increase with increasing L_5. This is due mainly to the fact that larger difference between the two water levels results in a reduction in the magnitude of hydrodynamic pressures in front of the wall, whereas the hydrostatic pressures acting on the backside of the wall simultaneously increase. It is also found that the influence of a variation in L_5 is much more significant on the magnitude of L when compared to the influences on F and M_{max}. L increases by about 102.9 ~ 138.1 % by increasing L_5 from 0.0 m to 1.0 m. In the case of L_5 = 1.0 m, the magnitude of M_{max} computed in static load case 1 exceeds those expected to occur under seismic loads. This could be explained by the fact that the point for zero shear force is located below the dredge line, that is z(= 6.86 m) > L_1 + L_2, and hence a portion of the seismic passive resistance acting on the front-side of the wall influences on the determination of M_{max}.

It may be concluded from the above results of analyses that where the differential is likely to be significant-for example, owing to rapid rise in the groundwater table after rainstorms or where the piling forms the wall in a dry dock-a careful selection of the sheet-pile section should be necessary.

Table 3. Effects of differential water levels(L_5)

		Static Load		k_h		
		case 1	case 2	0.05	0.10	0.15
L_5=0.0m	L	1.86	1.21	1.82	3.06	6.86
	F	54.14	49.59	51.31	62.51	77.54
	M_{max}	59.79	47.97	47.21	65.63	92.30
L_5=(L_1+L_2)/12 =0.417m	L	2.96	1.83	2.52	4.18	10.06
	F	69.37	60.10	59.61	72.70	88.24
	M_{max}	99.51	72.21	65.47	90.04	119.02
L_5=1.0m	L	4.95	2.87	3.73	6.21	16.33
	F	96.91	76.68	71.31	87.53	92.80
	M_{max}	197.09	117.41	95.50	130.89	129.24

Effects of Anchor Position(l)

The results of case studies in which l is varied are shown in Table 4. Both L and M_{max} are found to decrease with an increase in l, whereas F increases with increasing l. This phenomenon results from a change of moment directions as well as the relative magnitudes of hydrodynamic pressures and seismic effective soil pressures acting on the wall. It is also noted that the rate of a variation in the magnitudes of L, M_{max} and F decreases with increasing k_h. The results further indicate that a careful examination to select suitable sheet-pile section should not be disregarded.

Table 4. Effects of anchor position(l)

		Static Load		k_h		
		case 1	case 2	0.05	0.10	0.15
l=0.6m	L	3.29	2.06	2.73	4.47	10.45
	F	67.70	57.64	56.30	69.21	84.41
	M_{max}	121.39	88.65	79.08	107.74	141.51
l=1.0m	L	2.96	1.83	2.52	4.18	10.06
	F	69.37	60.10	59.61	72.70	88.24
	M_{max}	99.51	72.21	65.47	90.04	119.02
l=1.4m	L	2.56	1.56	2.25	3.83	9.59
	F	71.07	62.86	63.44	76.67	92.54
	M_{max}	76.49	54.79	51.01	71.27	95.19

Effects of Wall Friction(δ)

The consequences of a change in δ could be assessd by examining the results shown in Table 5. These results indicate that variations in δ have a greater influence on the magnitude of L when compared to the effects on F and M_{max}. L decreases by about 60.7 ~ 79.0 % with increasing δ for seismic loads. It is also expected that the magnitudes of L, F and M_{max} vary more significantly with a variation of δ as compared with a change in the value of FS_{static} from 2.0(static case 1) to 1.5(static case 2).

Table 5. Effects of wall friction(δ)

		Static Load		k_h		
		case 1	case 2	0.05	0.10	0.15
δ=0°	L	2.96	1.83	2.52	4.18	10.06
	F	69.37	60.10	59.61	72.70	88.24
	M_{max}	99.51	72.21	65.47	90.04	119.02
δ=ϕ'/2	L	1.73	1.16	1.52	2.26	3.75
	F	52.94	48.67	49.92	60.11	73.82
	M_{max}	63.62	52.52	49.90	65.84	89.75
δ=ϕ'	L	1.12	0.78	0.99	1.41	2.11
	F	43.98	41.63	43.89	52.84	64.30
	M_{max}	48.44	42.73	42.15	54.58	71.83

Effects of Dredge Line Slope(β)

An increase in the magnitudes of L, F and M_{max} is observed by sloping the dredge line and the results are presented in Table 6. As could be expected, this is due to the fact that the seismic passive resistance in front of the wall decreases with sloping the dredge line downward. For seismic loads with c-ϕ soils the magnitudes of M_{max} and z also increase with increasing the slope angle β downward. These observations are fully consistent with results of the model wall tests for static loads with sand backfills performed by Schroeder and Roumillac. Similar to the case of δ, the effect of sloping dredge line is seen to have the greatest effect on the magnitude of L.

Table 6. Effects of dredge line slope(β)

		Static Load		k_h		
		case 1	case 2	0.05	0.10	0.15
β=0°	L	1.73	1.16	1.52	2.26	3.15
	F	52.94	48.67	49.92	60.11	73.82
	z	4.27	4.01	3.63	3.72	3.84
	M_{max}	63.62	52.52	49.90	65.84	89.75
β=-5°	L	2.18	1.41	1.95	3.20	7.28
	F	56.31	50.59	51.72	63.68	81.06
	z	4.47	4.13	3.72	3.85	4.01
	M_{max}	73.02	57.39	54.56	75.32	109.21

Effects of Wall Adherence(a)

The effects of a variation in the ratio a/c' over the range from 0.0 to 0.7 are illuatrated in Table 7. In the case of a/c' = 0.0, the results show that much larger magnitude of M_{max} to which the sheet-pile is subjected is expected to occur under static load case 1, reflecting that careful considerations should be given in the selection of a sheet-pile section for reliable seismic designs. This could be explained by the fact that unlike the usual static cases, the point at which zero shear force takes place is located below the dredge line and hence a portion of the seismic passive resistance acting on the front-side of the wall influences on the determination of M_{max}. It is also noted that in the case of a/c' = 0.0 and k_h = 0.15, both magnitudes of F and M_{max} are significantly reduced as compared to those of the other cases. The reasons for this are that the magnitude of L required to satisfy moment equilibrium is extremely large, and hence a point for zero shear is located in the upper portion of the wall very close to the ground surface. These findings are considered quite important, giving rise to a necessity of further experimental verifications.

Table 7. Effects of wall adherence(a)

		Static Load		k_h		
		case 1	case 2	0.05	0.10	0.15
a =0	L	5.15	3.04	4.13	7.47	22.05
	F	89.94	72.35	70.19	86.39	53.83
	M_{max}	178.12	106.66	91.34	125.21	32.49
a/c' =0.5	L	2.96	1.83	2.52	4.18	10.06
	F	69.37	60.10	59.61	72.70	88.24
	M_{max}	99.51	72.21	65.47	90.04	119.02
a/c' =0.7	L	2.45	1.54	2.14	3.43	7.39
	F	64.39	57.05	56.76	68.63	85.36
	M_{max}	84.79	64.32	58.93	80.03	113.07

CONCLUSIONS

Based on the developed pseudostatic limit method of analysis which could estimate the seismic earth pressures exerted by c-ϕ soils, a procedure for the seismic design of anchored sheet-pile walls is proposed. Using the proposed solution procedure and typical input parameters likely to be adopted in practice, a wide range of parametric study is carried out. The results of analyses indicate that the effects of various design parameters could be considerable, and show general trends which imply that at relaticely low levels of seismic loads careful selection of a sheet-pile section is significantly important. It is believed, in the absence of an accepted seismic design procedure, that considerable judgements should be made based on the conditions prevailing for each structure. However, the present study would be of assitance in arriving at that judgement. Further studies, including a method of moment reduction for earthquake conditions, are planned to extend the finding over a wider range of conditions, and to establish a well documented seismic design procedure.

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