

Vertical Bearing Behavior of Monopod Caisson Foundation in Sandy Soils

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Abstract: This paper presents the results of numerical investigation on the vertical bearing behavior of monopod caisson foundations embedded in sandy soils and supporting offshore wind turbines. The effects of embedment depth and internal soil plug were investigated. The magnitude of lateral soil deformation at the foundation tip is noted to increase with skirt length and it remains greater in comparison with that of an embedded solid foundation. The internal soil plug is found to have a cushioning effect during the transfer of superstructure load to the soil mass underneath the foundation tip. This leads to a reduction in the end-bearing component of vertical capacity when compared with that of an embedded solid foundation. The failure of the soil at the tip resembles the shape of an inverted parabola in the case of caisson foundation and that of an upright parabola for the embedded solid foundation. Based on the results, predictive expressions have been proposed to determine the ultimate vertical bearing capacity of the caisson foundation and its settlement for the range of geometry and superstructure load considered in the study. DOI: [10.1061/\(ASCE\)WW.1943-5460.0000634](https://doi.org/10.1061/(ASCE)WW.1943-5460.0000634). © 2021 American Society of Civil Engineers.

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Introduction

Wind energy farms are moving seaward due to several factors: unavailability of suitable land locations, noise pollution produced by wind turbines during operation, visual impact on the natural environment, and availability of stronger and more stable winds offshore. Offshore wind turbine foundations are subjected to large lateral forces arising from wind forces and water waves, and the vertical self-weight loading is substantially less. The most commonly used monopile foundation system is expensive due to the necessary pile-driving equipment, and is also not recyclable after installation. The caisson foundation system has emerged as a practical alternative due to its cost-effective installation and retrieval methods. It is also known as suction caisson, bucket foundation, skirted caisson, or skirted foundation.

At the end of 2016, the first four leading countries in terms of installed cumulative offshore wind-farm capacity were the United Kingdom, Germany, China, and Denmark (GWEC 2017). The spread of the offshore wind industry to North America, East Asia, India, and elsewhere is picking up. In India, the first offshore wind farm project is being planned in an area of about 70 sq km, near the Gulf of Khambat, approximately 25 km off the coast of Gujarat, in water depth ranging from 10 to 18 m.

Usually, the locations set aside for offshore wind farms generally have water depths of up to 30 m, with 10 m being common. For adopting a monopod caisson foundation system, the selected sites are those without significant underwater seabed currents that can lead to scouring and cause reduction in the skirt length of the

caisson foundation. The monopod caisson foundation must resist vertical load, lateral load, and moment. The lateral load is produced due to the resultant action of wind force and water waves, and acts on the turbine tower at a particular loading height, measured vertically above the caisson lid level. At the caisson lid level, this eccentric resultant lateral load gives rise to an overturning moment. The vertically acting superstructure load of the wind turbine tower and its components is usually smaller in magnitude in comparison with the lateral load and overturning moment. The overturning moment generated due to the resultant water wave and wind force is a crucial aspect in the design of the foundation. For a sandy seabed, drained condition is reasonable considering the usual loading rates of the wave effects.

Offshore wind turbine development is continuing its increasing trend, and the industry continues to push toward larger turbines, seeking rated capacities greater than 8 MW in deeper waters. Larger turbine sizes are generally preferred in offshore applications because of fewer installations and lower maintenance, and many developers are anticipating continued turbine growth, with the expectation that the benefits of increased turbine size will continue to lower energy costs.

The behavior of skirted foundations under vertical loading in sandy soils has been studied through 1-g physical modeling (Byrne and Houlsby 2002; Villalobos 2007; Al-Aghbari and Dutta 2008; Wakil 2013; Haddad et al. 2019), through numerical modeling (Park et al. 2016), and through model tests and numerical analyses (Eid 2013; Barari et al. 2017).

Byrne and Houlsby (2002) found that the rate of cyclic and monotonic loading had little effect on the vertical response of suction caissons embedded in sand saturated with viscous silicon oil. Villalobos (2007) compared the results of laboratory experiments on scaled skirted foundations in dry sandy soils of different relative densities under pure vertical load with theoretical bearing capacity results, and observed that the usage of peak angle of internal friction overestimated the bearing capacity. Al-Aghbari and Dutta (2008) showed that the introduction of structural skirt enhances vertical bearing capacity and decreases settlement of square footing resting on dry sand, and proposed an equation to predict settlement reduction.

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Wakil (2013) reported that the enhancement of vertical bearing capacity due to the skirt of circular footing was found to be more effective in the case of dry loose sand as compared with that in dry medium dense and dense sands. Haddad et al. (2019) reported that the vertical bearing capacity of bucket foundation in loose saturated sand might be enhanced up to more than five times in comparison with surface footing with equivalent diameter depending on skirt length, and the corresponding settlement of bucket foundation at the failure load was found to increase with skirt length.

Park et al. (2016) studied the load transfer characteristics of bucket foundations in saturated sand through two-dimensional axisymmetric finite element analyses, and proposed predictive equations for shaft resistance and for a combined shape and depth factor of the base capacity. Eid (2013) presented correlations to determine the minimum bearing capacity and the maximum settlement of skirted square foundation of any aspect ratio on dry medium to dense sand from the corresponding magnitudes of surface square foundation. However, the bearing capacity ratios of the skirted foundation to that of the surface foundation obtained from the model testing were found to be much higher than those determined from the numerical analysis. It was also shown that the bearing capacity ratio of skirted foundation to that of solid foundation is approximately equal to 0.93, regardless of foundation size and sand relative density. Barari et al. (2017) compared the results of small-scale model tests and finite element analyses of bucket foundation in saturated dense sand, and proposed depth factor relationships to approximate the vertical capacity in terms of the capacity of surface foundation and embedment ratio. They also deduced that the bearing capacity of bucket foundation is slightly less than a solid foundation at the same embedment depth.

The behavior of skirted foundations in clayey soils under vertical loading has been studied through numerical simulations (Bransby and Randolph 1999; Mana et al. 2010), through centrifuge tests and numerical modeling (Mana et al. 2012; Yun and Bransby 2007), and through field tests and numerical analyses (Barari and Ibsen 2014).

Bransby and Randolph (1999) conducted finite element and upper-bound analyses to investigate the effect of embedment depth on the size and shape of yield locus for skirted strip foundation in undrained clay, and proposed design charts and equations to calculate the bearing capacity under general loading. Mana et al. (2010) presented a closed-form expression for the calculation of bearing capacity factors for circular skirted foundations in saturated clayey soil over a practical range of embedment ratio, skirt-soil interface roughness, and soil strength heterogeneity.

Later, Mana et al. (2012) reported that a comparison of the failure mechanisms predicted by finite element analyses with those observed in centrifuge tests showed some marked differences, though there was a close agreement of bearing capacity factors. Yun and Bransby (2007) proposed a design method to determine the vertical bearing capacity of skirted square foundations in normally consolidated clayey soil, which was then validated with centrifuge test results. Barari and Ibsen (2014) performed field tests and numerical analyses with monotonic loading on small-scale bucket foundations of different geometries in undrained clay, and presented a quadratic equation to predict vertical bearing capacity based on the embedment ratio.

The capacity of skirted foundations in sandy soils under combined vertical and lateral loading has been investigated through 1-g small-scale model tests (Byrne et al. 2003; Larsen et al. 2013; Ibsen et al. 2014), and through finite element simulations (Liu et al. 2014).

Byrne et al. (2003) developed an expression for moment capacity of shallow skirted foundations in dry sand from the test results

obtained by combined loading under very low vertical load of constant magnitude. Larsen et al. (2013) and Ibsen et al. (2014) developed an expression to describe the combined capacity of bucket foundations in saturated dense sand as a function of the tensile capacity and the inclination factor by calibrating the failure criteria. Liu et al. (2014) proposed new simplified calculation methods for the vertical load-bearing capacity and the overturning stability of wide-shallow caisson foundations in saturated silty sand, and a comparison with the results of finite element analysis showed that the proposed equations properly predicted the capacities.

The capacity of skirted foundations in clayey soils under combined vertical and lateral loading has been studied using finite element analyses (Gourvenec and Barnett 2011; Hung and Kim 2014; Mehravar et al. 2016; Fu et al. 2017).

Gourvenec and Barnett (2011) proposed a closed-form expression that enabled prediction of undrained bearing capacity of skirted foundations in clay under general in-plane loading, which was valid for a range of embedment ratios and soil shear strength heterogeneities. Hung and Kim (2014) proposed new equations to calculate the vertical, horizontal, and moment-bearing capacities of bucket foundations in nonhomogeneous clay, and also to define the capacity envelope under general combined loads. Mehravar et al. (2016) proposed equations to predict the ultimate vertical, maximum horizontal, and moment-bearing capacity factors of caisson foundations in homogeneous undrained clay, and presented the kinematic mechanism accompanying failure under uniaxial loadings for different embedment ratios. Fu et al. (2017) outlined a novel method to estimate the undrained ultimate uniaxial vertical, horizontal, and rotational capacity factors of skirted circular foundation in clay for any given embedment ratio and strength heterogeneity, by separating the contributions of soil above and beneath the skirt tip.

From the literature review, it is noted that many studies have been conducted to determine numerically the vertical bearing behavior of monopod caisson foundation supporting offshore wind turbine in clays. However, similar studies in sands are limited. The objective of this study is to obtain a better insight on the vertical load transfer characteristics, surrounding soil deformation, and failure mechanism, which are fundamental issues in its design and application in sandy soils.

Numerical Analysis

Three-Dimensional Finite Element Modeling

The vertical response of the three-dimensional monopod caisson foundation was simulated numerically using finite element software (Abaqus, version 6.10). The difficulties associated with caisson installation and its effect due to soil rearrangement were not considered in this study, and the simulations were performed assuming the caisson to be wished-in-place. Considering the geometric and loading symmetry, only half of the entire system was modeled, as shown in Fig. 1.

The displacements of the soil domain at the base boundary were fixed in every direction. In addition, normal displacements were constrained in the symmetric plane, and lateral displacements were restrained along the curved periphery of the soil domain. Hence, there was zero horizontal soil displacement on the lateral boundary and zero vertical soil displacement at the base. The soil and the caisson foundation were modeled using an eight-node linear brick element with reduced integration scheme (C3D8R). The foundation was modeled as an undeformable rigid body. In the vicinity of caisson, the soil domain was discretized with finer mesh

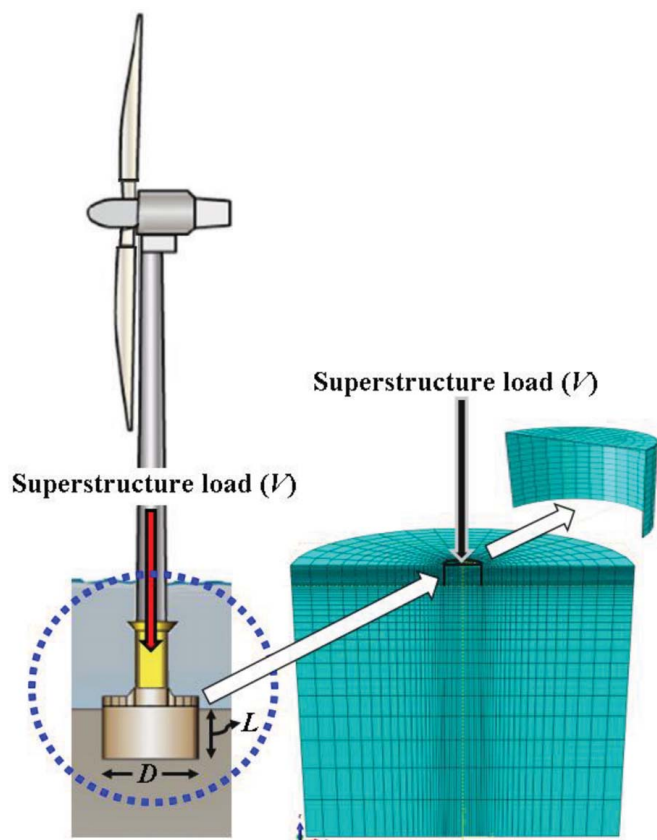


Fig. 1. Schematic diagram of monopod caisson foundation embedded in soil domain.

and the size of the mesh was increased gradually toward the domain boundary. This allowed accurate modeling at reduced computational expense.

The vertical and lateral extents of the soil domain were determined based on several trials to prevent any boundary effects on the response of the foundation. Once the domain size was fixed, a mesh convergence study was conducted to obtain the number of elements required for accurate results.

In the analysis, in the first step, geostatic load was applied to achieve the initial stress state in the whole soil domain. In the second step, contact between the caisson and soil was established. The interface between the caisson surface and the corresponding adjacent soil surface was modeled using a master–slave contact pair algorithm. The stiffer caisson foundation was chosen as the master surface and the soil surface was selected as the slave surface. The interface friction angle between master and slave surfaces was considered as two-thirds of the internal soil-friction angle. In the third step, during the simulation of vertical capacity, a displacement-controlled vertical load was applied at the center of the foundation top until failure was reached.

The experimental study on caisson foundation under vertical load indicated that the soil mass inside the caisson foundation gets compressed, with negligible vertical soil movement (Mana et al. 2012). Moreover, several numerical studies were also performed considering the rough interface between the internal skirt surface and the adjacent soil surface, which adequately modeled the limited deformation behavior of soil plug entrapped within the caisson foundation (Yun and Bransby 2007; Bransby and Yun 2009; Park et al. 2016). However, the interface between the external caisson skirt surface and the surrounding soil surface

Table 1. Soil parameters used in the study

Soil parameter	Medium dense sand	Very dense sand
Effective unit weight (γ'_{soil}), kN/m ³	9.0	11.0
Oedometric stiffness modulus, κ	400	600
Oedometric stiffness modulus, λ	0.60	0.55
Poisson's ratio, μ	0.25	0.25
Internal friction angle, ϕ'	35°	40°
Dilation angle, δ'	5°	10°
Cohesion (c'), kN/m ²	0.1	0.1

was modeled to allow relative movement due to significant deformation of the surrounding soil (Park et al. 2016).

To eliminate the effect of foundation self-weight on the vertical response, the unit weight of the foundation was taken as equal to the submerged unit weight of the surrounding soil (Gourvenec and Mana 2011; Mana et al. 2010, 2013), and this facilitated in setting up the geostatic stress field.

For the stress state in the initial stage, the sandy soil was modeled using the Mohr–Coulomb model. The development of plastic strains before and during failure due to nonlinear behavior of the soil was modeled with the stress-dependent modulus of elasticity (E_{soil}) of the soil mass, which varies with the mean stress as

$$E_{\text{Soil}} = \kappa \cdot \sigma_{at} \cdot \left(\frac{\sigma_m}{\sigma_{at}} \right)^{\lambda} \quad (1)$$

where $\sigma_{at} = 100$ kN/m² is reference stress; σ_m = mean principal stress; κ = the soil stiffness at the reference stress state; and λ = the stress dependency of the soil stiffness. The procedure for experimental determination of the parameters κ and λ is elaborated in the Appendix. The values of κ and λ utilized to simulate the nonlinearity of the medium and very dense sands have been taken from EAU (2004) and are presented in Table 1, along with other soil properties. However, it may be noted that this approach provides only an approximate match to the steady nonlinearity of the soil response and the final stress state.

Numerical Model Validation

The numerical model was validated under vertical load by comparing with the results of Liu et al. (2014). For the numerical validation, a caisson foundation of 30 m diameter and 15 m skirt length, embedded in soil domain comprised of normally consolidated silty sand, was modeled. The soil parameters, namely, modulus of elasticity, Poisson's ratio, effective unit weight, internal friction angle, and cohesion, were taken as 14.2 MPa, 0.35, 9.7 kN/m³, 30.5°, and 9 kN/m², respectively. The buoyant unit weight of the reinforced concrete caisson foundation was taken as 15.2 kN/m³. Thereafter, displacement-controlled lateral load was applied at the central lid location of the caisson foundation. Fig. 2 presents a comparison of vertical load capacity–settlement response of caisson foundation with that of the published results. It can be noted that there is a good agreement between them.

Parametric Study

The behavior of the caisson foundation was compared with that of surface circular and embedded solid foundations. Fig. 3 shows schematic diagrams of the three foundation types, which were assumed to be made up of the same steel material. The details of the foundation geometries are presented in Table 2. The foundation diameters (D) were taken as 12, 15, and 18 m, with aspect ratio varying from 0.5 to 1 in the case of caisson and embedded solid

foundations. The thicknesses of the surface circular foundation, caisson foundation lid, and caisson skirt were chosen as 0.1, 0.1, and 0.03 m, respectively.

Results and Discussion

Vertical Load–Settlement Response

In general, the ultimate or failure state of the foundation can be assumed to have reached when there is a continuous increase in settlement with no further increase in the load-carrying capacity. But in certain cases, even though a large settlement has been reached, the vertical load capacity continues to increase incrementally. For such cases, the ultimate capacity can be defined corresponding to a settlement, which is sufficiently high to cause collapse of the structure. In the present study, the ultimate vertical capacity of the considered foundations is taken as the vertical load applied at the top-center of the foundation, which causes a vertical settlement equal to 10% of the respective foundation diameter, measured at the top-center (Eid 2013; Park et al. 2016).

The variations of vertical load–settlement plots of surface circular, caisson, and embedded solid foundations are depicted in Figs. 4(a and b), respectively, for medium dense and very dense sandy seabeds. For all the foundation types, the settlement is noted to vary nonlinearly with applied vertical load. The surface circular foundation is noted to yield the least vertical capacity, whereas the maximum vertical capacity is obtained for the embedded solid foundation. Similar trends of vertical load capacity versus settlement plots are observed for the other foundation diameters, and

the ultimate vertical capacity is noted to be higher when embedded in very dense sand. The ultimate vertical capacities obtained for all three foundation types are presented in Table 3.

As compared with a surface circular foundation of the same diameter, the vertical capacity of a caisson foundation is noted to improve due to the inclusion of skirt. For example, in medium dense sand, the ultimate vertical bearing capacity of the surface circular foundation of 12 m diameter is obtained as 117.07 MN. For the caisson foundation of same diameter and skirt length of 6 m (aspect ratio of 0.5), the ultimate vertical capacity is noted to improve to 126.98 MN, that is, an increase in capacity by 8.47% over that of the surface foundation. For the solid foundation of same diameter and embedment length, the ultimate vertical capacity of 151.08 MN is obtained, that is, an increase in capacity by 18.98% over that of the caisson foundation. As the aspect ratio is increased to 0.75 and 1.0, the ultimate vertical capacity of caisson foundation of 12 m diameter is noted to increase by 9.86% and 14.87%, as compared with that of the surface circular foundation. Corresponding to the aspect ratios, the ultimate vertical capacity of the solid foundation of 12 m diameter is noted to increase by 23.93% and 26.96%, as compared with the caisson foundation.

In very dense sand, the ultimate vertical bearing capacity of the 12 m surface circular foundation is found to be 388.58 MN. As compared with the surface circular foundation, for the aspect ratios of 0.5, 0.75, and 1.0, the ultimate vertical capacity of the caisson foundation is observed to increase by 8.53%, 8.75%, and 11.01%, respectively, to values of 421.73, 422.59, and 431.35 MN. For the corresponding aspect ratios, the ultimate vertical capacity

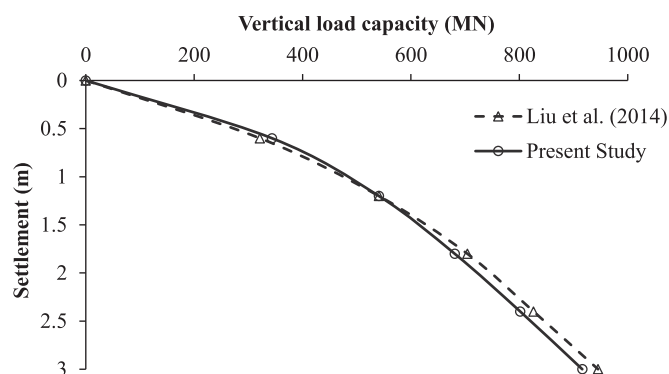


Fig. 2. Vertical load capacity–settlement response of caisson foundation embedded in normally consolidated silty sand.

Table 2. Geometric details of the foundations used in the study

Diameter, D (m)	Skirt length, L (m)			Aspect ratio (L/D)
	Surface circular foundation	Caisson foundation	Embedded solid foundation	
12	—	6	6	0.5
	—	9	9	0.75
	—	12	12	1
15	—	7.5	7.5	0.5
	—	11.25	11.25	0.75
	—	15	15	1
18	—	9	9	0.5
	—	13.5	13.5	0.75
	—	18	18	1

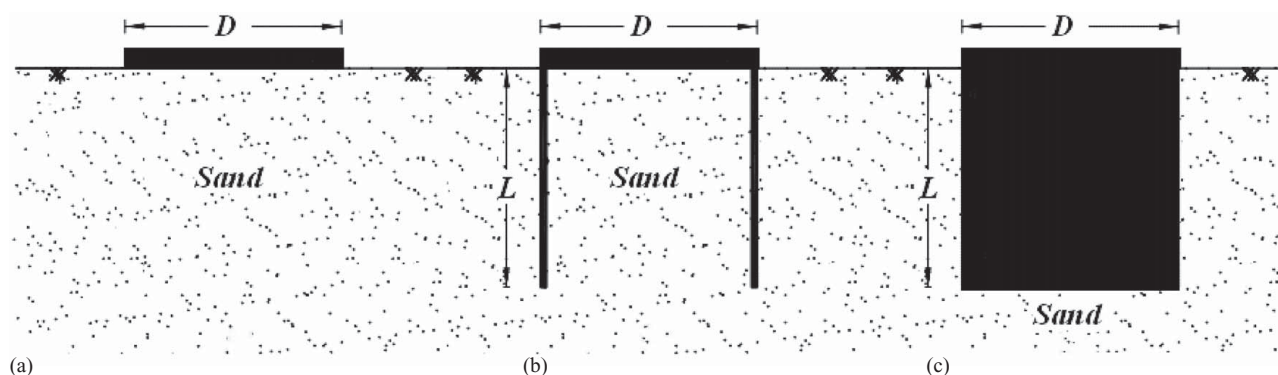


Fig. 3. Foundations used for numerical analysis: (a) surface circular foundation; (b) caisson foundation; and (c) embedded solid foundation.

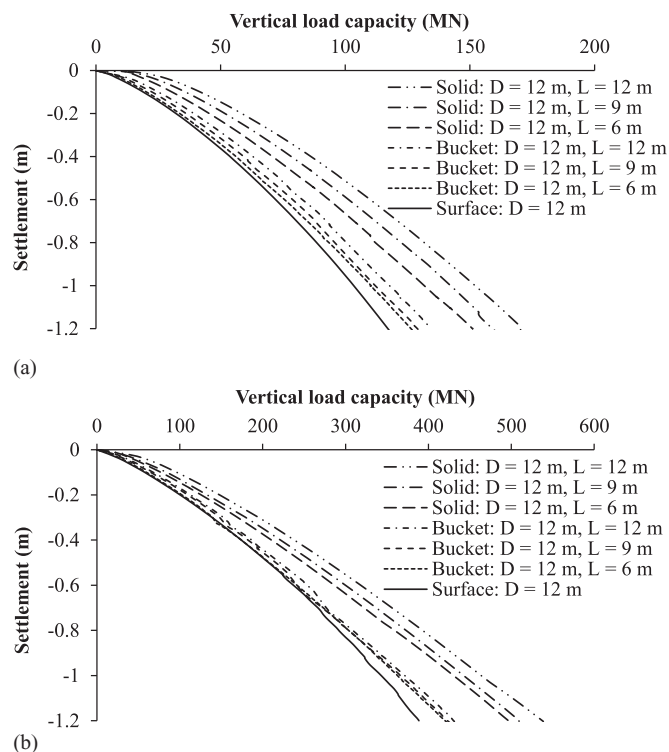


Fig. 4. Vertical load versus settlement response of the three foundation types ($D = 12$ m) in (a) medium dense sand; and (b) very dense sand.

Table 3. Ultimate vertical capacity of all foundation types

Sand type	Foundation type	Aspect ratio (L/D)	Ultimate vertical capacity (MN)		
			$D = 12$ m	$D = 15$ m	$D = 18$ m
Medium dense sand	Surface	0	117.07	193.51	296.90
	Caisson	0.5	126.98	206.56	308.40
		0.75	128.61	207.20	320.20
		1	134.36	218.76	335.48
	Solid	0.5	151.08	250.42	380.56
		0.75	159.39	270.59	411.49
Very dense sand	Surface	0	388.58	639.33	969.81
	Caisson	0.5	421.73	651.17	996.23
		0.75	422.59	669.14	1,010.26
		1	431.35	694.14	1,028.07
	Solid	0.5	498.07	806.63	1,198.65
		0.75	510.07	852.23	1,263.11
		1	538.68	873.82	1,317.41

values of the embedded solid foundation are 498.07, 510.07, and 538.68 MN, respectively, which are greater than that of the caisson foundation by 18.10%, 20.70%, and 24.88%.

Lateral Movement of Soil Mass near the Foundation Tip at Failure

At ultimate condition, below mid-depth of the embedment length, outward movement or lateral expulsion of the soil mass was noted for both caisson and embedded solid foundations. In the case of a caisson foundation, the expulsion of the soil mass increases considerably at depths below the embedment length of the foundation, indicated in terms of lateral displacement vector diagram in

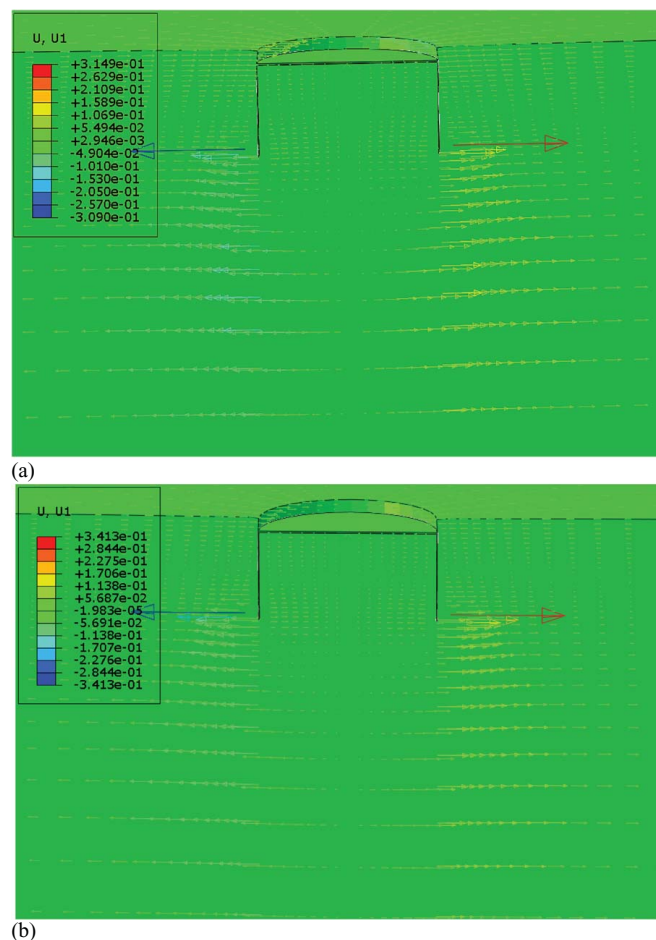


Fig. 5. Lateral displacement vector diagrams at failure for caisson foundation ($D = 12$ m and $L = 6$ m) in (a) medium dense sand; and (b) very dense sand.

Fig. 5. The lateral extents of soil movement at the tip of the two foundation types in both medium dense and very dense sands are listed in Table 4.

For the caisson foundation of 12 m diameter and aspect ratio of 0.5 in medium dense sand, the lateral expulsion of the soil mass is found to be 0.314 m away from the caisson tip [Fig. 5(a)]. As the aspect ratio is increased to 0.75 and 1, the lateral expulsion magnitude is noted to decrease to 0.284 and 0.257 m, respectively. The magnitude of lateral expulsion is noted to decrease with a higher aspect ratio in the case of caisson foundation. The lateral soil expulsion magnitude is noted to be lesser in case of embedded solid foundation as there is no soil plug inside. Further, in contrast to that of the caisson foundation, the lateral expulsion is noted to increase with aspect ratio. For the 12 m diameter embedded solid foundation, the lateral expulsion extents have been found as 0.127, 0.137, and 0.139 m for aspect ratios of 0.5, 0.75, and 1, respectively.

For the caisson foundation of 12 m diameter and aspect ratio of 0.5 in very dense sand, caisson–soil interface strength mobilization at failure is greater due to a higher angle of internal friction, and this gives rise to greater magnitude of lateral outward movement of soil mass at any location along the embedment length, including the tip [Fig. 5(b)], as compared with that of the caisson foundation embedded in medium dense sand [Fig. 5(a)]. For either foundation type, the magnitude of soil movement at the foundation tip is noted to be greater in the case of very dense sand as compared with medium dense sand.

Table 4. Lateral displacements of soil mass at foundation tip level at failure

Diameter	Aspect ratio (L/D)	In medium dense sand		In very dense sand	
		Caisson foundation (m)	Embedded solid foundation (m)	Caisson foundation (m)	Embedded solid foundation (m)
$D = 12$ m	0.5	0.314	0.127	0.341	0.137
	0.75	0.284	0.137	0.309	0.142
	1	0.257	0.139	0.279	0.141
$D = 15$ m	0.5	0.345	0.154	0.373	0.167
	0.75	0.253	0.168	0.265	0.168
	1	0.193	0.171	0.231	0.192
$D = 18$ m	0.5	0.279	0.196	0.308	0.198
	0.75	0.315	0.199	0.326	0.204
	1	0.239	0.204	0.305	0.209

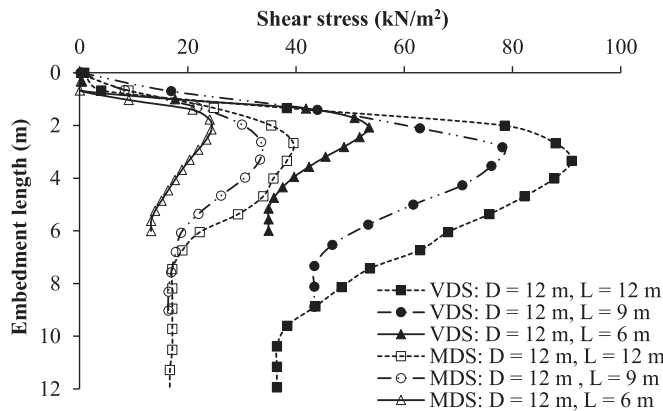


Fig. 6. Variation of shear stress along embedded length of caisson foundation ($D = 12$ m) at failure.

The variations of shear stress with embedment length of caisson foundation of 12 m diameter, embedded in both sand types, are shown in Fig. 6. At failure load, the shear stress is noted to increase parabolically along the embedment length, attains a peak, and then reduces gradually. For example, in case of the caisson foundation of 12 m diameter and aspect ratio of 0.5 embedded in medium dense sand, the shear stress increases gradually from bed level and attains peak stress of 24.38 kN/m² at a depth of 2.15 m. As the aspect ratio is increased to 0.75 and 1, the peak stresses increase to 33.53 and 39.50 kN/m² at greater depths of 2.63 and 2.67 m, respectively.

For a given sand type, the magnitude of peak shear stress is noted to increase with caisson geometry size (i.e., with embedment length for the same diameter and also with diameter for the same aspect ratio), and the magnitude is observed to be higher when embedded in very dense sand. The maximum shear stress is not found at the bottom portion of the embedment length of caisson, due to the lateral expulsion of soil away from the skirt surface at the bottom, which is noticeable from the lateral displacement vector diagram (Fig. 5).

Failure Mechanisms

Under the application of vertical load until failure, any given foundation settles, giving rise to plastic strains. The patterns of the generated plastic strains in medium dense sand for caisson and embedded solid foundations at failure are illustrated in Figs. 7(a and b). For both foundation types, the surrounding soil mass along the exterior interface of the skirt is noted to yield

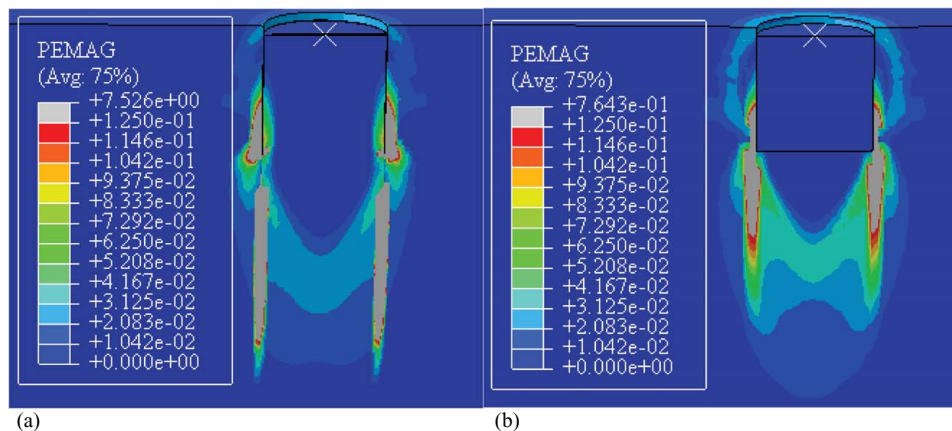
plastically in the bottom half of the embedment length. Both the foundations fail under punching shear, and a confined deep-flow mechanism is observed along with the formation of an elastic zone in the shape of a wedge. In the case of a caisson foundation, the interior soil plug also yields plastically along the interior interface of the skirt near the base of the foundation [Fig. 7(a)]. In the case of an embedded solid foundation, the region of plastically yielded soil extends downward from the foundation base to a considerable depth in the soil mass while surrounding an elastic wedge [Fig. 7(b)].

At failure, the movement of soil under vertical load is illustrated in terms of vertical displacement vectors in Figs. 8(a and b). The response of the soil plug inside the caisson foundation and the load transfer mechanism are highlighted by comparing the vertical displacement vector diagrams. The vertical displacement vector diagram beneath the caisson foundation resembles the shape of an inverted parabola [Fig. 8(a)], in which the minimum and maximum displacements of the soil are noted at the center and at the edges, respectively. This indicates that the end-bearing load transferred by the soil plug shows some softening or cushioning effect. The soil plug is noted to act as an integral part of the caisson foundation. In contrast, the vertical displacement vector diagram at the embedded foundation base resembles the shape of an upright parabola [Fig. 8(b)], with the maximum and minimum vertical displacements of the soil at the center and at the edges, respectively.

Effect of Soil Plug on End-Bearing Capacity

To determine the effect of a soil plug on the ultimate vertical bearing capacity of caisson foundations, numerical simulations were carried out by considering zero external shaft resistance by assigning a fully smooth soil–foundation interface. As a result, the vertical capacities obtained for both the caisson foundation and embedded solid foundation are only due to the end-bearing resistance. The end-bearing capacity of the caisson foundation is found to be less than that of the embedded solid foundation. The effect of a soil plug on the vertical bearing capacity of the caisson foundation is reported in Table 5 as the difference in the end-bearing capacities. The percentage reduction compared with that of the embedded solid foundation is also presented. This reduction in the end-bearing capacity is attributed to the cushioning effect of the soil plug inside the caisson foundation, as already observed from the vertical displacement vector diagram [Fig. 8(a)].

For a constant caisson diameter, the magnitude of reduction in the end-bearing capacity is noted to increase with aspect ratio. This is because of the narrower soil deformation zone that is involved in the failure mechanism with increasing aspect ratio (Table 4). Conversely, in the case of the embedded solid foundation,



the soil deformation zone becomes wider with the increase in aspect ratio.

For example, in medium dense sand, the end-bearing capacity reduces by 33.07 MN for a caisson foundation with diameter and aspect ratio of 12 m and 0.5, respectively. Further, for the same diameter, the difference of end-bearing capacity is noted to increase to 33.40 and 35.78 MN, respectively, for increased aspect ratio values of 0.75 and 1. In very dense sand, for a caisson foundation of 12 m diameter and aspect ratio of 0.5, the reduction in the end-bearing capacity is noted to be 88.58 MN. The difference increases to 94.51 and 92.11 MN, respectively, for aspect ratios of 0.75 and 1.0. With increasing caisson size, the reduction in end-bearing capacity is greater. The magnitude of reduction is also noted to be higher in very dense sand as compared with in medium dense sand.

Variation of Ultimate Vertical Bearing Capacity with Geometry

The variation of ultimate vertical capacity with skirt length for different caisson foundation diameters in medium dense and very dense sands are presented in Fig. 9(a), and with diameter for different skirt lengths are shown in Fig. 9(b). From the figures, the ultimate vertical capacity of caisson foundation is noted to increase with both skirt length and with diameter.

From Fig. 9(a), for a fixed skirt length of 12 m for the caisson foundation in medium dense sand, the ultimate vertical capacity increases from 134.36 MN to an interpolated value of 317.43 MN (a difference of 183.07 MN) as the diameter is varied by 6 m magnitude from 12 to 18 m. Similarly, from Fig. 9(b), for a fixed diameter of 12 m, it is noted that the ultimate vertical capacity decreases from 134.36 to 126.98 MN (a difference of 7.38 MN) as the skirt length is varied by 6 m magnitude from 12 to 6 m. It is evident that as compared with diameter, increasing skirt length has less influence on the ultimate vertical capacity. A similar trend is observed with substantially higher ultimate vertical capacity for caisson foundations embedded in very dense sand. The ultimate vertical capacity for any other geometry within the considered diameter range (12–18 m) and skirt length range (6–18 m) can be approximately estimated from the plots.

Predictive Expressions

The ultimate vertical capacities from Table 3 have been utilized to determine vertical capacity depth factor (D_{vc}), being defined as the ratio between the ultimate vertical bearing capacity of caisson

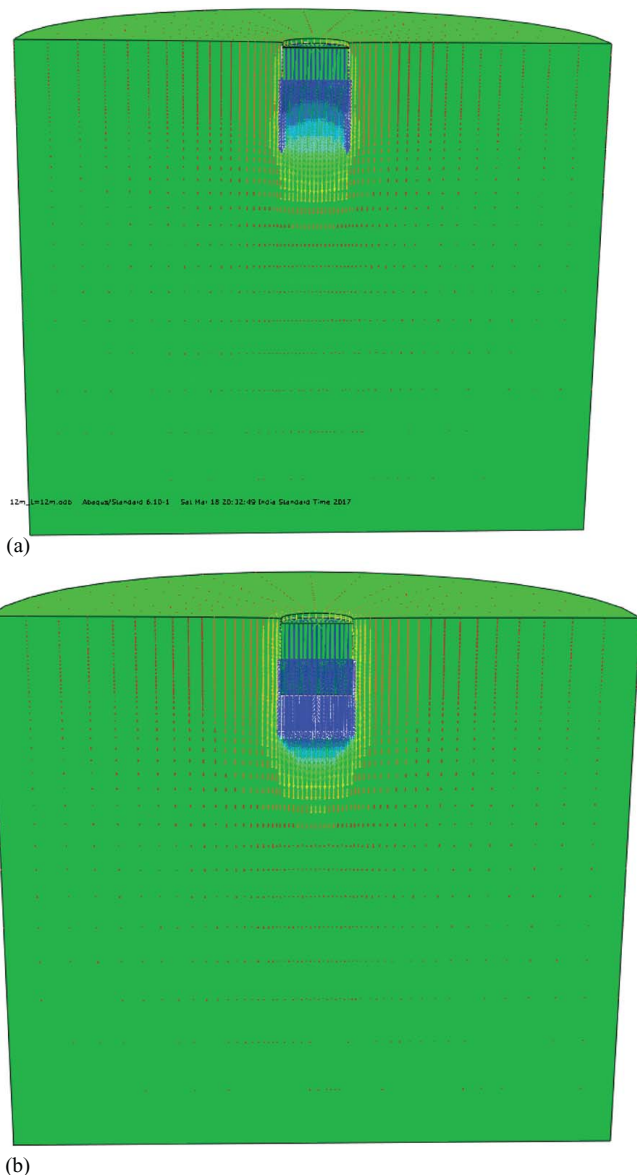
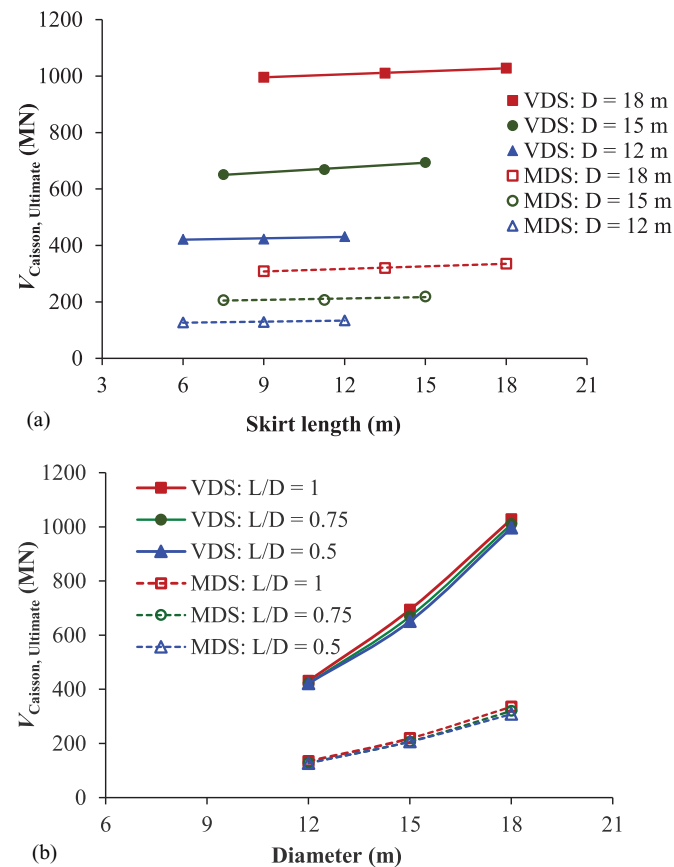


Fig. 8. Vertical displacement vector diagram in medium dense sand at failure of (a) caisson foundation ($D = 12$ m and $L = 12$ m); and (b) embedded solid foundation ($D = 12$ m and $L = 12$ m).

Table 5. Difference in end-bearing capacities of caisson and embedded solid foundations

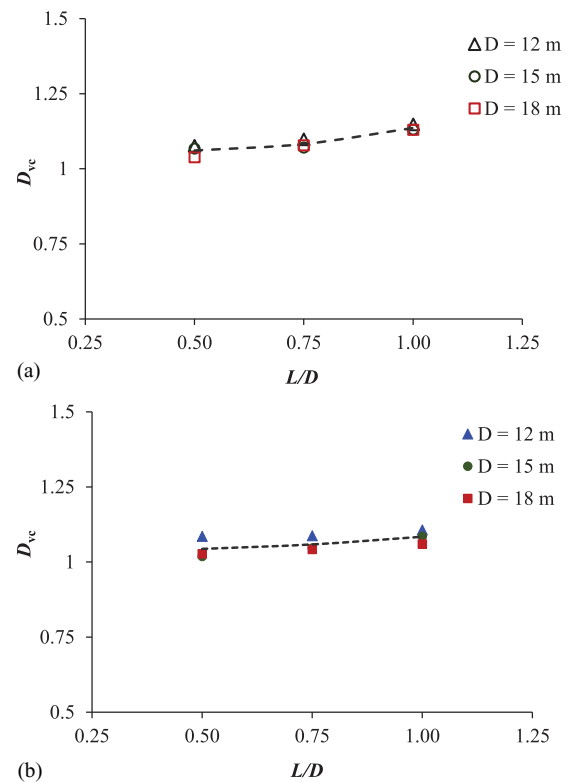
Diameter	Aspect ratio (L/D)	In medium dense sand		In very dense sand	
		Difference in end-bearing capacity (MN)	Reduction in end-bearing capacity (%)	Difference in end-bearing capacity (MN)	Reduction in end-bearing capacity (%)
$D = 12$ m	0.5	33.07	22.41	88.58	18.60
	0.75	33.40	21.59	94.51	19.43
	1	35.78	22.33	92.11	18.84
$D = 15$ m	0.5	49.70	20.49	136.84	17.44
	0.75	62.54	24.46	149.21	18.41
	1	59.75	22.01	163.71	19.64
$D = 18$ m	0.5	68.89	18.81	212.67	18.13
	0.75	93.25	23.94	238.16	19.77
	1	106.51	25.56	283.79	22.67

**Fig. 9.** Variation of ultimate vertical capacity of caisson foundation in medium dense and very dense sands: (a) with skirt length; and (b) with diameter.

foundation of a particular skirt depth to that of surface circular foundation of the same diameter:

$$D_{vc} = \frac{V_{\text{Caisson, Ultimate}}}{V_{\text{Surface, Ultimate}}} = a \left(\frac{L}{D} \right)^2 + b \left(\frac{L}{D} \right) + c \quad (2)$$

Figs. 10(a and b) show the variation of vertical capacity depth factor with aspect ratio in medium dense and very dense sands. Generally, as compared with diameter of the foundation, skirt length and soil-friction angle have greater influence on the vertical capacity depth factor. For both the sand types, the depth factor is noted to increase almost nonlinearly with aspect ratio. The data points in the figure have been utilized to obtain a best-fit curve

**Fig. 10.** Variation of vertical capacity depth factor with aspect ratio: (a) in medium dense sand; and (b) in very dense sand.**Table 6.** Values of coefficients for vertical capacity depth factor

Coefficient	Soil type	
	Medium dense sand	Very dense sand
a	0.2549	0.0852
b	−0.2324	−0.4680
c	1.1135	1.0458

represented by the general expression, as given in Eq. (2). From curve fitting analysis, the values of the coefficients for the predictive expression of vertical capacity depth factor in both sand types are presented in Table 6. This expression can be utilized to estimate the ultimate capacity of any caisson foundation geometry within the range of the parametric study, if the corresponding capacity of surface foundation is known.

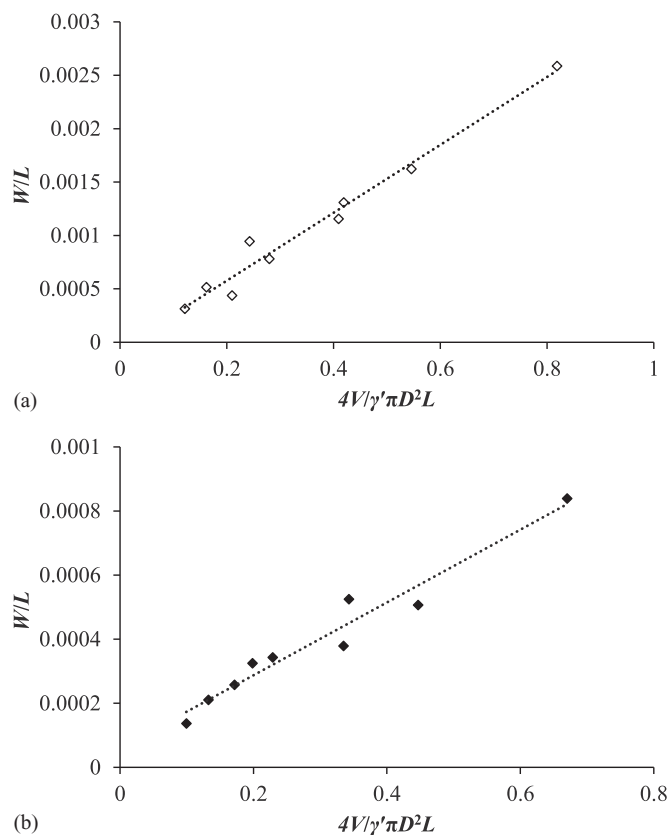


Fig. 11. Variation of normalized settlement with caisson foundation geometry under superstructure load of 5 MN: (a) in medium dense sand; and (b) in very dense sand.

Table 7. Values of coefficients to determine settlement under superstructure load

Soil type	Coefficient	Settlement under superstructure load (m)		
		$V = 5$ MN	$V = 10$ MN	$V = 15$ MN
Medium dense sand	a^*	0.0032	0.0032	0.0032
	b^*	0	-0.0001	-0.0002
Very dense sand	a^*	0.0011	0.0011	0.0012
	b^*	0	0.0001	0.0001

Typical self-weight of a 5 MW offshore wind turbine is 10 MN (Malhotra 2011; Achmus et al. 2013; Ahmed and Hawlader 2016). With increasing capacity and size of the wind turbine, the self-weight increases.

The settlement values are now normalized with respect to skirt length and the superstructure load is normalized with respect to the weight of the soil plug ($\gamma'\pi D^2 L/4$), and the data points are utilized to fit straight lines as in Fig. 11 for superstructure load of 5 MN in both medium dense and very dense sands. An expression has been arrived at for predicting the settlement for wind turbine superstructure load ranging from 5 to 15 MN, as in Eq. (3). The coefficients of the equation are presented in Table 7.

Eq. (3) can provide only an initial approximation to estimate the settlement under the range of superstructure loads for nine caisson geometries considered in this study. Further investigation is necessary considering a wide range of caisson geometries and superstructure loads to enable accurate prediction of settlement of

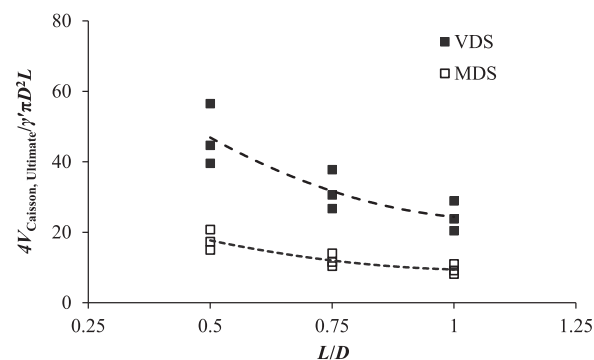


Fig. 12. Variation of normalized ultimate vertical capacity with aspect ratio of caisson foundation in medium dense and very dense sands.

Table 8. Values of coefficients to determine ultimate vertical capacity

Sl. no.	Coefficient	Ultimate vertical bearing capacity (MN)	
		Medium dense sand	Very dense sand
1	a'	25.135	63.096
2	b'	-54.211	-139.71
3	c'	38.511	100.98

caisson foundation.

$$\frac{W}{L} = \frac{4Va^*}{\gamma'\pi D^2 L} + b^* \quad (3)$$

The ultimate vertical bearing capacity of the caisson foundation has been normalized with respect to the weight of the soil plug ($\gamma'\pi D^2 L/4$) and plotted against aspect ratio, as shown in Fig. 12 for both medium dense and very dense sands. Based on the data points, an expression has been derived to determine the ultimate vertical bearing capacity of caisson foundation, given by Eq. (4). The values of the coefficients in the equation for the two sand types are presented in Table 8.

$$\frac{4V_{\text{Caisson, Ultimate}}}{\gamma'\pi D^2 L} = a' \left(\frac{L}{D} \right)^2 + b' \left(\frac{L}{D} \right) + c' \quad (4)$$

The expressions Eqs. (3) and (4) can serve as a base for preliminary design of the monopod caisson foundation of any geometry and embedment ratio within the range of the parametric study. Using Eq. (3), the settlement value of the foundation under the given superstructure load can be estimated, and its ultimate vertical capacity can be determined using Eq. (4). When using the results in practice, the submerged weight of the foundation minus the submerged weight of displaced soil should be subtracted from the computed vertical load capacity of the foundation at any settlement level. The remainder capacity is the superstructure load that the foundation can carry.

Conclusions

Based on the numerical study, the following main conclusions have been drawn:

1. Under the action of externally applied vertical load, the vertical bearing capacity of a caisson foundation for a given diameter is greater than that of a surface circular foundation and is lower than that of a solid foundation with the same skirt length. The

ultimate vertical capacity increases with aspect ratio and is also higher in very dense sand.

2. For a given diameter, greater lateral displacement of the soil mass away from the base tip is noted in the case of a caisson foundation compared with that of an embedded solid foundation.
3. The magnitude of maximum shear stress along embedment length obtained is noted to increase with caisson geometry size. The depth, where the maximum shear stress is encountered, increases with aspect ratio.
4. At failure, confined deep flow mechanism is observed for both caisson and embedded solid foundations with the formation of an elastic wedge zone. The failure of the soil at the tip resembles the shape of an inverted parabola in the case of caisson foundation and that of an upright parabola for the embedded solid foundation.
5. Due to the integral soil plug, the reduction in end-bearing component of vertical capacity of a caisson foundation is found to be less than that of an embedded solid foundation by 18%–25% in medium dense sand and by 17%–23% in very dense sand.
6. As compared with skirt length, increasing diameter showed higher influence on the ultimate vertical capacity of caisson foundation.
7. Predictive expressions have been proposed to determine the ultimate bearing capacity of the caisson foundation and its settlement for the range of geometry and superstructure load considered in the study.
8. The finite element analyses have provided insight into the vertical load bearing characteristics of the caisson foundation as well as about the soil deformation extent zones in the vicinity and the failure mechanism.

Appendix. Modeling of Soil Nonlinearity

In general, as soil is neither isotropic nor homogeneous, its behavior is nonuniform throughout a soil mass. The modulus of elasticity (E_{soil}) of the soil mass varies with the mean stress, and the values of κ and λ parameters utilized to simulate the nonlinearity of the soil can be determined from conventional laboratory drained triaxial shear tests performed for the determination of the shear strength parameters. In the first step, the test results are plotted in terms of deviator stress versus axial strain, as shown in Fig. 13, where σ_3 is the minor principal stress or confining pressure, σ_1 is major principal stress and ϵ is the axial strain. The initial elastic modulus (E_i) is the slope of the tangent line drawn from origin.

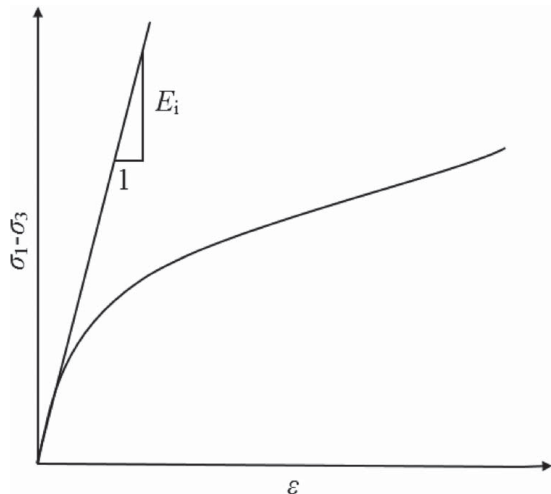


Fig. 13. Plot of $\sigma_1 - \sigma_3$ versus ϵ obtained from drained triaxial test.

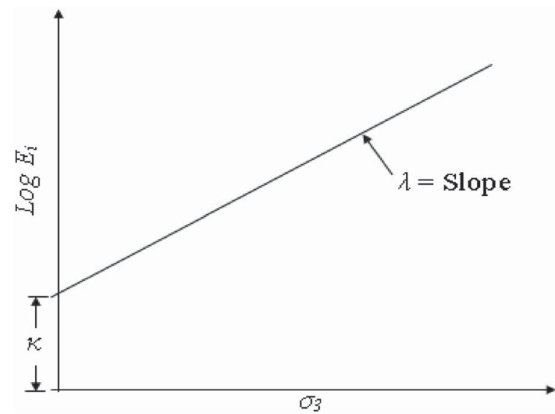


Fig. 14. Plot of σ_3 versus E_i in logarithmic scale.

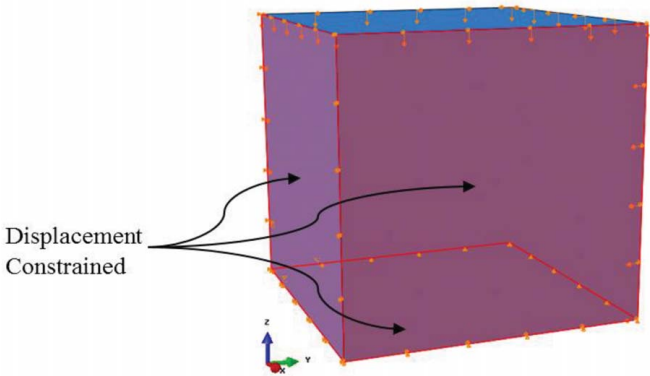


Fig. 15. Hydrostatic test considering a single element (C3D8R).

Table 9. Calculated mean stress and corresponding modulus of elasticity values used as input parameters in Abaqus (version 6.10)

Sl. no.	Mean stress (Pa) SDV2	E_{soil} (Pa) SDV1
1	0.01	8,475.23
2	2	156,213
3	4	228,710
4	6	285,848
5	8	334,850
6	10	378,574
7	12	418,505
8	14	455,535

In the second step, the test results for several confining pressures are plotted in terms of σ_3 versus logarithm of E_i , as shown in Fig. 14. In the third step, a straight line is fitted to the data points. The values of the nonlinear soil strength parameters, κ and λ , are respectively the intercept and the slope of this straight line.

Accordingly, in this study, in order to simulate the nonlinear behavior of sands realistically under a drained condition, the relationship between modulus of elasticity of soil (E_{soil}) of any element within the domain and the mean principal stress (σ_m), has been implemented as

$$E_{soil} = \kappa \cdot \sigma_{at} \cdot \left(\frac{\sigma_m}{\sigma_{at}} \right)^\lambda \quad (5)$$

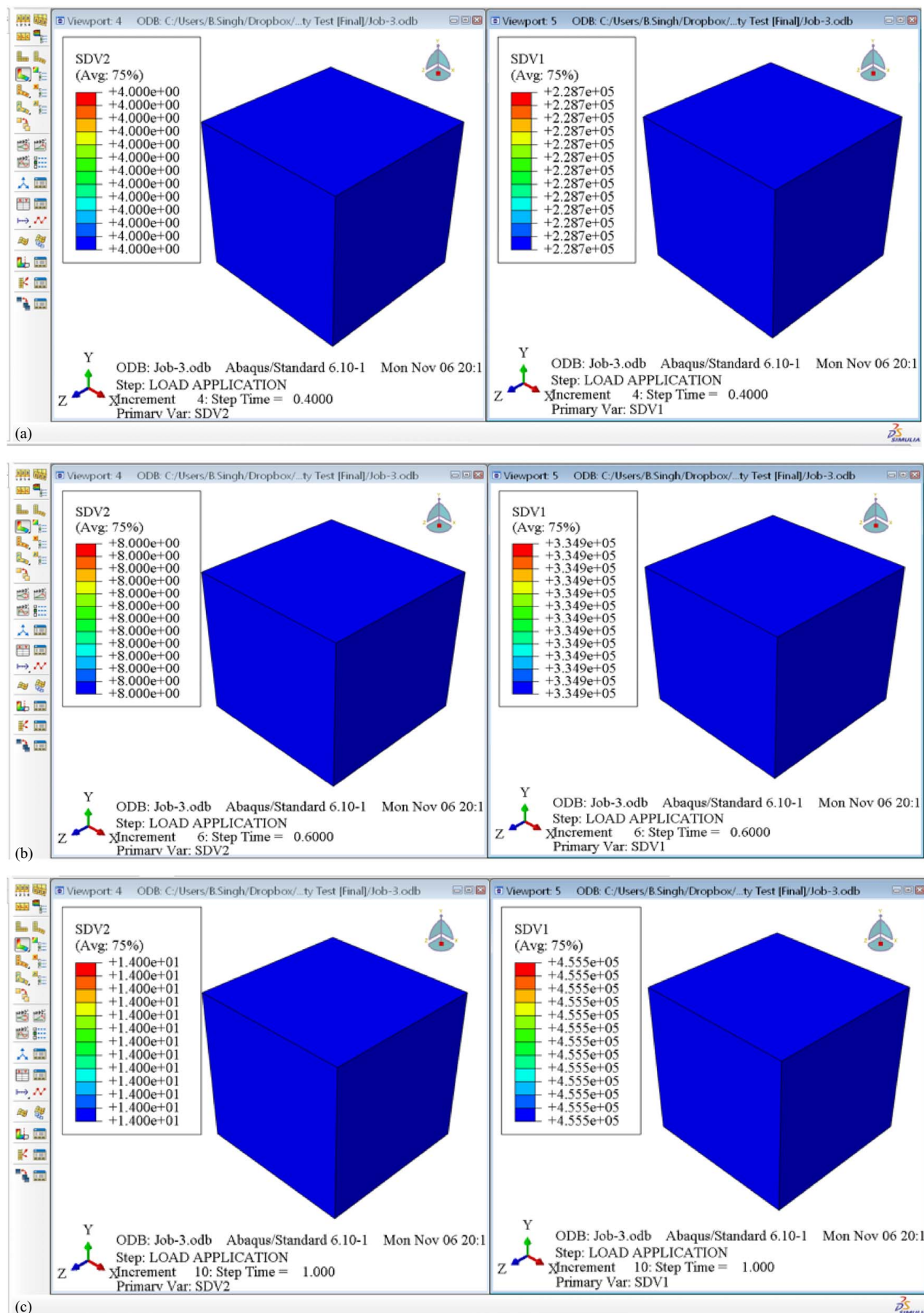


Fig. 16. Element test to check the variation of modulus of elasticity with mean stress of (a) 4 Pa; (b) 8 Pa; and (c) 14 Pa.

where $\sigma_{at} = 100 \text{ kN/m}^2$ is reference stress; κ = soil stiffness at the reference stress state; and λ = strength parameter determining the variation of soil stiffness with the mean principal stress. The values of κ and λ for sandy soils considered in the present study have been obtained from EAU (2004).

For the simulation of nonlinear behavior of the sandy soil in Abaqus (version 6.10), a user defined subroutine is introduced in which the modulus of elasticity is updated depending on the mean principal stress as a result of the vertical and lateral loads acting on the bucket foundation. The subroutine has been tested considering a single eight-node brick element with reduced integration (C3D8R) subjected to hydrostatic loading condition, as shown in Fig. 15. Boundary conditions constraining normal displacement are applied on three faces of the element. A compressive displacement-controlled load is then applied on the remaining opposite three faces of the element.

The modulus of elasticity values, calculated from different mean stresses using Eq. (5) and used as input parameters, are presented in Table 9. In other words, the modulus of elasticity (E_{soil}) is made to be dependent on the mean stress (σ_m). The values of mass density κ and λ used for verification of the subroutine program are $1,100 \text{ kg/m}^3$, 600 and 0.55, respectively.

When the execution of program starts, in the first iteration of a load increment, the subroutine is called twice. During the first call, the initial stiffness matrix is formed using the configuration of the model at the start of the increment. During the second call, a new stiffness matrix based on the updated configuration of the model is created. In subsequent iterations, the subroutine is called only once, and the corrections to the model's configuration are calculated using the stiffness matrix from the end of the previous iteration.

In this program, for all the calls of the subroutine, the stresses taken from the material points are used to calculate the mean stress, which is then saved as the value of field variable on which E_{soil} is dependent on. Corresponding to this mean stress value, the program calculates the value of modulus of elasticity from the material property deck (Table 9), which would then be used for stress calculation in the next increment.

As shown in Figs. 16(a–c), input values of 4, 8, and 14 Pa for the mean stress (SDV2) give the corresponding output values of 228,710, 334,850, and 455,535 Pa for the modulus of elasticity (SDV1). The output obtained from the visualization module of Abaqus indicates that the subroutine can effectively simulate the variation of modulus of elasticity with mean stress.

Data Availability Statement

Some or all data, models, or code that support the findings of this study are available from the corresponding author upon reasonable request.

- Subroutine code
- Numerical simulation files
- All the data obtained from numerical simulations relevant to this study

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