Numerical Simulation of the Tensile Resistance of Suction Buckets in Sand

Martin Achmus and Klaus Thieken
Institute for Geotechnical Engineering, Leibniz University Hannover
Hannover, Germany

The bearing behavior of a suction bucket foundation under tensile load is strongly influenced by the drainage conditions, which depend on the loading rate, the soil permeability, and the bucket dimensions. The paper presents numerical simulations based on a coupled pore fluid diffusion and stress analysis, which allows the description of the load-bearing behavior in terms of the mentioned parameters. Quite complex interaction effects between the suction pressure inside the bucket and the skin friction on the skirt perimeter become evident. A key challenge in the modeling is the consideration of soil liquefaction due to an upwards-directed seepage. Two approaches, which are assumed to be a lower and an upper bound solution, are presented and compared.

INTRODUCTION

The suction bucket is a relatively new foundation concept for offshore wind energy converters (OWECs). Bucket foundations seem to be favorable due to their relatively simple installation process and the avoidance of noise emission induced by pile driving. The latter will become more important and can severely affect the construction process when several wind farms will be installed at the same time. Two foundation concepts with suction buckets have to be discerned. Comparable to a monopile foundation, a single bucket (monopod) is predominantly loaded by horizontal forces (H) and bending moments (M), as described in Achmus et al. (2013). Several buckets can also be applied as foundation elements of lattice structures, e.g., tripods or jackets (multipod). In contrast to monopods, the loading of these buckets is predominately vertical. Figure 1 illustrates a schematic sketch of an OWEC on a jacket with a multipod suction bucket foundation. The buckets are placed with maximum feasible distance between each other to minimize the vertical loading due to wind and wave actions. The ratio of skirt length to bucket diameter, L/D, will probably be in a range between 0.5–1.0. Large ratios of L/D are favorable with respect to the drained tensile capacity, but limited by the installation process (Houlsby and Byrne, 2005).

For perfectly drained conditions, the tensile resistance of the bucket results from the dead weight of the bucket and the skin friction on the inner (\(\tau_i\)) and the outer (\(\tau_o\)) skirt perimeter. Under considerable tensile loads, a gap forms between the bucket lid and the soil surface. For perfectly undrained conditions, suction pressure inside the bucket prevents the formation of a gap. Consequently, the inner soil plug is uplifted, and moreover the outer soil is dragged beneath the bucket. In this case, the resistance in the surrounding soil becomes decisive. Senders (2008) termed this undrained behavior as “reverse end bearing,” which leads to a significantly larger resistance than that resulting from the drained (“purely frictional”) conditions (see Fig. 2). For partly drained conditions, the behavior is in between these basic conditions (“intermediate”). In terms of the loading rate, the bucket dimensions, and the permeability of the soil, the behavior has more in common with the perfectly drained or undrained behavior. Independent from the drainage conditions, the suction \(\Delta u\) is limited by the cavitation pressure, which amounts to \(u = u_c = -100\) kPa (average atmospheric pressure is about 100 kPa). The accessible suction pressure \(\Delta u_{\text{max}}\) therefore depends on the hydrostatic water pressure \(p_w\), which is the product of water depth \(h_o\) and the specific density of water \(\gamma_w = 10\) kN/m\(^3\) (see Fig. 1).

It is current practice in the design of this foundation type to determine the tensile capacity based on perfectly drained conditions. Even if model and field tests presented in different publications confirm that a significant increase in the resistance is to be expected for rapid tensile loading, these investigations also show that large heaves of the bucket are necessary to mobilize a considerable part of the additional resistance. Furthermore, the seepage into the bucket results in a degradation of suction pressure with time. Consequently, a large constant or repeated loading can only be transferred if the heave of the bucket increases steadily. Finally, it has to be ensured in the design that the heave of the bucket does not exceed the allowable heave (serviceability limit state design proof) for the entire lifetime of the wind turbine. Therefore, the admissible tensile reaction depends on a large number of influence parameters, such as the loading rate, the time period of loading, the soil permeability, etc. Until now, there has been no ready-to-use...
method to predict the tensile reaction in terms of these parameters. Moreover, there are large uncertainties with regard to the occurring mechanism in the load-bearing behavior for a repeated loading, as well as for a single loading event.

The paper presents results of numerical simulations that allow the description of the partially drained load-bearing behavior and the quantification of the tensile resistance. The simulations introduced here consider a single loading event, assuming a constant pullout rate (velocity of heave) \( V \). The authors suppose that the understanding of the occurring mechanism for a simple loading situation is a requirement for the prediction of the load-bearing behavior due to the actually quite complex loading situation of an OWEC.

**STATE OF THE ART**

The suction bucket foundation was originally developed for floating platforms in the oil and gas industry. The buckets were predominantly used in clay soils due to favorable effects in the installation procedure and the small permeability, which ensured a constant resistance for long-term loading conditions. Since the early nineties, suction buckets were also used for fixed jacket structures in sandy soils, as for the Draupner E gas platform in the North Sea. The platform was installed in dense-to-very dense sand and water depths of about 70 m (Bye et al., 1995). It is known that well as for a single loading event.

Therefore, it is not surprising that a large part of the previous investigations was performed with compression loading. For an OWEC, the compression loading is of minor importance. Instead, the tensile resistance is assumed to be driving the design for the installation. Unfortunately, the results of these investigations are confidential. With regard to the tensile resistance, only a short qualitative presentation on the influence of the pullout rate on the tensile resistance was given. It was stated that even a small increase in the pullout rate leads to significantly larger tensile capacities. It shall be noted that the ratio of maximum tensile resistance to compression loads for an OWEC is much larger than that for a gas platform due to the relatively small dead weight of OWECs. Therefore, it is not surprising that a large part of the previous investigations was performed with compression loading. For an OWEC, the compression loading is of minor importance. Instead, the tensile resistance is assumed to be driving the design for the bucket dimensions.

Iskander et al. (1993) conducted a model test for drained and for undrained conditions. The undrained test is used for the validation of the numerical model and is described in detail later. The first known investigation on the applicability of the multipod foundation concept for OWECs was given by Byrne (2000). He conducted pullout and cyclic vertical loading tests on small-scale buckets \( (D = 150 \text{ mm}, L = 50 \text{ mm}) \) in dense sand. Byrne concluded that the pullout rate has only a small influence on the initial stiffness, but increases the resistance for large heaves significantly. He also identified a close correlation between the cyclic loading results and the monotonic pullout tests. Based on this finding, Byrne assumed that the cyclic behavior (with respect to the accumulation of heave) could be inferred from the monotonic tests in future studies.

Feld (2001) conducted four small-scale model tests \( (D = 200 \text{ mm}, L = 100 \text{ mm}) \) with various pullout rates. The pullout tests were executed subsequent to an intensive dynamic loading. Feld assumed that the tensile resistance strongly increases with the pullout rate of the bucket. She also found out that the tensile resistance strongly increases when the duration of cyclic loading is doubled.

Houlsby et al. (2005) conducted pullout tests on small-scale buckets \( (D = 280 \text{ mm}, L = 180 \text{ mm}) \) installed by pushing. Independent of the load needed for complete penetration, the vertical load was increased to \( V = 35 \text{ kN} \). The four tests presented were conducted with pullout rates from \( V_z = 5 \text{ mm/s} \) to \( V_z = 100 \text{ mm/s} \) (0.56 times the skirt length \( L \) per second). The results of the tests indicate that the tensile capacity increases strongly with the pullout rate and a decreasing permeability of the soil. Houlsby et al. (2005) did not discuss the influence of the pre-loading on the test results. It can be assumed that the pullout tests, without dissipating the excess pore pressures resulting from the removal of the compression load, led to larger suction pressures and therefore to larger tensile resistance than that resulting for a single loading event. Based on the results of the model tests, they developed an approach to predict the ultimate tensile capacity of the bucket.

A field test with a suction-installed bucket \((D = 1.5 \text{ m}, L = 1.0 \text{ m})\) was presented by Houlsby et al. (2006). The test was conducted on a prepared test site at Luce Bay, Scotland. Prior to the pullout test, an intensive cyclic load was applied on the bucket, resulting in vertical displacements per cycle up to \( \Delta z = 12 \text{ mm} \). The field tests confirm that the tensile resistance under partly drained conditions is significantly larger than that for drained conditions, but also that a large heave is necessary to mobilize this resistance.

Senders (2008) conducted 38 centrifuge tests in dense sand with various pullout rates. He discovered that the resistance for large heaves is affected more by the pullout rate than by the initial stiffness. He developed a conceptual model with springs and a damper to assess the vertical load-bearing behavior. The realization of this model is problematic due to the assessment of a very large number of input parameters. As there are no strict guidelines to assess the input parameters (even the values in Senders’s own back-calculations are not stringent), the approach seems to be not yet suitable for a practical design.

Finally, a large-scale model test in silt, as given by Zhu et al. (2011), shall be mentioned. Based on three tests conducted, they identified a strongly nonlinear relationship between pullout rate and tensile resistance.

**NUMERICAL MODEL**

An axisymmetric finite element model of a single suction bucket foundation in very dense sand was developed. The finite element program Abaqus Version 6.12 (Dassault Systèmes, 2012) was used for the simulations. The numerical model is based on a coupled pore fluid diffusion and stress analysis, which allows a transient analysis of partially or fully saturated fluid-filled porous media. The finite element mesh used in the simulations is presented in Fig. 3. Four node axisymmetric pore pressure elements of type CAX4P were used to mesh the soil. The bucket elements were meshed with standard elements of type CAX4, which is acceptable due to the negligible permeability of steel. Preliminary sensitivity analysis was performed for model boundaries as well as for the mesh size. A model depth of 3.5 times the skirt length \( L \) and a model width of 6 times the bucket diameter \( D \) were found to be suitable to avoid an
water and soil, a direct bonding could be used. Between the water with material properties similar to water were generated. In Fig. 3, the water elements were modeled linear elastic with negligible stiffness (\(E_h = 10^{-3}\) kN/m\(^2\)), a Poisson’s ratio \(\nu = 0.499\), and a unit weight of \(\gamma_h = 10\) kN/m\(^3\). The permeability \(k\) was set as identical to the values of soil (see section on constitutive model below). A similar approach with poroelastic elements beneath the bucket skirt was already used in a study about the pullout capacity of suction buckets in soft clay by Cao et al. (2002). Mana et al. (2014) showed that the deviance of this approach to the correct solution decreases with the decreasing stiffness of the water elements.

The calculation was conducted in two steps. In the first step, the initial stress state was generated. Here, a coefficient of earth pressure at rest, \(k_0 = 0.47\), was used. In the second step, the bucket was raised with displacement controlled until a heave \(z = 200\) mm was reached. The load was applied on a reference node, which was bonded with the upper bucket surface. Since relatively large deflection can occur during uplift, geometric nonlinearity was considered, i.e., the coordinates of the element nodes were corrected for each load step with respect to the current deformations.

**Constitutive Model**

The modeling of the soil is challenging due to the complex stress situation and the importance of dilatancy on the effective stresses and the coupled permeability. For this purpose, the nonlinear hypoplastic material law according to von Wolffersdorff (1996) was used in this study. The rate-type formulation is defined by the tensorial function:

\[
\dot{T} = L(T, \dot{e}) : D + N(T, \dot{e}) \| \dot{D} \|
\]  

(1)

where \(\dot{T}\) is the objective stress rate, \(D\) is the strain rate, \(L\) is a fourth-order tensor, and \(N\) is a second-order tensor that depends on the actual Cauchy stress \(T\) and the void ratio \(e\). The soil model used is able to account for dilatancy, barotropy, and pycnotropy of granular soils. Furthermore, the implementation in a rate-type formulation allows the consideration of unloading and reloading paths realistically. Hypoplastic parameters for the IGtH sand used are presented in Table 1. The determination of the parameters is described in detail by tom Wörden (2010). IGtH sand is a poorly-graded medium quartz sand with rounded grains (char. grain size \(d_{50} = 0.52\) mm, coefficient of uniformity \(U = 2.3\), coefficient of curvature \(C = 0.9\)).

For the very dense state considered, a buoyant unit weight \(\gamma' = 10.31\) kN/m\(^3\) and a void ratio \(e = 0.6\) were assumed. The results of numerical triaxial tests for IGtH sand in medium dense state are given in Fig. 4.

**Soil Permeability**

In the simulations, the semiempirical and semitheoretical approach according to Kozeny-Carman (Kozeny, 1927; Carman, 1937; Carman, 1956) is used for the determination of the soil permeability. The approach accounts for the grain size distribution, the void ratio, and the fluid viscosity, and is assumed to be quite accurate (Carrier, 2003). The permeability according to Kozeny-Carman, \(k_{K-C}\), can be derived by Eq. 2:

\[
k_{K-C} = \left(\frac{\gamma_u}{\mu}\right) \cdot \left(\frac{1}{C_{K-C}}\right) \cdot \left(\frac{1}{S_i}\right) \cdot \left(\frac{e \cdot 2}{1 + e}\right)
\]

(2)

where \(\gamma_u = \) unit weight of pore fluid (kN/m\(^3\)), \(\mu = \) viscosity of pore fluid (kN·s/m\(^2\)), \(C_{K-C} = \) Kozeny-Carman empirical coefficient (1), \(S_i = \) specific surface area per unit volume of particles (1/m),

<table>
<thead>
<tr>
<th>Description</th>
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<tr>
<td>Critical state friction angle</td>
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<td>Maximum void ratio</td>
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<tr>
<td>Exponent</td>
<td>(\beta)</td>
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</tr>
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</table>

Table 1 Hypoplastic material parameters for IGtH-Sand
and \( e = \) void ratio (1). Carman (1956) reported the value of \( C_{K–C} \) as being equal to \( 4.8 \pm 0.3 \). Here, \( C_{K–C} \) is set equal to 5.0, as it is common practice. For the assumed water temperature of \( 10^\circ \text{C} \), the term \( \gamma_p/\mu_w \) is equal to \( 7.645 \cdot 10^4 \) (1/m·s). The specific surface area per unit volume of particles is calculated, based on the grain size distribution according to Carrier (2003), as \( \Delta_0 = 1.633 \cdot 10^4 \) (1/m). For the scheduled initial void ratio \( e = 0.6 \), Eq. 2 yields an initial permeability \( k_{\text{init}} = 7.73 \cdot 10^{-4} \text{ m/s} \). To account for the change in void ratio during uplift, the permeability is recalculated stepwise for each element. Therefore, the increase in permeability with loosening and the decrease with densification is accounted for. This behavior is also implemented for the water elements between the soil and the bucket.

### Consideration of Soil Liquefaction

Soil liquefaction occurs in noncohesive soils if the critical gradient \( i_{\text{crit}} \) of an upwards-directed seepage is exceeded, and a sufficient volumetric dilatation is possible to free the soil particles. The occurrence of soil liquefaction comes along with a total loss of effective stresses and an increase of permeability as the soil–water mixture behaves like a fluid. For rapid tensile loading, it is assumed that the seepage into the bucket is able to reach the vertical resistance is underestimated, since the assumption that sufficient volume dilation is possible to induce the permeability increase seems conservative.

### Limitations of the Numerical Model

All numerical models have their limitations, which must be considered in the assessment of simulation results. Important limitations of the presented model are:

- The numerical model does not account for effects of the installation, i.e., loosening of the soil inside the bucket due to the upwards-directed seepage near the critical gradient is neglected.
- The usage of an axisymmetric model does not allow the consideration of an additional horizontal (inclined) loading. It is to be assumed that there is an influence of an additional horizontal loading on the vertical load-bearing behavior due to a change in the effective stresses, similar to how it is described for pile foundations by Achmus and Thieken (2010). Additionally, a horizontal load affects the void ratio and therefore also the permeability of the soil in the region of the bucket skirt. At worst, a gap forms between the skirt and the outer soil, which reduces the seepage path and thus the suction resistance significantly.
- In the simulation, complete saturation for the soil, as well as for the water elements, is assumed. Due to the compressibility of locked air, the actual suction resistance due to uplift can be somewhat smaller than the calculated values. The deviation depends on the level of saturation of the soil and the appearance of air bubbles beneath the bucket lid.
- Quasi-static system behavior is assumed in the simulations. Therefore, no inertia effects or damping can be considered, which are assumed to cause an increasing model error with increasing pullout rate. However, for realistic loading conditions of offshore wind energy converters, the influences of the dynamic effects are assumed to be of minor importance. It shall be noted that it is common practice in the design of other types of OWEC foundations to neglect such dynamic effects.

### VALIDATION OF THE NUMERICAL MODEL

In order to validate the numerical model, back-calculations of reported tests in the literature were carried out. It is preferred to use large-scale field tests instead of small-scale laboratory tests. However, reports about field tests relevant to the problem under consideration are very scarce. Only one field test conducted by Houlshys et al. (2006) is known (see section on state of the art). However, due to major uncertainties regarding the soil permeability, the pullout rate, and the previous load history applied in this test, it is not seen as reasonable for the validation. Therefore, the validation is realized with a well-documented model test from
Iskander et al. (1993). The results of the test were given in terms of a load-displacement curve. Additionally, the pore pressures were measured at the top, middle, and tip on the inside of the bucket skirt. The experimental results, as well as the results of the numerical back-calculations, are presented in Fig. 5. For the back-calculation, a bucket with a diameter \( D = 100 \text{ mm} \) and a skirt length \( L = 194 \text{ mm} \) was modeled and uplifted with a constant pullout rate \( V_z = 7.6 \text{ mm/s} \), as it was specified in the publication. Due to the fact that no further information about the hypoplastic parameters of the soil used was available, the fine sand in a very dense state was modeled with parameters of IGtH sand. This seems to be feasible, as the given parameters of the grain size distribution, the minimum and maximum void ratio, the grain shape, and the friction angle in a very dense state are fairly comparable to the parameters of IGtH sand. The permeability was calibrated to the given initial value \( k_{\text{init}} = 10^{-5} \text{ m/s} \), but was assumed to vary during uplift with regard to a change in the void ratio \( \varepsilon \).

First, it must be stated that no deviations between the upper and the lower bound solution occur for the system analyzed. This is related to the applied pullout rate and is explained in the following sections. From Fig. 5, it becomes apparent that, except for the initial part of the test, both the resistance-heave relation and the pore pressure measurements are in good agreement with the numerical simulations. Looking at the initial part of the load-displacement curve of the model test, it could be assumed that the heave measurement was not calibrated correctly to zero. After compensating for this by shifting the model test results about 2 mm to the left, the results are completely in compliance.

The back-calculation of just one test is, of course, not sufficient for a complete model validation, especially as the correctness of the soil parameters used cannot be ensured. However, the results indicate that the model yields reasonable results and can thus be used in studies of the load-bearing behavior of buckets under tensile load. In the following sections, results from the upper bound and the lower bound solutions with respect to occurrence of liquefaction are presented and compared.

**RESULTS FOR THE UPPER BOUND SOLUTION**

Numerical simulations were first performed for the upper bound solution. A suction bucket with a skirt length \( L = 12 \text{ m} \), a diameter \( D = 12 \text{ m} \) \((L/D = 1.0)\), and a wall thickness \( t = 30 \text{ mm} \) in very dense sand was investigated. These dimensions agree well with realistic parameters for large OWECs in the North Sea.

First, the results of the simulations are presented in terms of load-displacement curves (refer to Fig. 6). Here, the vertical load given includes the dead weight of the bucket of about \( V_B = 0.92 \text{ MN} \). The constant pullout rate \( V_z \) is varied in a range between 0.01 mm/s and 100 mm/s. It becomes obvious that the vertical resistance strongly depends on the pullout rate. The initial stiffness is less influenced than the resistance for larger heaves. To mobilize the ultimate capacity, very large heaves are necessary. In the case of pullout rates \( V_z = 10 \text{ mm/s} \) and \( V_z = 100 \text{ mm/s} \), the maximum resistances were not reached even for a heave \( z = 200 \text{ mm} \). It shall be noted that, due to distortions of the finite element mesh, a further increase of the heave is hardly possible.

The corresponding resistance from suction pressure is presented in Fig. 7. Almost no suction resistance occurs for the slowest pullout rate, which denotes a behavior under nearly drained conditions. This can be confirmed by the load-displacement curve that indicates a bilinear course typical for load transfer by skin friction only. For larger pullout rates, the suction resistance increases similarly to the total resistance. As the suction pressure is constant for the whole area of the bucket lid (see Fig. 8), the same diagram can be used for presentation of occurring suction pressures. The knowledge about
the suction pressure is important with regard to the occurrence of cavitation. As stated before, the suction pressure $\Delta u$ is limited by the occurrence of cavitation. Regarding the results presented here, this means that the suction resistance for a water depth $h_w = 0$ m is limited to $V_s = 11.31$ MN and to $V_s = 22.62$ MN for $h_w = 10$ m (see Fig. 7).

The distributions of suction pressure due to uplift for two pullout rates are presented in Fig. 8. For a pullout rate $V_z = 1$ mm/s, the suction pressure mainly dissipates inside the bucket, as it is known from flow net calculations that no soil deformation is considered. For the pullout rate $V_z = 100$ mm/s, a more complex situation of suction pressures occurs, as the soil plug inside the bucket is uplifted with the bucket. This behavior can be explained by Fig. 9 and Fig. 10.

In Fig. 9, the expansion of the gap (respective to the water elements) beneath the bucket lid due to vertical uplift of the bucket is presented. Here, it can be seen that for small pullout rates, the gap is almost completely expanded. The seepage into the bucket is sufficient to compensate for the suction pressure such that it remains so small that no uplift of the inner soil plug occurs. In contrast, for large pullout rates, the gap is almost completely unexpanded, as the seepage is too small.

Consequently, the inner soil plug is uplifted, as already described by Senders (2008) (see Fig. 2). This is visualized in Fig. 10 by the vertical displacements of the soil for a heave $z = 200$ mm due to pullout rates of $V_z = 100$ mm/s and $V_z = 1$ mm/s. For a pullout rate $V_z = 1$ mm/s, only a partial uplift inside and directly beneath the bucket occurs. In contrast, for a pullout rate $V_z = 100$ mm/s, the inner soil plug is raised similarly with the bucket. Moreover, the soil outside the bucket is dragged by the suction pressure beneath the bucket (see also Fig. 2, left), which causes a settlement beside the bucket.

The occurring soil deformations affect the suction pressure distributions (see Fig. 8). For the pullout rate $V_z = 1$ mm/s, the dilation due to uplift occurs almost completely in the gap beneath the bucket lid and in the inner soil plug. Consequently, the suction pressure dissipation occurs mainly in these areas. For a pullout rate of $V_z = 100$ mm/s, almost no dilation occurs in these areas, which causes a much smaller dissipation of suction pressures. The dissipation therefore has to occur in a wider area (see Fig. 10, left). It is remarkable that the soil deformations reduce the effective stresses on the outside of the bucket. This can be seen inversely by an increase in the pore pressure, which compensates for this reduction (see Fig. 8, left).

The occurring suction pressures also affect the magnitude of skin friction on the bucket skirt. In theory, a downwards-directed seepage increases the effective stresses, whereas an upwards-directed seepage decreases them. The outer skin friction is presented for various pullout rates in Fig. 11 (left). For small pullout rates, a drained response is observed, as it was already found for the total resistance. For larger pullout rates, the outer skin friction increases, as expected, due to the downwards-directed seepage. However, the skin friction is smaller for a pullout rate $V_z = 100$ mm/s than for slower pullout rates. This behavior is caused by the soil deformations and the corresponding reduction of effective stresses beside the bucket mentioned earlier. Based on the contact formulation used, this leads directly to a reduction in skin friction. In conclusion, two counteracting effects influence the magnitude of the outer skin friction, depending on the pullout rate.
The skin friction for drained conditions on the inside is somewhat smaller than that on the outside due to the larger effect of reduced effective stresses by the upwards-directed friction stresses and the smaller coverage inside (see Fig. 11, right). For larger pullout rates, the upwards-directed seepage results in a decrease in effective stresses and therefore in skin friction. When the dilation of the gap and the soil inside the bucket becomes smaller with increasing pullout rates, the upwards-directed seepage is reduced, which consequently leads to larger effective stresses. This caused the increase in skin friction for the larger pullout rates. Thus, for the inner skin friction, two counteracting effects occur.

RESULTS FOR THE LOWER BOUND SOLUTION

The load-displacement curves for the lower bound solution are presented in Fig. 12. The results are almost identical to the results from the upper bound solution, with the exception of a pullout rate $V_z = 100 \text{ mm/s}$ (see Fig. 6). Here, an almost identical resistance to the pullout rate $V_z = 10 \text{ mm/s}$ can be obtained.

Likewise, the results for the suction resistance are similar, with the exception of the fastest pullout rate. Here, the suction resistance is larger than that for $V_z = 10 \text{ mm/s}$, but the increase is significantly smaller than what results from the upper bound solution (see Fig. 7).

The occurring differences have to be analyzed in conjunction with the limitation in permeability due to Eq. 3. For this purpose, the distributions of soil permeability for the upper and lower bound solutions are given in Fig. 14. The results are valid for a pullout rate $V_z = 100 \text{ mm/s}$ and a heave $z = 200 \text{ mm}$. It must be noted that the permeability is scaled by the cube root to enable a meaningful presentation. As stated earlier, the initial permeability was set to $k_{init} = 7.74 \cdot 10^{-4} \text{ m/s}$.

Regarding the results for the upper bound solution, the permeability is only increased considerably in a small region below the bucket tip. Here, the loosening of the soil due to uplift of the bucket results in an increase in the void ratio, and also in the permeability to a maximum dimension of about $k = 1.70 \cdot 10^{-3} \text{ m/s}$. For the lower bound solution, the increase in permeability is significantly larger. Here, the permeability is increased to a maximum dimension of $k = 1.67 \cdot 10^{-1} \text{ m/s}$ (216 times the initial value). It is remarkable that the limitation of permeability due to Eq. 3 becomes leading in a wide area beneath the bucket. It is understood that this larger permeability leads to smaller suction pressures, and therefore to a smaller total resistance of the bucket, as it was already presented in Fig. 12 and Fig. 13.

The suction pressure distribution from the lower bound solution for a pullout rate $V_z = 100 \text{ mm/s}$ is shown in Fig. 15 (left). Compared to the suction pressure distribution for the upper bound solution, a much larger suction dissipation inside the bucket is found. This is caused by the greater permeability, which leads to a greater expansion of the water elements beneath the bucket lid and the inner soil plug. Consequently, the behavior becomes more similar to smaller pullout rates.

The distributions of skin friction for the lower bound solution are presented in Fig. 16. Here, the outer skin friction for all introduced pullout rates is almost identical to the upper bound solution. The distribution for the fastest pullout rate $V_z = 100 \text{ mm/s}$ is almost not at all influenced, as two counteracting influences occur. On the one hand, the downwards-directed seepage is reduced due to the larger pore pressure dissipation inside the bucket. On the other hand, the expansion of the gap and the inner soil plug becomes smaller, which causes a smaller decrease in effective stresses and therefore of skin friction.
Whereas for the upper bound solution, the skin friction is increasing with the pullout rate. Here, the skin friction becomes zero for large pullout rates. Again, this behavior is caused by the increasing suction dissipation inside the bucket due to the larger permeability. The larger dissipation leads to a larger decrease in effective stresses. In Fig. 15 (right), the distribution of effective stresses is presented for a pullout rate \( V_z = 100 \text{ mm/s} \) and a heave \( z = 200 \text{ mm} \). It can be seen that the stresses become very small inside the bucket and directly beneath it. For the upper bound solution, considerable stresses can be found for identical conditions.

The presentation of effective stresses also indicates that the limitation of permeability (see Fig. 14) becomes leading in a region where the effective stresses are by no means zero. The effective stresses would prevent soil liquefaction and therefore an increase in permeability as implemented for the lower bound solution in this area.

**EVALUATION OF SOLUTIONS**

First, it should be noted that the results presented here are valid for one specific bucket soil system. Differing bucket dimensions will significantly change the influence of the suction on the load-bearing behavior for a certain pullout rate. In this context, an extended parametric study based on the upper bound solution was published by Thieken et al. (2014). However, it is assumed that the qualitative effects regarding the consideration of soil liquefaction can be captured by taking into account one system only.

Based on the results achieved, it can be concluded that the actual load-bearing behavior is in between the presented upper and lower bound solutions. It is not known to what extent the soil particles get freed, and therefore to what extent soil liquefaction will actually occur. However, it is remarkable that the influence of the soil liquefaction becomes relevant only in cases of very large pullout rates in conjunction with very large heaves. Therefore, it has to be considered that the dilation in soil occurring for large pullout rates is rather small due to the approximation for an undrained behavior. Based on these findings, it is believed that the upper bound solution will be more accurate than the lower bound solution. Nevertheless, further investigations (especially experimental) must be performed to clarify to what extent the particles get freed during uplift.

**CONCLUSIONS**

Suction buckets are a promising foundation concept for offshore wind energy converters. A numerical model is presented that is able to account for the partly drained load-bearing behavior of a single suction bucket under tensile loading conditions. The following main conclusions can be drawn from the results of the numerical simulations:

- An increase in the pullout rate leads to a significant increase in the tensile resistance. To mobilize a considerable portion of this additional resistance, a large heave of the bucket is necessary.
- For small pullout rates, drained behavior occurs, whereas for larger pullout rates, suction pressures inside the bucket arise. At very large pullout rates, the vertical resistance is limited by the occurrence of cavitation or the undrained soil behavior, whichever is reached first.
- In partly drained cases, the distribution of the total resistance to inner and outer skin friction and suction forces is strongly dependent on pullout rate and the current absolute heave value. A strong and complex interaction of suction pressure, bucket heave, soil deformation, and skin friction is observed.
- The consideration of soil liquefaction is challenging due to the modeling of the actual permeability. Two solutions, which are assumed to be an upper and a lower bound solution, are compared. It can be shown that the influence is limited to large (probably inadmissible) heaves of the bucket and very large pullout rates. It is stated that the upper bound solution is believed to be more accurate than the lower bound solution.

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