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Nonlinear seismic behaviour of gravity waterfront walls

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ABSTRACT : This paper presents a model for computing the earthquake induced permanent displacement of a gravity waterfront wall.The conventional,Mononobe-Okabe method dosn't take into account that during earthquake the acting forces developed in a system normally exceed the resistant forces , making permanent displacements to occure because of yielding of the materials.The proposal model consists of water,wall and soil.For considering sliding effect of wall,joint element is used and soil is considered as an elasto - rigid plastic material with neglible tensile strength.Wall is taken into account as a rigid element and water as an added mass and a mass connected by spring (modified Housner model for analysing of tanks during earthquake).At the end , by parametric study the effect of amplitude and frequency content of earthquake and mass of the wall on the displacement are studied.

1 Introduction

Gravity waterfront walls are commonly encountered in marine structures as seawalls or quaywalls . One of the important force for designing of these walls is dynamic seismic force . The most common method for seismic design of gravity walls is Mononobe - Okabe procedure . This method is discussed extensively by Seed and Whitman (6).

The inertial forces of the wall itself are usually neglected in the conventional design methods.However ,Richards and Elms (5) showed that as the inertial forces which are as large as dynamic earth pressures are taken into account in the analysis , the design of wall will be uneconomical and sometimes impossible.Hence,the wall must be allowed to slid to some allowable extent during earthquake. Although in the Richards and Elms procedure , the displacement of wall is considered but there are some shortcomings in their models which this proposal model try to eliminate them . The main object of this model is determining the relative displacement between the wall and its supporting soil in a simplified manner.

2 Evaluation of Mononobe-Okabe and Richards-Elms' methods

Since 1920 , the common practice in seismic design of gravity retaining walls has been the use of the Mononobe (2) and Okabe (4) analysis . This methode is an extension of Coulomb's theory of static earth pressures, where the inertial forces of backfill soil are taken into account (Figure 1).

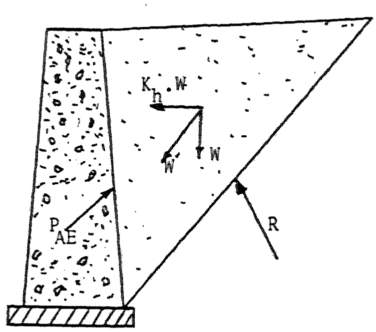

Figure 1. Wall ,the wedge failure and forces acting on it

As the Mononobe-Okabe's method is essentially for dry cohensionless soil , so for saturated soil behind of waterfront wall , the method must be modified as below :

1. It must be used submerged density instead of dry density
2. The half of δ (friction between wall and soil) must be used for determining dynamic earth pressure
3. The seismic coefficient (K_h) must be modified as below :

$$K_h' = K_h \left(\frac{\gamma_{sat}}{\gamma_{sat} - 1} \right) \left(\frac{1}{1 \pm K_v} \right)$$

γ_{sat} : saturated specific gravity of soil
K_h : horizontal seismic coefficient
K_v : vertical seismic coefficient

The shortcomings of this method are discussed by Richards and Elms (5) . They proposed a model (Figure 2) which is similar to model of sliding block , proposed by Newmark (3) . In his model a soil body is replaced by a rigid block which slides on a friction floor . In this procedure , on contrast to Mononobe-Okabe's method ,the wall is designed for a limit acceleration which normally is less the expected peak ground acceleration .In their procedure , the design criteria is to keep the earthquake - induced relative displacement between the wall and its supporting soil less than an allowable amount . In the Richards - Elms' method,the relative displacement is depended on the maximum acceleration and velocity of earth motion and it dosen't consider other characteristic of earth motion and wall geometry . Therfore this method isn't applicable for waterfront walls which the soil behind it , is saturated . So , in this paper it will be tried to eliminate some of their shortcomings .

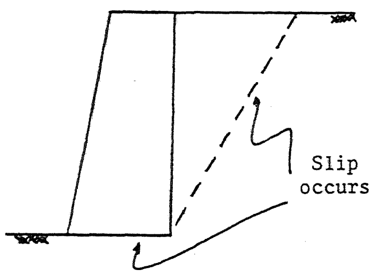

Figure 2. Sliding block of soil and wall

3 Proposed model

The proposed model allows one to compute the amount of permanent displacement of a gravity waterfront wall to earth motion very precisely in a simple manner . Figure 3 shows this proposed model . It consist the elements as below :

Mass M_1 : due to sum of mass of wall (M_w) and added mass of water in front of wall (M_0)

Mass M_2 : due to effective mass of soil behind of wall

Mass M_3 : due to slashing motion of water in front of wall

Spring K_1 :stiffness of soil behind of wall

Spring K_2 :stiffness of soil behind wedge failure

Spring K_3 :stiffness of slashing motion of water

Spring K_4 :stiffness of wall against sliding

The magnitude of M_0,M_3 and K_3 are computed on the baises of Housner model for rectangular tank (1) in the case that length of tank tends to infinite.

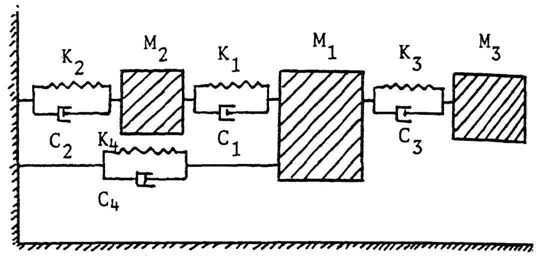

Figure 3. Proposed model

The magnitude of M_2 , K_1 , K_2 and K_4 are determined on the basis of the following assumptions :

1. The wall is stable against overturning and the mode of failure is sliding
2. The modified formula of Mononobe - Okabe for determining of dynamic pressure of soil is valid at the bigining of sliding
3. The earthquake has only horizontal component and its vertical component is ignored .

The relationship between force and displacement for springs K_1 , K_2 and K_4 are showen in figure 4. (behaviours of K_2 and K_4 springs are apparently similar but with different characteristics) . The response of 3 D.O.F Proposed model is computed by Wilson-θ method.

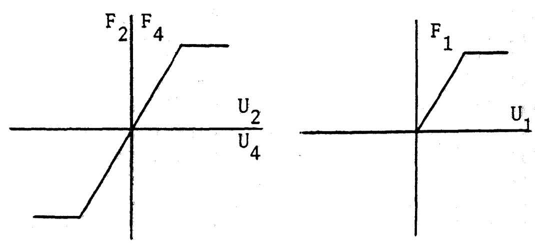

Figure 4. Force - displacement relationship of springs K_1,K_2 and K_4

4 Numerical application (parametric study)

The wall of figure 5 is studied by the proposed model . The safety factor against sliding in dry static case is 1.5 and the active force in dry and saturated condition is respectively 209 and 105 kN / m . The limit acceleration of earthquake which causes

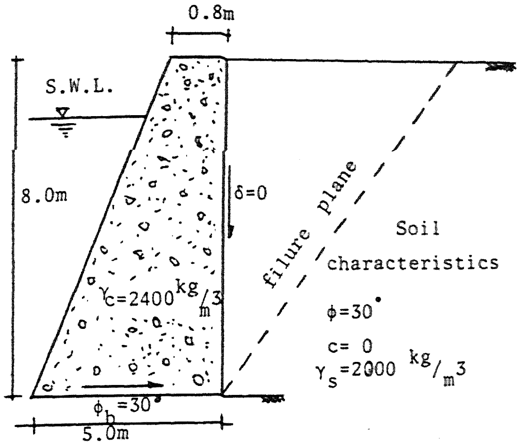

Figure 5. Geometry characteristic of the example wall

sliding in dry and saturated condition is 0.112g and 0.045g .

4.1 Effect of saturation

The response of the wall is computed for harmonic excitation of base (with 3 cycles) for three amplitude of acceleration (0.1g , 0.2g and 0.3g) in dry and saturated cases.The results are shown in figure 6. It is observed in the saturation case that wall displacement is increased in comparison with dry case and the difference between them is considerable with due to acceleration amplitude .

In table 1 the final wall displacement in dry condition is compared with Richards-Elms method . Difference in results is due to amplification of base motion by soil behind wall . This effect is ignored in Richards-Elms method.

Table 1. Comparison of wall displacement (cm) in dry and saturated case with results of Richards - Elms' method .

CASE	Ampl.of ground Accel.		
	0.1 g	0.2 g	0.3 g
Saturated Soil	0.2	3.9	14.1
Dry Soil	0.1	1.2	5.3
Richards - Elms	0.006	0.176	1.34

4.2 Effect of acceleration amplitude

Figure 7 shows the wall displacement in saturated condition for harmonic excitation of ground (f = 5 Hz) with 0.1g ,0.2g and 0.3g amplitude . Obviously , with increasing of acceleration amplitude ,the wall displacement increased drastically . So the amplitude of acceleration in a site is an important parameter for displacement determination .

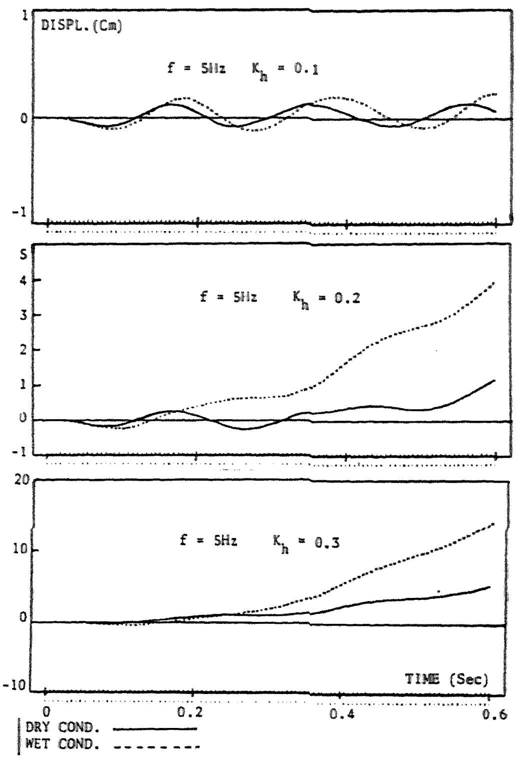

Figure 6. Wall displacements' variations in dry and saturated soils .

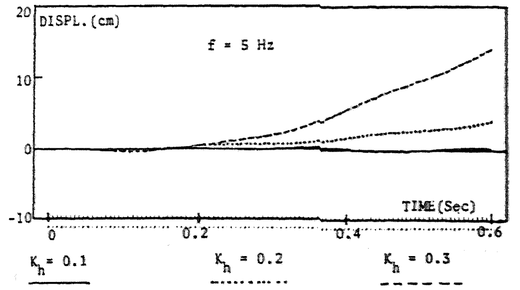

Figure 7. Wall displacements' variations as compared with acceleration amplitude

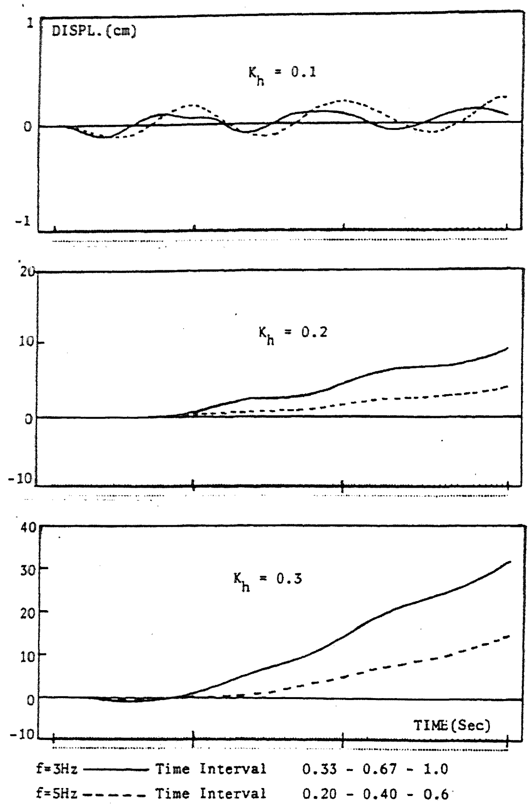

Figure 8. Wall displacements' variations for different frequency contents in various cases

4.3 Effect of frequency content of ground motion

Figure 8 shows the variation of wall displacement in saturated condition due to the 3 cycles harmonic ground motion with maximum acceleration amplitude of 0.1g , 0.2g and 0.3g for different frequency contents (3 and 5 Hz).The results are summarized in table 2 .Upon the results,this parameter has also significant effect.

Table 2. Final displacement of wall (cm) for different frequency of motion

Frequency of Motions	Amplitude of ground Acceleration		
	0.1 g	0.2 g	0.3 g
3 Hertz	0.1	9.1	31.8
5 Hertz	0.2	3.9	14.1

4.4 Effect of damping ratio

The results of response due to the harmonic motion of ground with frequencies of 3 and 5 Hz and maximum acceleration amplitude of 0.3g for 10% and 20% damping ratios are shown in figure 9 and table 3.

Table 3. Effect of damping on final wall displacement

DAMPING RATIO	Frequency of Motions	
	3 Hz	5 Hz
10 percentage	31.8	14.1
20 percentage	18.0	9.0

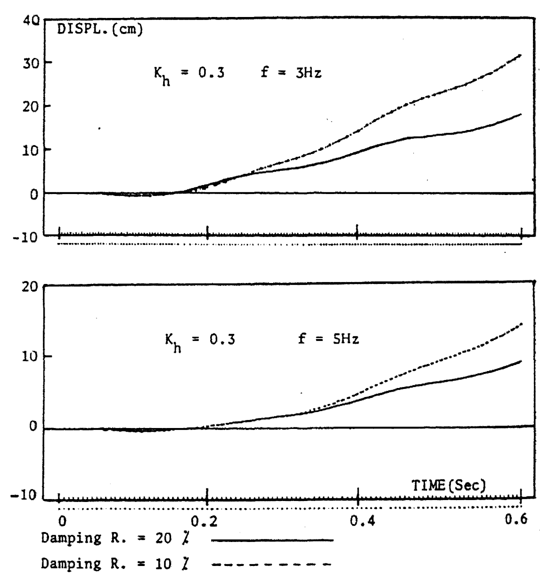

Figure 9. Wall displacements' variations for different damping ratios

4.5 Effect of wall wieght

Figure 10 shows the displacement variation of two walls with different weights due to the harmonic motion of ground with 5 Hz frequency and 0.2g,0.3g maximum acceleration amplitudes It is observed , with increasing the weight of wall , the final displacement is decreased considerably.

Table 4. Effect of wall weight increase on final wall displacement

WALL MASS	Amplitude of ground Acceleration	
	0.2 g	0.3 g
55680 Kgm	3.9	14.1
131211 Kgm	1.2	4.5

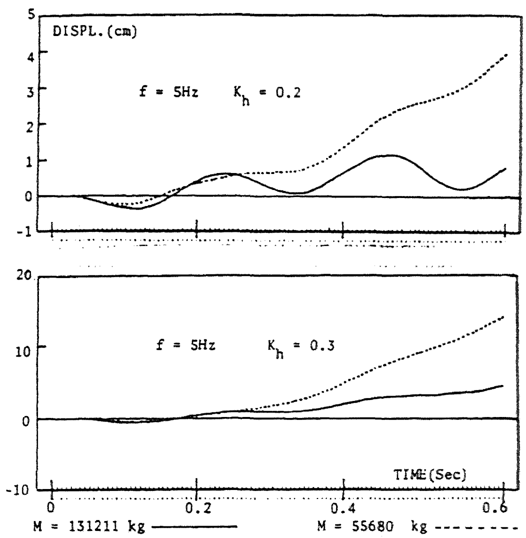

Figure 10. Wall displacements' variations for the different wall weights

5 Conclusion

As the convention design method which are based on prevention of sliding of walls lead to uneconomical and sometimes impossible design . So it is necessary to predict the relative wall displacement as accurate as possible .Although Richards and Elms suggest a new design procedure , but there are some shortcomings in their model . Hence in this paper , it was presented a three D.O.F system for computing permanent displacement of wall accurately in a simple manner .In this study it was founded that the amplitude of acceleration , frequency of ground motion, weight of walls and presence of water have major effect on the magnitude of permanent relativ wall displacement .

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