VERTICAL CAPACITY OF BUCKET FOUNDATIONS IN UNDRAINED SOIL

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Abstract. Offshore wind turbine structures are traditionally founded on gravity concrete foundations or mono-piles. Bucket foundations were developed for the offshore oil and gas industry and are now being used in wind turbine construction. The loading in this application is characterized by a vertical load due to the slender construction combined with horizontal forces inducing a large overturning moment. Field tests on bucket foundations were performed to gain insight into the vertical load response of bucket foundations in clay soils. The field tests were accompanied by finite element numerical simulations in order to provide a better understanding of the parameters influencing bucket foundation behaviour.

Keywords: bucket foundation, vertical loading, bearing capacity, Mohr-Coulomb model, finite element.

Introduction

Increasing attention has recently been focused on the development of environmentally safer means of energy production. Wind turbines represent one of the most efficient energy production technologies and their use has rapidly expanded due to environmental considerations. Many projects have recently been initiated to expand wind energy production across the world. It is common practice to place wind turbines offshore. A major design issue concerning these offshore structures is the turbine foundation, which can consume up to 30 percent of the total construction cost. Several techniques are available for wind farm foundation construction including gravity foundations (Fig. 1a), bucket foundations (Fig. 1b), monopile foundations (Andersen *et al.* 2012) (Fig. 1c), and tripod foundations (Fig. 1d). Among the aforementioned foundation types, the bucket foundation system is unique for its convenient installation method and reusability (Ibsen *et al.* 2004). A bucket foundation is essentially an upside-down bucket-like steel structure which is lowered to the seabed under its own weight and then inserted into the soil by pumping the water from the interior of the bucket. Other advantages of bucket foundations include reduced construction costs and the availability of passive suction forces to counteract uplift.

Recent comprehensive research and development projects have proven that bucket foundations are effective given suitable soil conditions such as fine sand or clay materials in water depths from near-shore to approximately 55 meters (Ibsen *et al.* 2004; Achmus, Abdel-

Rahman 2005; Abdel-Rahman, Achmus 2005; API 2000; EAU 2004; Feld 2001; Ibsen 2008) .The bucket foundation design reduces the required amount of steel by half compared to traditional monopile solutions, and bucket foundations are much easier to install and do not require heavy installation equipment (Ibsen *et al.* 2004; Achmus, Abdel-Rahman 2005; Abdel-Rahman, Achmus 2005). The suction installation technology was originally introduced by Shell and is widely used for anchor piles and skirted foundations on offshore structures (Tjelta 1995; Ebrich, Tjelta 1999).
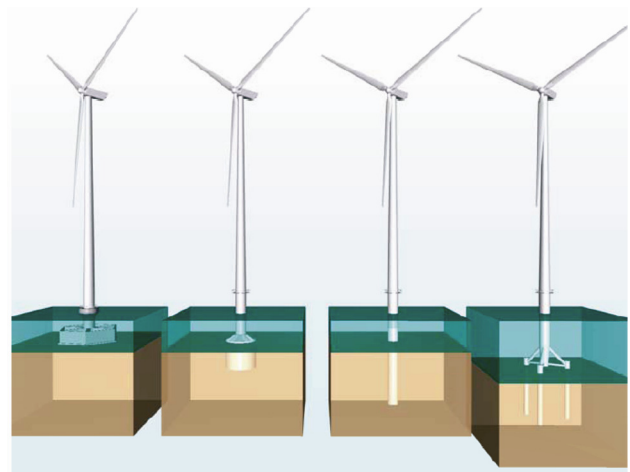

Fig. 1. Offshore foundations employed in wind turbine construction, from left to right: a) gravity foundation; b) bucket foundation; c) monopole foundation; d) tripod foundation

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The choice of foundation type depends on several factors such as soil condition, water depth, structure of the wind turbine, environmental conditions, economics, and politics (Feld 2001). Advantages and disadvantages of the foundation types in relation to the factors presented above are outlined by Liingaard (2006) and Ibsen *et al.* (2003).

Due to their advantages, bucket foundations have recently attracted the attention of oil and wind turbine companies for additional uses such as gravity platform jackets, jack-ups, offshore wind turbines, subsea systems, and seabed protection structures (API 2000; EAU 2004; Feld 2001, 2006; Ibsen 2008; Senepere, Auvergne 1982; Tjelta 1995; Ebrich, Tjelta 1999; Liingaard 2006; Ibsen *et al.* 2003; Dyme, Houlsby 1998; Aas, Andersen 1992; Clukey *et al.* 1995; Allersma *et al.* 1997, 2000; Houlsby, Byrne 2000; Byrne *et al.* 2002; Byrne, Houlsby 2004; Andersen, Jostad 1999; Ibsen *et al.* 2012).

Following extensive research examining the performance of bucket foundations in sand, several models have been developed to describe their behaviour. Feld (2001) investigated the design procedures for bucket foundations within frictional materials by employing combined loading experimental tests as well as non-linear finite element simulations. The tests were performed on a half bucket placed against a transparent wall identifying the net flow during tension and the failure surface at pullout, corresponding to the maximum tensile capacity of the foundation. The buckling of large diameter bucket foundations during installation in sand was analysed by LeBlanc (2009), since the bucket foundation is a thin shell structure and is therefore potentially vulnerable to structural buckling as a result of hydrostatic loading during suction installation. A modified theoretical relationship for bucket foundations analogous to vertical loading of circular foundations in homogeneous and isotropic sand was successfully developed by Ibsen *et al.* (2012).

Taiebat and Carter (2000) performed finite-element analyses of circular foundations on a homogeneous, purely cohesive soil to investigate the shape of the undrained failure locus for the foundation. The results of the numerical analyses were compared with some of the available theoretical solutions for the undrained bearing capacity of the foundation.

Wang *et al.* (2006) utilized three-dimensional finite element analysis to model the soil-structure interaction of suction caissons. They modelled the suction caisson and supporting soil as a nonlinear/plasticity problem and presented case studies of several geometries to demonstrate the accuracy of their model.

Ding *et al.* (2003) estimated soil liquefaction during ice-induced vibration of bucket foundations in the Bohai Sea, China. They analyzed the foundation vibrations using self-excited vibration theory to obtain the maximum ice-induced dynamic shear stresses in the soil.

Zhang *et al.* (2007) performed a series of centrifuge model tests to investigate the behaviour of suction bucket foundations to be used on a tension leg platform in the Bohai Sea. Lateral loadings were included to study the characteristics of structural vibration-induced soil lique-

faction. They constructed a centrifuge model comprising two layers of soil: a 210 mm-thick silt layer at the top and a 20 mm-thick coarse sand layer at the bottom with the water level approximately 20 mm above the soil surface. Excess pore water pressures reached a maximum at depths of 1–1.5 m.

Zhang *et al.* (2004) presented a finite element simulation of the seepage field generated by suction penetration of a bucket foundation into the sea floor. The water head and its gradient on both sides of the bucket foundation skirt were computed in order to calculate the changes in penetration resistance with penetration depth.

Although some researchers have investigated the behaviour and bearing capacity of bucket foundations in clay (Ibsen *et al.* 2004; Achmus, Abdel-Rahman 2005; Abdel-Rahman, Achmus 2005; API 2000; EAU 2004; Feld 2001; Ibsen 2008), the full response of these structures under various loading conditions has not been extensively examined.

In the present study, field tests were performed on bucket foundations installed in Baltic clay to obtain insight into the vertical load response. The field tests were complemented by finite element numerical simulations to provide a better understanding of the parameters influencing the foundation behaviour.

1. The clay test site

Clay minerals are the main constituents of most marine sediments. In order to study the behaviour of bucket foundations under various loading conditions, a series of field tests were carried out at the Aalborg University clay test site. The test site is located in Northern Jutland on the outskirts of Aalborg (Grinsted), Denmark on a marine plain formed during the Yoldia transgression period, in connection with the ice retreat from Northern Jutland (approximately 13,000–9,000 years B.C). The clay is a deposit from the Yoldia Sea and is often referred to as Baltic clay. To a depth of approximately 2 m the soil consists of a brownish-gray clay with a high silt content containing mixtures of silt, sand, and chalk in horizontal stripes of approximately 1 mm. High percentages of illite and low kaolinite/chlorite and quartz/feldspar ratios are characteristic of this marine deposit. Clay mineral assemblages are characterized by an increase in kaolinite/chlorite ratios moving from late glacial to Holocene sediments (Gingele, Leipe 1997). In contrast to other marine sediments such as those in the Mediterranean, the Baltic Sea may be considered as a temporarily inundated portion of the European continent, with salinities ranging from 20–30% in the Belt Sea to less than 3% in the Bothnian Bay (Matthaus 1992). The location of the clay test site and the soil layering are depicted in Figures 2 and 3.

2. Experimental work

Experiments were conducted to determine the strength and deformation of the clay, including in-situ cone penetration (CPTU) and vane shear tests. The CPTU used in the field tests is illustrated in Figure 4, and the test results are summarized in Tables 1 and 2.

Fig. 2. Study area in Grinsted, north Jutland

Fig. 3. Illustration of the layering in the soil in Grindsted

Classification experiments were also carried out in the soil mechanics laboratory at Aalborg University according to the "Guidelines on the performance of geotechnical classification" by Wiley Lund. These included water content, grain density, void ratio, Atterberg limits, and hydrometer tests. Table 3 lists the characteristic values for Yoldia clay. A comparison of the Yoldia clay values with the average values obtained in the classification experiments revealed excellent agreement (Table 4). The hydrometer analysis resulted in poor graded curve (Fig. 5).

Bucket foundations are often subjected to a combination of loading conditions including vertical loads and moments. The experiments were performed on a small scale bucket foundation, and separate tests were performed in which vertical loads and bending moments (Barari, Ibsen 2012) were imposed on the foundation via a special loading apparatus designed specifically for bucket foundations.
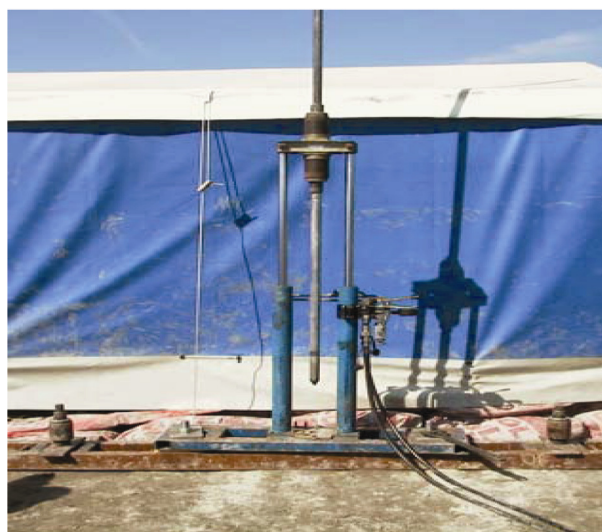

Fig. 4. CPTU test apparatus

Table 1. Experimental c_u values (Ibsen, Barari 2011)

Test	(1)	(2)	(3)	(4)	(5)
Borehole A- c_u (kPa)	74.36	74.41	76.24	82.06	–
Borehole B- c_u (kPa)	88.4	99.21	91.55	83.09	87.33
Borehole C- c_u (kPa)	108.47	93.41	98.51	108.38	108.47
Borehole D- c_u (kPa)	88.18	88.73	–	–	–
Borehole E- c_u (kPa)	105.85	99.46	99.43	103.08	96.47

Table 2. Data obtained from vane shear tests (Ibsen, Barari 2011)

Depth (m)	c_v [kPa]	c_{vr} [kPa]	S_t (sensitivity)
0.2	147.45	33.82	4.36
0.6	145.72	36.99	3.94
1	143.31	39.07	3.67

Table 3. Characteristic properties of Yoldia clay

G_s	γ	e	W	LL %	PL %	I_p %
2.7–2.75	19–20	0.8–1	28–30	45–65	20–25	25–40

Table 4. Characteristic properties of Grindsted clay

G_s	γ	e	W	LL %	PL %	I_p %
2.73	18.61	0.85	28.27	55.91	26.62	29.29

The foundations were installed by applying pressure to the top of the bucket using a hydraulic cylinder to ensure penetration at a predefined rate of 1.4 mm/s (Fig. 6). Loading of the bucket continued until the bucket skirt fully penetrated the soil and the top plate was in contact with the ground surface. The installation rate was the

same as the rate employed for previous tests in sand. After installation, the hydraulic cylinder was removed and the engine was mounted to the red beams in Figure 7. The experimental apparatus consisted of two 1.5 m vertical legs on which were placed two horizontal beams with spans of 2.25 m. The legs were inserted into the soil to a depth of approximately 1.5 m. The hydraulic cylinder and bucket foundation were attached to the beams. The vertical loading experiments required addition of a brace (Fig. 8) to the apparatus to prevent rotation of the threaded rod.
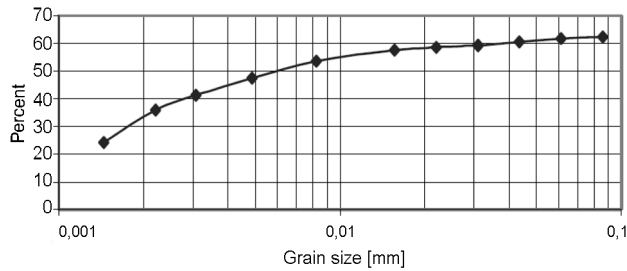

Fig. 5. Average size distribution of eight soil samples from test site

Fig. 6. Installation of bucket foundation ($D = 30$ cm, embedment ratio 0.25)

The tests were performed on level ground in order to ensure uniform load transmission from the bucket to the foundation during soil penetration. Levelling of the test area was achieved by embedding metal bars in the soil, levelling the bars, and grading the soil surface by passing a metal scraper along the bars.

Fig. 7. Bucket load test device used for performing vertical load tests

Fig. 8. Hanger bar reinforcement for vertical load tests

Bucket diameters of 20, 30, and 40 cm and various embedment ratios were considered for the mechanical tests. During previous plate load tests using 40 cm buckets, failure occurred around the soil edges at loads of approximately 50 kN, which resulted in the structure slowly rising from the surface (Fig. 9). Load tests were therefore conducted on bucket foundations with 20 and 30 cm diameters in an experimental area of 100×150 cm^2.

3. Results of field tests

The bucket foundations were loaded at the same strain rate as had been used in tests of circular surface foundations (Ibsen, Barari 2011), corresponding to a penetration rate of 44 mm/h for the 20 cm buckets and 60 mm/h for the 30 cm buckets.

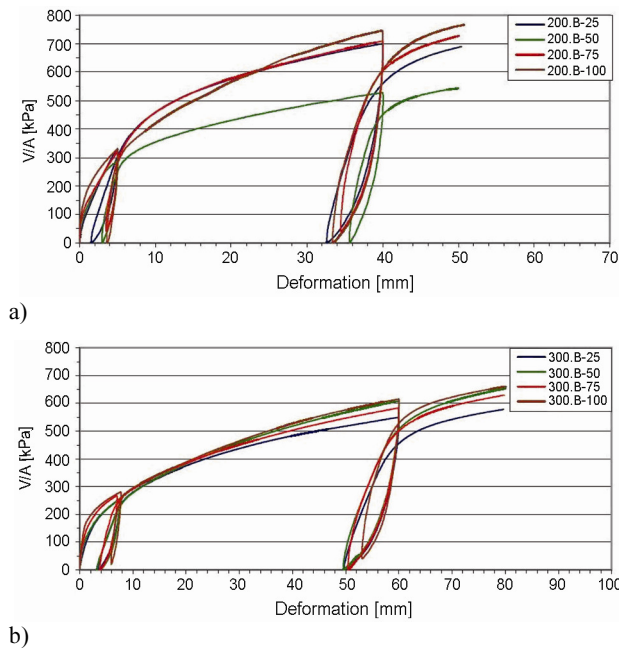Fig. 9. Plate load test on 40 cm circular surface foundations

a)

b)

Fig. 10. Stress-displacement curves obtained during 20- and 30-cm diameter bucket tests

The mechanical properties required for the numerical simulations were obtained from the tests. Figure 10 contains the stress-displacement curves from the 20 cm and 30 cm diameter bucket tests. Except in the case of the 20 cm bucket with 0.5 embedment ratio, the variation of skirt length had little effect on the bearing capacity. The results observed using the 20 cm bucket with an embedment ratio of 0.5 may be attributed to silt stripes present in the vicinity of the test area. The modulus of elasticity E was extracted from the loading test results for use in the numerical analyses (Tables 5 and 6).

Table 5. *E*-moduli calculated from experiments using 20-cm bucket foundations

Models	E_{ur} [kPa]	E_{25} [kPa]	E_{30} [kPa]	E_{40} [kPa]	E_{50} [kPa]
$D = 20$ cm, $d / D = 0.25$	31920	8795	8263	7329	6582
$D = 20$ cm, $d / D = 0.5$	33286	10720	9433	7618	6078
$D = 20$ cm, $d / D = 0.75$	33779	8340	6877	5602	4855
$D = 20$ cm, $d / D = 1$	32727	13087	9419	5660	3978

The incidence of failures resulting in earth movement around the loaded buckets decreased with increasing embedment ratio. For 20 cm buckets the failure lines were approximately 1.7 cm, 1.5 cm, 1 cm, and 2 mm at embedment ratios of 0.25, 0.50, 0.75, and 1. For 30 cm buckets almost no movements in excess of 2 mm were observed (Fig. 11).

Table 6. *E*-moduli calculated from experiments using 30-cm bucket foundations

Models	E_{ur} [kPa]	E_{25} [kPa]	E_{30} [kPa]	E_{40} [kPa]	E_{50} [kPa]
$D = 30$ cm, $d / D = 0.25$	32250	10345	9242	6520	4690
$D = 30$ cm, $d / D = 0.5$	23402	9411	7609	4939	3730
$D = 30$ cm, $d / D = 0.75$	32133	13569	10320	6252	3831
$D = 30$ cm, $d / D = 1$	33766	19107	12535	5730	3294

Fig. 11. Failure line in surrounding soil after testing 30 cm foundation

3.1. Strength of transition zone using α-method

As the bucket is inserted in the soil, a thin transition zone forms between the bucket surface and the surrounding area. An accurate numerical model of the bucket foundation should incorporate a realistic estimate of the strength of this zone, which may be determined using the α-method. This approach was initially used to determine the undrained bearing capacity of offshore piles. In pile loading tests the total capacity of the pile Q_T is initially determined, and Q_m is calculated by subtracting the rough calculated resistance Q_p from Q_T.

The shaft resistance may be expressed as:

$$f = Q_m / A_m = \alpha c_u . \qquad (1)$$

Several authors have developed formulas or curves for the variation of α with c_u (Peck *et al.* 1953; Woodward *et al.* 1961; Kerisel 1961). There are large differences in the α-values values determined using different formulas (Fig. 12).

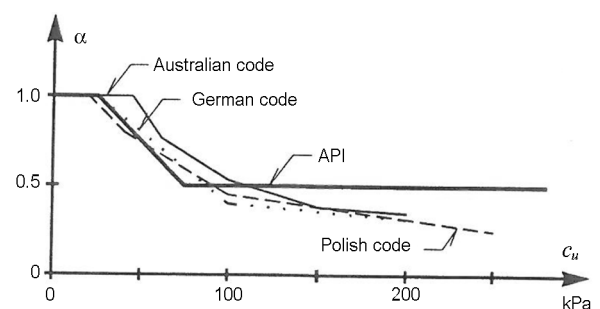Fig. 12. *α*-values obtained using different codes

Table 7. Undrained shear strengths calculated analytically using bearing capacity formula

	20 cm-d/D = 0.25	20 cm-d/D = 0.5	20 cm-d/D = 0.75	20 cm-d/D = 1	30 cm-d/D = 0.25	30 cm-d/D = 0.5	30 cm-d/D = 0.75	30 cm-d/D = 1
c_u [kPa]	113.05	85.08	114.31	120.1	88.61	97.97	93.59	98.61
$c_{u,dc}$ [kPa]	97.7	68.36	85.82	88.99	76.58	78.71	70.27	73.07

Although the codes are in reasonable agreement with pile loading test results, it is clear that additional factors should be included. For instance, many authors have noted that α depends on the overconsolidation ratio (OCR) (McCelland 1974; Semple, Rigden 1984). Figure 13 is a plot of the α values derived from our results as a function of undrained clay shear strength.
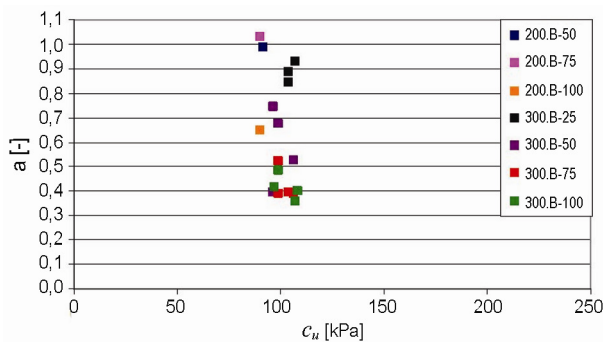

Fig. 13. α-values with respect to undrained shear strength of clay

3.2. Effect of embedment on the shear strength values

A general bearing capacity equation was developed based on the load test results. The equation models the bucket as a surface foundation in which the foundation surface is located at a certain depth below the soil surface. The general bearing capacity equation is:

$$b = \frac{Q}{A} = c_{u,dc} N_c S_c i_c d_c + q \,. \qquad (2)$$

The undrained shear strength of the buckets was measured twice in order to investigate the influence of depth factor. The depth factor was calculated from Brinch Hansen's (1961, 1970) expression:

$$d_c = 1 + 0.35 \frac{\overline{D}}{D}, \qquad (3)$$

in which $\overline{D}$ is the effective depth and D is the diameter of the foundation. The effective depth is the minimum depth below the surface at which the soil possesses at least the same strength as the soil beneath the foundation plate. The effective depth was initially equal to the skirt length and increased with increasing vertical load on the foundation. The depth factor should not assume values greater than 1.35. The undrained shear strengths calculated using Eqn (2) are listed in Table 7.

4. Finite element model

4.1. Mesh

Two-dimensional finite element models of the test apparatus were developed in order to obtain realistic numerical models of the bucket foundation system. In order to take advantage of symmetry, the model reflected only half of the physical structure.

A number of mesh densities were investigated to achieve a time-efficient model without compromising accuracy. Since there is a high stress and strain concentration near the bucket foundation and the intensity of stress and strain diminishes further from the bucket, a special discretization mesh was developed. The mesh in Figure 14 comprises three distinct regions differing in mesh size, beginning with a very fine six-node triangular mesh in a trapezoidal region near the bucket and moving to coarser mesh regions away from the bucket while retaining the six-node triangular elements. The axisymmetric model consisted of approximately 1400 elements. Boundary conditions were selected so that both sides were restrained against horizontal movements and the bottom horizontal boundary was restrained against horizontal as well as vertical movements. The node at the left vertical boundary representing the bucket foundation was restrained against rotation.
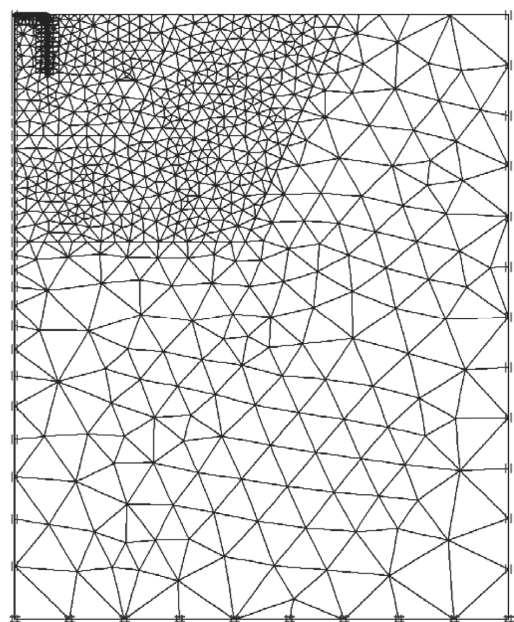

Fig. 14. Finite element mesh of bucket foundation under vertical loading

4.2. Material properties

The soil conditions were modelled using an elastic-purely plastic Mohr-Coulomb constitutive model with a uniform undrained shear strength/depth profile. Although the Mohr-Coulomb model is unable to capture post-yield behaviours such as hardening or softening, this was unimportant since the interest of this study was to determine the failure stress level. The bucket foundation was assumed to exhibit linear elastic behaviour.

The physical and mechanical properties of the clay including E, c_u, $v = 0.495$ and $\gamma = 19$ kN/m^3 were determined from laboratory tests performed on field samples. The properties of the bucket foundation simulations were based on the properties of the buckets used in the field tests (Table 8). The undrained shear strength and elasticity modulus of the soil were varied in a parametric analysis in order to investigate their effect on the bearing capacity of the bucket foundation under vertical loading.

Table 8. Material parameters for bucket foundations

Diameter	EA (kN/m)	EI (kN.m²/m)	W (kN/m)	v
20 cm	6.594×10^6	16493	0	0.3
30 cm	1.4844×10^7	79521	0	0.3

5. Analyses

A series of parametric studies were performed to obtain the best fit curves for vertical bearing capacity of bucket foundations employing various moduli. In this section the finite element simulations are compared with the behaviour observed during field tests. In the vertical loading simulations, the load-displacement curves were fit by varying the soil parameters E and C_u in the simulations. The interface between the soil and the foundation was assumed to be fully bonded, which is a relevant consideration in offshore foundations. The suction developed within the soil plug during loading provides a capacity to resist tensile loading conditions. Undrained conditions may be present for months or years due to the large size of offshore foundations (i.e. 15000 m^2 in plane area).

The load-displacement relationship is illustrated in Figures 15 and 16. In the numerical simulations, the value of c_u was modified to achieve the best fit with the experimental data. A similar procedure was followed with the modulus of the soil, in which values of 25%, 30%, 40%, and 50% and E_{25} is then chosen to reach the best agreement with the field data. The results of the vertical loading analyses are summarized in Tables 9 and 10.

5.1. Ultimate loads

In order to demonstrate the effects of embedment ratio and foundation diameter, failure values under pure vertical loading were analyzed. Tables 11 and 12 contain the values of the ultimate vertical load (F_{Vult}) for 20 and 30 cm surface and bucket foundations on Yoldia clay evaluated using finite element analysis, as well as relevant literature data. Bucket foundations with a skirt length ratio of 0.5 were selected for validation. The calculated ultimate bearing capacities under pure vertical loads (F_{Vult}) for circular surface foundations on Yoldia clay were 76.6 kN for 20 cm foundations and 132.5 kN for 30 cm foundations, very close to the exact solutions proposed by Shield (1955) and the finite element results of Gourvenec and Randolph (2002).

The behaviour presented in the figures and Tables highlights the significant sensitivity of the bearing capacity to the skirt length ratio (e.g. a 37% increase in pure bearing capacity for the two cases moving from $d/D = 0$ to $d/D = 0.5$).
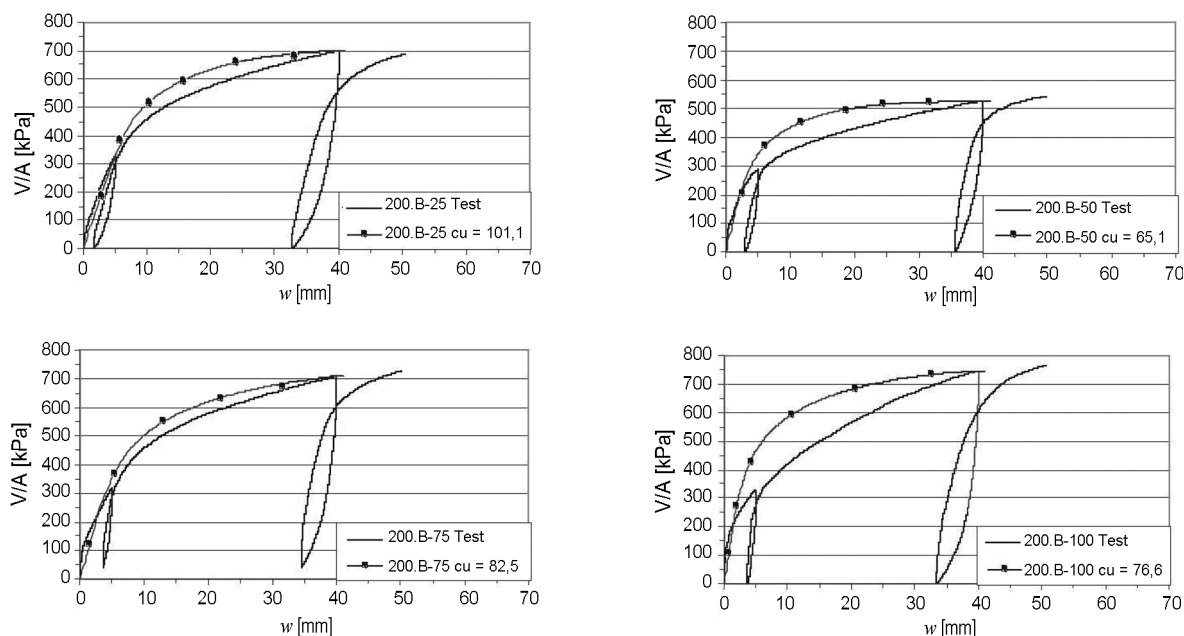

Fig. 15. Best fit curve for load-displacement data from field tests for vertical loading of 20-cm bucket foundation
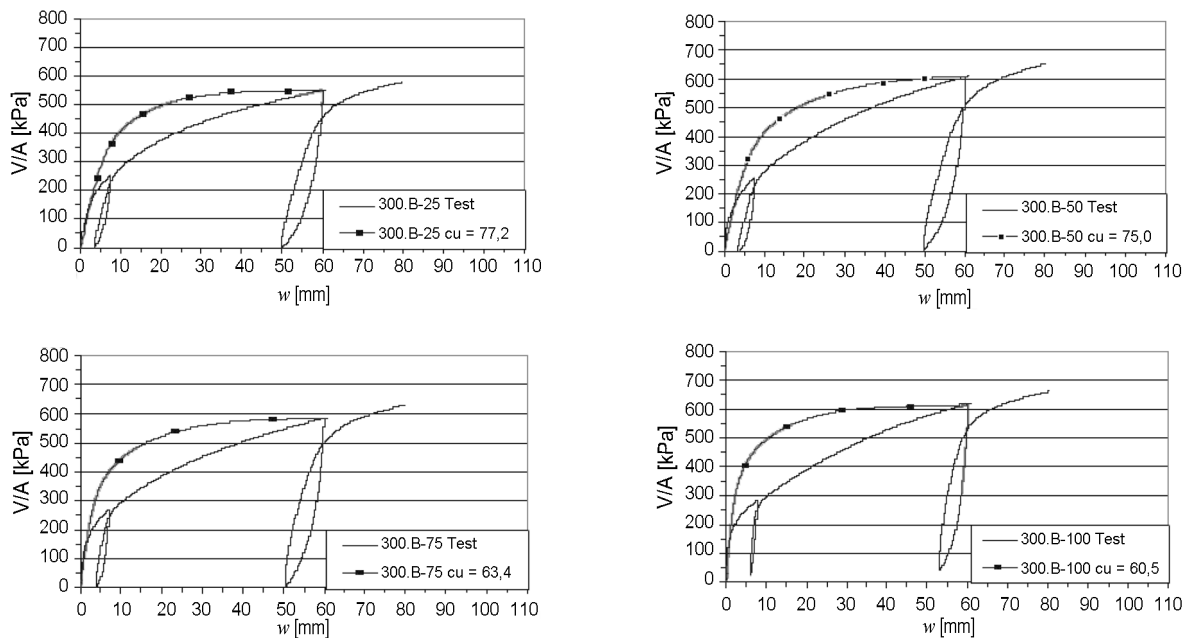
Fig. 16. Best fit curve for load-displacement data from field tests for vertical loading of 30-cm bucket foundation

Table 9. Results of best-fit finite element simulations

Type of Foundation	Type of Simulation	Deformation (mm)	Failure Value [kPa]	Error (Deviation)
D = 20 cm, d / D = 0.25	FEM c_u =101.2 [kPa]	39.98	699.79	+0.087
	FEM c_u =101.1 [kPa]	39.98	699.15	−0.004
	Experiment	39.98	699.18	–
D = 20 cm, d / D = 0.5	FEM c_u = 65.1 [kPa]	40	527.13	−0.064
	FEM c_u = 65.2 [kPa]	40	527.9	0.082
	Experiment	40	527.47	–
D = 20 cm, d / D = 0.75	FEM c_u = 82.5 [kPa]	39.98	708.58	−0.021
	FEM c_u = 82.6 [kPa]	39.98	709.32	+0.083
	Experiment	39.98	708.73	–
D = 20 cm, d / D = 1	FEM c_u = 76.5 [kPa]	39.98	744.55	−0.102
	FEM c_u = 76.6 [kPa]	39.98	745.44	0.017
	Experiment	39.98	745.31	–

5.2. Comparison with the stress characteristics method

According to plasticity theory (Davis 1968), the stress characteristic method is capable of an exact solution for a rigid, perfectly plastic material under plane strain conditions. With undrained saturated clay, the normality condition is reasonably postulated as $\phi = \psi = 0°$. The same technique may be used for axisymmetric problems, although the relationship of the solution to the exact one for a rigid, perfectly plastic material is no longer clear be-

cause of the failure to satisfy the normality condition (Houlsby, Wroth 1983).

The numerical analysis failure values obtained for the bucket foundations exhibit excellent agreement with the plasticity stress characteristic values presented by Houlsby and Wroth (1983). The current solutions should be considered upper bounds, unless it can be shown that lower bounds exist with the same failure values (Tables 11 and 12). In addition, the finite element results for bucket foundations were validated against the plasticity results presented by Martin (2001) for circular foundations. Sur-

prisingly, the collapse loads for circular foundations obtained using plasticity solutions fall just below the numerical and experimental simulation results obtained for bucket foundations.

5.3. Ultimate vertical capacity depth factors

Figure 17 illustrates the variation in normalized vertical limit state ($N_{cV} = (\frac{V_{ult}}{Dc_u})$) with respect to embedment ratio for bucket foundations, as well as a comparison with previous numerical analyses in the literature for circular foundations (Bransby, Randolph 1999; Gourvenec 2008). The results are normalized with respect to foundation area (i.e. for plane strain the area is equal to the diameter), and undrained shear strength.

Embedment increases the vertical capacity as the failure regions are forced deeper within the soil mass. Classical bearing capacity theory uses depth factors proposed by Skempton (1951) and Brinch Hansen (1970) to modify the uniaxial vertical bearing capacity, and recommended industry practices are based on these factors (e.g. Eqn (4)):

$$d_{cV} = 0.3e^{-0.5kD/c_{u0}} \arctan(d/D), \qquad (4)$$

k is the gradient of the undrained shear strength profile (equal to zero for homogeneous deposits). Classical depth factors were originally derived for smooth-sided circular foundations but are also widely used for rough, smooth-sided, strip, and three dimensional foundations.

Table 10. Results of best-fit finite element simulations

Type of Foundation	Type of Simulation	Deformation (mm)	Failure Value [kPa]	Error (Deviation)
D = 30 cm, d/D = 0.25	FEM c_u = 77.1 [kPa]	59.96	548.60	–0.113
	FEM c_u = 77.2 [kPa]	59.96	549.28	0.011
	Experiment	59.96	549.22	–
D = 30 cm, d/D = 0.5	FEM c_u = 75 [kPa]	59.99	608.38	–0.015
	FEM c_u = 75.1 [kPa]	59.99	609.13	0.108
	Experiment	59.99	608.47	–
D = 30 cm, d/D = 0.75	FEM c_u = 63.3 [kPa]	59.92	581.99	–0.134
	FEM c_u = 63.4 [kPa]	59.92	582.85	+0.014
	Experiment	59.92	582.77	–
D = 30 cm, d/D = 1	FEM c_u = 60.4 [kPa]	60.01	614.42	–0.112
	FEM c_u = 60.5 [kPa]	60.01	615.37	0.042
	Experiment	60.01	615.11	–

Table 11. Comparison to published data

	Current research	Gourvenec, Randolph (2002)	Shield (1955)	Houlsby, Wroth (1983)	Martin (2001)
Bucket Foundation (20 cm) d/D = 0.5, c_u = 65.1 [kPa]	105.42	97.12	–	99.082	99.34
Circular Footing (20 cm) c_u = 65.1 [kPa]	76.6	76.94	78.77	–	–

Table 12. Comparison to published data

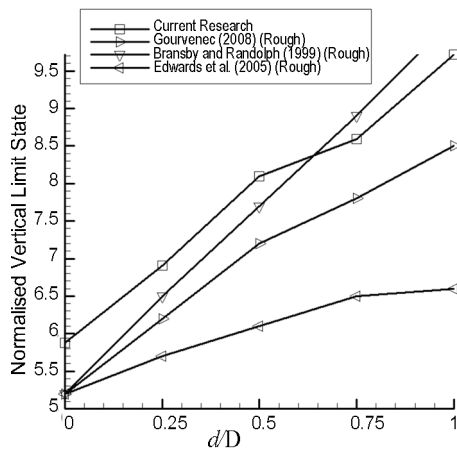|	Current research	Gourvenec, Randolph (2002)	Shield (1955)	Houlsby, Wroth (1983)	Martin (2001)
Bucket Foundation (30 cm), d/D = 0.5, c_u = 75 [kPa]	182.51	167.85	–	171.22	171.67
Circular Footing (30 cm) c_u = 75 [kPa]	132.5	132.97	136.12	–	–

Fig. 17. Ultimate vertical bearing capacity as a function of embedment ratio for Yoldia clay

For pure vertical loading, Meyerhof (1953) and Brinch Hansen (1970) also approximated the depth factor for strip or circular foundations as a linear fraction relative to *d/D*:

$$d_{cV} = 1 + n\left(\frac{d}{D}\right),$$ (5)

$$0.2 \subseteq n \subseteq 0.4.$$

The analyses describe a quadratic relationship between the ultimate uniaxial vertical load and the embedment ratio, which can be presented in terms of a depth factor:

$$d_{cV} = 1.0036 + 0.7138\frac{d}{D} - 0.0766\left(\frac{d}{D}\right)^2.$$ (6)

Eqn (6) has been verified only for skirt lengths between 0 and 1. Tani and Craig (1995) carried out lower bound plasticity analyses and centrifuge tests to investigate the vertical capacity of skirted foundations in non-homogeneous soils. They showed that, the soil above the level of the skirt tips for strip footings does not contribute to the vertical bearing capacity, while for circular footings its contribution was small.

Given the conclusions of Tani and Craig (1995), analyses of small scale bucket foundations suggested that, for pure vertical bearing capacity, the load-displacement curves are dependent on the foundation type and embedment depths.

Recent research has challenged the use of these conventional modification factors and explored the extent to which capacity is enhanced and the failure mode is changed with increasing embedment ratio. The trend of increasing capacity with increasing embedment does not continue indefinitely owing to a change in failure mode (Fig. 18), confirming the behaviour proposed by Yun and Bransby (2007) for rough embedded strip foundations.

Similar trends have been observed in other model simulations. It has also been noted that a failure mechanism similar to that of a shallow foundation is produced in bucket loading. Measurements have demonstrated that the failure surface beneath the bucket skirt extends to a depth of approximately *D*/2.
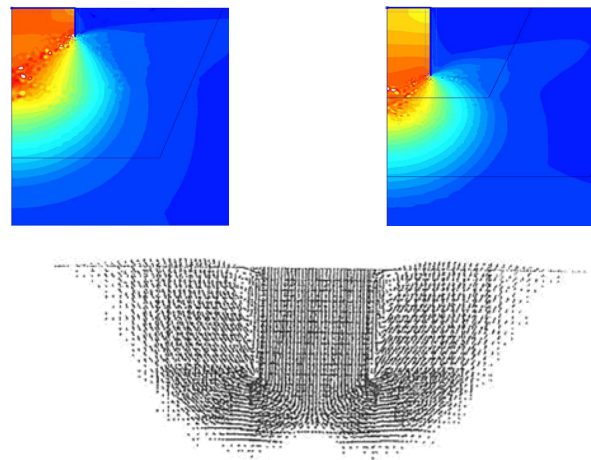

Fig. 18. Comparison of kinematic mechanisms accompanying failure of bucket foundations in homogeneous soil and skirted foundations in normally consolidated soils: a) bucket with embedment ratio 0.25; b) bucket with embedment ratio 1; c) skirted foundation with embedment ratio 1 (Yun, Bransby 2007)

Conclusions

A suction bucket foundation is a closed-top steel tube that is lowered to the seafloor, allowed to penetrate the bottom sediments under its own weight, then pushed to full depth under the suction force produced by pumping water out of the interior. Suction foundations are attractive because of their convenient installation and reusability, as well as the fact that they may withstand a significant amount of reverse loading through passive suction during uplift.

Extensive research has been conducted on bucket foundations in sand, leading to a host of theories describing their behaviour in this soil type. However, there has been relatively little research examining bucket foundation behaviour in clay soils. In the present study, we carried out experimental and numerical analyses of bucket foundations under vertical loading in clay soils.

In order to examine the effect of foundation geometry on the load transfer mechanism and the stress distribution within the soil, bucket diameters of 20 cm and 30 cm and embedment ratios of 0.25, 0.50, 0.75, and 1 were tested.

The Mohr-Coulomb model was sufficient for modelling the initial loading condition, but could not be used to describe post-yield behaviour such as hardening and softening. However, the strains corresponding to peak strength indicate that failure is reached prior to softening, and therefore the fundamental behaviour leading to failure can be effectively modelled using constitutive approaches such as the Mohr-Coulomb model.

Although Tani and Craig (1995) suggested that the bearing capacity of a shallow foundation was independent of the soil strength above the base of the footing (i.e. no embedment effects) for undrained conditions, this finding has not been verified using vertical monotonic loading or finite element analysis while a 37% increase in pure bearing capacity for the two samples ranging from *d/D* = 0 to *d/D* = 0.5 is determined.

The results were also compared to plasticity models in order to verify the accuracy of the simulations. A comparison of the variation in normalized vertical limit state

for circular and bucket foundations was also presented, and the quadratic relationship between ultimate vertical load and embedment ratio was discussed in detail.

In contrast to soils with significant strength heterogeneity (such as normally consolidated deposits) in which a Hill-type mechanism governs failure, soils with a uniform shear strength profile failure are described by a Prandtl-type failure mechanism in which the soil plug tends to move as a rigid block under uniaxial vertical loads. In this case the type of embedment has little effect on bearing capacity.

Nomenclature

c_u	Undrained shear strength [kPa]
S_c	Dimensionless factor, equal to 1.2 for circular foundations
N_c	dimensionless factor
i_c	Dimensionless factor, taken as 1 with H = 0
d_c	Depth factor, taken as 1.35 for D / B > 1 (Brinch Hansen (1961, 1970) [m]
Q	Total load at the bucket tip [kPa]
A	Area of bucket tip
Q_T	Total capacity of the pile
G_s	Specific gravity
γ	Unit weight
e	Void ratio
LL	Liquid limit
PL	Plastic limit
I_p	Plasticity index
W	Water content
f	Shaft resistance
c_{u0}	Undrained shear strength of the soil at foundation level
c_v	Shear strength from vane shear test

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