Stability of waterfront retaining walls in seismic conditions

Stabilité de parois de quais du type a caisson dans conditions sismiques

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ABSTRACT

This paper analyses the stability of rigid waterfront structures, as caisson quay walls, by using the traditional pseudo-static method. The current procedures to calculate the forces acting on such structures under seismic condition are discussed. The case of partially submerged backfill is particularly focused and a new approach is proposed. Referring to stability against a sliding mode of failure of a typical waterfront wall, a comparison among different methods of analysis is presented and the effects of water to wall height ratio, excess pore water pressure and horizontal seismic coefficient are also highlighted.

RÉSUMÉ

Cet article traite avec la stabilité de structures rigides de soutènement réalisées au bord de mer, comme quais du type à caisson, en utilisant la traditionnelle méthode pseudo-statique. Les procédures actuelles pour calculer les forces agissant sur de ces ouvrages au cours du séisme sont discutées; en particulier le cas de sol de remblayage partiellement submergé est examiné et une nouvelle approche analytique est proposée. Enfin le coefficient de sécurité contre le glissement du mur est calculé en utilisant différentes méthodes et les effets de niveau d'eau, pressions interstitielles excessives et coefficient sismique horizontal sont analysés.

Keywords : Quay walls, earthquake, seismic design, inertial forces, dynamic thrust

1 INTRODUCTION

Recent failures of waterfront retaining walls under seismic conditions highlighted the importance of a proper design of these structures. It is now widely acknowledged that the port facilities should be carefully designed to guarantee their survival during a strong earthquake. Significant efforts have been made during the past decades to develop rational methods and guidelines for the analysis and design of port structures as caisson quay walls (Ebeling & Morrison, 1992; PIANC, 2001; Kim et al., 2004; Kim et al., 2005; Choudhury & Ahmad, 2007; Dakoulas & Gazetas, 2008; among others).

The methods for analysis of retaining structures can be subdivided into three categories (PIANC, 2001): (a) simplified methods, based on the conventional pseudo-static approach, (b) simplified dynamic methods, including those based on the Newmark sliding block concept and (c) dynamic methods. Although the dynamic analyses can be considered the most complete tool available to predict seismic response of a geotechnical system, they require specific knowledge of earthquake geotechnical engineering. Moreover, dynamic analyses require not only much effort and time but also proper values for the various input parameters, which are difficult to obtain.

Therefore in engineering practice most of designs are still based on a pseudo-static approach that has been proved to be quite realistic in many cases. Despite the considerable simplification of their complex actual behavior, gravity walls designed by the traditional approach have generally performed quite well in earthquakes (Kramer, 1996).

For waterfront structures a correct evaluation of forces acting on both sides of wall is fundamental. Many studies were devoted to evaluate earth pressure or dynamic water pressure during earthquake shaking, but a very few literature analyzed the stability of waterfront retaining walls under the combined action of both earth and water pressure under seismic conditions. Moreover, the evaluation of seismic earth thrust is generally addressed with reference to dry soil (Okabe, 1926; Mononobe & Matsuo, 1929) or fully submerged soil (Matsusawa et al., 1985; Eurocode 8). For partially submerged soil, which is the most frequent condition of waterfront retaining walls, the analysis is more complicated and some studies available in the literature present misleading results, as discussed by Bellezza & Fentini (2008).

In this paper current procedures to compute pseudo-static forces acting on waterfront structures are discussed, with emphasis to a partially submerged backfill. A comparison among different approaches, including the trial wedge method, is also presented, referring to the stability against sliding of a typical caisson quay wall.

2 EVALUATION OF FORCES

A typical waterfront retaining wall with vertical face, width '*b*', height '*H*' and unit weight γ_c is shown in Fig. 1. It retains backfill to its full height on the land-ward side and water to a height '*h*' on the sea-ward side. The ground surface of the backfill is assumed to be horizontal and it is submerged to same level '*h*'.

Basically, the wall is subjected to three kinds of forces: the seismic earth force, the forces due to water and the inertial forces of the wall.

2.1 *Seismic earth thrust*

The seismic active earth thrust (static plus dynamic), P_{AE}, on the wall is calculated using the pseudo-static Mononobe-Okabe approach, as modified by Matsusawa et al. (1985) and Ebeling & Morrison (1992) to account for full or partial submergence of the backfill:

$$P_{AE} = 0.5 K_{AE} \gamma^{*} (1 \pm k_v) H^2 \tag{1}$$

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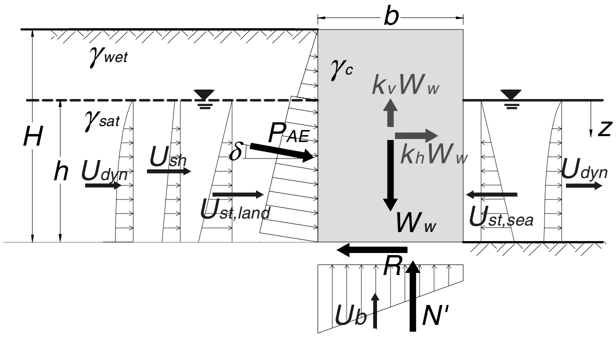

Figure 1. Forces acting on a typical waterfront retaining wall.

where k_v is the vertical seismic acceleration coefficient, K_{AE} is the seismic active earth pressure coefficient and γ^* is the equivalent unit weight of the backfill.

For a wall with vertical face and horizontal backfill (Fig.1), K_{AE} is given by :

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

with ϕ = soil shear resistance angle; δ = backfill-wall friction angle and ψ = seismic inertia angle.

According to Eurocode 8, which agrees with the study of Matsusawa et al. (1985), the value of γ^* in eq. (1) depends on the water table position within the backfill; in particular, for the soil above the water level γ^* coincides with the wet unit weight ($\gamma^* = \gamma_{wet}$) whereas for the soil below the water table γ^* is the submerged unit weight ($\gamma^* = \gamma_b = \gamma_{sat} - \gamma_w$). For partially submerged soil Eurocode 8 does not indicate how to select γ^*; in such a case the seismic soil thrust can be calculated starting from the active soil pressures acting above (K_{AE1}) and below (K_{AE2}) the water table, obtained by the Rankine theory.

For partially submerged backfill, PIANC (2001) suggests to calculate γ^* as an *average unit weight* based on the volumes of soil within the active wedge that are below and above the water table:

$$\gamma^* = \gamma_b\lambda^2 + \gamma_{wet}\left(1-\lambda^2\right) \tag{3}$$

where $\lambda = h/H$ (see Fig. 1).

In the Ebeling & Morrison approach (Ebeling & Morrison, 1992) the excess of pore water pressure due to shaking is also included:

$$\gamma^* = \gamma_b\lambda^2(1-r_u) + \gamma_{wet}\left(1-\lambda^2\right) \tag{4}$$

where r_u is the ratio between the excess of pore water pressure Δu, to the initial vertical effective stress, σ'_{vo} ($r_u = \Delta u/\sigma'_{vo}$). Although r_u can actually depend on depth, Ebeling & Morrison (1992) assume r_u constant throughout the submerged portion of the backfill.

The seismic inertia angle ψ in eq. (2) depends on the permeability (k) of the backfill. For $k < 5\cdot10^{-4}$ m/s (Eurocode 8) the water moves with the solid skeleton (*restrained water case*) and Ebeling & Morrison (1992) calculate ψ as:

$$\psi_{restr} = tan^{-1}\left(\frac{\gamma_{sat}}{\gamma_b\lambda^2(1-r_u)+\gamma_{wet}\left(1-\lambda^2\right)}\frac{k_h}{1\pm k_v}\right) \tag{5}$$

where k_h is horizontal seismic acceleration coefficient.

PIANC (2001) suggests a modified expression, which neglects the excess pore water pressure and vertical acceleration:

$$\psi_{restr} = tan^{-1}\left(\frac{\gamma_{sat}\lambda^2+\gamma_{wet}\left(1-\lambda^2\right)}{\gamma_b\lambda^2+\gamma_{wet}\left(1-\lambda^2\right)}k_h\right) \tag{6}$$

Including r_u and k_v in (6) a revised expression of the seismic angle ψ can be written:

$$\psi_{restr} = tan^{-1}\left(\frac{\gamma_{sat}\lambda^2+\gamma_{wet}\left(1-\lambda^2\right)}{\gamma_b\lambda^2(1-r_u)+\gamma_{wet}\left(1-\lambda^2\right)}\frac{k_h}{1\pm k_v}\right) \tag{7}$$

Eq. (7) combined with eq. (4) can be viewed as a new method to calculate the seismic soil thrust for partially submerged backfill.

According to Eurocode 8 it is necessary to distinguish the seismic inertia angle above the water table [$\psi = \tan^{-1}(k_h/(1\pm k_v))$] from that below the water table obtained by assuming $\lambda = 1$ and $r_u = 0$ in eq. (5) or eq. (7).

For highly permeable backfill soils [Matsusawa et al. (1985) indicate $k > 10^{-2}$ m/s], pore water can move independently of the solid skeleton (*free water condition*). In such a case, the horizontal inertial force of soil is assumed to be proportional to the dry unit weight (γ_d), whereas the vertical component is assumed to be related to submerged soil unit weight, γ_b. Therefore eqs. (5)-(7) can be used by substituting γ_{sat} with γ_d (Matsusawa et al. 1985; Ebeling & Morrison 1992, Eurocode 8, PIANC, 2001).

2.2 *Water thrust*

In the general case, the force due to water can be distinguished in three different forces: the hydrostatic force (U_{st}), the force due to excess pore water pressure (U_{sh}) and the hydrodynamic force (U_{dyn}).

The hydrostatic force acts both at the landward ($U_{st,land}$) and the seaward side ($U_{st,sea}$) at a height of $h/3$ from the base of the wall (Fig. 1):

$$U_{st,land} = 0.5\gamma_w h^2 = 0.5\gamma_w\lambda^2 H^2 \tag{8}$$

The Ebeling & Morrison approach considers the force due to excess water pressure as the resultant of a trapezoidal pressure distribution acting on the backfill (Fig. 1):

$$U_{sh} = 0.5\left[2\gamma_{wet}(H-h)+\gamma_b h\right]r_u h \tag{9}$$

For a fully submerged backfill ($\lambda = 1$) the sum of U_{st} and U_{sh} is equivalent to a hydrostatic force calculated with an increased water unit weight $\gamma_{we} = \gamma_w + r_u\gamma_b$ (Kramer, 1996).

Finally, the hydrodynamic force is usually calculated according to Westergaard's approach (Westergaard, 1933):

$$U_{dyn} = \pm\int_{z=0}^{z=h}\frac{7}{8}k_h\gamma_w\sqrt{hz}\,dz = \pm\frac{7}{12}k_h\gamma_w h^2 \tag{10}$$

For a waterfront structure this force, included in all approaches, is assumed to act always at the sea-ward side in a direction opposite to the direction of the hydrostatic force ($U_{st,sea}$), whereas at the land-ward side it should be considered only for the *free water case* in the same direction of the hydrostatic force (Fig. 1). The point of application of U_{dyn} is at $0.4h$ above the base of the wall. It is worth to note that the procedure followed in the *free water case* is not totally consistent since the effect of the increased pore pressures due to the dynamic water pressure is neglected in the computation of the thrust due to soil skeleton (Ebeling & Morrison 1992; PIANC, 2001).

2.3 *Trial wedge method*

For a partially submerged backfill the active soil thrust P_{AE} can be obtained by imposing vertical and horizontal equilibrium of a soil wedge inclined at angle α to the horizontal. In Figure 2 the forces acting on the wedge for the *restrained water condition* is shown. Using a Mohr Coulomb failure criteria along the planar slip surface ($T = N'\tan\phi$), the seismic soil thrust is the maximum value of the following expression:

$$P_{AE}(\alpha)=\frac{\left\{\begin{array}{l}[W-F_V-(U_{st}+U_{sh})cot\,\alpha](sin\,\alpha-tan\,\phi\,cos\,\alpha)+\\+F_H(cos\,\alpha+tan\,\phi\,sin\,\alpha)\end{array}\right\}}{cos\,\delta(cos\,\alpha+tan\,\phi\,sin\,\alpha)+sin\,\delta(sin\,\alpha-tan\,\phi\,cos\,\alpha)} \quad (11)$$

According to Matsusawa et al. (1985), for the *restrained water case*, the wedge horizontal inertia force F_H is proportional to the *total* weight of wedge [$F_H = k_h\,(V_{sub}\gamma_{sat} + (V_{tot} - V_{sub})\,\gamma_{wet})$], whereas the wedge vertical inertia force F_V is assumed to depend on γ_b [$F_V = k_v\,(V_{sub}\gamma_b + (V_{tot} - V_{sub})\gamma_{wet})$].

The values of P_{AE} obtained by the trial wedge method are compared in Table 1 with those obtained by other procedures for a typical set of backfill parameters. It can be noted that the present approach, represented by eqs. (4) and (7), is in good agreement with the trial wedge method; the two approaches coincide for $r_u = 0$. On the other hand, Eurocode 8 and PIANC procedures overestimate the seismic active thrust, even for $r_u = 0$.

Table 1. Values of P_{AE} for H = 10 m; h = 8 m; ϕ = 36°; δ = 24°; γ_{wet} = 18 kN/m^3; γ_{sat} =19 kN/m^3; γ_w =10 kN/m^3; k_h = 0.15; k_v = 0.075.

approach	P_{AE} (kN/m)		notes
	$r_u = 0$	$r_u = 0.2$	
Ebeling & Morrison (1992)	237.5	228.9	eqs. (4) & (5)
PIANC (2001)	243.1	243.1	eqs. (3) & (6)
Eurocode 8	289.3	289.3	$r_u = 0$; $k_h \neq 0$
Trial wedge method	234.9	217.4	eq.(11)
Present approach	234.9	226.1	eqs.(4) & (7)

2.4 *Inertial force of the wall*

As far as inertial forces of the wall are concerned, they are proportional to the total weight of the wall, W_w. Specifically, the horizontal component, k_hW_w, acts in the direction of seismic soil thrust P_{AE}, while the vertical component, k_vW_w, is directed upward (Fig. 1). The seismic coefficients are assumed to be the same acting on the active wedge, with k_v taken as one half of k_h (Eurocode 8).

3 GLOBAL FACTOR OF SAFETY AGAINST SLIDING

The global factor of safety against sliding is defined as the ratio between the resisting force to the driving force. For a waterfront structure the following expression is generally accepted (Ebeling & Morrison, 1992, Bellezza & Fentini, 2008):

$$F_s=\frac{\tan\delta_b\left[W_w(1\pm k_v)+P_{AE}\sin\delta-U_b\right]}{P_{AE}\cos\delta+U_{st,land}+U_{sh}+\chi U_{dyn}-U_{st,sea}+W_wk_h} \quad (12)$$

where δ_b is the friction angle at the bottom of the wall, U_b is the resultant of pore pressure acting along the base of the wall, χ is a numerical coefficient (χ = 1 for the *restrained pore water case* and χ = 2 for the *free pore water case*). In using eq. (12) it is implicitly assumed that soil and wall inertial forces peak simultaneously. Moreover, in the hypothesis that the water level is the same in the backfill and outboard of the wall, eq. (12) is simpler because $U_{st,land} = U_{st,sea}$.

The force U_b depends on the pore pressure distribution along the base. Assuming a linear pressure diagram (PIANC, 2001); U_b can be calculated on the basis on the pore pressure acting at the two edges of the base:

$$U_b=0.5b\left\lfloor u_{st,land}+u_{sh,land}+u_{dyn,land}(\chi-1)+u_{st,sea}-u_{dyn,sea}\right\rfloor \quad (13)$$

According to Ebeling & Morrison (1992) and PIANC (2001), the effect of hydrodynamic pressure is here neglected. This assumption is on the safe side for the *restrained water condition* and neutral for the *free water condition* (if the water level is the same on both sides).

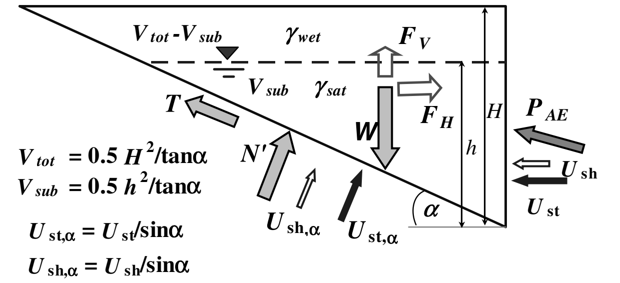

Figure 2. Seismic active wedge.

4 RESULTS AND DISCUSSION

Referring to a typical waterfront retaining wall under seismic conditions, the stability against sliding is evaluated in the following example according to Eurocode 8, Ebeling & Morrison (1992), PIANC (2001), the trial wedge method and the present approach.

The base parameters considered in the example are: λ = 0.8; b/H = 0.9; ϕ = 36°; δ = 24°;δ_b = 31°; γ_c/γ_w = 2.2; γ_{wet}/γ_w = 1.8; γ_{sat}/γ_w = 1.9; k_h = 0.15; k_v = k_h/2; r_u = 0.2. The water in the backfill is assumed to be in the *restrained condition*.

4.1 *Effect of water level*

Figure 3 shows the factor of safety obtained by varying the level of submergence of the backfill, all other parameters being equal. It can be observed that the fully submerged backfill (λ = 1) represents the most critical condition in terms of sliding stability. The comparison among the different approaches shows that the proposed approach, the Ebeling & Morrison approach and the trial wedge method give practically coincident results. On the other hand PIANC approach is on the unsafe side, mainly because it neglects the excess pore water pressures and vertical seismic acceleration. Also the approach based on Eurocode 8 results in a overestimate of the factor of safety of about 20%. For the analyzed case, the waterfront wall is stable according to PIANC and Eurocode 8 procedure, while the other procedures clearly indicate that it fails for λ > 0.76÷0.77.

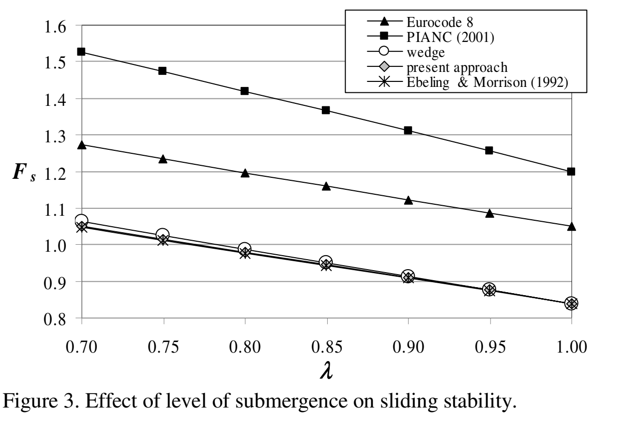

Figure 3. Effect of level of submergence on sliding stability.

4.2 *Effect of excess pore water pressure*

The factor of safety against sliding is plotted in Figure 4 by varying the excess pore pressure ratio r_u. It can be observed that factor of safety calculated by Eurocode 8 or PIANC does not change with r_u, because these procedures neglect the excess water pressure (i.e. r_u = 0).

According to the approaches that include the effect of excess pore water pressure, the stability of the wall decreases as r_u increases. Specifically, at increasing r_u the seismic soil thrust P_{AE} slightly decreases, while U_{sh} and U_b increase. The combined effect of these variations results in a decrease of the global factor of safety.

The results showed in Fig.4 highlight the importance of a proper selection of the r_u value, which should be estimated on the basis of several factors, including the properties of the submerged soil, the maximum amplitude of ground acceleration as well as the magnitude of the design earthquake (Kim et al. 2005).

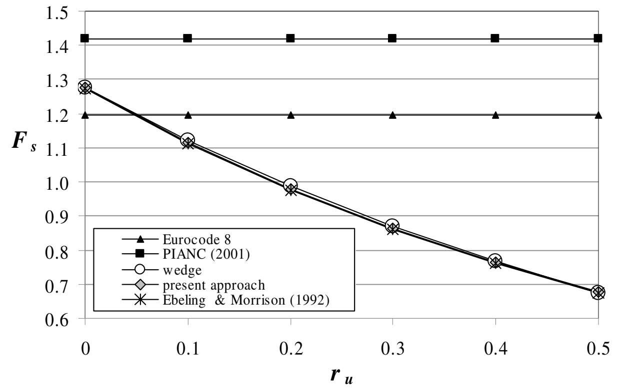

Figure 4. Effect of excess pore water ratio r_u on sliding stability.

4.3 *Effect of the horizontal seismic coefficient*

The factor of safety is plotted in Figure 5 as a function of the horizontal seismic coefficient k_h, which is directly proportional to the peak ground acceleration a_{max}, although the correlation is not the same for all procedures (PIANC, 2001; Eurocode 8; Nozu et al., 2004).

For an earthquake with a given magnitude the excess pore water pressures within the backfill depend on the a_{max}; hence r_u is expected to increase with k_h. In order to eliminate the effect of the excess pore water pressures, the comparison among different procedures is made in Fig. 5 assuming $r_u = 0$, regardless of the k_h value. In such a way all methods neglect the excess pore water pressures in the backfill.

As expected, F_s decreases as k_h increases, mainly because of the increased horizontal inertial forces on both active wedge and wall. Similarly to Figs. 3 and 4, the procedure suggested by PIANC (2001) overestimates the factor of safety; conversely, the Eurocode 8 is on the safe side. It can be observed that the trend of the factor of safety calculated according to Eurocode 8 slightly changes at k_h equal to about 0.3. This is due to fact that for k_h greater than ≈0.3 the seismic angle ψ relevant to the submerged part of the backfill exceeds ϕ, so that eq. (2) is not applicable and the active pressures must be calculated using an alternative expression suggested by Eurocode 8. The other procedures used in this study do not present numerical problems.

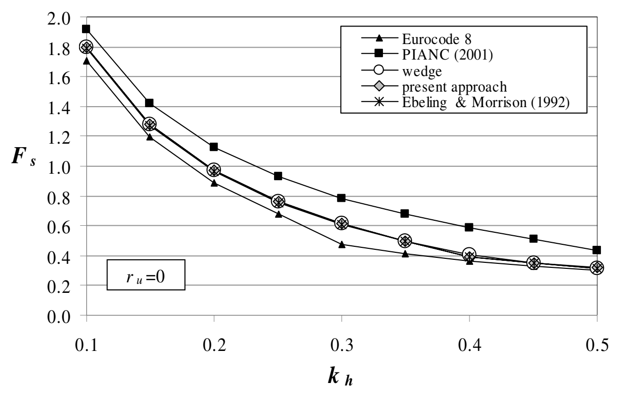

Figure 5. Effect of k_h on sliding stability.

5 CONCLUSIONS

Current procedures to analyze rigid waterfront retaining walls in the presence of partially submerged backfill have been discussed.

A new method to calculate the soil seismic thrust has been proposed that accounts for vertical seismic acceleration and excess pore water pressure. By this method the seismic soil thrust is in good agreement with those obtained by the trial wedge method and by Ebeling & Morrison (1992).

Referring to the sliding stability of a typical waterfront retaining wall a parametric study has been carried out. The results indicate that the fully submerged backfill represents the most critical condition in terms of sliding stability, regardless of the method of analysis.

As far as the comparison among different procedures is concerned, the global factors of safety calculated by the proposed approach practically coincide with those obtained by the approach of Ebeling & Morrison (1992) and the trial wedge method.

For a typical set of backfill parameters and a given value of the horizontal seismic coefficient k_h the procedure suggested by PIANC (2001) always overestimates the factor of safety. The procedure based on the Annex E of Eurocode 8 is on the safe side whenever it is assumed that shaking causes no associated buildup of excess pore water pressure in the backfill ($r_u = 0$). Conversely, Eurocode 8 can result in unsafe design in the presence of excess pore water pressures generated by cyclic shaking. In such a case the value of r_u is found to strongly affect the sliding stability of the structure and its proper selection must be considered as a key step in the analysis.

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