PSEUDO-STATIC ANALYSIS OF DIAPHRAGM WALLS – LIMITATIONS AND ALTERNATIVES BASED ON PRACTICE AND MODELLING

G. Pereira[1], T. Jeanmaire[2]

[1] *Principal Engineer at the Group Design Office, Soletanche Bachy, gustavo.pereira@soletanche-bachy.com*
[2] *Director of the Group Design Office, Soletanche Bachy, thierry.jeanmaire@soletanche-bachy.com*

Abstract

Due to simplicity and ease of use, the pseudo-static Mononobe-Okabe method is included in the current format of Eurocode 8 Part 5, and it is likely that it will remain as a recommended method in the revised version of this standard. This paper reviews the proposed method and its basic assumptions, identifies its limitations, and presents alternative hypothesis. These will focus on aspects such as the passive resistance of retaining walls, the "r" factor for reduction of seismic earth pressures and the incoherence of the seismic action impacting the structure. By means of a case study of an anchored quay wall, the impact of these assumptions on the design will be presented, along with a comparison with the results of a time domain Finite Element Method analysis.

Keywords: Retaining structures; pseudo-static; seismic earth pressure; Eurocode 8; performance-based design

1. Introduction

With the availability of increasingly powerful computers and codes, the use of non-linear time domain analysis in earthquake geotechnical engineering is no longer limited to research applications or to especially complex projects, and it has recently become popular in the design of earth retaining structures, and flexible retaining walls in particular.

Nevertheless, dynamic numerical analysis does require significant time and expertise, and the associated costs are often an obstacle to its use in medium and small scale projects, which are typically designed with simplified pseudo-static methods. Furthermore, pseudo-static methods also prove valuable in the checking and troubleshooting of more sophisticated analysis, and as such remain an important tool for designers.

A pseudo-static approach is included in the current version of Eurocode 8 Part 5 (EN1998-5, 2004), and it is likely that it will remain a recommended feature in the new version of this standard. The current approach is, similarly to several other standards, based on the Mononobe-Okabe method, and its application is relatively straightforward. However, a significant number of hypothesis are not always well defined, leading to a certain variability of practice among the geotechnical community. Furthermore, the standard seems to put retaining walls (i.e. non-gravity retaining structures) at a disadvantage in some aspects. The purpose of this paper is to stimulate discussion and to point to possible ways forward.

2. Review of the Eurocode 8 Part 5 approach

The simplified approach in EN1998-5 (2004) is based on the method established by Okabe (1926) and Mononobe and Matsuo (1929). The method, an extension of Coulomb's limit earth pressure theory, is well known and will not be detailed here. A comprehensive review of Mononobe-Okabe's method can be found in Ebeling & Morrison (1992). In the paragraphs below, aspects of particular interest for the design of diaphragm walls, as well as of other non-gravity walls, will be discussed.

2.1 Alternative assumptions regarding passive resistance

The contribution of passive resistance to the stability of gravity walls is generally small or non-existent. On the contrary, diaphragm walls and other retaining walls rely on passive pressure for both horizontal and rotational equilibrium, and any earthquake-induced reduction of the available passive resistance may have a severe impact on the stability of the structure.

EN1998-5 states, in §7.3.2.3(6)P, that the static and dynamic passive pressures "shall be taken to act with an inclination with respect to a direction normal to the wall […] equal to zero." While this is justifiable when using the Coulomb and, consequently, Mononobe-Okabe methods (when the ground-wall interface angle is significantly different from zero, these methods, based on planar failure wedges, may vastly overestimate passive resistance), other methods are able to simulate the passive wedge curvature and could therefore be used to obtain reasonable estimates.

One alternative procedure to determine dynamic passive resistance coefficients is to simulate the loading with a Limit Equilibrium Method (LEM) software; another alternative is to resort to the Finite Element Method (FEM). Fig. 1 shows, for a submerged frictional material (friction angle equal to 40^o, saturated unit weight equal to 20kN/m^3) and for different assumptions of wall/soil shearing resistance, the results of a FEM analysis for various levels of pseudo-static horizontal acceleration (Pereira et al., 2018).

The plots show a coherent trend of diminishing capacity with increasing acceleration, and it can also be observed that the FEM results show good agreement with the reference cases (namely, the Kerisel and Absi (1990) static results and Eurocode 8's Mononobe-Okabe formula for δ_d=0); both observations seem to validate the results obtained with FEM analysis for larger wall/soil shearing resistance. It is our opinion that alternative methods should not be limited to a zero shearing resistance between the soil and the wall (δ_d=0).

If reasonable methods for estimating dynamic passive resistance are adopted, the authors suggest that wall/soil shearing resistance up to $\delta_d = 2/3\varphi'_d$ is allowed, leading to dynamic K_{pd} which are often more than double of that estimated with Eurocode 8. This has a large impact in the design of diaphragm walls and can lead to significant economies in retaining wall design.

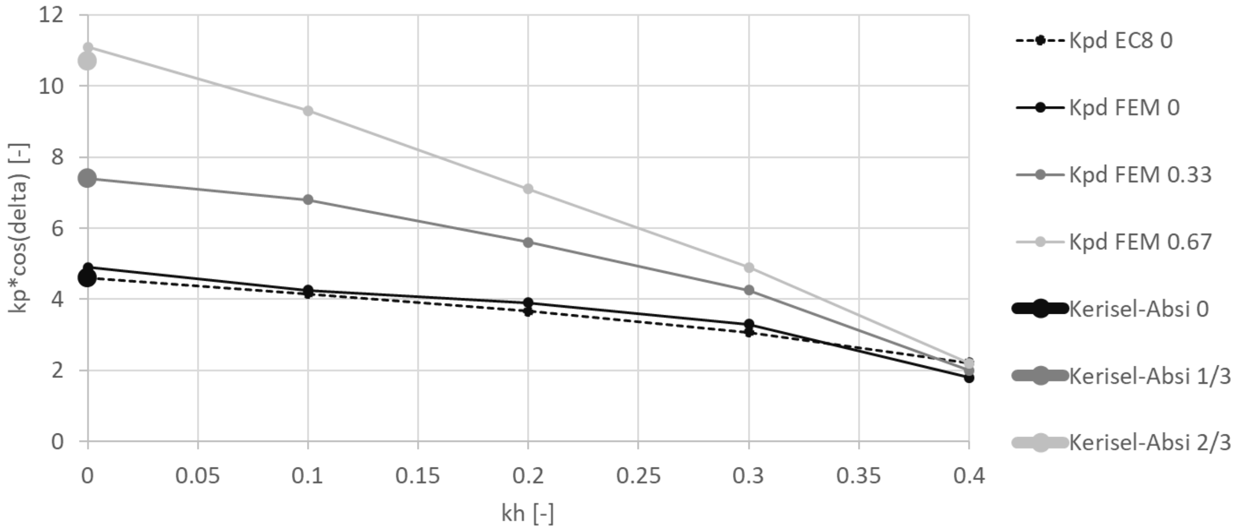

Fig. 1 – Comparison of passive coefficients for different ground/wall interface angles (δ=0°, $\delta=\varphi'/3$ and $\delta=2/3\varphi'$). Case of submerged frictional material (γ_{sat}=20kN/m^3, φ'=40°)

2.2 Evaluation of height-dependent seismic coefficient for walls above 10m in height

The incoherence of ground motions makes that acceleration levels tend to vary both along vertical and horizontal directions; as a result, the representative seismic coefficient for the retaining wall will tend to be lower. EN1998-5 (2004), in §7.3.2.2(4)P, §7.3.2.2(6) and appendix E2, partly addresses this by allowing the averaging of accelerations along the height of walls taller than 10m, if a one-dimensional vertical shear wave propagation analysis is performed. It does not, however, consider incoherence in the horizontal direction, nor does it allow replacing the 1D propagation with other methods.

Nevertheless, several references (NCHRP, 2008, FHWA, 2011 and NTC, 2018) allow significant reduction in seismic coefficients through simplified methods, by considering the spectral characteristics of the ground motion, the wall height and the ground characteristics. For example, the NCHRP method (see left hand side of Fig. 2, below) defines Scaling Factors for three typical United States spectra of varying frequency content. For a 20m height retaining wall, one derives that the Scaling Factor can be as low as 0.4 for the Lower Bound (higher frequency) spectrum, corresponding to a 60% reduction of the seismic coefficient. In the case of the Upper Bound (lower frequency) spectrum, however, the reduction is 15% instead. To estimate the reduction associated to a specific Eurocode 8 spectrum, it suffices to normalize it by Peak Ground Acceleration, and to compare it with the three NCHRP response spectra in order to determine which of these shows a better match (see example in right hand side of Fig. 2, below).

In order to facilitate the design of smaller structures, it is our opinion that §7.3.2.2(4)P should allow the reduction of seismic action of taller walls via simplified procedures such as those described here.

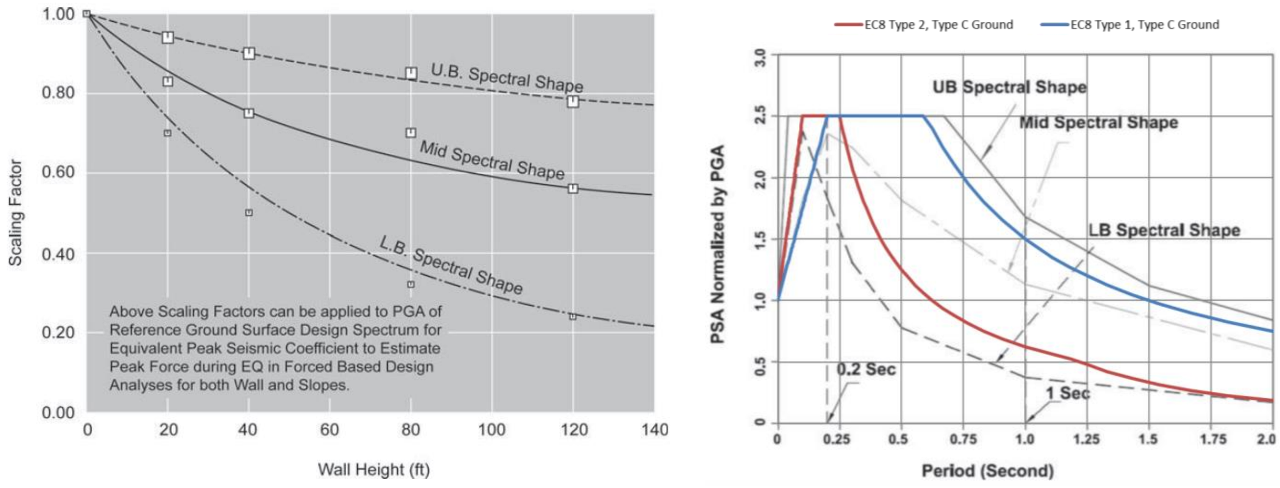

Fig. 2 – Left: spatial incoherence scaling factors for reference response spectra, as per NCHRP (2008). Right: normalized reference spectra, superposed with examples of normalized Eurocode 8 spectra

2.3 Reduction of horizontal seismic coefficients through the acceptance of residual displacements in the retaining structure

If a sufficiently ductile structure is designed for a given acceleration level and said level is exceeded during an earthquake, is widely accepted that the structure will not fail, and will instead accumulate residual displacements every time the input acceleration exceeds the design seismic coefficient. The accumulated residual displacements are usually assessed with methods based on Newmark-type analysis, performed for several representative ground motions and for different values of the k_y/k_{max} ratio (the ratio between the yield acceleration of the structure and the maximum acceleration of the motion).

EN1998-5 (2004) does take into account reductions in seismic action if the structure accepts some degree of displacements, and a reduction factor r is defined in §7.3.2.2(4)P. However, it is explicitly stated that only gravity structures are allowed to use a reduction factor r larger than 1, while flexural reinforced concrete walls and anchored walls, among others, are imposed r=1 (i.e. no reduction in seismic coefficient k_h).

It should be stressed that this puts diaphragm walls, and other non-gravity retaining walls, at a marked disadvantage relatively to gravity walls, especially because the magnitude of residual displacements indicated in EN1998-5 (2004) is generally compatible with the adequate behaviour of diaphragm walls (see Table 1 below).

Table 1 – Displacement limits and reduction factors for gravity walls (adapted from EN1998-5, 2004)

Case	**Limit displ. αS=0.15g* [mm]**	**Limit displ. αS=0.30g* [mm]**	**$r=1/W_d$ [-]**	**$W_d=k_y/k_{max}$ [-]**
d>200αS*	30	60	1.5	0.7
d>300αS*	45	90	2.0	0.5

* αS corresponds to the seismic coefficient k_h (acceleration at surface) and is therefore the same as k_{max}.

Furthermore, these provisions seem to disagree with several international references, which frequently indicate reduction factors in excess of r=3, as long as the ductility of the retaining system is guaranteed. Pereira et al. (2018) present a comparative analysis of the reduction factors according to several references, as well as the equations used to estimate the residual displacements as a function of the k_y/k_{max} ratio. It is our opinion that the residual displacements associated with reduction factors in excess of r=2 are acceptable for many diaphragm walls. As long as the ductility of the retaining wall (and its supports) can be proved, the design standard should not limit the reduction factor to r=1.

3. Case study

3.1 General characteristics

The case study represents a quay wall in a moderately seismic area. The wall, destined to the servicing of very large container ships, has a retaining height of approximately 20m. Ground conditions consist in vibrocompacted, quarried, granular backfill, sitting on top of silts and silty sands. These materials overlay the bedrock which consists of extremely compact gravel sediments, frequently cemented.

The design was based on the static performance criteria, which led to a 1200mm thick diaphragm wall embedded at a depth of 29m. The wall is stabilized by a single layer of passive anchors, consisting each of a Ø110mm steel tie rod, connected to a reaction steel tubular pile. The reference seismic acceleration for type A ground is a_g=0.11g (ULS conditions for a 475 years return period). Variable loads during seismic condition consist of a 20kPa surcharge on the apron. Fig. 3 below schematically represents the main components of the quay wall.

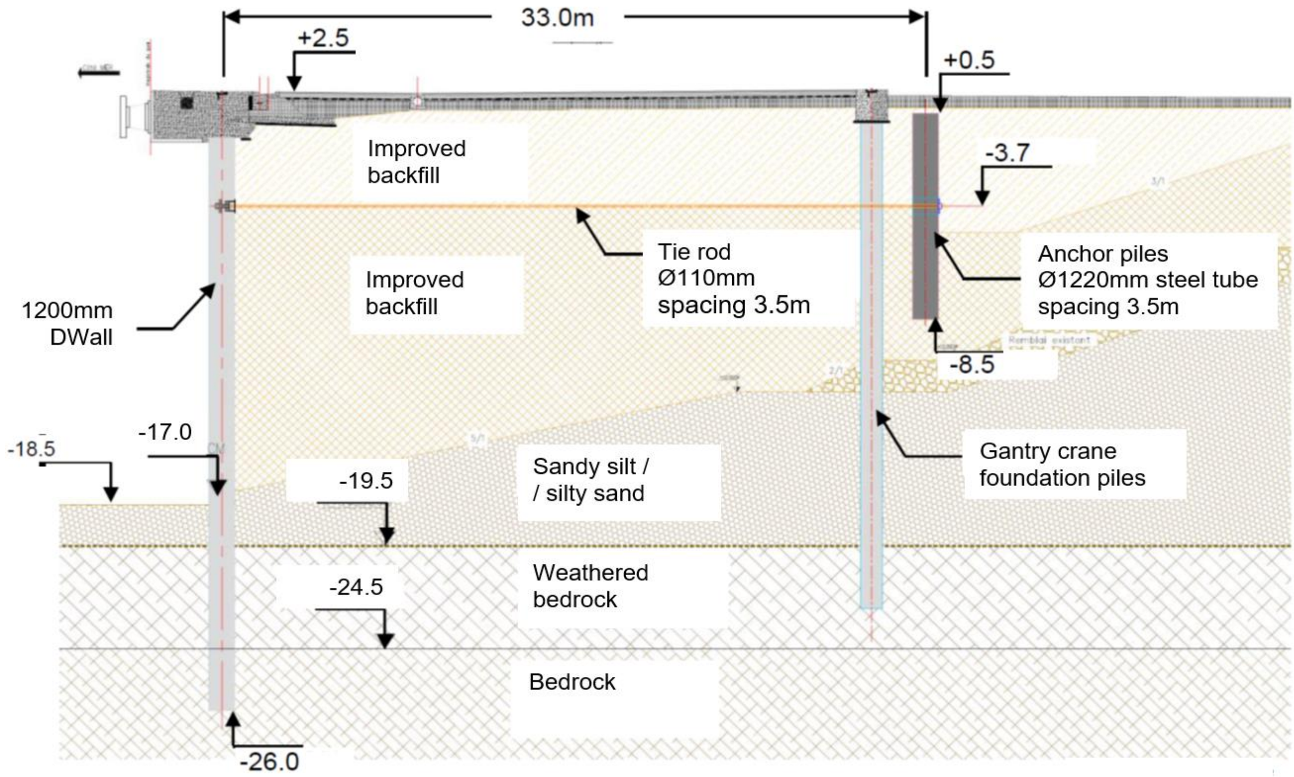

Fig. 3 – Case study: passively anchored diaphragm wall quay

3.2 Pseudo-static analysis

Pseudo-static calculations were, except where stated, performed according to the simplified procedure indicated EN1998-5 (2004). In particular, the partial safety factors on ground strength parameters as per §3.1 (3) (5) were applied to the tangent of the friction angle (γ_{φ}'=1.25).

Taking into account the retaining height of the structure, the equivalent seismic coefficient was determined from an equivalent-linear, one-dimensional wave propagation analysis, performed with DEEPSOIL 6.1 (Hashash et al., 2016) for a suite of 3 accelerograms, as required by EN1998-1 (2004). The accelerograms were based on real ground motions representative of the local seismicity, scaled to the relevant PGA, and the frequency content matches the target spectrum according to the requirements of EN1998-1 (2004). Although the provisions of NCHRP611 (2008) and NTC (2018) were used in earlier stages of the design, a specific site analysis with DEEPSOIL provided an optimized value of k_h=0.09g, determined from the average acceleration along the diaphragm wall height for the worst case accelerogram.

Hydrodynamic effects, due to free water, were estimated according to Westergaard's formulation, as per EN1998-5 (2004). In the case of the pseudo-static analysis, these effects are modelled as pressures, cumulated with soil static and dynamic pressures (an admittedly conservative assumption, since it does not seem likely that ground pressures and hydrodynamic pressures are in phase, causing maximum effects at the same moment in time).

The seismic increment on the exposed height was applied as a uniform pressure; seismic pressures on the embedded depth were modelled by replacing the static active and passive coefficients by pseudo-static values (in agreement with FHWA, 2011).

As presented in previous paragraphs, the passive resistance provided by the embedded depth of the diaphragm wall was estimated with TALREN5 (limit equilibrium software, developed by TERRASOL) for spiral-log failure surfaces, considering δ_d=2/3φ'_d. This led to passive resistance coefficients substantially in excess of those obtained with EN1998-5 (2004) (see Table 2 below). It should be noted that the passive resistance according to EN1998-5 (2004), considering δ_d=0, would not provide sufficient resistance to the wall toe, and that consequently the diaphragm wall embedment would need to be increased by more than 60% (at least 5m in additional depth), with very substantial economic impacts.

It should be noted that, as allowed by §7.3.2.2 (7) for the case of non-gravity walls, the effects of vertical acceleration were neglected throughout the analysis. This hypothesis, while not unanimous, is accepted by many references, among others NTC (2018), NCHRP (2008), FHWA (2011) and OCDI (2009). Further discussion can be found in Pereira et al. (2018).

Table 2 –Passive resistance coefficients for pseudo-static calculations

Layer	**φ'_k**	**φ'_d**	**$k_{pd}\cdot\cos\delta_d$; δ_d=0 (*)**	**$k_{pd}\cdot\cos\delta_d$; δ_d=2/3φ'_d (**)**
	[deg]	**[deg]**	**[-]**	**[-]**
Sandy silt / silty sand	30	25	2.13	3.01
Weathered bedrock	34	30	2.66	4.34
Improved backfill	38	32	2.98	5.06
Bedrock	40	34	3.21	5.97

* As per EN1998-5 (2004) Mononobe-Okabe formula

** As per TALREN5 Limit Equilibrium Method calculations

The main results for the first calculation (case PSa) can be found in Table 3 and Fig. 4 below. As for the static load case (PS0, also in Table 3), the pseudo-static soil-structure interaction calculations were performed with Soletanche Bachy's software PARIS, which simulates soil-structure interaction though a beam-on-springs approach. It is generally accepted that pseudo-static displacement estimates are not reliable, and this opinion is shared by the authors of this paper. This is even more the case for calculations using factored material parameters, such as required by EN1998-5 (2004). The capping beam displacements presented in Table 4 are therefore intended solely as a means of comparison between cases and as rough guidance.

Table 3 – Main results for PARIS analysis: static (PS0) and pseudo-static cases

Case	**PS0**	**PSa**	**PSb**	**PS1**
Material sets	Unfactored	Factored	Factored	Unfactored
PGA [g]	0 (static)	0.11 (k_h=0.09g)	0.11 (k_h=0.09g)	0.11 (k_h=0.09g)
Seismic increment	n.r.	Rectangular	Triangular	Triangular
M_{max} [kNm/m]	1420	3620	3870	2350
M_{min} [kNm/m]	-580	-900	-750	-760
F_{anchor} [kN/m]	460	1000	970	740
$displ_{capbm}$ [mm]	22	54	48	40

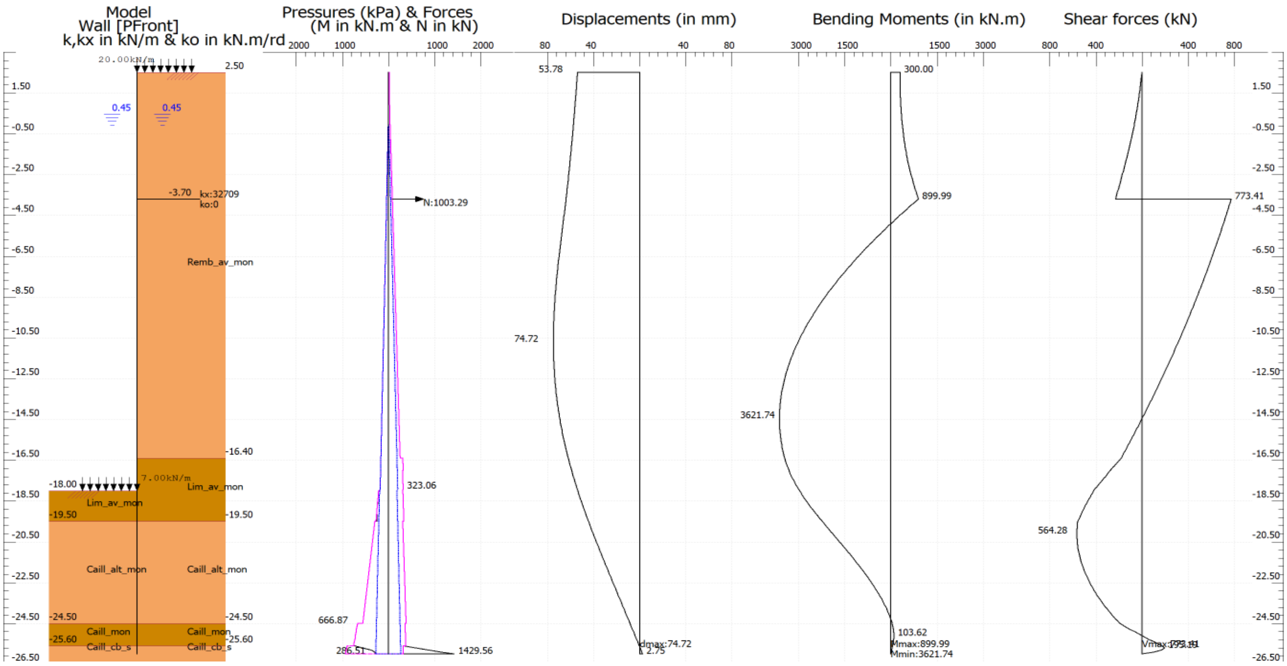

Fig. 4 – PARIS software outputs for pseudo-static conditions (case PSa). From left to right: schematic section; mobilized pressures and anchor force; horizontal displacements; bending moments; shear forces

Instead of the standard rectangular increments prescribed by EN1998-5 for standard retaining structures, an additional case (PSb) was calculated using triangular seismic increments (i.e. the same pressure distribution as in the static case), as permitted by EN1998-5 for structures which are free to rotate about their toe. It is interesting to note, however, that the impact on the results is small. For convenience, the triangular distribution was used in all forthcoming calculations.

In order to establish a comparison between the pseudo-static calculations and the time domain analysis (for which characteristic soil parameters are considered), an additional calculation was performed (case PS1), in which no partial safety factor was applied to the tangent of the friction angle ($\gamma_{\varphi'}$=1.00). As would be expected, anchor force and bending moments are very significantly reduced. By comparing cases PS1 and PSb it can be observed that, by suppressing the partial safety factor on material strength, the effect of the earthquake on anchor forces is more than halved (increment of 230kN/m vs 510kN/m), the same happening to the effect on peak bending moments (increment of 930kNm/m vs 2450kNm/m).

3.3 Non-linear time domain analysis

The non-linear time domain analysis was implemented with the ZSOIL finite element code (developed by ZACE Services Ltd). Dynamic water pressures were modelled as added masses applied to the plate elements, with the masses being calculated according to Westergaard's formula, as before for the pseudo-static case. The accelerograms used for the one-dimensional wave propagation were deconvoluted, with DEEPSOIL, from the bedrock outcrop to the base of the ZSOIL model. Soils were modelled with Hardening Soil Small Strain model (Benz et al., 2009). Being a time domain analysis, characteristic ground strength parameters were used (no partial safety factors).

Calculations were performed for the 3 accelerograms, with the main results (maximum and minimum bending moments, anchor forces, displacements at capping beam level) for each of the signals being within a ±10% difference envelope. For succinctness, only the results of the first accelerogram (which consistently showed intermediate results) are shown here.

Calculations started with the static case (TD0), the results of which are generally very similar from those obtained with PARIS (PS0), making for a good base for comparison between pseudo static and dynamic analysis. In addition to the static case and the reference acceleration case (PGA=0.11g, case TD1), additional calculations were performed for doubled acceleration (PGA=0.22g, case TD2) and trebled acceleration (PGA=0.33g, case TD3). Results are summarized in Table 4, while Fig. 5 shows the mesh and the displacement contours for one of the cases.

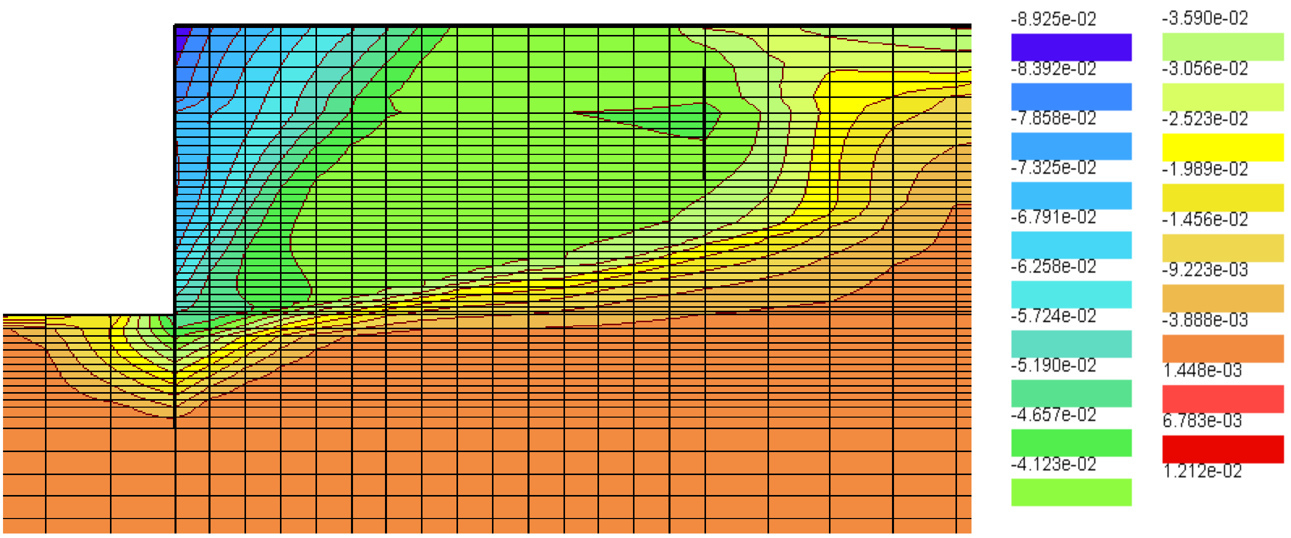

Fig. 5 – ZSOIL software outputs (case TD1ww): horizontal displacement contours and element mesh

Firstly, it can be observed that the calculation for PGA=0.11g (TD1) leads to results that are much more favourable than those obtained by application of EN1998-5's provisions, even if some of these provisions were already optimized according to this paper (case PSa in Table 2). It seems that a large part of the difference comes from factoring the shear resistance in the pseudo-static calculation, and not doing so in the time domain case. It can also be observed that calculation TD1 compares well with its pseudo-static equivalent (PS1), although this partly proves to be a coincidence, as will be explained further below.

As expected, displacement results differ substantially from the rough estimates of pseudo-static methods. While the displacements are non-negligible (total capping beam displacement for case TD1 is 103mm, of which 55mm occurred during earthquake) they are compatible with the operating conditions of many quay walls. Finally, it can be observed that the increment in forces and displacements is approximately constant between cases (from TD0 to TD3), which suggests that the yield acceleration of the system was not attained. In conclusion, it seems that, even for PGA=0.33g, the passive resistance of the wall – and the global resistance of the soil-structure system – is not yet exhausted, and that displacements remain at levels compatible with the behaviour of many types of maritime structures (140mm at the capping beam). It is also interesting to note that the forces of case TD3 are not very different from those of the factored pseudo-static case PSa, corresponding to a peak ground acceleration 3 times lower.

Table 4 – Main results for time domain ZSOIL analysis: static case (TD0), dynamic cases with hydrodynamic effects (TD1 to TD3) and without hydrodynamic effects (TD1ww to TD3ww)

Case	**TD0**	**TD1**	**TD2**	**TD3**	**TD1ww**	**TD2ww**	**TD3ww**
PGA [g]	0	0.11	0.22	0.33	0.11	0.22	0.33
M_{max} [kNm/m]	1440	2130	3120	4100	2300	3300	4300
M_{min} [kNm/m]	-700	-1100	-1230	-1250	-850	-1000	-1150
F_{anchor} [kN/m]	500	700	830	940	630	760	890
$displ_{capbm}$ [mm]	48	103	140	175	90	123	165

In order to separately assess the effect of the hydrodynamic effects, a series of calculations was performed without considering the Westergaard added masses (calculations TD1ww to TD3ww). The results are shown on the three right hand side columns in Table 4. It can be observed that, apart from negative bending moments (the magnitude of which is always small) and from a moderate increase in displacements (less than 14%), hydrodynamic effects tend to be minor, with peak bending moments and anchor forces systematically showing less than 10% differences. This suggests that the peak hydrodynamic and ground pressures are not likely to be in phase. While these results broadly agree with the findings of Gazetas et al. (2016), they are contrary to the usual practice, in pseudo-static modelling, of cumulating the effects of dynamic soil and water pressures.

3.4 Comparison of pseudo-static and time domain analysis

With the purpose of further comparing the pseudo-static and the time domain analyses, the calculations presented in Table 4 were replicated with the pseudo-static method in PARIS. The results can be found in Table 5 (calculations PS1 to PS3, and PS1ww to PS3ww) and it becomes evident that, instead of the progressive incremental behaviour obtained with the successive time domain models, increasing the acceleration levels in the pseudo-static models induces markedly larger effects: the cases for doubled PGA behave significantly worse, and the cases with trebled acceleration (PS3 and PS3ww) did not converge, due to insufficient passive resistance.

Table 5 – Main results for unfactored pseudo-static PARIS analysis: static case (PS0), pseudo-static cases with hydrodynamic effects (PS1 to PS3) and without hydrodynamic effects (PS1ww to PS3ww)

Case	PS0	PS1	PS2	PS3	PS1ww	PS2ww	PS3ww
PGA [g]	0	0.11	0.22	0.33	0.11	0.22	0.33
M_{max} [kNm/m]	1420	2350	4830	-	1770	3690	-
M_{min} [kNm/m]	-580	-760	-820	-	-700	-810	-
F_{anchor} [kN/m]	460	740	1210	-	590	970	-
$displ_{capbm}$ [mm]	22	40	62	-	32	51	-

For the cases without hydrodynamic effects, it can be observed that, for PGA (case PS1ww), peak bending moments are lower than those obtained with ZSOIL for case TD1ww (1770kNm/m vs 2300kNm/m; or 125% vs 160% of peak static moments from cases PS0 and TD0), while anchor forces match well (590kN/m vs 630kN/m; or 128% vs 126% of static forces). Comparing the cases of doubled PGA, it is apparent that the pseudo-static case is more unfavourable for peak moments (260% vs 229% of static moments, respectively for cases PS1ww and TD1ww), and even more so for anchor forces (211% vs 153%).

For the cases including hydrodynamic effects, it appears that, for PGA (case PS1), bending moments match relatively well those obtained with time domain analysis for case TD1 (2350kNm/m vs 2130kNm/m; or 165% vs 148% of peak static moments from cases PS0 and TD0); it should be noted that the moments match well essentially because pseudo-static calculations are very sensitive to hydrodynamic effects (applied as pressures) and that, conversely, the time domain analysis are not significantly affected by the added water mass. Regarding anchor forces, the results match well (740kN/m vs 700kN/m; or 161% vs 140% of static forces). Comparing the cases of doubled PGA, it is apparent that the pseudo-static case is much more unfavourable for both peak moments (340% vs 217% of static moments, respectively for cases PS1 and TD1) and for anchor forces (263% vs 166%).

While a better match between pseudo-static and time domain could have been obtained, namely by using variable seismic coefficients for the upper and lower parts of the wall (the bending moments seem very sensitive to the exhaustion of passive resistance along the embedment depth), the results for moderate acceleration levels (PGA=0.11g) are relatively well matched, especially for anchor forces. On the other hand, the unfavourable behaviour of the pseudo-static calculations for doubled and trebled acceleration levels is less satisfactory: while the time domain calculations show reserves of capacity even at trebled PGA=0.33g, the pseudo-static results show that the available passive resistance is almost completely mobilized at 0.22g. More work is required to assess the reasons for this discrepancy, with reduction in seismic coefficients due to displacement and other non-linearities being the most likely explanations.

4. Conclusions

This paper presents aspects of particular interest in the pseudo-static analysis of diaphragm walls, and suggests how some of the provisions of the current EN1998-5 could be updated, in order to better approximate the seismic behaviour of flexible retaining walls and to minimise the negative economic impacts associated with the current provisions.

The paper focused on optimised methods for the computation of passive resistance, on the evaluation of the impacts of the spatial incoherence of the ground motion, and on the acceptability (or otherwise) of post-earthquake residual displacements for non-gravity retaining walls.

A quay wall case study, comparing several pseudo-static and dynamic analysis, showed that pseudo-static analysis is generally conservative, even when the suggestions indicated in this paper are implemented.

Attention is drawn to the apparent conservatism associated with factoring the soil's strength parameters (as required by the current version of EN1998-5) and with the pseudo-static cumulus of peak hydrodynamic pressures and dynamic soil pressures, which do not seem likely in real conditions.

While pseudo-static methods cannot – and should not – replace time domain calculations, the adoption of optimised hypothesis can take simplified methods closer to more sophisticated analysis.

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