Discussion: Seismic design factor for sliding of waterfront retaining wall

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The paper by Ahmad and Choudhury (2009) presents a parametric study to analyse the stability of waterfront retaining walls under seismic conditions using a pseudodynamic approach.

The approach proposed by the authors seems to be too approximated and hybrid. The soil thrust P_{ae} of a partially submerged backfill is calculated using the same pseudodynamic approach as proposed for dry backfill (by substituting only the soil unit weight). The wall inertia forces (q_h and q_v) are also evaluated by a pseudodynamic approach. However, the authors assume that the seismic active soil thrust and the seismic wall inertia forces peak simultaneously, whereas P_{ae} , q_h and q_v reach their maximum values at different times. On the other hand, the dynamic water thrust is calculated by a pseudostatic approach, neglecting its variation over time.

Besides these general drawbacks, the paper contains further inaccuracies, as detailed in the following.

- (a) Equation 3 is valid only for a completely saturated backfill (i.e. $h_{wd}/H = 1$). When the backfill is *partially* submerged ($h_{wd} < H$), P_{std} can be subdivided into a static component (equal to $0.5\gamma_w h_{wd}^2$) and a component depending on the coefficient r_u , called U_{shear} by Ebeling and Morrison (1992). According to the definition of r_u , U_{shear} is calculated as the resultant of a trapezoidal distribution, given by

$$22. \quad U_{shear} = 0.5[2\gamma_d(H - h_{wd}) + \gamma_b h_{wd}] h_{wd} r_u$$

- (b) Equation 6 does not correspond to the equation proposed by Choudhury and Nimbalkar (2006). In particular, the term containing r_v should be negative, and $\cos(\alpha - \phi)$ must be replaced by $\sin(\alpha - \phi)$.
- (c) Equation 9 is clearly wrong. If the backfill is completely saturated (i.e. $h_{wd}/H = 1$), using Equation 9 and Equation 3

with $r_u = 0$, the soil thrust would be proportional to γ_{sat} , which represents an absurdity in soil mechanics. In the correct expression, reported for example by Ebeling and Morrison (1992) or Dakoulas and Gazetas (2008), γ_{sat} must be replaced by γ_{sub} .

$$23. \quad \bar{\gamma} = \left(\frac{h_{wd}}{H}\right)^2 \gamma_{sub} + \left[1 - \left(\frac{h_{wd}}{H}\right)^2\right] \gamma_d$$

When the pore pressure ratio r_u is considered, it acts only in a submerged part of the backfill, and Equation 23 becomes (Ebeling and Morrison, 1992; Kramer, 1996)

$$24. \quad \bar{\gamma} = \left(\frac{h_{wd}}{H}\right)^2 \gamma_{sub}(1 - r_u) + \left[1 - \left(\frac{h_{wd}}{H}\right)^2\right] \gamma_d$$

Actually, Kramer (1996) also indicated the same expression as proposed by the authors, but it is probably a printing error, because Kramer himself, in numerical example 11.3 of his book (p. 488), used Equation 24, not Equation 9.

- (d) Equation 5 is not correct. As the pore pressure ratio is inserted in the average unit weight $\bar{\gamma}$ (Equation 24), Equation 5 should be written without the term $(1 - r_u)$, to give

$$5b. \quad P_{ae} = 0.5 K_{ac} \bar{\gamma} H^2$$

Equation 5, as proposed by the authors, is valid only for *completely saturated* backfill, provided that $\bar{\gamma} = \gamma_{sub}$ (see item (c)).

- (e) The main drawback of the paper is that the authors neglect the resultant of pore water pressure U_b along the base of the wall; this assumption is wrong in both static and dynamic conditions, and it results in a very significant overestimation of the resisting force against sliding. Equation 16 is valid

only for *dry backfill*, and this is surely not the case for a waterfront structure. The force U_b can be calculated assuming a linear distribution of the pore water pressure along the base of the wall, from

$$25. \quad U_b = 0.5(u_{\text{land}} + u_{\text{sea}})b$$

where u_{land} is the pore water pressure at the base of the wall on the landward side; u_{sea} is the pore water pressure at the base of the wall on the seaward side; and b is the width of the wall.

Even in the static case ($k_h = r_u = 0$) and for $h_{\text{wd}} = h_{\text{wu}} = h$, U_b is not zero, but $U_b = \gamma_w h b$ (i.e. U_b coincides with the buoyancy).

- (f) Many of the figures in the paper are obtained by assuming that the water level on the landward side is different from that on the seaward side. In such a case the actual distribution of pore water pressures over depth is not hydrostatic. The authors do not explain why this aspect is neglected in the paper.
- (g) The friction angle at the base of the wall, δ_b , is assumed to be equal to the soil strength resistance angle of the backfill (ϕ). In design it is generally assumed that $\delta_b < \phi$ (e.g. Ebeling and Morrison, 1992). Therefore the assumption made by the authors is on the unsafe side.
- (h) It is not clear which soil unit weight γ must be inserted in the denominator of Equation 19 (γ_d , γ_{sub} , γ_{sat} , $\bar{\gamma}$, ...).
- (i) In Figures 2–7 the authors select a constant value of r_u ($= 0.2$) for values of the seismic acceleration coefficient k_h varying in the range 0–0.4. In this way it is implicitly assumed that r_u is independent of k_h (i.e. a_{max}) and this clearly represents an oversimplification of the actual behaviour. Moreover, the authors provide no indications on how to select a proper value of r_u in practice. Finally, it is considered that $r_u = 0.2$ even when $k_h = 0$; obviously, this assumption ($k_h = 0$ and $r_u = 0.2$) does not represent the static condition; it is not clear if the authors refer to a post-earthquake condition, when residual excess pore pressures can exist also in the absence of earthquake forces.

In order to highlight the implications of the above inaccuracies for the results presented in the paper, we calculated the value of F_{Wwet} for $H = 10$ m, $h_{\text{wd}}/H = h_{\text{wu}}/H = 0.75$, $\phi = \delta_b = 30^\circ$, $\delta = 15^\circ$, $k_h = 0.2$, $k_v = 0.1$, $r_u = 0.2$, $V_s = 100$ m/s, $V_p = 187$ m/s, $V_{\text{sw}} = 2800$ m/s, $V_{\text{pw}} = 4300$ m/s, $\gamma_c = 24$ kN/m³, $\gamma_d = 16$ kN/m³, $\gamma_{\text{sat}} = 19$ kN/m³ and $\gamma_w = 10$ kN/m³. Using Equations 5b, 22, 24 and 25, and assuming that the backfill and the wall peak simultaneously, the value of F_{Wwet} is 10.06, whereas the authors indicated $F_{\text{Wwet}} = 4.27$.

Authors' reply

The authors thank the discussers for showing a keen interest in the published work, and going through it in detail to report some

interesting observations. The authors would like to address the discussers' concerns thoroughly. However, at the outset, the authors do not agree with the very first concern of the discussers, that the original work is an approximation and hybrid. There are some approximations that have been made in the study presented by the authors, but these were actually required to be made to provide simplifications in the approach, especially as it is required to handle such a complex problem as the seismic stability of waterfront retaining structures. The authors have presented a new approach, and probably have been successful in showing a new research area of application of the pseudodynamic approach over the conventional and more simplified pseudostatic approach for analysing waterfront retaining structures.

In view of the above statement, the authors would refrain from using the word 'approximation', for in their opinion they are actually 'simplifications', and offer scope for future research on this recent topic. The first simplification is the use of existing seismic earth pressure theory for the wet backfill case. The authors are aware that by merely considering the submerged backfill unit weight (instead of the dry unit weight) in the expression for calculating the seismic earth pressure (with any approach, whether pseudostatic or pseudodynamic), one would not obtain an exact replication of the field scenario, and the value so calculated will differ. This difference occurs because of the seismic acceleration coefficients in the horizontal and vertical directions (k_h and k_v) and the soil properties, which are affected by the presence of water. However, in the absence of any relevant comprehensive study that covers this aspect in detail, the authors resorted to the conventional method, and replaced the dry unit weight with the submerged unit weight of the backfill. At this stage, the authors would like to make the point that the pseudodynamic method (for calculation of the seismic earth pressure) is basically an extension of the seismic coefficient method. Although the seismic coefficient method fails in truly replicating the dynamic interaction of the soil, water and waterfront retaining wall, it is quite widely accepted as a semi-empirical method for the analysis and design of waterfront retaining structures (US Army Corps of Engineers, 1995). Another point that was stressed in the original paper, and is again highlighted here, is the fact that the approach adopted by the authors is that of considering worst-case scenarios with respect to loads and their points of applications. With this in view, it has been considered that the seismic earth pressure (P_{se}) and the seismic inertia forces q_h and q_v act simultaneously; this may be slightly unrealistic, but is not a totally unlikely behaviour. The hydrodynamic force has been calculated using the pseudostatic approach, which to date is the most widely used approach, perhaps because of its straightforwardness and simplicity.

In the study proposed by the authors, the hydrostatic force (P_{std}) for the partially submerged backfill case has not been broken into a static component and a component involving the pore pressure ratio r_u (Equation 3). If one goes by the yardstick of being extremely strict, then this simplification may be wrong, but if one

adopts a fairly practical approach, then in view of the authors this simplification does not compromise the integrity of the proposed methodology. This is because of the manner in which the hydrostatic force has been calculated, by using the equivalent unit weight of water (i.e. γ_{we}) and not the unit weight of water (i.e. γ_w). A close look at Equation 4 reveals that γ_{we} is always greater than γ_w (except for the case when the pore pressure ratio, i.e. r_u , is considered to be zero, which is never the case in the analysis presented by the authors). Further, if a detailed computation of the hydrostatic force (P_{std}) is made independently by considering the simplified approach presented by the authors, and that in which the component due to the pore pressure ratio is considered, one would observe that there is very little difference between the two values. The same has been detailed in Table 2; column 3 shows the values of the hydrostatic force calculated by using the approach in which both the static component and the component involving the pore pressure ratio have been considered, that is

$$P_{std} = \frac{1}{2} \gamma_w (h_{wd})^2 + 0.5 [2\gamma_d (H - h_{wd}) + (\gamma_{sat} - \gamma_w) h_{wd}] h_{wd} r_u$$

Equation 6 does not seem to contain any errors – neither in concept, nor in typing. It is correct. However, for the case of submerged backfill, the expression for the computation of the equivalent unit weight has to be the same, as has been mentioned by the discussers. This aspect was overlooked in the study presented by the authors. The authors, though, would not be in agreement with the discussers that the expression for the computation of the seismic active earth pressure also needs to be modified. This will contain the term $(1 - r_u)$, in line with the approach presented in previous works (e.g. Ebeling and Morrison, 1992; Choudhury and Ahmad, 2008; Ahmad and Choudhury, 2010).

At the time of carrying out the work presented by the authors, a simplification with regard to the non-consideration of the pore pressure force at the base of the foundation was made, and this can be considered to be a limitation of their study. However, the authors are currently engaged in considering these forces along the base of the wall, and are attempting to modify their pseudodynamic approach, as well by considering refraction analysis, which will make the methodology more realistic, and closer to the exact field conditions. The base has been considered to be quite frictional, so that full mobilisation of the resisting force at the base of the waterfront retaining wall is achieved (similar to the work done by the US Army Corps of Engineers, 1995). This is actually a debatable topic for future research, with consideration of a particular value that is suitable for use as the coefficient of base friction, and it depends on an individual's choice and engineering judgement. Further, the authors would like the discussers to look again at Equation 14, in which the coefficient

h_{wd}/H (1)	P_{std} (by Equation 3): kN/m ² (2)	P_{std} (by Equation 26): kN/m ² (3)
0.00	0.00	0.00
0.75	3.32	3.46
1.00	5.90	5.90

Table 2. Comparison of hydrostatic force P_{std} for $\gamma_w = 10 \text{ kN/m}^3$, $\gamma_d = 16 \text{ kN/m}^3$, $\gamma_{sat} = 19 \text{ kN/m}^3$ and $r_u = 0.2$

of base friction has been taken to be equal to $\tan \phi$, and not simply ϕ .

Equation 19 presents a ratio in terms of the static earth pressure, calculated by using the dry unit weight of the soil. Thus this seems to be a typographical error committed by the authors, and γ shown in Equation 19 should actually be γ_d .

The consideration of pore pressure ratio even for the case when $k_h = 0$ (i.e. no earthquake condition) has been done with the intention of simulating the scenario that is most often observed after a major earthquake has occurred and has impacted on a waterfront retaining structure. Moreover, finding a relevant dependence of the pore pressure ratio on the seismic acceleration coefficients and vice versa is a topic of the authors' current research, and thus for the time being it was considered that these two entities are independent of each other, and any interaction between them was disregarded.

The authors agree with the general theme presented by the discussers, and are in partial agreement with them. But at the same time the authors stress that the topic covered by them was a new one, and like any new scientific study, it also had some initial simplifications. Now, based on the discussion of and response to this new emerging field of research, further detailed study (including but not limited to experiments) needs to be carried out to ascertain the reliability and limits of the proposed methodology.

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