Available online at www.sciencedirect.com
ScienceDirect
Ocean Engineering 34 (2007) 1947–1954
OCEAN ENGINEERING
www.elsevier.com/locate/oceaneng

Stability of waterfront retaining wall subjected to pseudo-static earthquake forces

Deepankar Choudhury*, Syed Mohd. Ahmad

Department of Civil Engineering, Indian Institute of Technology Bombay, Powai, Mumbai 400 076, India

Received 30 October 2006; accepted 26 March 2007
Available online 12 April 2007

Abstract

Waterfront retaining walls supporting dry backfill are subjected to hydrostatic pressure on upstream face and earth pressure on the downstream face. Under seismic conditions, if such a wall retains a submerged backfill, additional hydrodynamic pressures are generated. This paper pertains to a study in which the effect of earthquakes along with the hydrodynamic pressure including inertial forces on such a retaining wall is observed. The hydrodynamic pressure is calculated using Westergaard's approach, while the earth pressure is calculated using Mononobe-Okabe's pseudo-static analysis. It is observed that when the horizontal seismic acceleration coefficient is increased from 0 to 0.2, there is a 57% decrease in the factor of safety of the retaining wall in sliding mode. For investigating the effect of different parameters, a parametric study is also done. It is observed that if ϕ is increased from 30° to 35°, there is an increase in the factor of safety in the sliding mode by 20.4%. Similar observations were made for other parameters as well. Comparison of results obtained from the present approach with [Ebeling, R.M., Morrison Jr, E.E., 1992. The seismic design of waterfront retaining structures. US Army Technical Report ITL-92-11. Washington DC] reveal that the factor of safety for static condition ($k_h = 0$), calculated by both the approaches, is 1.60 while for an earthquake with $k_h = 0.2$, they differ by 22.5% due to the consideration of wall inertia in the present study.

Keywords: Hydrodynamic pressure; Seismic active earth pressure; Design; Wall inertia; Sliding; Overturning; Factor of safety; Soil and wall friction angle

1. Introduction

Waterfront retaining structures are extensively used across the world and the design of these retaining structures is an important topic of research for civil engineers. Again, the devastating effects of the earthquakes make the problem more complicated compared to the static design procedure for the waterfront retaining wall. Hence, the stability of the waterfront retaining wall under the earthquake conditions must be studied carefully. Under the static condition, for a typical waterfront retaining wall, supporting a dry backfill, the only disturbing force for the stability of the wall is the lateral earth pressure from the downstream side, while on the upstream side, the disturbing force is the hydrostatic pressure. However, the situation changes when such a waterfront retaining wall retains a submerged backfill and is subjected to an earthquake, because additional hydrodynamic pressure gets generated along with the seismic lateral earth pressure on the downstream side of the wall. Several researchers in the recent past had given solutions for the computation of the seismic lateral earth pressure acting on a rigid retaining wall. The pioneering work by Okabe (1924) and Mononobe and Matsuo (1929), which is commonly known as Mononobe–Okabe method (see Kramer, 1996) by considering the pseudo-static seismic accelerations, is still being used worldwide, to compute the seismic lateral earth pressure. The work done by Ebeling and Morrison (1992) considered both the seismic active earth pressure and hydrodynamic pressure for the design of the waterfront retaining walls. Again, the hydrodynamic pressure, which tends to destabilize the wall, was described by Kim et al. (2005). A study of such a waterfront retaining wall and its behaviour under the action of the above-mentioned forces needs to be carried out to assess its stability.

*Corresponding author. Tel.: +91 22 2576 7335; fax: +91 22 2576 7302.
E-mail address: dc@civil.iitb.ac.in (D. Choudhury).

0029-8018/$ - see front matter © 2007 Elsevier Ltd. All rights reserved.
doi:10.1016/j.oceaneng.2007.03.005

Nomenclature

b, H	width and height of the wall
h	height of the water on upstream side
F_{d_r}, F_{d_f}	driving force for restrained and free water conditions
F_r	resisting force
$FS_{overturning_r}$	factor of safety for overturning mode of failure for restrained water condition
$FS_{overturning_f}$	factor of safety for overturning mode of failure for free water condition
$FS_{sliding_r}$	factor of safety for sliding mode of failure for restrained water condition
$FS_{sliding_f}$	factor of safety for sliding mode of failure for free water condition
k	hydraulic conductivity of the soil
k_h, k_v	horizontal and vertical seismic acceleration coefficient
K_{ae}	seismic active earth pressure coefficient
P_{ae}	seismic active earth thrust
P_{dyn}	hydrodynamic pressure
P_{stat}, P'_{stat}	hydrostatic pressure on upstream and downstream side
P_w	pressure due to water
r_u	pore pressure ratio
W	weight of the wall
y	point of application of P_{ae}
β	ground inclination with respect to horizontal
δ	wall friction angle
γ_w, γ_s, γ_c	specific weight of water, soil, and concrete
$\bar{\gamma}$, γ_{we}	equivalent specific weight of the soil and water due to submergence
γ_{sat}, γ_d	saturated and dry specific weight of the soil
ϕ	soil friction angle
μ	coefficient of base friction
θ	wall inclination with respect to vertical
ψ	seismic inertia angle

However, a very few literature proposed the analysis of waterfront retaining wall under the combined action of forces due to water and seismic earth pressure, as most of the literature deals with the individual forces acting on the waterfront retaining wall. For example, the effect of wave action on caisson, vertical and sloping walls and other coastal structures had been studied by Kirkgoz and Mengi (1987) and by Kirkgoz (1990, 1991 and 1995). Muller and Whittaker (1993) investigated the effect of wave impact on the sloping walls, while a comparative study for the evaluation of the design wave impact pressure is again reported by Muller and Whittaker (1996). Experimental studies to assess the behaviour of the vertical wall were reported by Ramsden (1996) with the details of the development of an empirical expression for calculating the forces and moments on a vertical wall due to long waves, bores and surges. For the studies related to the hydrodynamic pressure, Chakrabarti et al. (1978) had shown its effect on cellular type cofferdams. New method of analysis for the quay wall including the effect of hydrodynamic pressure was described by Nozu et al. (2004). Again, the seismic active earth pressures acting on the rigid retaining wall for dry soil were computed by using different methods of analyses like the limit equilibrium method (Seed and Whitman, 1970; Richards and Elms, 1979; Choudhury and Singh, 2006; Choudhury and Nimbalkar, 2006, 2007; Nimbalkar and Choudhury, 2007), approximate elastic solutions (Matsuo and Ohara, 1960), two-dimensional wave propagation theory or shear-beam model (Scott, 1973; Veletsos and Younan, 1994; Wu and Finn, 1999), finite element techniques (Nadim and Whitman, 1983; Gazetas et al., 2004), numerical simulation by using geotechnical software FLAC (Green et al., 2003). But none of the above solutions considered the effect of hydrodynamic pressure.

Steps for the analysis of the rigid retaining wall by considering the hydrodynamic pressure generated due to the submerged backfill along with the seismic active earth pressures were given only by Ebeling and Morrison (1992). However, one of the important aspect of considering the wall inertia, the effect of which on the stability of a retaining wall has already been well established, as is reported by Richards and Elms (1979), Choudhury and Nimbalkar (2007) and Nimbalkar and Choudhury (2007) is not addressed properly in the above-mentioned analysis. Hence, till today, the complete solution for the combined effect of seismic active earth pressure and hydrodynamic pressure on the waterfront retaining wall with the consideration of wall inertia is scarce.

The present method completely describes the behaviour of a waterfront retaining wall from the stability consideration in terms of the sliding and overturning modes of failure under earthquake condition. This study is extremely essential for the design purpose of the waterfront retaining wall under seismic condition. A generalized case of a waterfront retaining wall, supporting a submerged backfill on one side and water on the other side, under seismic conditions including seismic inertial forces is considered.

2. Method of analysis

A typical waterfront retaining wall with vertical face (i.e., $\theta = 0°$), width 'b' and height 'H' is shown in Fig. 1. It retains backfill to its full height on one side, referred to as the 'downstream side', and water to a height of 'h' on the other side, called as the 'upstream side' of the wall. The ground surface of the backfill is assumed to be horizontal (i.e., $\beta = 0°$) and is submerged to the same level (i.e., 'h') up to which the water is standing on the upstream side of the retaining wall. A free body diagram of the wall showing

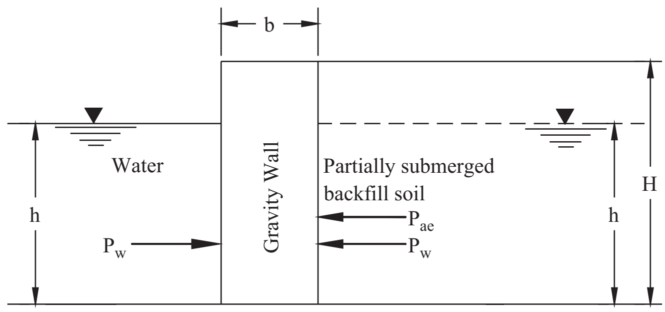

Fig. 1. A typical gravity type waterfront retaining wall.

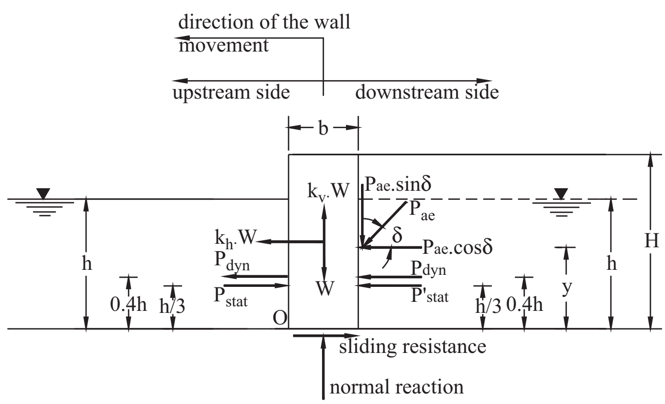

Fig. 2. Free body diagram of the wall subjected to different forces.

different forces coming onto it from soil, water, and due to seismicity along with their respective points of applications is shown in Fig. 2. Pseudo-static seismic accelerations with acceleration coefficients k_h and k_v in the horizontal and vertical directions, respectively, are assumed to act. Basically, the wall is subjected to three kinds of forces viz., the seismic earth pressure force, the inertia force on the wall and force due to the presence of water (both on the upstream and downstream sides) and each of these are calculated as follows.

2.1. Seismic earth pressure

The seismic active earth pressure on the wall is calculated using the pseudo-static Mononobe–Okabe's approach. Similar to the analysis of Ebeling and Morrison (1992) and the expression given by Kramer (1996), the basic expression for the calculation of the total seismic active earth thrust (P_{ae}) has been modified to consider the effect of submergence in the backfill and the existence of excess pore pressure, and is given as,

$$P_{ae} = \tfrac{1}{2} K_{ae} H^2 \bar{\gamma}(1 - k_v)(1 - r_u), \tag{1}$$

where

$$K_{ae} = \frac{\cos^2(\phi - \theta - \psi)}{\cos\psi \cos^2\theta \cos(\delta + \theta + \psi)\left[1 + \sqrt{\sin(\delta + \phi)\sin(\phi - \beta - \psi)/\cos(\delta + \theta + \psi)\cos(\beta - \theta)}\right]^2}, \tag{2}$$

$$\psi = \tan^{-1}\frac{\gamma_{sat} k_h}{\bar{\gamma}(1 - k_v)}, \tag{3}$$

$$\bar{\gamma} = \left(\frac{h}{H}\right)^2 \gamma_{sat} + \left(1 - \left(\frac{h}{H}\right)^2\right)\gamma_d. \tag{4}$$

It is to be noted that the pore pressure ratio 'r_u', which is defined as the ratio of excess pore pressure to the initial vertical stress, incorporated in Eq. (1) above is a simplified way (as per Ebeling and Morrison, 1992) of simulating the effect of the excess pore pressure generated due to cyclic shaking of the soil during an earthquake.

2.2. Seismic inertia forces on the wall

Due to earthquake, additional inertia forces will be developed in the wall and for vertical and horizontal directions, these forces are given by $k_v \cdot W$ and $k_h \cdot W$ respectively. Though different combinations of these inertia forces with respect to the direction of vertical and horizontal seismic acceleration coefficients k_v and k_h are considered, only the critical combination resulting in maximum seismic active earth pressure, which needs to be considered for the design is shown in Fig. 2.

2.3. Forces on the wall due to water

The forces acting on the wall due to the presence of water, both on the upstream and downstream sides are calculated as follows.

2.3.1. Hydrostatic force

The hydrostatic force (P_{stat}) due to the standing water on the face of the wall is given by,

$$P_{stat} = \tfrac{1}{2}\gamma_w h^2. \tag{5}$$

It acts at a height of $h/3$ from the base of the wall. However, for calculating the hydrostatic pressure on the wall from the downstream side (P'_{stat}), γ_w in Eq. (5) is replaced by γ_{we} (as given by Ebeling and Morrison, 1992), and can be calculated as

$$\gamma_{we} = \gamma_w + (\bar{\gamma} - \gamma_w) r_u. \tag{6}$$

Thus,

$$P'_{stat} = \tfrac{1}{2}\gamma_{we} h^2. \tag{7}$$

2.3.2. Hydrodynamic force

The hydrodynamic force (P_{dyn}) acting on the vertical face of the wall is calculated using the Westergaard's (1933) approach and is given as

$$P_{dyn} = \tfrac{7}{12} k_h \gamma_w h^2. \tag{8}$$

It acts at a height of $0.4h$ from the base of the wall. On the upstream side, this hydrodynamic force acts in a direction opposite to the direction of the hydrostatic force (Ebeling and Morrison, 1992), while on the downstream side the direction of both the hydrostatic and hydrodynamic forces would be towards the wall, thus creating a worst possible combination with respect to both the sliding and overturning modes of failure of the wall. Though Matsuo and Ohara (1965) had suggested the hydrodynamic pressure on the downstream side to be around 70% of that on the upstream side, but to consider the worst possible combination of forces for the design of the wall, similar to the consideration of Ebeling and Morrison (1992), here in the present study, the same amount of the hydrodynamic pressure is considered both on the downstream and upstream side.

3. Stability of the wall

Under the action of the above-mentioned forces, the stability of the wall is checked for both the sliding and overturning modes of failure using limit equilibrium method. Depending on the hydraulic conductivity (k) of the soil, two different cases viz., *restrained water case* ($k \sim 10^{-3}$ cm/sec) and *free water case* ($k\sim$very high) may arise for the generation of the hydrodynamic pressure in the backfill soil (Kramer, 1996). For the restrained water case, the movement of water is assumed to be with the movement of the backfill soil particles and thus it is assumed that the hydrodynamic pressure is not present. However, for the free water case, the water is having enough space to move freely within the soil, hence, the additional hydrodynamic pressure is considered. Expressions for finding out the factor of safety against the sliding and overturning modes of failure are detailed in the following section.

3.1. Factor of safety against sliding mode of failure

Considering the equilibrium of all the forces acting in the horizontal direction (Fig. 2), one can write

$$\text{Total resisting force,} \quad F_{\mathrm{r}} = P_{\mathrm{stat}} + \mu[(1-k_{\mathrm{v}})W + P_{\mathrm{ae}}\sin\delta]. \tag{9}$$

And, the total driving force for the restrained water case is

$$F_{d_{\mathrm{r}}} = P'_{\mathrm{stat}} + P_{\mathrm{dyn}} + P_{\mathrm{ae}}\cos\delta + k_{\mathrm{h}}\cdot W. \tag{10}$$

Similarly, for the free water case, due to the additional hydrodynamic force, there would be an extra component of the same and the total driving force will be

$$F_{d_{\mathrm{f}}} = P'_{\mathrm{stat}} + 2P_{\mathrm{dyn}} + P_{\mathrm{ae}}\cdot\cos\delta + k_{\mathrm{h}}\cdot W. \tag{11}$$

The respective factor of safety of the wall against sliding mode of failure for both the restrained and free water cases are then given as

$$\mathrm{FS}_{\mathrm{sliding}_{\mathrm{r}}} = \frac{F_{\mathrm{r}}}{F_{d_{\mathrm{r}}}} = \frac{P_{\mathrm{stat}} + \mu[(1-k_{\mathrm{v}})W + P_{\mathrm{ae}}\cdot\sin\delta]}{P'_{\mathrm{stat}} + P_{\mathrm{dyn}} + P_{\mathrm{ae}}\cdot\cos\delta + k_{\mathrm{h}}\cdot W} \tag{12}$$

and

$$\mathrm{FS}_{\mathrm{sliding}_{\mathrm{f}}} = \frac{F_{\mathrm{r}}}{F_{d_{\mathrm{f}}}} = \frac{P_{\mathrm{stat}} + \mu[(1-k_{\mathrm{v}})W + P_{\mathrm{ae}}\cdot\sin\delta]}{P'_{\mathrm{stat}} + 2P_{\mathrm{dyn}} + P_{\mathrm{ae}}\cdot\cos\delta + k_{\mathrm{h}}\cdot W} \tag{13}$$

where

$$\mu = \text{coefficient of base friction} = \tan\phi, \tag{14}$$

$$W = \text{weight of the wall} = bH\gamma_{\mathrm{c}}. \tag{15}$$

For the generalized design purpose, Eqs. (12) and (13) can be rewritten in the non-dimensional form as follows:

$$\mathrm{FS}_{\mathrm{sliding}_{\mathrm{r}}} = \frac{\frac{1}{2}\gamma_{\mathrm{w}}(h/H)^2 + \mu\left((1-k_{\mathrm{v}})(b/H)\gamma_{\mathrm{c}} + \frac{1}{2}K_{\mathrm{ae}}\bar{\gamma}\cdot\sin\delta\right)}{\frac{1}{2}\gamma_{\mathrm{we}}(h/H)^2 + \frac{7}{12}k_h\gamma_w(h/H)^2 + \frac{1}{2}K_{\mathrm{ae}}\bar{\gamma}\cdot\cos\delta + k_{\mathrm{h}}(b/H)\gamma_{\mathrm{c}}}, \tag{16}$$

$$\mathrm{FS}_{\mathrm{sliding}_{\mathrm{f}}} = \frac{\frac{1}{2}\gamma_{\mathrm{w}}(h/H)^2 + \mu\left((1-k_{\mathrm{v}})(b/H)\gamma_{\mathrm{c}} + \frac{1}{2}K_{\mathrm{ae}}\bar{\gamma}\cdot\sin\delta\right)}{\frac{1}{2}\gamma_{\mathrm{we}}(h/H)^2 + \frac{7}{6}k_{\mathrm{h}}\gamma_{\mathrm{w}}(h/H)^2 + \frac{1}{2}K_{\mathrm{ae}}\bar{\gamma}\cdot\cos\delta + k_{\mathrm{h}}(b/H)\gamma_{\mathrm{c}}}. \tag{17}$$

3.2. Factor of safety against overturning mode of failure

Similarly, by assuming that the seismic active earth pressure (P_{ae}) acts at $y = 0.5\mathrm{H}$ above the base of the wall (Ebeling and Morrison, 1992), the factor of safety against the overturning mode of failure for both the restrained and free water cases, respectively, are given as

$$\mathrm{FS}_{\mathrm{overturning}_{\mathrm{r}}} = \frac{\frac{1}{6}\gamma_{\mathrm{w}}(h/H)^3 + \frac{1}{2}(b/H)^2(1-k_{\mathrm{v}})\gamma_{\mathrm{c}} + \frac{1}{2}K_{\mathrm{ae}}\bar{\gamma}(b/H)\sin\delta}{\frac{1}{6}\gamma_{\mathrm{we}}(h/H)^3 + (2.8/12)k_{\mathrm{h}}\gamma_{\mathrm{w}}(h/H)^3 + \frac{1}{4}K_{\mathrm{ae}}\bar{\gamma}\cos\delta + \frac{1}{2}k_{\mathrm{h}}(b/H)\gamma_{\mathrm{c}}}. \tag{18}$$

and

$$\mathrm{FS}_{\mathrm{overturning}_{\mathrm{f}}} = \frac{\frac{1}{6}\gamma_{\mathrm{w}}(h/H)^3 + \frac{1}{2}(b/H)^2(1-k_{\mathrm{v}})\gamma_{\mathrm{c}} + \frac{1}{2}K_{\mathrm{ae}}\bar{\gamma}(b/H)\sin\delta}{\frac{1}{6}\gamma_{\mathrm{we}}(h/H)^3 + \frac{5.6}{12}k_{\mathrm{h}}\gamma_{\mathrm{w}}(h/H)^3 + \frac{1}{4}K_{\mathrm{ae}}\bar{\gamma}\cos\delta + \frac{1}{2}k_{\mathrm{h}}(b/H)\gamma_{\mathrm{c}}}. \tag{19}$$

Using the above proposed Eqs. (16), (17), (18) and (19), one can easily design the section of the waterfront retaining wall subjected to the combined earthquake and hydrodynamic forces.

4. Results and discussions

Combination of the different parameters involved in Eqs. (16)–(19) can predict the stability of the waterfront wall under earthquake. However, the chosen values must be of practical significance and the phenomenon of shear fluidization must be avoided. To avoid the phenomenon of shear fluidization, as proposed by Richards et al. (1990) for the dry soil, and subsequently modified by Ebeling and Morrison (1992) for the wet soil, the following expression

Table 1
Values/range of different parameters chosen for the present study

Parameter	Value/range
b/H	0.4
h/H	0, 0.4, 0.8
k_h	0, 0.1, 0.2, 0.3, 0.4
k_v	0, $k_h/2$, k_h
r_u	0.2
γ_c, γ_{sat}, γ_d, γ_w	25, 19, 16 and 10 kN/m^3 respectively
ϕ (degree)	25, 30, 35, 40
δ (degree)	$-\phi/2$, 0, $\phi/2$

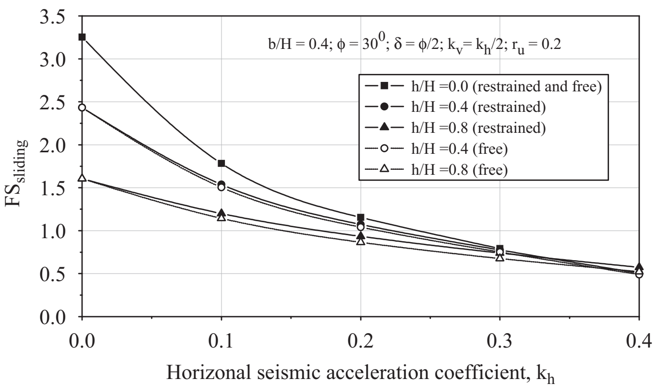

Fig. 3. Factor of safety in sliding mode for different h/H values.

need to be satisfied for the present study.

$$\psi < \phi \tag{20}$$

Different values of the parameters and their ranges considered in the present analysis are given in Table 1. Effect of the various parameters on the sliding and overturning stability of the wall with respect to the value of water to wall height ratio (h/H), soil friction angle (ϕ), wall friction angle (δ), and the coefficients of horizontal and vertical seismic accelerations (k_h and k_v) are discussed in Figs. 3–10.

4.1. Effect of the horizontal seismic acceleration coefficient (k_h)

From Fig. 3 it is observed that for a particular value of h/H ratio, the factor of safety in sliding mode decreases drastically with an increase in the value of horizontal seismic acceleration coefficient k_h. It shows that a stable waterfront retaining wall can fail with the increase in horizontal seismic acceleration. As an illustration, for chosen values of $b/H = 0.4$, $h/H = 0.4$, $\phi = 30^\circ$, $\delta = \phi/2$, $k_v = k_h/2$ and $r_u = 0.2$, the factor of safety against sliding both for the restrained and free water cases is 2.43 when there is no earthquake (i.e., $k_h = 0$); while the same reduces to 1.07 for the restrained water case and to 1.04 for the free water case when the value of k_h is increased to 0.2. Hence, a

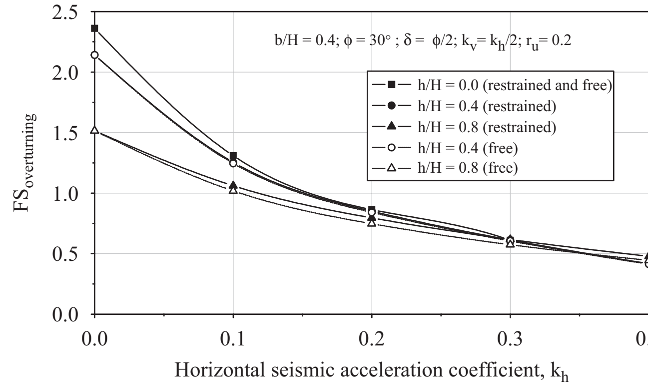

Fig. 4. Factor of safety in overturning mode for different h/H values.

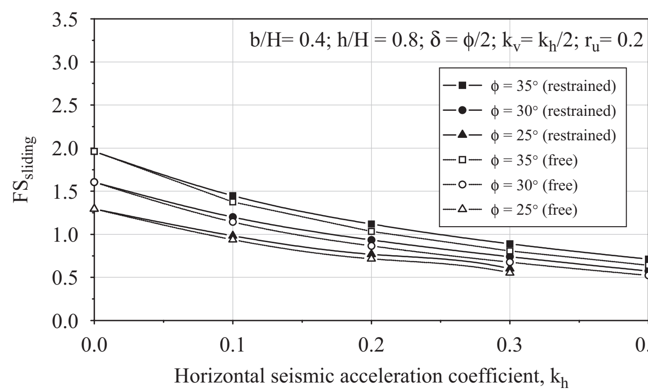

Fig. 5. Effect of ϕ on sliding stability.

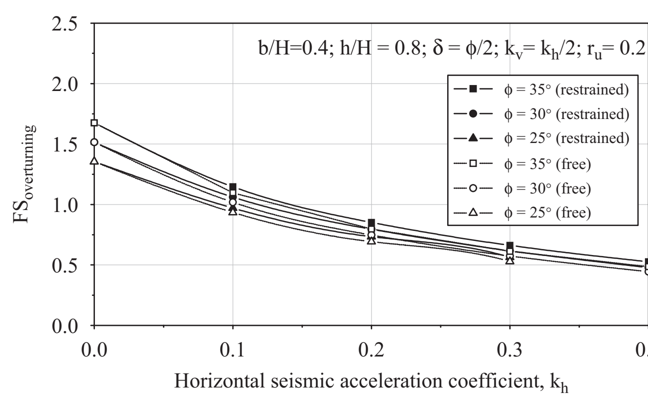

Fig. 6. Effect of ϕ on overturning stability.

decrease in the factor of safety by about 57% for an increase in k_h from 0 to 0.2 is observed for both the cases. Similar trend is noted for the overturning mode also (Fig. 4). Another important observation which can be made from Figs. 3 and 4 is that the effect of h/H ratio on the factor of safety for lower k_h values is significant as can be observed from a large difference in values of factor of safety for a particular k_h value, say 0 or 0.1. But the effect of h/H ratio on the factor of safety in both sliding and overturning modes of failure is marginal at higher values of k_h. However, it may be noted that at higher values of k_h,

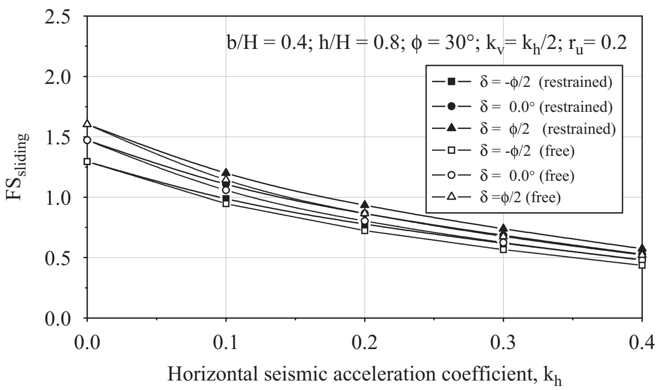

Fig. 7. Effect of δ on sliding stability.

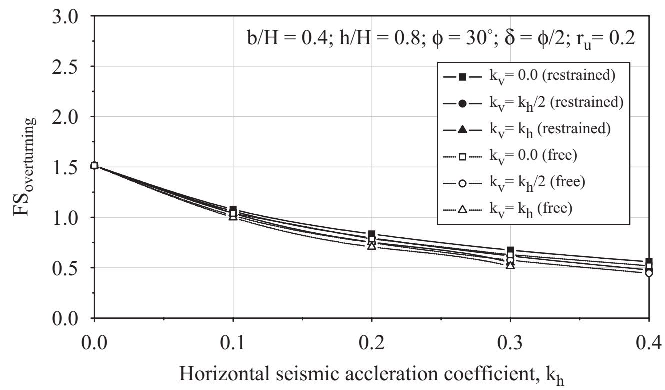

Fig. 10. Effect of k_v on overturning stability.

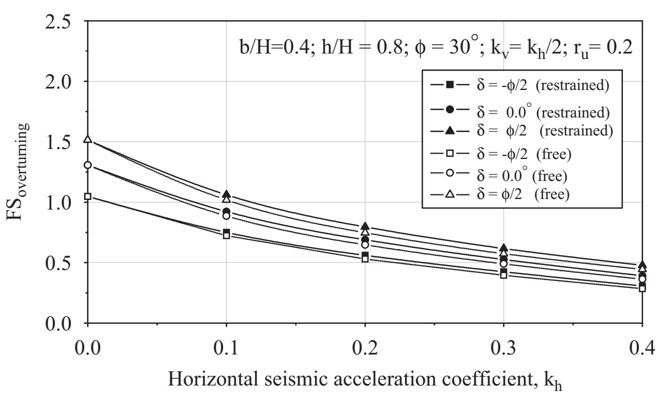

Fig. 8. Effect of δ on overturning stability.

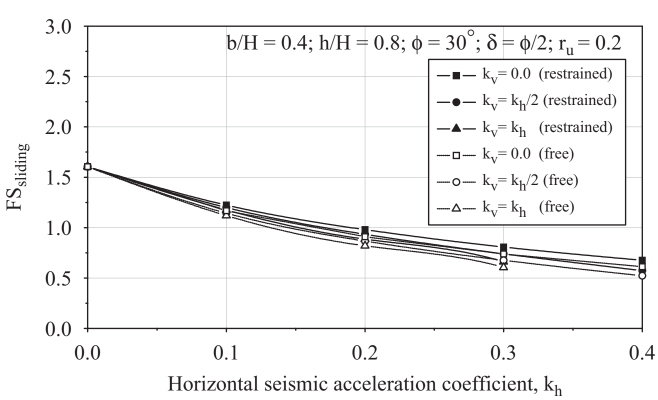

Fig. 9. Effect of k_v on sliding stability.

the stable wall under static condition has failed in both the sliding and overturning modes (FS < 1 in Figs. 3 and 4).

4.2. Effect of soil friction angle (ϕ)

Figs. 5 and 6 respectively show the variation of factor of safety in the sliding and overturning modes of failure for different ϕ values. With the increase in the value of ϕ from 30° to 35°, there is 20.4% increase in the factor of safety against the sliding mode of failure (Fig. 5) for $k_h = 0.1$, $b/H = 0.4$, $h/H = 0.8$, $\delta = \phi/2$, $k_v = k_h/2$ and $r_u = 0.2$. The rate of decrease in factor of safety value with decrease in the value of soil friction angle ϕ is nearly constant for all values of k_h and is true for the overturning case also (Fig. 6). Also, the trend is similar both for the free and restrained water cases, except for the fact that the value for free water case is slightly lower than the value of restrained water case. Another important observation from these figures is that with an increase in ϕ, the shear fluidization phenomenon can be avoided.

4.3. Effect of wall friction angle (δ)

From Figs. 7 and 8, the effect of wall friction angle (δ) on the sliding and overturning stability of the wall is observed. It is found that as δ increases from 0 to $\phi/2$, the stability of the wall increases. For example, for $k_h = 0.1$, $b/H = 0.4$, $h/H = 0.8$, $\phi = 30°$, $k_v = k_h/2$ and $r_u = 0.2$ in Fig. 7, the factor of safety against the sliding mode is 1.11 when δ is 0 and increases to 1.20 when δ is changed to $\phi/2$, i.e., an increase of about 8.1% in the factor of safety value for a change in δ from 0 to $\phi/2$ for the restrained water case. For the same data, in case of free water, the increase in the factor of safety against sliding is about 8.0% for a change in δ from 0 to $\phi/2$. For the overturning mode (Fig. 8), the similar trend is observed for the factor of safety value with change in δ.

4.4. Effect of the vertical seismic coefficient (k_v)

As shown in Fig. 9, with the increase in the vertical seismic acceleration coefficient, k_v from 0 to k_h, the factor of safety against sliding mode reduces (for $k_h = 0.1$, $b/H = 0.4$, $h/H = 0.8$, $\phi = 30°$ and $r_u = 0.2$) by about 4%, which may be considered as marginal. Overturning mode of failure of the wall shows the similar behaviour as can be seen from Fig. 10.

5. Comparison of results

For the purpose of verifying the present methodology for design, the results obtained must be compared with existing works. However, as already mentioned, for the waterfront retaining wall subjected to combined hydrodynamic

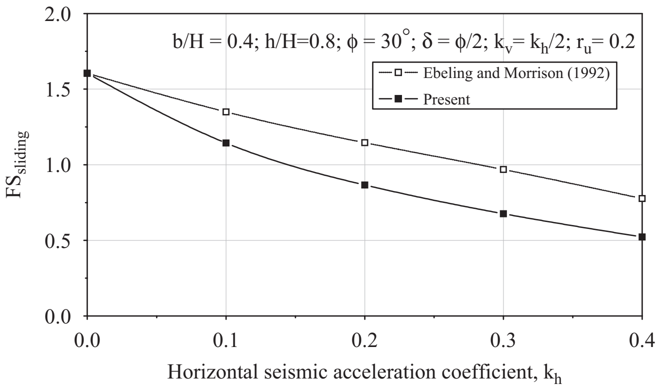

Fig. 11. Comparison between present analysis and the one adopted by Ebeling and Morrison (1992) for sliding stability.

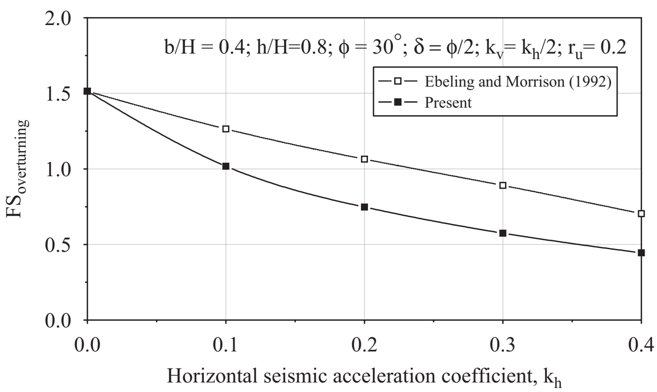

Fig. 12. Comparison between present analysis and the one adopted by Ebeling and Morrison (1992) for overturning stability.

pressure and seismic active earth pressure, the only work which can found out is the one by Ebeling and Morrison (1992). Figs. 11 and 12 present a comparison between the results of the present study and the one obtained by the approach used by Ebeling and Morrison (1992). A keen observation of the expressions for the factor of safety (Eqs. (16)–(19)) would show that by the present work, because of the consideration of the additional seismic wall inertia forces, the factor of safety would be lower when compared with the one calculated using the approach adopted by Ebeling and Morrison (1992), where no such wall inertia was considered for the design. For the purpose of illustration, the results for free water case have been plotted in Figs. 11 and 12 for the sliding and overturning modes of failure respectively. From Fig. 11, the factor of safety in sliding mode of failure for no earthquake condition i.e. $k_h = 0$, and with $b/H = 0.4$, $h/H = 0.8$, $\phi = 30^\circ$, $\delta = \phi/2$, $k_v = k_h/2$ and $r_u = 0.2$, calculated by both the approaches comes out to be same (around 1.60) as expected; however, under seismic condition, say for $k_h = 0.2$, the factor of safety calculated from the Ebeling and Morrison's (1992) approach and by present approach is 1.14 and 0.86 respectively. This shows that the same wall when analysed using the Ebeling and Morrison's (1992) approach is stable while the present analysis shows that it has failed. This difference is due to the presence of additional inertial force considered in the present analysis which is more logical. Similar observation can be made for the overturning case also as shown in Fig. 12.

6. Conclusions

The study shows the importance to develop a separate design methodology for a waterfront retaining wall under earthquake condition. The stability of the wall decreases significantly during an earthquake. An easy methodology to design the section of the waterfront retaining wall subjected to the combined action of hydrodynamic pressure and seismic active earth pressure is described through the closed-form solutions to obtain the factor of safety against sliding and overturning modes of failure. From the typical results it is observed that for a given wall section and other soil and water parameters remaining constant, the factor of safety in overturning mode is less than the factor of safety in sliding mode under earthquake condition. Parameters like soil friction angle (ϕ), wall friction angle (δ), and horizontal and vertical seismic accelerations (k_h and k_v), water to wall height ratio (h/H) have significant effect on the stability of the wall, and out of these, the factor of safety value is very much sensitive to the ϕ and δ. Comparison of the present results with those obtained by Ebeling and Morrison (1992) suggests that wall inertia needs to be considered as it has a significant effect on the stability of the wall.

Acknowledgement

The authors gratefully acknowledge the critical reviews and helpful suggestions made by the anonymous reviewers.

References

Chakrabarti, S., Husak, A.D., Christiano, P.P., Troxell, D.E., 1978. Development of seismic design criteria for category I cofferdams. Nuclear Engineering and Design 45, 277–283.

Choudhury, D., Nimbalkar, S.S., 2006. Pseudo-dynamic approach of seismic active earth pressure behind retaining wall. Geotechnical and Geological Engineering 24, 1103–1113.

Choudhury, D., Nimbalkar, S., 2007. Seismic rotational displacement of gravity walls by pseudo-dynamic method: passive case. Soil Dynamics and Earthquake Engineering 27, 242–249.

Choudhury, D., Singh, S., 2006. New approach for estimation of static and seismic active earth pressure. Geotechnical and Geological Engineering 24 (1), 117–127.

Ebeling, R.M., Morrison Jr., E.E., 1992. The seismic design of waterfront retaining structures. US Army Technical Report ITL-92-11. Washington DC.

Gazetas, G., Psarropoulos, P.N., Anastasopoulos, I., Gerolymos, N., 2004. Seismic behaviour of flexible retaining systems subjected to short-duration moderately strong excitation. Soil Dynamics and Earthquake Engineering 24, 537–550.

Green, R.A., Olgun, C.G., Ebeling, R.M., Cameron, W.I., 2003. Seismically induced lateral earth pressures on a cantilever retaining wall. ASCE, Earthquake Engineering 133, 946–955.

Kim, S., Jang, I., Chung, C., Kim, M., 2005. Evaluation of seismic displacements of quay walls. Proceedings of the International Conference on Geotechnical Engineering for Disaster Mitigation & Rehabilitation (GEDMR-2005) Singapore, pp. 84–93.

Kirkgoz, M.S., 1990. An experimental investigation of a vertical wall response to breaking wave impact. Ocean Engineering 12 (4), 379–391.

Kirkgoz, M.S., 1991. Impact pressure of breaking waves on vertical and sloping walls. Ocean Engineering 18 (1/2), 45–59.

Kirkgoz, M.S., 1995. Breaking wave impact on vertical and sloping coastal structures. Ocean Engineering 22 (1), 35–48.

Kirkgoz, M.S., Mengi, Y., 1987. Design of a caisson plate under wave impact. Ocean Engineering 14 (4), 275–283.

Kramer, S.L., 1996. Geotechnical Earthquake Engineering. Pearson Education Inc., New Jersey.

Matsuo, H., Ohara, S., 1960. Lateral earthquake pressure and stability of quay walls during earthquakes. Proceedings of Second World Conference on Earthquake Engineering, vol. 2, pp. 165–173.

Matsuo, H., Ohara, S., 1965. Dynamic pore water pressure acting on quay walls during earthquakes. Proceedings of Third World Conference on Earthquake Engineering, vol. 1, pp. 130–140.

Mononobe, N., Matsuo, H., 1929. On the determination of earth pressures during earthquakes. Proceeding of the World Engineering Congress, vol. 9, pp. 177–185.

Muller, G.U., Whittaker, T.J.T., 1993. An investigation of breaking wave pressures on inclined walls. Ocean Engineering 20 (4), 349–358.

Muller, G.U., Whittaker, T.J.T., 1996. Evaluation of design wave impact pressures. Journal of Waterway, Port, Coastal and Ocean Engineering, ASCE 122 (1), 55–58.

Nadim, F., Whitman, R.V., 1983. Seismically induced movements of retaining walls. Journal of Geotechnical Engineering, ASCE 109 (7), 915–931.

Nimbalkar, S., Choudhury, D., 2007. Sliding stability and seismic design of retaining wall by pseudo-dynamic method for passive case. Soil Dynamics and Earthquake Engineering 27, 497–505.

Nozu, A., Ichii, K., Sugano, T., 2004. Seismic design of port structures. Journal of Japan Association for Earthquake Engineering 4 (3), 195–208 (special issue).

Okabe, S., 1924. General theory of earth pressure and seismic stability of retaining wall and dam. Journal of the Japanese Society of Civil Engineers 10 (5), 1277–1323.

Ramsden, J.D., 1996. Forces on a vertical wall due to long waves, bores, and dry-bed surges. Journal of Waterway, Port, Coastal and Ocean Engineering, ASCE 122 (3), 134–141.

Richards Jr., R., Elms, D.G., 1979. Seismic behavior of gravity retaining walls. Journal of Geotechnical Engineering, ASCE 105 (4), 449–469.

Richards Jr., R., Elms, D.G., Budhu, M., 1990. Dynamic fluidization of soils. Journal of Geotechnical Engineering, ASCE 116 (5), 740–759.

Scott, R.F., 1973. Earthquake-induced pressures on retaining walls. Proceedings of Fifth World Conference on Earthquake Engineering, International Association of Earthquake Engineering, Tokyo, Japan, vol. II, pp. 1611–1620.

Seed, H.B., Whitman, R.V., 1970. Design of earth retaining structures for dynamic loads. ASCE Speciality Conference on Lateral Stresses in the Ground and Design of Earth Retaining Structures, pp. 103–147.

Veletsos, A.S., Younan, A.H., 1994. Dynamic soil pressures on rigid vertical walls. Earthquake Engineering & Structural Dynamics 23 (3), 275–301.

Westergaard, H.M., 1933. Water pressures on dams during earthquakes. Transactions, ASCE 98, 418–433.

Wu, G., Finn, W.D., 1999. Seismic lateral pressures for design of rigid walls. Canadian Geotechnical Journal 36, 509–522.