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a literature review

Foglia, Aligi; Ibsen, Lars Bo

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by

Aligi Foglia
Lars Bo Ibsen

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Bucket foundations: a literature review

Aligi Foglia and Lars Bo Ibsen

Department of Civil Engineering, Aalborg University

In this report, bearing behaviour and installation of bucket foundations are reviewed. Different methods and standards are compared with the experimental data presented in Foglia and Ibsen (2014a). The most important studies on these topics are suggested. The review is focussed on the response of monopod bucket foundations supporting offshore wind turbines.

1 Introduction

Settlements and bearing capacity of shallow foundations have been studied for over one century and yet many issues are still to be addressed and resolved. This technical report covers some of the fundamental topics that were experimentally and/or theoretically explored throughout the experimental campaign conducted by Foglia and Ibsen (2014a). This literature review compares different approaches and, when relevant, the comparison is integrated with the experimental results collected in Foglia and Ibsen (2014a).

The bearing capacity of rigid flat footings is the necessary starting point to understand the response of bucket foundations under general loading. The focus is then shifted towards the bearing capacity of bucket foundations, as these are the main object of the experimental work (Foglia and Ibsen, 2014a). Two methods are used to predict the bearing capacity of the experi-

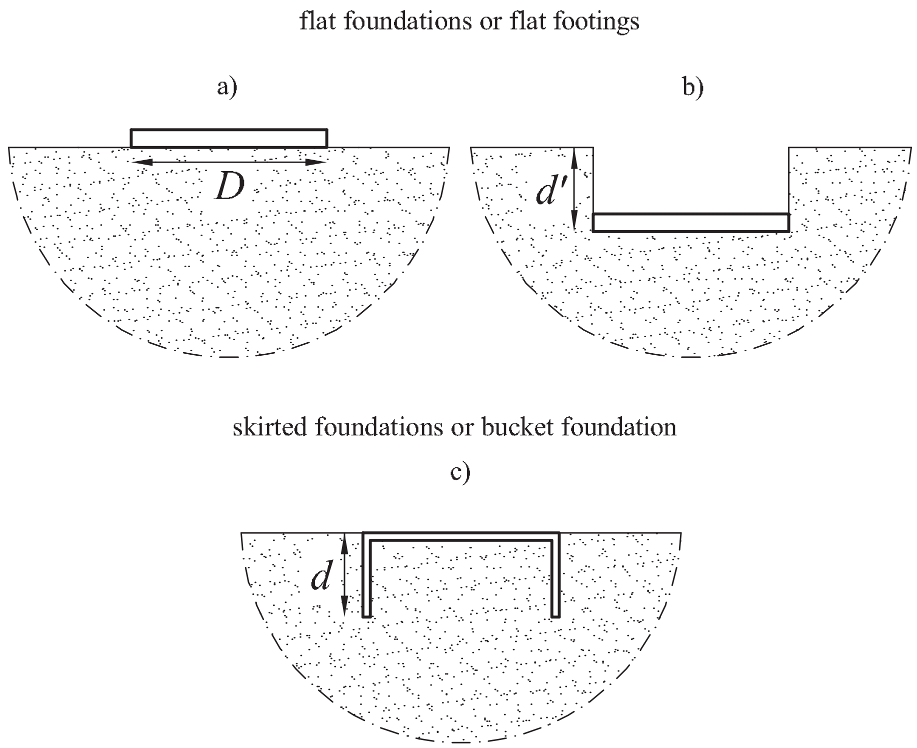

Figure 1: a) standard flat footing; b) buried or embedded footing; c) skirted foundation or bucket foundation

mental tests. Innovative and more traditional methods to evaluate the bearing capacity of bucket foundations under general loading are discussed. The installation process is described and three methods are used to interpret the jacked installation of a small-scale foundation.

Figure 1 illustrates the types of shallow foundations examined in this study. Throughout the report, the terms bucket foundation and skirted foundation are used interchangeably.

2 Bearing capacity under vertical loading

2.1 Flat footings

Shallow foundations under pure vertical loading are traditionally designed on the base of the classic bearing capacity theory proposed by Terzaghi (1943). For a flat embedded footing with width, D, and area, $A = DL$, the bearing capacity can be expressed as:

$$q_u = \frac{V_\mathrm{u}}{A} = cN_c s_c + qN_q + 0.5\gamma' D N_\gamma s_\gamma \tag{1}$$

where N_c, N_q and N_γ are the bearing capacity factors, c is the cohesion of the material, q is the surcharge ($q = \sigma'_\mathrm{v}(d') = \gamma' d'$; where d' is the depth of excavation), γ' is the effective unit weight of the soil and s_c and s_γ are the shape factors that account for rectangular and circular shapes of the foundation. For most of the authors, the shape factors are functions of D, L, and, for some calculation methods (Brinch Hansen, 1970; Vesić, 1973), also of the friction angle, ϕ'. Circular and square footings have $D = L$ and thus their shape is considered to affect the bearing capacity in the same manner (CEN, 2004; Fang, 1991).

By multiplying q_u by the area of the foundation, the ultimate vertical load of the footing, V_u, can be obtained. In practice, equation 1, uncouples and superimposes the three terms influencing the bearing capacity. The solution proposed by Terzaghi (1943) is based on the work conducted by Prandtl (1920) who adopted the theory of plasticity to analytically solve the problem of a rigid body penetrating into a granular material. The bearing capacity factors are by definition functions of the friction angle and, after Terzaghi (1943), many authors have proposed new formulations for their estimation (Meyerhof, 1963; Brinch Hansen, 1970; Vesić, 1973). Among the authors there is general agreement about the value of the factors N_c and N_q. On the contrary, N_γ can vary significantly, especially for friction angles larger than 40° (Bowles, 1996). Meyerhof (1963) Brinch Hansen (1970) and Vesić (1973) propose also that the depth factors, d_c, d_q and d_γ, and one further shape factor, s_q, are to be included in equation 1. Though, the depth factors are not included in current standards (CEN, 2004; DNV, 2014).

More recently, Bolton and Lau (1993) and Martin (2005) have used the method of characteristics to obtain the exact value of the bearing capacity factors for strip and circular footings with rough and smooth interface. In Bolton and Lau (1993) and Martin (2005) the depth and shape factors

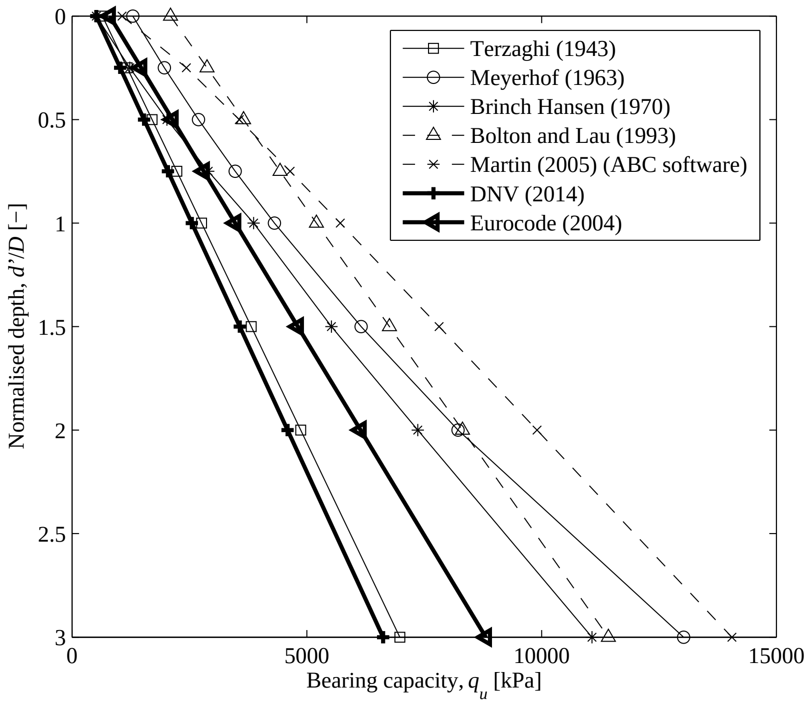

Figure 2: Estimation of the bearing capacity of a flat circular footing with seven different methods

are not evaluated since the bearing capacity factors obtained with the method of characteristics already embrace the effects of shape and depth. Exact values of the vertical bearing capacity of shallow foundations can be obtained with the software ABC developed by Martin (2003) and based on the method of characteristics. Houlsby and Martin (2003) used the same method to estimate the bearing capacity factors of spudcan foundations on clays considering the effects of embedment, roughness, strength heterogeneity and cone angle.

In Figure 2 the evaluation of the bearing capacity of a circular foundation ($D = 5$ m) on sand ($\phi' = 35°$) with seven different methods is illustrated. A rough soil-footing interface is chosen for the estimation. In Figure 2 it can be observed that the bearing capacity equation given by DNV (2014) seems to be the most conservative. Furthermore, depending on the normalised depth, the approaches of Martin (2005) and Bolton and Lau (1993) give the largest value of q_u.

2.2 Skirted foundations

As mentioned by Villalobos (2006), when the ultimate vertical load of a bucket foundation, V_s, is being investigated, multiple issues emerge. For example, the soil plug inside the foundation can be assumed to be rigid or flexible. If the soil plug is assumed to act as a rigid block, the bearing capacity is calculated at the level of embedment ($d = d'$; where d is the length of the skirt):

$$\frac{V_\mathrm{s}}{A} = qN_q d_q s_q + 0.5\gamma' D N_\gamma d_\gamma s_\gamma \tag{2}$$

Equation 2 is written for a skirted foundation in non-cohesive soil.

Clearly, assuming rigid skirt and flexible soil plug would be more realistic. In case of pure vertical loading though, the result would not change dramatically. Conversely, in case of combined loading, Bransby and Yun (2009) showed that due to a failure mechanism inside the skirt, the capacity of skirted foundations with flexible soil plug could be significantly lower than that of solid embedded foundations. For this reason, as recommended in Randolph and Gourvenec (2011), internal skirts should be included in the bucket foundation design to ensure a non-flexible soil plug.

Another issue is related to the effect of installation on the volume of material surrounding the foundations. This aspect is discussed in Chapter 4.

The contribution of the friction on the outer surface of the skirt should also be taken into account. A straightforward estimation of the skin friction resistance, V_f, can be obtained by integrating a constant shear stress, τ_o, over the skirt length d:

$$V_\mathrm{f} = 2\pi R \int_0^d \tau_\mathrm{o} dz = \pi R \gamma' K tan(\delta) d^2 \tag{3}$$

where τ_o is the shear stress on the outer surface of the skirt, R is the outer radius of the bucket, K is the lateral earth pressure coefficient and δ is the interface friction angle.

In an attempt to estimate the vertical bearing capacity of bucket foundations, small-scale vertical loading tests until failure were carried out at different scales and on different sands by Villalobos (2006) and Larsen (2008). Villalobos (2006) run displacement controlled vertical loading tests of buckets with D = 50.9 mm and with seven different embedment ratios (d/D from 0 to 2), on loose and dense sand samples. As expected, he found punching shear mechanism for the loose samples and general shear mechanism for the dense samples. He interpreted his results with the bearing capacity equation:

$$V_{\mathrm{s}} = D\pi \int_0^d \tau_0 dz + A\left(qN_q + 0.5\gamma' DN_\gamma\right) \tag{4}$$

where it was assumed K = 2 and δ= 16°. N_q and N_γ were calculated for smooth interface according to Bolton and Lau (1993) and to Martin (2005), respectively. He found that by using the peak friction angle, the estimation of V_{s} overestimates the experimental results for both loose and dense sample.

Larsen (2008) carried out several vertical loading tests of buckets with diameter varying between 50 and 200 mm and four different embedment ratios (d/D from 0 to 1). Larsen (2008) calculated V_{s} as a linear function of d/D and V_{u}:

$$\frac{V_{\mathrm{s}}}{V_{\mathrm{u}}} = 1 + c\left(\frac{d}{D}\right) \tag{5}$$

Larsen (2008) estimated the parameter c as 2.9 while the bearing capacity factors for V_{u} were deducted according to Martin (2005). Equation 5 was first put forward by Byrne and Houlsby (1999) who estimated c as 0.89.

In Foglia and Ibsen (2014a) the results of two vertical loading tests until failure performed with a novel experimental rig, are presented. A detailed description of the test setup is given in Vaitkunaite et al. (2014). Two buckets with D = 300 mm were tested. One foundation had d/D = 1 (test S64) and the other had d/D = 0.75 (test S63). It is worth to emphasise that, given the dimension of the foundations tested, laboratory tests of such a kind are rare. The

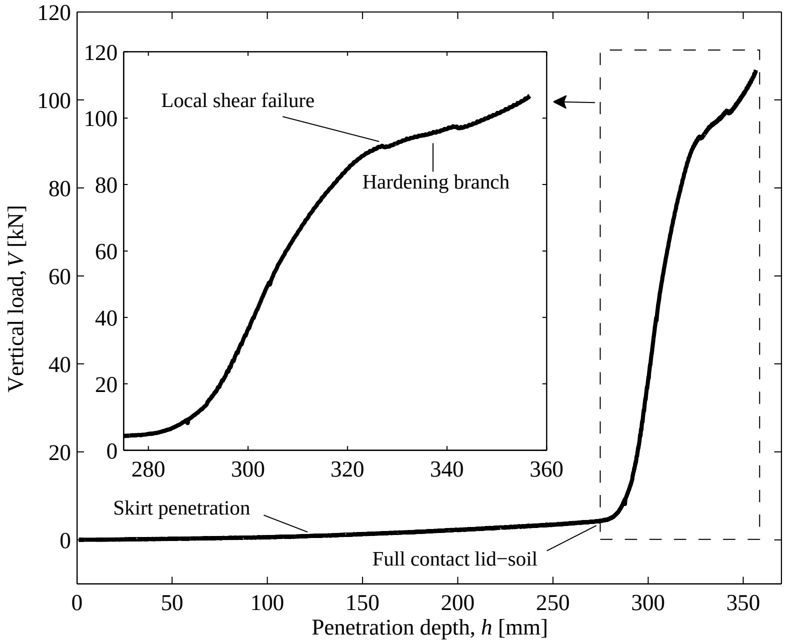

Figure 3: Installation and bearing capacity test until failure, test S64

relative density, D_r, of the sand sample was estimated with a small-scale cone penetration test as 77%. The $V - h$ curve of test S63 is shown in Figure 3, where h is the penetration depth of the foundation. In the figure, the part of the curve after the full contact lid-soil (full skirt penetration) is shown in a magnified inner plot. The entire curve can be divided in two different parts. In the first part the increase in V is due only to the skirt resistance. This part of the curve is, in reality, the jacked installation phase, which is analysed in section 4.2. Once full contact lid-soil (full skirt penetration) is established, the penetration curve has a sudden stiffness increase caused by the lid which becomes the predominant bearer. According to Vesić (1973), the soil supporting a footing under vertical load can fail following three mechanisms: general shear, local shear and punching shear. Figure 3 clearly shows that no general shear failure of the soil occurred. During the test, soil bulging was observed meaning that the soil around the foundation (unloaded soil) was visibly involved in the failure mechanism. According to this observation a local shear failure of the soil appears to have occurred (Vesić, 1973). As already mentioned, general failure was reported by Villalobos (2006) in all the tests on dense sand ($D_r = 88\%$ and $D_r = 83\%$). This difference in failure mechanism can be attributed to the different scale of the physical models or to the discrepancy in relative density.

The ultimate bearing capacity gained with S63 and S64 is plotted in Figure 4 together with equation 4 and equation 5. The critical friction angle of the sand used in the test is reported in Larsen (2008) to be equal to $\phi_{cr} = 31°$. According to Bolton (1986) that would give a peak friction angle, ϕ_{peak}, of 39.6°. In Figure 4, it can be seen that equation 4 captures very well the bearing capacities trend with an unexpectedly high value of the friction angle, ϕ'= 45°. Equation 5, with the empirical parameter c proposed by Larsen (2008) and a friction angle close to the critical one (ϕ'= 39°), predicts the result of test S63 but overestimates the bearing capacity of test S64.

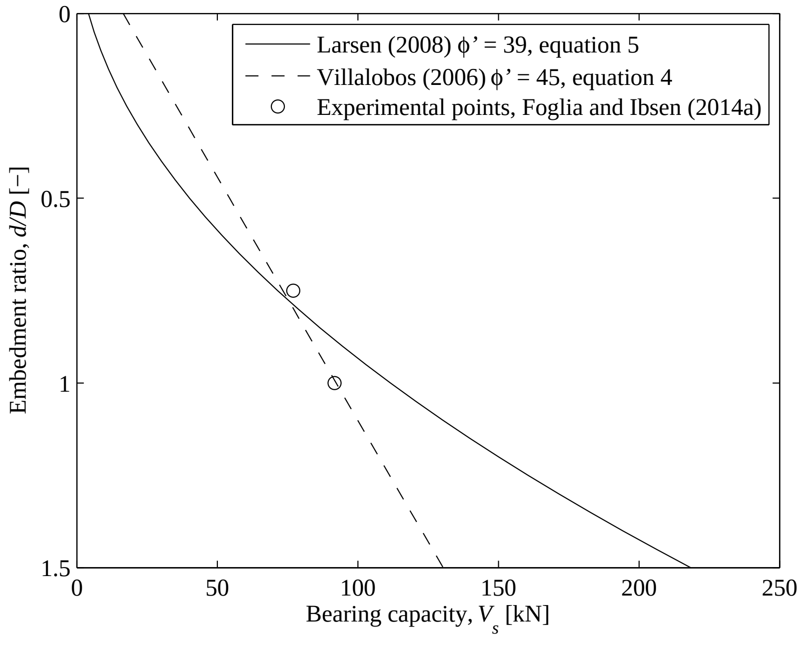

Figure 4: Bearing capacity of bucket foundations estimated with two different methods and experimental results

3 Bearing capacity under general loading

3.1 Flat footings

While most onshore foundations are characterised by predominant vertical loading, V, offshore foundations must withstand general loading with significant components of, horizontal load, H, and moment, M. Well-established design criteria for onshore foundations are not always suitable for offshore systems. For instance, the ultimate bearing capacity of shallow foundations for onshore systems is often unlikely to occur. Conversely, the ultimate bearing capacity of offshore structures (and particularly that of offshore wind turbines) could be breached owing to exceptionally large overturning moments, and cannot therefore be overlooked.

Following the classic bearing capacity theory, when a shallow foundation is subjected to general loading conditions, an array of empirically derived coefficients reduces V_{u}. For flat footings on sand under pure vertical loading, equation 1 becomes:

$$\frac{V_{\mathrm{u}}}{A} = 0.5\gamma' D N_\gamma s_\gamma \tag{6}$$

If the foundation is subjected to general loading, the effect of M is taken care of by reducing the foundation area as a function of the eccentricity induced by the overturning moment ($e = M/V$). Besides, the effect of the horizontal load is introduced through the inclination factor i_γ. As a result of that, the ultimate vertical load of flat foundations on sand under general loading is calculated as:

$$\frac{V_{\mathrm{gu}}}{A'} = 0.5\gamma' D N_\gamma s_\gamma i_\gamma \tag{7}$$

where A' is the effective foundation area calculated as a function of e. A number of authors attempted the assessment of the i_γ coefficient by using analytical and empirical methods. Gottardi (1992) conducted a detailed review of the different expressions proposed in literature. The most used coefficients in engineering practice are those of Meyerhof (1953) and Brinch Hansen

(1970). According to Meyerhof (1953) the inclination factor can be written as:

$$i_\gamma = \left(1 - \frac{\theta}{\phi'}\right)^2 \tag{8}$$

where θ is the angle of inclination of the resultant force, $\theta = arctan(H/V)$. The expression of Brinch Hansen (1970) for i_γ, does not include the friction angle and is written as:

$$i_\gamma = \left(1 - 0.7\frac{H}{V}\right)^5 \tag{9}$$

Similarly, DNV (2014) expresses i_γ as:

$$i_\gamma = \left(1 - \frac{H}{V}\right)^2 \tag{10}$$

Note that Meyerhof (1953) includes the friction angle in the definition of the inclination factor. By using these traditional approaches, the non-linearity of the geotechnical problem, which is rather significant for general loading, is simplistically considered through a superposition of different effects. To reflect properly the non-linearity of the system and consider directly the interaction between V,H and M, interaction diagrams (or failure envelopes) were conceived. Interaction diagrams encompass a region of the three-dimensional load space within which the foundation does not violate the failure criterion. Roscoe and Schofield (1956) and Butterfield and Ticof (1979) were pioneers of this technique which is used today as fundamental element for macro-models (Gottardi et al., 1999; Cremer et al., 2001; Houlsby and Cassidy, 2002; Bienen et al., 2006).

Expressions of the $H-V$ interaction from the inclination factors of Meyerhof (1953), Brinch Hansen (1970) and DNV (2014) can be simply obtained by including i_γ in the bearing capacity formula and expressing H as a function of V (Gottardi, 1992; Byrne, 2000). In Figure 5 the experimentally deduced interaction diagrams of Butterfield and Gottardi (1994) and Houlsby and Cassidy (2002) (Model C) are plotted together with the classic bearing capacity methods.

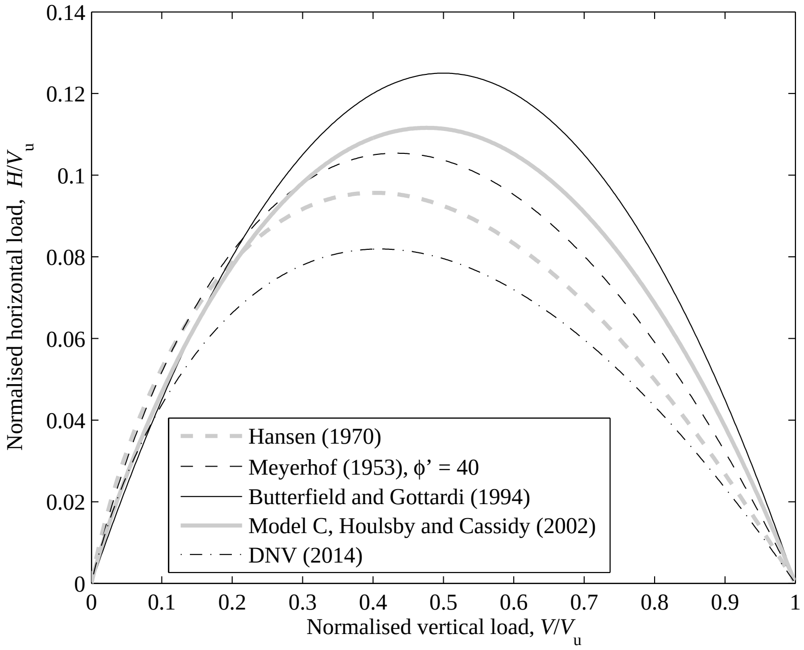

Figure 5: Comparison of different interaction diagrams for flat footings in the normalised load plane

As similarly pointed out by Byrne (2000), the classic methods of Meyerhof (1953) and Brinch Hansen (1970) are conservative for $V/V_{\mathrm{u}} > 0.3$. More importantly, the four envelopes are alike for $V/V_{\mathrm{u}} < 0.3$. Note that this is also the region of the load space relevant for offshore wind turbines. It is also worth to note that the DNV (2014) method gives the most conservative failure envelope and agrees with the other curves only for $V/V_{\mathrm{u}} < 0.1$.

Even though the interaction diagrams appear to agree with the traditional methods in the region of interest, their importance is undeniable. In fact, they form the base of macro-models and are thereby essential to model sophisticated problems regarding the interaction between soil, foundation and superstructure. An analogue plot to Figure 5 could be obtained also for M. Though, the envelopes of Meyerhof (1953), Brinch Hansen (1970) and DNV (2014) would be equal as they all use the same approach to account for the presence of M.

Failure envelopes have been lately incorporated in the API standards (API, 2011). Other well-known failure envelopes for shallow foundations are: Saleçon and Pecker (1995), for footings on clay; Martin and Houlsby (2000), for spudcan foundations on clay; Byrne and Houlsby (2001), for footings on carbonate sand; Randolph and Puzrin (2003), for circular foundations on clay (upper bound solution); Bienen et al. (2006), for footings in six degrees of freedom.

3.2 Skirted foundations

The same principle explained for flat footings is applicable to skirted foundations as well. When a skirted foundation on sand is subjected to general loading, the sustainable vertical load, V_{gs}, can be evaluated as:

$$\frac{V_{\mathrm{gs}}}{A'} = qN_q i_q s_q d_q + 0.5\gamma' D N_\gamma i_\gamma s_\gamma d_\gamma \tag{11}$$

According to Meyerhof (1953) the inclination factor, i_q, can be written as:

$$i_q = \left(1 - \frac{\theta}{90^\circ}\right)^2 \tag{12}$$

The equation of Brinch Hansen (1970) for i_q is:

$$i_q = \left(1 - 0.5\frac{H}{V}\right)^5 \tag{13}$$

The DNV (2014) recommends that i_q is calculated according to:

$$i_q = \left(1 - \frac{H}{V}\right)^4 \tag{14}$$

Since the surcharge component increases the degree of non-linearity of the problem, closed analytical solutions for H to plot the interaction diagram for the methods of Meyerhof (1953), Brinch Hansen (1970) and DNV (2014), cannot be obtained for skirted foundations. Numerical solutions are however obtainable and these are shown in Figure 6 together with the experimentally derived failure envelope of Ibsen et al. (2014).

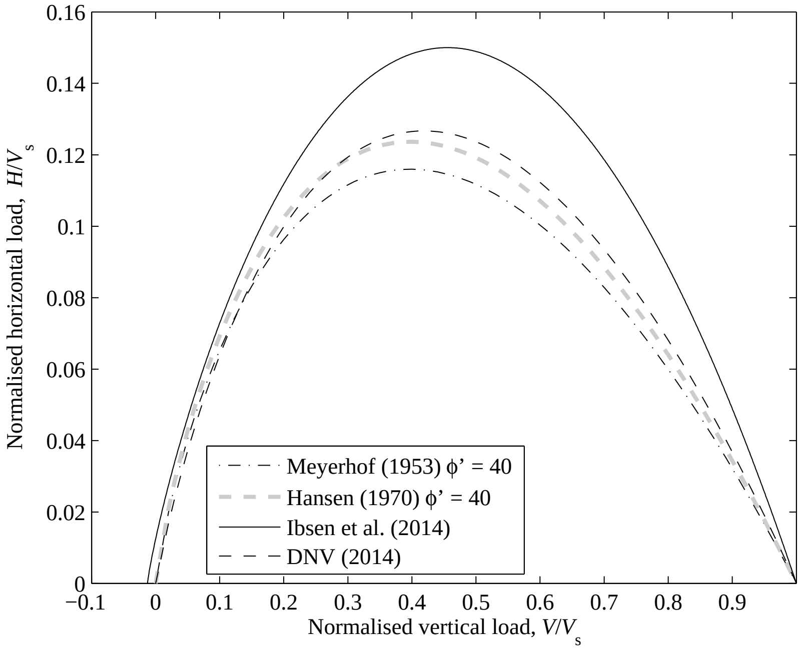

Figure 6: Comparison of different interaction diagrams for skirted foundations in the normalised load plane

A yielding surface for bucket foundations was experimentally investigated by Villalobos (2006) (see also Villalobos et al. (2009)). The ellipsoid extrapolated by Villalobos (2006) has equation:

$$f = \left(\frac{H}{V_0 h_0}\right)^2 + \left(\frac{M}{DV_0 m_0}\right)^2 - 2e_0 \frac{H}{V_0 h_0}\frac{M}{DV_0 m_0} - \beta_{12}^2 \left(\frac{V}{V_0} + t_0\right)^{2\beta_1} \left(1 - \frac{V}{V_0}\right)^{2\beta_2} \tag{15}$$

where V_0 is the preconsolidation vertical load, t_0 is the tension parameter ($t_0 = V/V_0$), h_0, m_0, e_0, β_1 and β_2 are the non-dimensional parameters and β_{12} is defined as:

$$\beta_{12} = \frac{(\beta_1 + \beta_2)^{(\beta_1+\beta_2)}}{\beta_1^{\beta_1}\beta_2^{\beta_2}(t_0 + 1)^{(\beta_1+\beta_2)}} \tag{16}$$

Ibsen et al. (2014) (see also Larsen, 2008) proposed a failure envelope on the base of the yielding surface of Villalobos (2006). The failure envelope of Ibsen et al. (2014) has the form of equation 15 but with V_s instead of V_0. In this report we are interested in the ultimate resistance of the foundation and the envelope proposed by Ibsen et al. (2014) is therefore used.

Note that, in the legend of Figure 6, the friction angle is indicated also for Brinch Hansen (1970). This is because the d_γ proposed by Brinch Hansen (1970) depends on ϕ'. Instead, as mentioned earlier, depth factors are not included in the formulation of DNV (2014). In Figure 6 it is seen that, for skirted foundations, the three classic bearing capacity approaches give a rather similar representation of the failure load. In a similar fashion to flat footings, the failure envelope derived experimentally gives the largest prediction of bearing capacity. As in Figure 5, in the relevant region for offshore wind turbines, all the methods predict a similar bearing capacity. The classic methods seem to be particularly conservative for $0.3 < V/V_\mathrm{s} < 0.9$.

Eight monotonic tests until failure of a bucket foundation with $d/D = 1$ and $D = 300$ mm, are presented in Foglia et al. (2014). The tests were conducted with $V/V_\mathrm{s} = 0.0026$ and with five different $M/(HD)$ ratios. The failure points of this test series are represented in Figure 7 together with the interaction diagram of Ibsen et al. (2014).

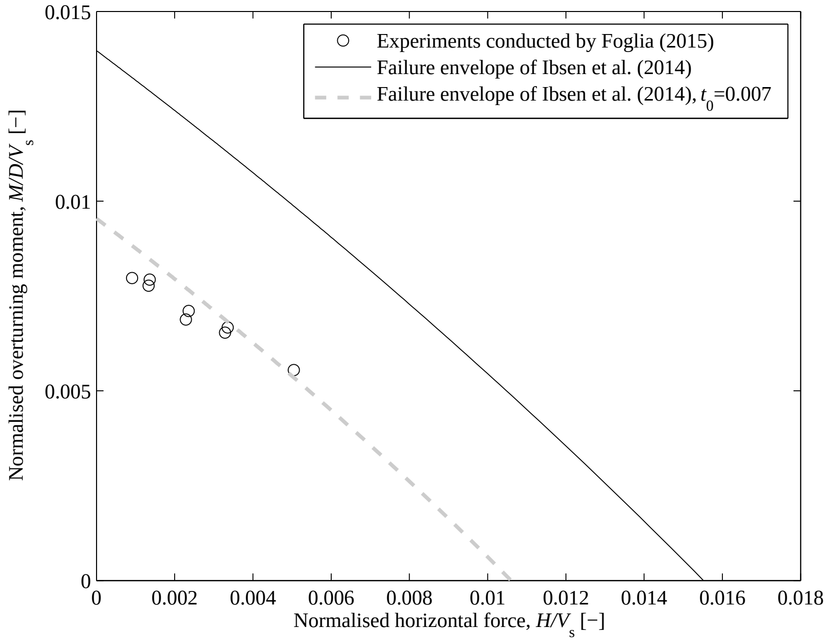

Figure 7: Experimental results of a bucket foundation $(d/D = 1)$ against the original and the modified interaction diagram of Ibsen et al. (2014)

The experimental points are overestimated by the failure envelope. This is attributed to the fact that the failure envelope of Ibsen et al. (2014) was calibrated only over tests with $V/V_{\mathrm{s}} = 0.5$. As shown in Foglia et al. (2014), by setting $t_0 = 0.007$ the curve matches well the experimental results. The choice of adapting the failure surface by changing $t_0 = 0.007$ is not randomly made. t_0 is in fact a rather straightforward parameter to be evaluated as explained in Foglia et al. (2014).

Recently, another interaction diagram on the $(M - H)$ load plane has been numerically derived in Achmus et al. (2013a). The numerical simulations were calibrated against large scale tests. According to Achmus et al. (2013a) the normalised ultimate horizontal load in very dense sand can be expressed by:

$$\left(\frac{H_{\mathrm{u}}}{\gamma' d^2 D}\right) = -0.011 \left(\frac{d}{d_{\mathrm{ref}}}\right) (M'_{\mathrm{u}})^2 - 0.43 \left(\frac{d}{d_{\mathrm{ref}}}\right)^{0.2} M'_{\mathrm{u}} + 14.1 \left(\frac{d_{\mathrm{ref}}}{d}\right)^{0.6} \tag{17}$$

where d_{ref} is a reference embedment length equal to 1 m and M'_{u} is expressed by:

$$M'_{\mathrm{u}} = \left(\frac{M_{\mathrm{u}}}{\gamma' d^3 D}\right) \left(\frac{d}{d_{ref}}\right)^{0.8} \tag{18}$$

In a similar way, H'_u is defined as:

$$H'_{\mathrm{u}} = \left(\frac{H_{\mathrm{u}}}{\gamma' d^2 D}\right) \left(\frac{d}{d_{ref}}\right)^{0.6} \tag{19}$$

The failure envelope expressed by equations 17-19 can be compared with the envelope of Ibsen et al. (2014). In order to obtain M'_{u} and H'_{u} values from the failure criteria of Ibsen et al. (2014), it is necessary to estimate the vertical bearing capacity of the bucket foundation, V_{s}. The foundation considered for the calculation has $D = 16$ m, $d = 12$ m and is subjected to $V = 20$ MN. V_{s} is calculated with the software ABC in a non-cohesive soil with $\gamma' = 10$ kN/m and for three values of the friction angle. The comparison is shown in Figure 8.

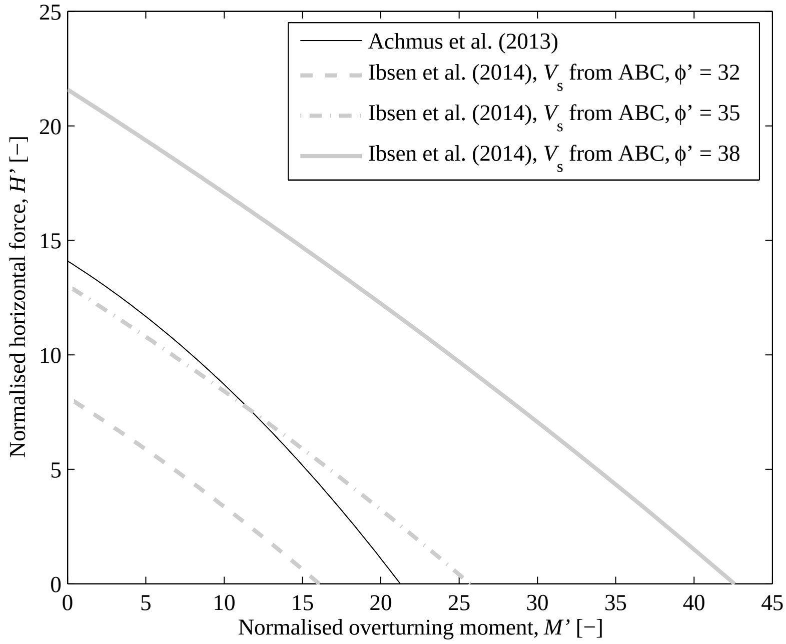

Figure 8: Interaction diagrams of Ibsen et al. (2014) against failure criteria of Achmus et al. (2013a)

Note that the axes of Figure 8 are H' and M'. These are defined as equations 18 and 19 but with H and M instead of H_{u} and M_{u}. The curves shown in Figure 8 from Ibsen et al. (2014) are quite influenced by the choice of V and by the type of soil. In spite of this, it is remarkable that for $\phi' = 35°$ the two methods give similar predictions.

Beside the failure envelope, Achmus et al. (2013a) formulated an expression for the initial stiffness. Furthermore, the numerical simulations revealed an interesting feature of the bearing behaviour: when a bucket foundation approaches failure, a gap between lid and soil occurs. This detachment between soil and structure induces the skirt to bear all the load. The latter information is crucial and would technically implicate that the traditional bearing capacity methods are inadequate instruments to evaluate the capacity of bucket foundations under predominant general loading. Nevertheless, from Figure 6 it is clear that these methods give a fairly similar result to small-scale experiments.

3.3 Additional literature

Failure envelopes Other failure envelopes for skirted foundation can be found in: Mangal (1999), exploration of the foundation behaviour in partially drained conditions; Bransby and Randolph (1998), Bransby and Yun (2009), Gourvenec (2007) and Gourvenec and Barnett (2011), investigation on combined loading of bucket foundations in undrained condition with numerical and analytical methods; Cassidy et al. (2006), development of a plasticity model for skirted foundations in clay.

Monopod bucket foundations for offshore wind turbines Since the monopod bucket foundation has been considered a cost-competitive option for offshore wind turbine sub-structures (Ibsen, 2008), great attention has been given to the cyclic lateral response of skirted foundations. The main publications on this topic are: Kelly et al. (2006), field tests compared with $1g$ laboratory tests; Achmus et al. (2013b), numerical simulations; Zhu et al. (2013) and Foglia and Ibsen (2014b), $1g$ physical models. Interesting are also the contour diagrams for suction bucket under lateral loading foundations in silt extrapolated by Watson and Randolph (2006) on the base of centrifuge experiments.

Tensile capacity, offshore wind turbines Jacket sub-structures supporting offshore wind turbines can be founded on driven piles or bucket foundations. The load transferred to the foundations is in this case axial, in tension and compression. Bucket foundations for jacket sub-structures have been widely investigated. Feld (2001) performed small-scale 1*g* tensile loading tests with different loading rates. These tests were compared to numerical models and a simple analytical model. The tensile capacity was found to be greatly influenced by the loading rate. Byrne and Houlsby (2002) undertook $1g$ cyclic and monotonic tensile loading tests. To model the appropriate drainage time, a viscous pore fluid was chosen to saturate the soil sample. The experiments revealed that the rate-dependency becomes significant only at large displacements. Centrifuge tests exploring monotonic and cyclic uplift of bucket foundations were carried out by Senders (2008) who also developed a theoretical model to calculate the pull-out resistance. Interestingly, he observed that unless the cyclic magnitude exceeds the frictional resistance, cyclic degradation does not occur. Very recently, Thieken et al. (2014) have reported a number of numerical simulations of bucket foundations under transient tensile loading. In terms of rate-dependency and sustained loading (equivalent to cyclic loading in this case), the simulations corroborated what was found experimentally by previous studies. Thieken et al. (2014) also found that, as opposite to the drained up-lift capacity (frictional resistance), lid and skirt are equally involved in the partially undrained resistance. Pullout field tests on clay and on sand are respectively presented in Houlsby et al. (2005) and Houlsby et al. (2006).

Bucket foundations for oil and gas platforms Bucket foundations have been mostly used as foundations for jacket structures supporting oil and gas platforms or as anchoring systems for tension leg platforms or floating platforms. Bucket foundations for floating platforms and tension leg platforms are often named suction anchors as their embedment length is larger than the diameter.

According to the type of sub-structure or mooring system (jacket, catenary, taut line) the foundations are subjected to different loading conditions. For jackets and for mooring systems in vertical configuration, the tensile loading governs the foundation design. Experimental tests on tensile loading were overtaken for example by: Wang et al. (1977), breakout capacity in three different soils; Steensen-Bach (1992), monotonic loading in clay and sand; Andersen et al. (1992), pull-out capacity method based on laboratory tests and validated against field tests; Clukey et al. (1995), centrifuge study on monotonic and static tensile resistance in clay; Whittle

et al. (1998), static and sustained loading in clay; El-Gharbawy and Olson (1998), monotonic and cyclic loading in clay.
When floating platforms are connected to the seabed through taut lines, the suction anchor is subjected to combined horizontal load and vertical load in tension. Instead, in case catenary moorings are adopted, the suction anchors have to withstand horizontal load only. Early studies on these issues are Hogervost (1980) and Larsen (1989). More recently, Andersen et al. (2005) wrote a compendium on design and analysis of suction anchors in clay. Supachawarote et al. (2004) run numerical simulations of suction anchors in clay deriving the failure envelope in the $(V - H)$ load plane identifying the optimum load attachment position.
The knowledge contained in these papers will perhaps turn out to be valuable when designing anchoring systems for floating offshore wind turbines or wave energy devices.

4 Installation

4.1 Bucket installation by suction

The first documents on the installation of bucket foundations have been published more than half a century ago (Goodman et al., 1961; Sato, 1965). One of the first offshore structures supported by skirted foundations is Gullfaks C (Tjelta et al., 1988). This was a very heavy structure to be installed in relatively soft soil. In order to avoid a significant enlargement of the foundation area, concrete skirts of 22 m were provided to the structure. To prove the penetrability of long concrete skirts, large-scale tests of two steel cylinders connected through a concrete panel were performed (Tjelta et al., 1986). To help the consolidation process this structure was provided with an active drainage system consisting of filters mounted on the skirt wall. Information on the monitoring campaign regarding Gulfaks C is given in Tjelta et al. (1992).
As explained in dedicated sections in Lesny (2011) and Randolph and Gourvenec (2011), the installation of bucket foundations can be divided into two main phases. The first phase consists

of self-weight penetration into the superficial layer of the seabed. The penetration achievable during this installation stage depends on the properties of the soil and on the weight of the upper structure. In the second phase, a pumping system pumps out water from inside the bucket creating suction (or under pressure). Frequently, to ensure a fully controlled penetration, the suction is combined with water injection at the skirt tip. A comprehensive study on this technique was undertaken by Cotter (2010). The suction applied within the foundation produces two phenomena: seepage flows around and inside the bucket and differential pressure acting on the lid. In soils with low permeability (fine grained), the decisive effect is the differential pressure. In soils with high permeability, the action of the seepage flows is predominant. Seepage flows are directed towards the lid within the soil plug and towards the skirt tip in the soil surrounding the foundation. In addition, the seepage flows reduce significantly the end bearing resistance of the skirt tip. Evidence of this effect is given for instance in Bye et al. (1995) and Tjelta (1994) where, previous to the installation of the Europipe 16/11-E Riser jacket, field tests on a steel cylinder were performed.

As underlined by Tjelta (2014), many issues could be encountered during the installation of bucket foundations. According to Tjelta (2014), possible problems during the installation phases could relate to soil limitations, structural limitations or pumping system limitations.

Soil limitations are mainly two: soil plug heave and piping channels. When the under pressure is applied to permeable soils, piping channels will occur if the critical hydraulic gradient is exceeded. Soil plug heave, instead, may occur in fine grained soils if the under pressure is larger than the resistance of the soil plug. A simple method to estimate the maximum under pressure allowed before soil plug heave, is described in Randolph and Gourvenec (2011).

Structural limitations concern strength of the top plate, buckling of the shell and buckling of the top plate. The effect of geometric imperfections on buckling is analysed in Madsen et al. (2013). In Figure 9 a picture of the large-scale installation tests conducted in 2012 in Frederikshavn

Figure 9: Field tests of the installation of a bucket foundation with $d = 4$ m and $D = 4$ m, Frederikshavn 2012. On the right-hand side the pumping system

is illustrated. Note the multi-shield (anti-buckling) shape of the cross section as opposed to standard circular cross sections. Pumping system-related issues can be cavitation of the water and pump leakages. To avoid cavitation, the suction applied does not have to exceed the vapour pressure of the water. The deeper the water the more pressure can be applied before breaching the vapour pressure limit.

Small-scale and real-scale studies addressing installation issues are numerous in literature (Senpere and Auvergne, 1982; Rusaas et al., 1995; Alhayari, 1998; Solhjell et al., 1998; Chen and Randolph, 2004; Tran et al., 2004; Houlsby et al., 2005). A complete procedure for suction-assisted penetration design is described, and proved against real measurement and small-scale tests, in Houlsby and Byrne (2005). Villalobos (2006) examines the penetration of small-scale bucket pointing out the differences in bearing behaviour between jacked and suction installation. For bucket foundations the installation phases are important parts of the design process. Scrupulous installation analysis should be conducted for every new site. Besides, to mitigate the risk, small-scale or large-scale experiments could be considered. A picture of one field test of a

Figure 10: Field tests of installation and bearing capacity of a bucket foundation with $d = 2$ m and $D = 2$ m, Frederikshavn 2002

bucket foundation with diameter 2 m and embedded length 2 m is depicted in Figure 10.

4.2 Bucket installation by pushing

Although penetration by pushing (or jacking) has relatively little applicability to real cases, it is of interest to analyse this phenomenon in the context of small-scale experimental tests. Test C41, presented in Foglia and Ibsen (2014a), is the representative experiment used for the installation comparisons. The bucket used in the test has $D = 300$ mm, $d = 300$ mm and wall thickness, $t = 1.5$ mm.

A straightforward interpretation of the total installation force of the physical experiments, V_{i}, is possible by using a simple linear model. The contribution of the skirt tip end bearing, V_{end}, can be simply superimposed to that of the internal and external frictional resistance acting on the skirt, V_{skirt}, as follows:

$$V_{\text{i}} = V_{\text{end}} + V_{\text{skirt}} \tag{20}$$

The skirt tip end bearing resistance can be calculated by assuming a footing of width equal to the skirt thickness t, and length equal to $\pi(D + D_i)/2$:

$$V_{\text{end}} = t\pi\frac{(D + D_{\text{i}})}{2}\left(0.5tN_{\gamma}\gamma' + h\gamma' N_q\right) \tag{21}$$

where h is the given penetration depth and D_{i} is the internal diameter of the bucket foundation. Villalobos (2006) calculated V_{end} considering the penetration of two corps into the sand:

$$V_{\text{end}} = t\pi\frac{(D + D_{\text{i}})}{2}\left(tN_{\gamma}\gamma' + 2h\gamma' N_q\right) \tag{22}$$

The difference between the two approaches for the foundation used in test C41, is shown in Figure 11. The plot shows that the choice of how to calculate V_{end} is not negligible. Houlsby and Byrne (2005) also adopted equation 21. V_{skirt} can be calculated by summing the internal and the external shear resistance acting on the skirt wall:

$$V_{\text{skirt}} = D_i\pi\int_0^h \tau_{\text{i}} dz + D\pi\int_0^h \tau_{\text{o}} dz \tag{23}$$

The shear stresses are calculated as:

$$\tau_{\text{i}} = \tau_{\text{o}} = K\sigma'_v tan(\delta) \tag{24}$$

where σ'_v is the vertical earth pressure at the given penetration depth, δ is the interface friction angle taken equal to $\phi'/3$ and K is the passive coefficient of horizontal earth pressure calculated according to Villalobos (2006):

$$K = \frac{2 - cos^2\phi'}{cos^2\phi'} \tag{25}$$

This value of K is derived taking into account the soil arching effect caused by the shear stresses acting on the surface of a skirt penetrating into the soil.

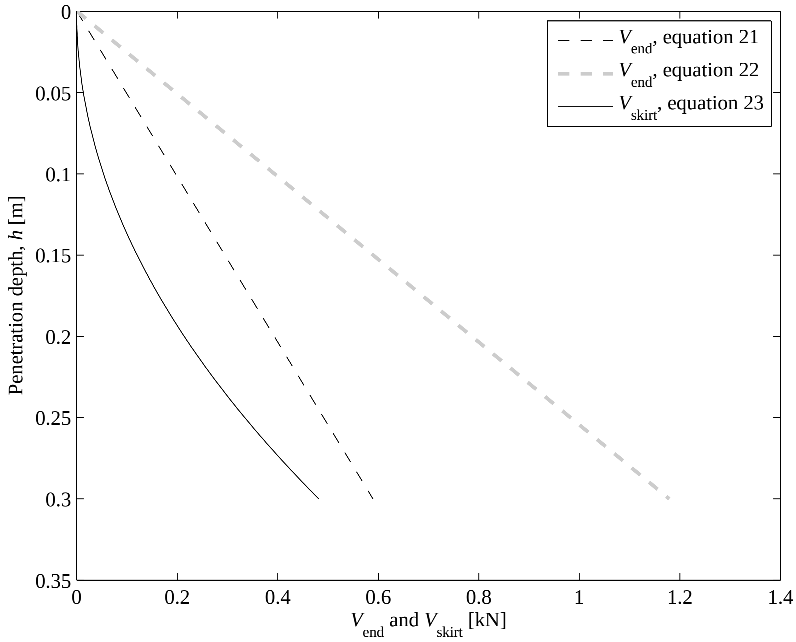

Figure 11: Comparison of the contributions to V_{i}. Calculations performed with $\phi' = 40°$

In Figure 11 it can be seen that, as expected, the frictional force caused by the shear stresses on the skirt surface has a smaller contribution to the penetration resistance than the skirt tip end bearing. Besides, it should be pointed out that the higher the friction angle the larger the discrepancy between V_{end} and V_{skirt}. The installation curve of test C41 against three linear model curves, are shown in Figure 12. The linear model, with an input friction angle of $\phi' = 44$, gives a good estimation until 100 mm of penetration. In general though, the linear model is not able to predict the experimental observations.

Two more advanced non-linear theoretical methods to obtain the jacked penetration curves of bucket foundations are suggested in Houlsby and Byrne (2005). These models have been proven valid by a number of studies and they embrace the effect of increase in stresses due to the frictional forces acting on the skirt during penetration. The first model considers a constant increment of stresses with depth. The second model allows the stresses to vary linearly with

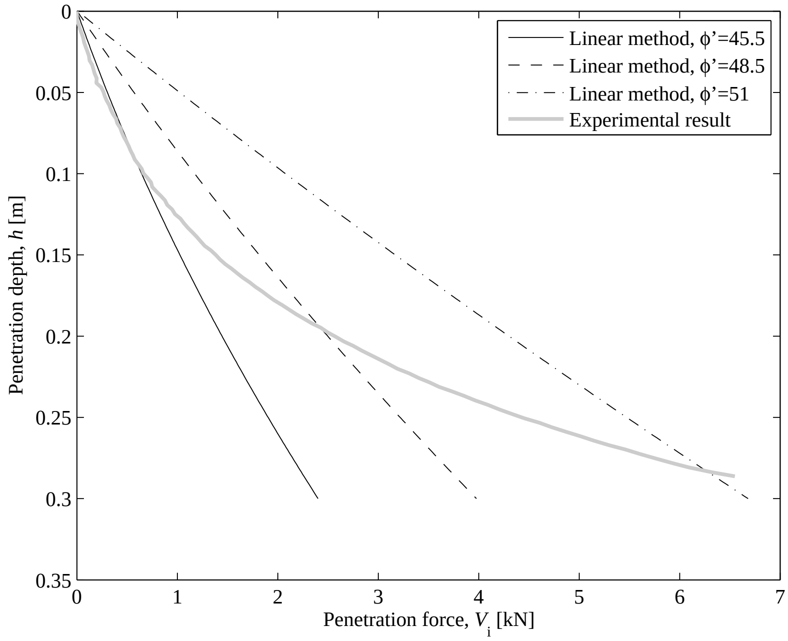

Figure 12: Linear estimation of the penetration resistance with three different friction angles against experimental curve

depth. In the following, these two models are referred to as: non-linear model 1 and non-linear model 2. Further details on the models are not mentioned here. The reader should refer to Houlsby and Byrne (2005) and Villalobos (2006) for theoretical explanations and numerical implementation

Senders (2008) investigated the behaviour of bucket foundations supporting tripods. He conducted centrifuge tests addressing installation and vertical cyclic response of the foundations. Senders (2008) implemented the second non-linear method of Houlsby and Byrne (2005) to interpret centrifuge experimental data. He concluded that with adequate input parameters the method is able to predict the experimental behaviour.

Cotter (2010) conducted numerous installation tests on three different soil samples. He mainly investigated the installation process with respect to the suction needed for the penetration and

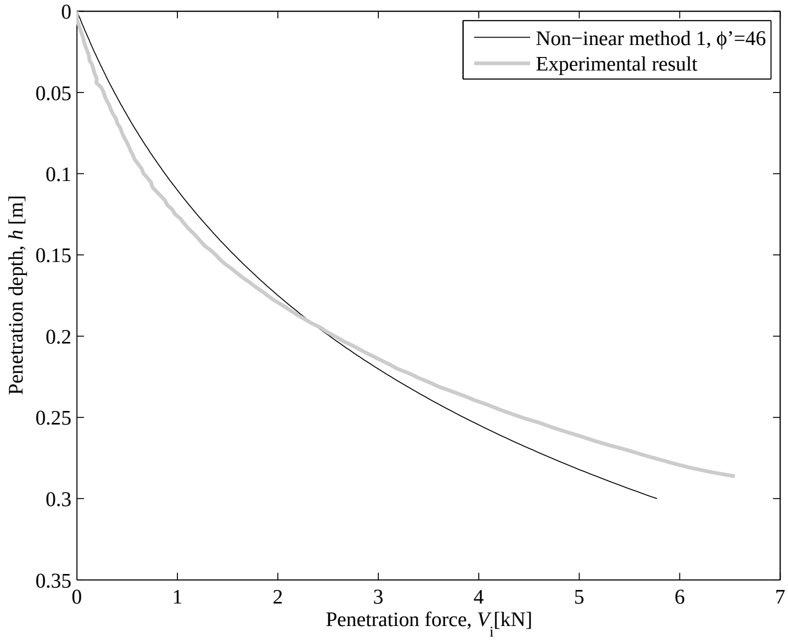

Figure 13: Non-linear model 1 against experimental results

to the skirt tip injection for steering the bucket into the ground. Cotter (2010) chose the second non-linear method of Houlsby and Byrne (2005) to predict the experimental data during self-penetration of the bucket foundation. Also Villalobos (2006) successfully implemented the approaches presented in Houlsby and Byrne (2005). The non-linear models are plotted together with the installation curve of test C41 in Figures 13 and 14. Simulations for several values of ϕ' were run. In the figures the best result achieved for one value of the friction angle is shown. The calculation factors chosen for the simulations were those suggested by the previous studies mentioned above: $m = 2$ (for non-linear model 1), $f_1 = 1$ and $f_2 = 2$ (for non-linear model 2). Note, in Figure 14, a discontinuity in correspondence to $h = 150$ mm owing to a change in the solutions of non-linear model 2 when $h \geq D_\mathrm{i}/2f_1$.

The non-linear model 2 interprets the experimental trend better than the linear model. However, non-linear model 1 seems to fit best the experimental observations. Of course, by choosing

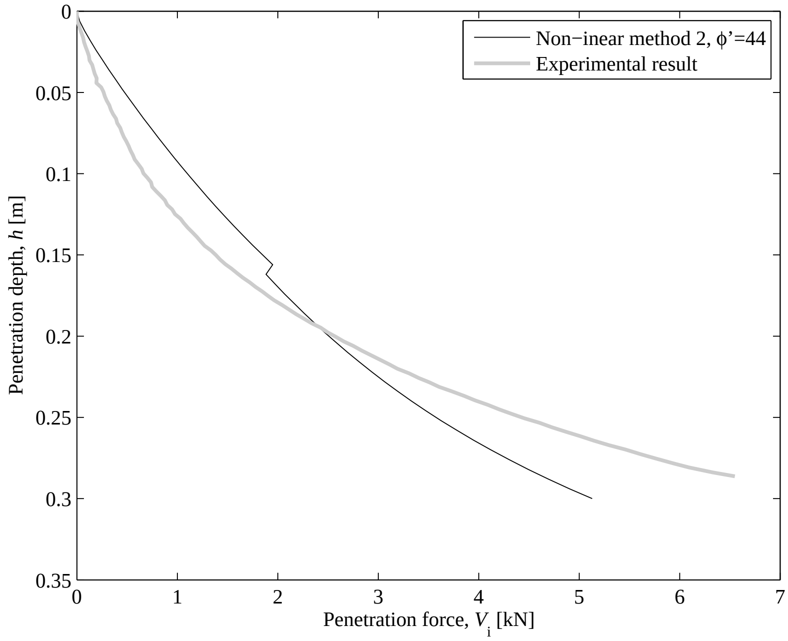

Figure 14: Non-linear model 2 against experimental results

another set of input parameters (K, δ, f_1 and f_2) non-linear model 1 might be able to better interpret the experimental results.

5 Conclusions

The bearing capacity of a flat footing is estimated with seven different methods. The formula given by DNV (2014) seems to give the most conservative estimation. Two methods to estimate the bearing capacity of bucket foundations are compared against experimental results. The method proposed by Villalobos (2006) predicts well the experimental data for a very high value of the friction angle. The method proposed by Larsen (2008), with a friction angle similar to the peak friction angle, predicts one experimental point but seems to overestimate the trend shown by the experimental data.

Interaction diagrams for flat footings are presented and evaluated against classic methods. The bearing capacity calculated with DNV (2014) gives the smallest prediction. However, in the relevant region for offshore wind turbines, full agreement between the methods is found. Also for skirted foundations, interaction diagrams and classic approaches are compared. Similarly to what observed for flat footings, the largest discrepancy between classic methods and interaction diagrams is seen out of the relevant region for offshore wind turbines. The failure envelope derived by Ibsen et al. (2014) is shown to overestimate the experimental results at small V/V_s. The tensile parameter t_0 can however be modified to obtain a better description of the experimental points. The interaction diagram for bucket foundations proposed by Achmus et al. (2013a) is proven to be reasonably in agreement with experimentally derived envelopes and appears thereby to be a powerful preliminary design tool.

Three methods to estimate the jacked installation of bucket foundations are adopted to interpret one experimental curve. As expected, the non-linear models show better prediction abilities than the linear model.

From the literature review of the bucket bearing behaviour it is clear that a large amount of knowledge has been collected on bucket foundation supporting floating structures and substructures with multiple foundations. Only recently, the research focus has turned to monopod bucket foundation.

The authors would like to emphasize that real-scale installation of bucket foundations has been proven over the last 30 years in many soil conditions. Therefore, this design and construction phase should not be an issue any longer. More rational research directions include the behaviour of buckets under predominant overturning moment and the dynamic properties of the foundation. Finally, the monopod bucket foundation concept will have proper industry recognition once its bearing behaviour will be proven in real offshore environment.

Nomenclature

A	area of the foundation
A'	effective area of the foundation
D	foundation width (diameter for circular cross section)
D_{i}	internal diameter of bucket foundations
D_{r}	relative density
H	horizontal load
H'	normalised horizontal load
H'_{u}	normalised ultimate horizontal load
K	coefficient of lateral earth pressure
L	length of the foundation
M	moment
M'	normalised moment
M'_{u}	normalised ultimate moment
N_c, N_q, N_γ	bearing capacity factors
R	outer radius of the bucket
V	vertical load
V_{u}	ultimate vertical load of flat footings
V_{s}	ultimate vertical load of skirted foundations
V_{f}	vertical contribution of the frictional resistance of the skirt
V_{gu}	ultimate vertical load of flat footings under general loading
V_{gs}	ultimate vertical load of skirted foundations under general loading
V_{i}	penetration resistance during jacked installation
V_{end}	contribution of tip end bearing to the installation resistance
V_{skirt}	contribution of the skirt to the installation resistance
V_0	preconsolidation vertical load
c	cohesion
d'	depth pf excavation
d	length of the skirt
d_{ref}	reference skirt length
d_c, d_q, d_γ	depth factors
e	load eccentricity
h	penetration depth
h_0, m_0, t_0, e_0	dimensionless parameters of the failure surface
i_q, i_γ	load inclination factors
m, f_1, f_2	dimensionless parameters of the non linear installation models
q	surcharge
q_u	ultimate bearing capacity of flat footings
s_c, s_q, s_γ	shape factors
t	thickness of the skirt

β_1, β_2, β_{12}	dimensionless parameters of the failure surface
δ	interface friction angle
ϕ'	effective soil friction angle
γ'	effective unit weight of the soil
θ	angle between H and V
τ_o	shear stress on the outer skirt
τ_i	shear stress on the inner skirt

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