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Numerical investigation on the capacity decrease of monopiles in sand due to cyclic loading

D. Frick, K. Abdel-Rahman, N. Goldau, M. Achmus*

Institute for Geotechnical Engineering, Leibniz University Hannover, Hannover, Germany

**achmus@igth.uni-hannover.de (corresponding author)*

ABSTRACT: Monopiles are the most commonly used type of foundation for offshore wind turbines. Due to the intensive repeated loading by wind and wave loads, the influence of cyclic loads on the load-bearing behavior is of great importance. For monopile foundations in sandy soils, among other things, a reduced horizontal capacity is subject to great uncertainties. It is largely unclear how system properties (monopile geometry, soil parameters) and the cyclic loading conditions (mean load, load amplitude, load frequency and number of load cycles) affect the reduction. An explicit calculation concept was developed at the authors' institute, which enables the estimation of the load-bearing capacity reduction by finite element simulations using the results of contour diagrams derived from cyclic direct simple shear tests. The excess pore water pressure dissipation during the storm event is also taken into account by applying pore pressure decay curves derived from a consolidation analysis. With this calculation concept, the capacity reduction due to a given storm load scenario is first determined for a reference case. Then, based on this reference case, the above-mentioned system properties (in particular the hydraulic conductivity of the sandy soil) and the cyclic loading conditions are varied in order to identify their influence on the capacity reduction. The results are evaluated and compared in particular with the capacity reduction resulting from the p-y approach according to API guidelines for cyclic loading. It is shown that the reduction in capacity is strongly dependent on the magnitude of the cyclic load and in particular on the hydraulic conductivity of the in-situ soil and that in individual cases the reduction may well be greater than that determined using the API approach.

Keywords: monopiles, cyclic loading, load-bearing capacity, excess pore water pressure, finite element simulations

1 INTRODUCTION

Cyclic loading of an offshore structure may, depending on the geometry, soil and loading conditions, result in the accumulation of excess pore pressures around the foundation even in sandy soils. The build-up of excess pore pressures is accompanied by a reduction in the effective stresses within the soil, which can impair the load-bearing capacity of the foundation structure, particularly during heavy storm events. This is different from piles in non-saturated or even dry soils, where no excess pore pressures occur and cyclic loading can lead to an increase in pile capacity due to induced soil compaction (e.g. Nicolai et al., 2017). Despite the common requirement to account for cyclic degradation effects on bearing capacity due to excess pore pressures in the design process (e.g., DNV-RP-C212, 2019), the relevant geotechnical offshore guidelines or standards do not provide a generally applicable and accepted method for the calculative verification. According to design regulations (e.g. the German standard DIN 18088-1), the cyclic loads representing a certain "design storm" should be considered in the determination of the foundation structure's bearing capacity. Although, in principle, the estimation of excess pore pressures resulting from cyclic loading is possible using advanced implicit numerical methods, the application of these is often too complex and costly for practical use, especially if a large number of locations with different soil conditions must be considered and several design loops have to be performed. For this reason, simplified approaches and assumptions are usually applied in practice. For instance, monopiles in sandy soils under extreme loads (ultimate limit state) are often designed using "cyclic" p-y curves such as those proposed by API (2014) or methods based on them (e.g., Sørensen, 2012). By doing so, it is assumed that the general reduction of the maximum bedding resistance of the "cyclic" p-y curves compared to the static curves takes into account the possible degradation in bearing capacity, e.g., due to pore water pressure accumulations or other cyclic effects. However, as the degradation of the static API p-y curves is achieved by a single empirical adjustment factor, no other input variables such as soil properties or loading conditions are taken into account. In addition, the empirical reduction factor is valid only for approximately 100 load cycles due to the underlying test data on long and slender piles (Cox et al., 1974) and was not originally intended to take account of pore water pressure accumulations. Accordingly, it must be stated that this approach can at best lead to very conservative or, under particular circumstances, even uncertain results. To overcome these shortcomings, new methods for predicting excess pore water pressures and their effects on bearing capacity are needed that meet the requirements of practical applications.

In the following, a new explicit method for the prediction of excess pore water pressures due to cyclic loading of soils and the associated capacity decrease of foundation structures is briefly introduced. The simple and comprehensible method is termed Excess Pore Pressure Estimation (EPPE) approach and was developed and optimized with regard to practical requirements and ease of application, i.e., simple estimation of input parameters and short calculation time. In order to demonstrate the possibilities of the EPPE approach results for a reference monopile are presented before a parametric study is introduced. In the end, the outcomes from application of the EPPE approach are compared with those obtained using the commonly used cyclic API p-y approach described above.

2 THE EPPE APPROACH

The general concept of the EPPE approach was first outlined in Achmus et al. (2018) and has since been further developed, see in particular Saathoff (2023). The EPPE approach in general involves four primary calculation steps, all performed within a numerical finite element (FE) simulation incorporating the results of cyclic laboratory element tests. The individual steps for a reference procedure are briefly explained below. However, it should be noted that the EPPE approach is a modular procedure, meaning that all steps can be interchanged for more advanced analysis. For further information on alternative options, the underlying assumptions or a more detailed description of the procedure, please refer to Saathoff (2023).

Step 1: Load application

In the initial calculation phase, the numerical model is created based on site specific soil conditions and the intended foundation geometry (the method is applicable for all types of foundation). The numerical model of the foundation is then subjected to the mean load F_{mean} that corresponds to the cyclic loading conditions being considered. The mean load is thereby treated as a long-term load, so drained conditions are assumed. Subsequently, the lateral load is increased by the cyclic load amplitude F_{cyc} to achieve the maximum load of the first load cycle $F_{max} = F_{mean} + F_{cyc}$. Although the load amplitude should realistically be applied in an undrained manner, for the EPPE approach it seems sufficient to perform a drained calculation. Saathoff (2023) showed by comparison that for monopiles in sand application of the load amplitude under assumption of undrained or drained conditions leads to very similar results of the EPPE calculation. After load application, the stress components for both calculation steps, i.e., the mean and the maximum load application, are read from the integration points of the finite element model and stored in a database.

Step 2: Derivation of the undrained excess pore pressure

In the second step, the stresses extracted from the drained finite element calculation in the previous step need to be processed, i.e., the effective octahedral stresses (or mean normal stresses) σ'_{oct} as well as the equivalent octahedral shear stresses τ_{oct} (Eq. 1 & 2) are calculated as representative values for normal stress and shear stress of a three-dimensional (3D) problem.

$$\sigma'_{oct} = \frac{\sigma'_1 + \sigma'_2 + \sigma'_3}{3} \tag{1}$$

$$\tau_{oct} = \sqrt{\frac{2}{9}} \sqrt{\frac{1}{2}\begin{bmatrix}(\sigma'_{xx} - \sigma'_{yy})^2 + \\ (\sigma'_{yy} - \sigma'_{zz})^2 + \\ (\sigma'_{zz} - \sigma'_{xx})^2\end{bmatrix} + 3\begin{bmatrix}\tau_{xy}^2 + \\ \tau_{yz}^2 + \\ \tau_{xz}^2\end{bmatrix}} \tag{2}$$

Based on these quantities, mean and cyclic stress ratios according to Eq. 3 & 4 are derived.

$$CSR_{FE} = \frac{\tau_{oct,Fmax} - \tau_{oct,Fmean}}{\sigma'_{oct,Fmean}} \tag{3}$$

$$MSR_{FE} = \frac{\tau_{oct,Fmean} - \tau_{oct,F=0}}{\sigma'_{oct,Fmean}} \tag{4}$$

For the estimation of the excess pore pressures, contour plots calibrated from load-controlled and undrained (constant volume) cyclic direct simple shear (DSS) tests, see e.g., Figure 1, are utilized to determine the normalized excess pore pressure ratio R_u (Eq. 5) which depends on load cycle number N based on the previously determined CSR and MSR values.

$$R_u = \frac{\Delta u}{\sigma'_{oct}} \tag{5}$$

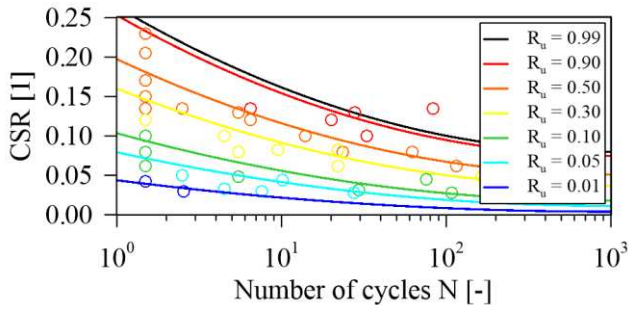

Figure 1 – Contour plot for MSR = 0 with CSR over number of cycles N for a relative density of $D_r = 0.85$.

However, before the excess pore pressure ratios R_u can be read from the contour plot or equations describing it (c.f., e.g., Saathoff & Achmus, 2023), the CSR_{FE} and MSR_{FE} values from the numerical model must be converted to the specific stress state within a DSS test (CSR_{DSS} and MSR_{DSS}) by consideration of the earth pressure coefficient at rest $k_0 = 1 - \sin\varphi'$, see Eq. 6 and 7. This simplified approach is valid for normally consolidated conditions. It is acknowledged that this may not represent overconsolidated soil states, where adapted k_0-values should be applied.

$$CSR_{DSS} = CSR_{FE}\frac{1+2k_o}{3} \tag{6}$$

$$MSR_{DSS} = MSR_{FE}\frac{1+2k_o}{3} \tag{7}$$

After determination of the R_u values for each integration point in the numerical model, the excess pore pressures $\Delta u(N=1)$ after the application of only one load cycle under the given loading conditions and assuming undrained behaviour can be calculated from the normalised pore pressure ratios R_u read from the contour plot by multiplication with the corresponding octahedral stresses $\sigma'_{oct,Fmax}$. In addition to these, also $R_u(N)$ curves are determined for the number of load cycles to be considered and stored in a database as input for the next EPPE step.

Step 3: Dissipation and analytical superposition

To account for dissipation under partially drained conditions and to derive the response due to several load cycles N, the previously derived excess pore pressure field $\Delta u(N=1)$, same as the geostatic stress tensor σ reduced by excess pore pressure (to keep equilibrium), are applied as initial conditions to the numerical model and a consolidation analysis is performed. In this way, element specific dissipation curves $\Delta u(t)$ are derived, considering the given foundation geometry, flow paths and soil properties. In a further calculation step, normalized decay curves $\Delta u(t)/\Delta u(t=0)$ are calculated and stored for each element. The normalized decay curves are - together with the undrained $R_u(N)$ curves (cf. Step 2) - the essential inputs for the following analytical superposition and the consideration of load cycle numbers $N > 1$ under partially drained conditions.

The analytical superposition can be done by different methods. Two of these, both later compared in the parametric study, are exemplarily illustrated in Figure 2 and briefly described hereafter. For all superposition approaches, a representative storm load period T has to be chosen, e.g., $T = 10$ s. According to the storm load period, decay values are read from the normalized decay curves and applied for the analytical superposition.

The first superposition approach, termed horizontal shifting, is illustrated in Figure 2 (a). Here, first the undrained R_u value for $N = 1$ is determined, which is then reduced due to dissipation by application of the corresponding value of the decay curve after one cycle period T (exemplarily assumed to be 50%). Afterwards, the undrained $R_u(N)$ curve is horizontally shifted to this point and another cycle is added. The resulting (undrained) R_u value is again reduced by 50%. This approach primarily considers the stiffness of the initial segment of the undrained $R_u(N)$ curve and is based on the assumption that the residual pore pressure ratio R_u after dissipation is back-calculated to a new number of equivalent load cycles within the same $R_u(N)$ curve.

The second superposition approach (vertical shifting) was already introduced in Achmus et al. (2018) and is visualized in Figure 2 (b). It is assumed that the increase in $R_u(N)$ under partially drained conditions is the same as under undrained conditions. Consequently, the total partially drained pore pressure

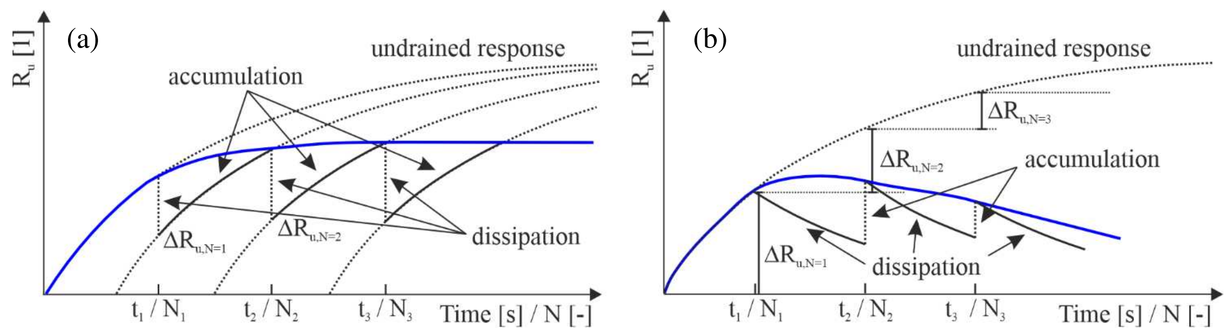

Figure 2 – Methods for analytical superposition: (a) horizontal shifting and (b) vertical shifting of the $R_u(N)$ curve.

ratio R_u decreases as the number of cycles increases, since the added increment in (undrained) R_u diminishes with each additional cycle. This approach contrasts with the initially presented method. With the vertical shifting approach, the resulting partially drained pore pressure ratio $R_u(N)$ begins to build up and then decreases, as from a certain number of cycles the dissipation starts to exceed the additional increments in R_u. Consequently, there may be a point during the calculation where the pore pressure ratio is higher than at the end of the storm, i.e., the considered number of load cycles N. However, this state must not occur for all elements at the same number of cycles N.

Step 4: Post-cyclic calculation

The residual partially drained excess pore pressure ratio after the considered storm event (derived by analytical superposition) is utilized to analyse the post-cyclic foundation response. In situ, due to the accumulated excess pore pressures, the octahedral stresses are reduced, resulting in a lower shear stress capacity until failure occurs. To consider this, in a new post-cyclic finite element model (identical to those previously used), the earlier derived excess pore pressure ratios R_u for each integration point accounting for partially drained conditions are employed to derive an equivalent (reduced) angle of friction (Eq. 8).

$$\varphi'_{red} = \tan^{-1}[(1 - R_u)\tan(\varphi')] \qquad (8)$$

By application of these friction angles to the corresponding elements in the FE model, the shear strength is diminished, allowing the determination of the post cyclic foundation capacity. Herein, also the dilatancy angles of the soil were adapted with $\psi = \varphi'_{red} - 30° \geq 0°$. The reduced foundation capacity can finally be assessed from the calculated load-displacement curve, e.g., using a deformation criterion such as $0.1D$ (D = pile diameter) typical for pile foundations.

3 PARAMETRIC STUDY

3.1 Scope of analysis and boundary conditions

To demonstrate the capabilities of the previously described EPPE approach and to quantify the effects of excess pore pressure accumulations due to cyclic loading on the post-cyclic bearing capacity, a parametric study for a typical laterally loaded monopile foundation (cf. section 3.2) was carried out. The storm load scenario considered was assumed to be represented by an equivalent and regular load package (constant mean load and amplitude) with $N_{eq} = 40$ load cycles of pure one-way loading with complete unloading in each cycle and a period of $T = 10$ s. The cyclic load magnitude $\zeta_b = F_{max}/F_{ref}$, where F_{ref} is the drained horizontal load-bearing capacity of the foundation at a pile head displacement of $0.1D$, was varied in a range of 0.1 – 0.5. In addition, for $\zeta_b = 0.3$, the influence of a change in soil permeability k_f was analysed for a bandwidth of $3.7\text{x}10^{-4}$ m/s to $3.7\text{x}10^{-5}$ m/s. To estimate the undrained cyclic soil response (EPPE step 2), both the equation proposed by Saathoff (2023) and the corresponding regression parameters for a very dense sand (relative density $D_r = 0.85$) were used in the present calculations to parameterize the required contour plots. In order to highlight the possible effect of the chosen superposition approach (EPPE step 3), comparative calculations were carried out using the two methods of horizontal shifting and vertical shifting already described in section 2. As a benchmark, the post-cyclic capacity reductions (at a pile head displacement of 0.1 D) derived by the EPPE approach relative to the static drained case (F_{cyc}/F_{stat}) were derived and compared with those resulting from the frequently used p-y method according to API.

3.2 Numerical model

The numerical analysis was performed using the finite element software ABAQUS. The three-dimensional model of a monopile with a diameter of $D = 8$ m, a wall thickness of $t = 0.1$ m and an embedded length of $L = 32$ m includes approximately 35,000 C3D8(P) elements. Exploiting the symmetry of the problem, only one half was modelled to reduce computational effort. The mesh resolution and model dimensions were optimized by a preliminary sensitivity study. The chosen dimensions of the half-cylindrical soil domain are 12-times the pile diameter D in width (radius $r = 6D$) with a height of 1.5-times the pile embedment length L. Constraints were applied to the bottom of the model that was fixed in all degrees of freedom, the outer perimeter and at the symmetry plane, both fixed in normal direction. The monopile was modelled with linear-elastic behaviour, characterized by a Young's modulus of $E = 2.1 \times 10^8$ kN/m², a Poisson's ratio $\nu = 0.27$ and a buoyant unit weight $\gamma'_{steel} = 68$ kN/m³. Lateral loading was applied on a reference node at height $h = 40$ m above soil surface, which is connected to the monopile via a kinematic coupling.

The soil parameters for the reference case are summarized in Table 1 (k_f varied in the parametric study). The initial horizontal earth pressure at rest was calculated by $k_0 = 1 - \sin\varphi'$ and the dilatancy angle was determined using $\psi = \varphi' - 30°$. For modelling

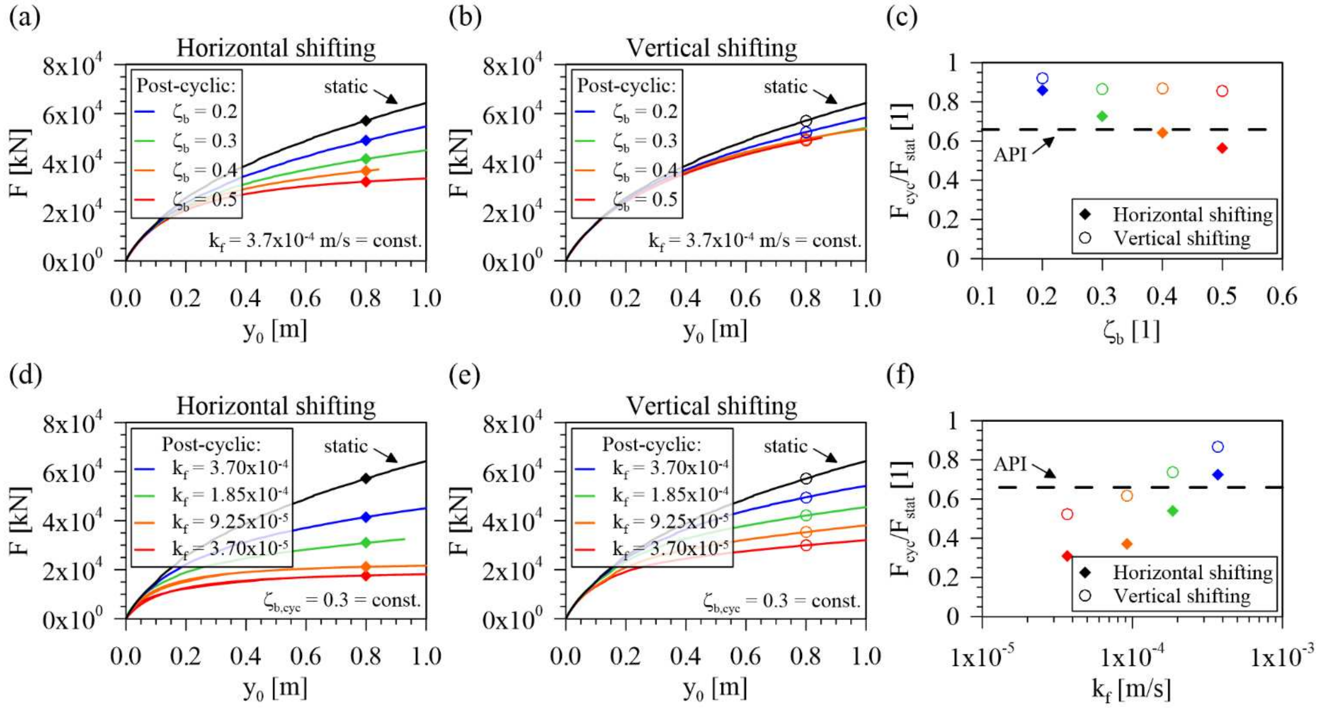

Figure 3 – Results of the parametric study: comparison of static and post-cyclic load-displacement curves with a) and b) varying load magnitude ζ_b of the considered storm event or d) & e) varying soil permeability k_f and c) and f) resulting capacity reductions F_{cyc}/F_{stat} including results according to the API p-y method.

the soil, an elasto-plastic material law based on Mohr-Coulomb failure citerion in conjunction with a stress-dependent stiffness was employed:

$$E_s = \kappa\, p_{ref} \left(\frac{\sigma'_{oct}}{p'_{ref}}\right)^{\lambda} \tag{9}$$

where, E_s is the oedometric stiffness, κ and λ are soil dependent stiffness parameters, $p'_{ref} = 100$ kPa is a reference stress and σ'_{oct} is the current octahedral effective stress.

Table 1. Soil properties for numerical calculation.

κ [1]	λ [1]	ν [1]	φ' [°]	c' [kPa]	k_f [m/s]	ψ [°]	γ' [kN/m³]
670	0.5	0.25	38	0.1	3.7E^{-4}	8	11

For contact modelling, an elasto-plastic master-slave approach was applied between the monopile and the surrounding soil, allowing for both a connection and relative displacement between soil and structure. The maximum coefficient of friction in the sand-steel interface was set to $\delta = 2/3\varphi'$ and it was linearly mobilized within an elastic slip value of Δu_{el} =1 mm.

The calculation itself was conducted in several stages. First, the initial conditions were set, whereby a k_0-procedure was applied. Afterwards, the monopile and contact elements were activated (wished-in-place). Following this, the mean lateral load and its associated moment were applied, and finally, the maximum lateral load.

For the consolidation analysis (cf. EPPE step 3), the ABAQUS model was extended to enable coupled pore fluid and stress analysis. This involved conversion of the drained model into a linear elastic coupled model by changing the element type to C3D8P. The boundary conditions were modified to accommodate the additional degree of freedom. The pore fluid weight was set to γ_{water} =10 kN/m³ and for the bulk modulus of the pore fluild a value of $K_w = 2.092 \times 10^6$ kPa (for $T = 10$°C) was chosen.

3.3 Results

The most relevant results of the parametric study are shown in Figure 3. Subplots (a) and (d) of Figure 3 present a comparison of the derived load-displacement curves (y_0 = horizontal pile head displacement) with variations either in load magnitude ζ_b or soil permeability k_f both resulting from the horizontal shifting method. Figures 3 (b) and (e) illustrate the corresponding results for the vertical shifting approach. In Figures 3 (c) and (f) the determined capacity reductions F_{cyc}/F_{stat} for both superposition methods are given as a function of either ζ_b or k_f. Additionally, the capacity reduction due to cyclic loading based on the p-y method according to API is also indicated (dashed lines).

For the drained static case, a capacity of $F(y_0 = 0.1\,D) = 57\,MN$ was determined (cf., e.g., Figure 3 (a)). In contrast, considerably smaller post-cyclic capacities were obtained considering the given storm load scenarios using the EPPE approach, depending on the load magnitude ζ_b and soil permeability k_f. The results in Figure 3 (c) and (f) show that particularly when applying the horizontal shifting method, and for load magnitudes $\zeta_b > 0.4$ (for $k_f = 3.7\text{x}10^{-4}$ m/s) or soil permeabilities $k_f < 1.85\text{x}10^{-4}$ m/s (at $\zeta_b = 0.3$) significantly greater reductions in capacity are observed compared to the corresponding value according to the API method.

4 DISCUSSION

The presented results demonstrate that the application of cyclic p-y curves according to API is unsuitable for reliably accounting for the influence of excess pore pressure accumulation due to a storm event on the load-bearing capacity in the ultimate limit state. For the monopile investigated, the EPPE approach provides post-cyclic capacities that are partly deviating significantly, with some values exceeding and others falling well below the API predictions, therefore rendering the API method unconservative. The results further illustrate that the EPPE approach, in contrast to the cyclic p-y curves according to API, is capable of considering the influence of relevant boundary conditions, e.g., foundation geometry, magnitude of cyclic storm loading or soil permeability, while delivering plausible results. However, when applying the EPPE approach, the choice of the superposition method (step 3, cf. section 2) appears to have significant impact on the outcome. The reason for this is that both methods applied are based on simplifying assumptions, and that, particularly when applying the vertical shifting method, it may occur that the increase in pore water pressure within a cycle becomes smaller than the simultaneously occurring dissipation in a given element, cf. Figure 2 (b). As a result, the excess pore water pressure distribution at the end of cyclic loading determined using the vertical shifting approach does not necessarily reflect the state of maximum excess pore water pressure occurring during the cyclic loading phase for each element. This is different for the horizontal shifting method, where the determined excess pore water pressure always will approach a limit value with increasing number of load cycles, cf. Figure 2 (a). Even though this behaviour is not necessarily realistic, only the horizontal shifting method can be recommended initially for a conservative design. Further research and validation against pore water pressure data from model tests or field measurements is required to improve the EPPE approach, especially the superposition method.

5 CONCLUSIONS

The response of a foundation structure to cyclic loading in saturated sand may be largely affected by the drainage conditions and the potential accumulation of excess pore pressures. Calculations using the presented EPPE approach for a typical large diameter monopile subjected to an equivalent cyclic storm load show the possible extent of the post-cyclic capacity reduction and thus the importance of considering pore water pressure accumulations in the ultimate limit state design.

AUTHOR CONTRIBUTION STATEMENT

D. Frick: Conceptualization, Investigation, Visualization, Writing- Original draft. **K. Abdel-Rahman**.: Software. **N. Goldau**: Writing – Reviewing and Editing. **M. Achmus**: Supervision, Funding acquisition, Writing- Reviewing and Editing.

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