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Numerical analysis of the contribution of side resistance to caisson bearing capacity

Bárbara Estéfany Pereira[1], Jean Rodrigo Garcia[1#]

Article

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Large tip (bell)
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Abstract

The use of deep foundations is a common practice in geotechnical civil engineering designs, in which the bearing capacity of these foundations occurs by side resistance, tip, or through the combination of both. In the case of caisson, the bearing capacity is often obtained by considering only the resistance of the lower end due to its bell-shaped geometry, neglecting the skin friction resistance of the shaft, which may represent an oversizing in some cases. In this context, this paper analyzed the behavior of nine caisson prototypes laid at 10 m, 15 m and 20 m deep. At each depth, three types of caissons were analyzed, with and without an expanded base, and a third type with deformable material at the top of the base. The axisymmetric numerical analyses were conducted by using the finite element method considering an isotropic medium. Thus, it was found that with increasing depth, the skin frictional resistance of the surrounding soil of shaft contributes significantly to the bearing capacity of the caisson suggesting that little load would reach the base of the caisson in situations that would negligible the side resistance of the shaft in the design phase. This may be an important consideration in foundation design using caisson, as it would reduce risks to human life, as well as reduce material consumption and the generation of carbon released into the atmosphere.

1. Introduction

According to Carneiro (1999) estimating the bearing capacity of a foundation element is essential so that it can proper behave to the needs of the design for which it was intended. In this case, there is the need to know this behavior through theoretical procedures and through accumulated knowledge from the experience of designers, the use of field and laboratory correlations, which represent the essence of the empirical and semi-empirical methods established in Brazil.

The bearing capacity of caisson foundations may consider in its composition, the resistance coming from side resistance, provided a length equal to a diameter of the base, immediately above it, is disregarded for load transfer calculation purposes. This consideration of the friction resistance coming from the caisson shaft may be interesting in certain cases for optimization, economy and safety of the geotechnical design, because depending on the depth of laying of the caisson base, the resistance of the shaft may represent a significant portion of the load capacity of the foundation element (ABNT, 2019).

2. Expressions for calculation caisson foundation

Theoretical, semi-empirical, and prototype load tests can be used to calculate the bearing capacity of tubular foundations, but due to the large dimensions of this type of foundation, the bearing capacity is high, requiring highly resistant equipment that provides for the application of loads of great magnitude on the foundation, which increases the cost of load tests. Therefore, it is opted to use established methods for such determination. Theoretical methods applied to sandy soils are used. Meyerhof (1951) proposed an expression for calculation of the bearing capacity of deep foundations, analogous to the equation proposed by Terzaghi in 1943 (Albuquerque & Garcia, 2020), as shown in the Equation 1.

According to several authors, the theoretical expression proposed by Meyerhof can lead to optimistic results regarding the bearing capacity, but not too divergent from reality for the cases observed in sandy soils. Another way to find the bearing capacity is through semi-empirical methods based on correlations between soil strength properties (SPT and CPT) and settlement, which mostly do not consider the side resistance by skin friction, as seen in Alonso (1983) and Aoki & Velloso (1975). Besides these, other renowned methods in the Brazilian scenario for bearing capacity of deep foundations are those proposed by Décourt & Quaresma (1978), Velloso (1981), Décourt (1989) and Albiero & Cintra (2016), which according to Albuquerque & Garcia (2020), based on the estimation for shallow presented a proposal that can be extended to the case of deep foundations by including the effect of depth in effective terms, geostatic stress, at the support quota of the caisson base, as shown in the Equations 2, 3 and 4.

#Corresponding author. E-mail address: jean.garcia@ufu.br
[1]Universidade Federal de Uberlândia, Faculdade de Engenharia Civil, Uberlândia, MG, Brasil.

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Terzaghi (1943) $\sigma_{rup} = c \cdot N_c + q \cdot N_q + 0.5 \cdot \gamma \cdot B \cdot N_\gamma$ (1)

Décourt (1989) $\sigma_{adm} = 25 \cdot \overline{N}_{SPT} + \sigma'_{vb}$ (in kPa) (2)

Albiero & Cintra (2016) $\sigma_{adm} = 20 \cdot \overline{N}_{SPT} + \sigma'_{vb}$ (in kPa) (3)

Alonso (1983) $\sigma_{adm} = 33.33 \cdot \overline{N}_{SPT}$ (in kPa) (4)

In theoretical and semi-empirical applications, it has been usual practice to assume that the lateral stress along the shaft is null, and thus the load of the structure is transferred to the subsoil by the support of the base. However, several load tests performed on deep caissons indicate that, for small displacements, the friction resistance of deep wells is significant and develops fully for displacements of the order of 5 to 10 mm, regardless of the diameter of the shaft. However, the full mobilization of the base resistance is only effective for large displacements, on the order of 10% to 20% of the base diameter. Therefore, for the allowable load, the caisson may present a behavior different from that foreseen in the design if the friction resistance is not considered (Hachich & Falconi, 1998).

According to Teixeira (1997) the influence of skin friction on the bearing capacity of a caisson is very important, even in shorter caissons with lengths of approximately 4 meters, because the instrumentation performed in the tests indicated an average of 48% of skin friction and 52% for the base.

Carneiro (1999) performed load tests on open caisson under humidity in situ and flooded conditions. This author states that the portion coming from skin friction is of utmost importance when considering the bearing capacity calculation of these foundations, since there is an effective collaboration, especially when it comes to collapsible soils, where his conclusion is in line with that reported by Mello (1975) who, through load tests performed on prototypes in London clays and sands in the United States, shows that such friction should not be disregarded.

The magnitude of settlements due to the allowable load is low and the displacement records come basically from the elastic deformations of the concrete (shaft). According to Falconi et al. (2016), when it comes to estimating settlements for tubulars, we find great difficulties to perform such a calculation, because numerous factors such as the scarcity of information in geotechnical literature with respect to calibration of the various methods available, the high cost to perform load tests due to the order of magnitude of the element and the several variations characteristic of soils, These factors can lead to erroneous results regarding the actual behavior of the soil underlying the base, since skin friction at the caisson shaft may reduce the load on the base, whether or not this was considered in the calculations.

3. Foundations and characteristics analyzed

Three types of caisson foundations were analyzed, and each type was also analyzed at three different depths of 10 m, 15 m and 20 m (Figure 1). Thus, the increase of the skin frictional resistance could be evaluated in relation to the base resistance. Therefore, the values for shaft diameter (*d*), base diameter (*B*), shaft height (*L*), laying depth (*H*), and widened base height (h_B) were established based on values commonly employed in caisson foundation designs (Table 1).

For the analyses, the length "*L*" of the shafts was varied in 10, 15 and 20 meters, keeping the other geometries unchanged. The region with failure material, i.e., with low strength employed above the base of the defective caisson. Thus, a cylindrical segment of 0.5 m height (*y*) and diameter (*d*) equal to the caisson shaft was adopted. The use of this element aims to obtain the full mobilization of skin frictional resistance before the beginning of the mobilization of the widened base. In this situation, the caisson base would only be mobilized for more significant deformations, on the order of 10% to 20% of the base diameter, as recommended by (Hachich & Falconi, 1998).

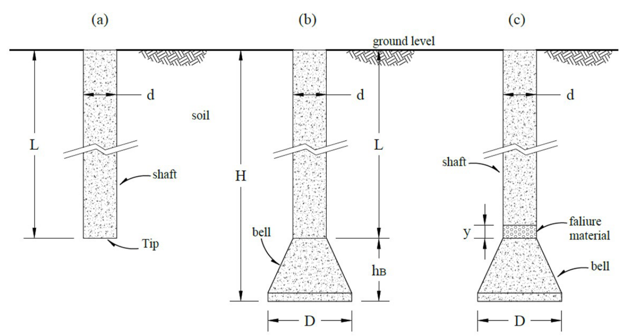


Figure 1. Schematic of the analyzed foundation types: a) pile (no bell); b) unimpaired caisson (with bell); c) defective caisson (with bell).

Table 1. Geometric characteristics of the analyzed foundations.

Types of foundations	*d* (m)	*D* (m)	*L* (m)	*H* (m)	h_B (m)
Pile-10	0.80	-	10	10	-
Caisson-10	0.80	2.70	10	11.80	1.80
Defective Caisson-10	0.80	2.70	10	11.80	1.80
Pile-15	0.80	-	15	-	-
Caisson-15	0.80	2.70	15	16.80	1.80
Defective Caisson-15	0.80	2.70	15	16.80	1.80
Pile-20	0.80	-	20	-	-
Caisson-20	0.80	2.70	20	21.80	1.80
Defective Caisson-20	0.80	2.70	20	21.80	1.80

With the adopted geometry it was ensured that the inclination of the widened base is greater than 60 º so that tensile stresses at the bottom of the base can be neglected without the need for reinforcement for this purpose.

Therefore, a total of nine analyses were performed, being three analyses for each depth and foundation type, with variation of the shaft length (*L*) in 10, 15 and 20 meters, according to Table 1.

4. Axisymmetric FEM 2D model

An axisymmetric geometry was used to numerically simulate the caisson foundation prototypes, since the shaft and base have the same axis of symmetry (Figure 2). This technique allows representing the three-dimensional behavior of foundations by means of a plane model. Based on this, 2D finite element mesh elements of the triangular type were used, composed of 6 nodes, 3 nodes at each vertex and 3 nodes at the midpoint of each edge. The convergence of the model with the number of degrees of freedom was evaluated. Both the mesh density and the discretization of the model contour impact the number of degrees of freedom. Therefore, all models were subjected to convergence tests to achieve the necessary accuracy to them be used in the analyses. The numerical simulation phases are composed of a) RS Modeler (preprocessing); b) RS Compute (processing) and c) RS Interpret (postprocessing). The resolution of the numerical models follows the Absolute Force and Energy criterion, in which the simultaneous convergence of the values of the internal and external forces acting on the nodes of the elements is evaluated. The use of this criterion increases the accuracy of the simulation results.

The half-space considered in the analysis has adequate dimensions that validate the established boundary conditions. On the side boundaries of the half-space, the displacements are released in the vertical direction and restricted in the horizontal direction. At the base of the half-space, the displacement constraints were assigned in both directions (Figure 2). The same concept was employed for the numerical models of the pile (Figure 2a), pipe (Figure 2b) and defective caisson (Figure 2c). Thus, the dimensions of 30 m x 30 m were adopted for the domain of the three foundation models, preventing the extension of the domain from affecting the comparison between the simulation results.

The load was applied axially to compression at the top of the shaft of the foundation elements, being subdivided into 11 load stages, with the increment equal to 10% of the maximum estimated load, until reaching a displacement that would characterize the failure of the foundation element. In this case, as recommended by the international literature, a value of 10% of the shaft diameter (i.e., a settlement of 80 mm) was fixed as the displacement that corresponds to the failure. The displacements were measured in the same plane of load application, from an average of five measurements.

To reproduce the material failure, the Mohr-Coulomb criterion was used, which has been employed in numerical analyses to relate shear and normal stress acting in a plane:

$$\tau = c + \sigma \cdot \tan \phi \tag{5}$$

Where σ is the normal stress; *c* is the cohesion intercept and ϕ is the friction angle.

5. Material properties

For the simulation of the cases analyzed, the geotechnical parameters were defined as constant in depth for a homogeneous and isotropic soil, allowing the analysis of the stresses caused by the contact between the foundation element and the soil, without the influence of the characteristics of the soil type variation. The soil input parameters were based on the characterization performed by Oliveira (2022) for a silty-clayey sand (Table 2).

The concrete with a compressive strength of 20 MPa was adopted for the caisson material, which is the most used in the concreting of this type of foundation. The elastic modulus of concrete was obtained by following the procedures of NBR 6118 (ABNT, 2014). The input properties in the Mohr-Coulomb model for concrete and low strength material with high deformability were obtained based on the work of Ardiaca (2009).

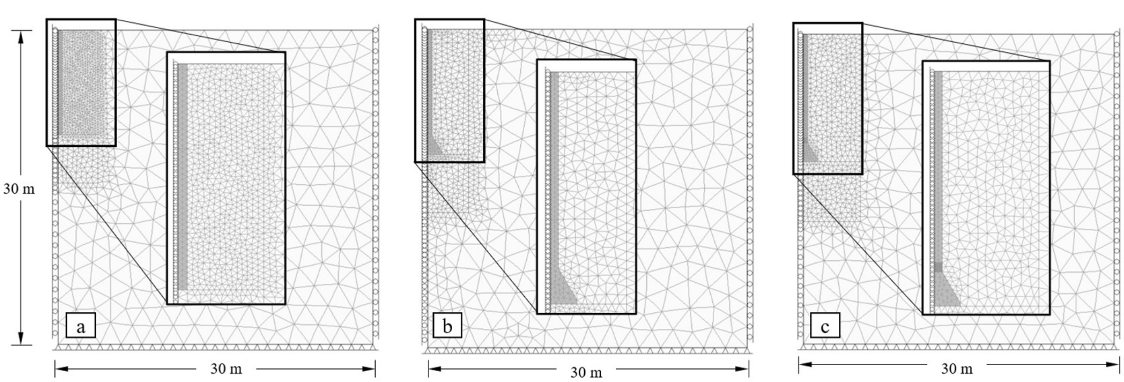


Figure 2. Numerically simulated axisymmetric models of: a) pile; b) pipe; c) defective caisson.

Table 2. Parameters for the Mohr-Coulomb criteria.

Material	E (MPa)	ν	K_0	γ (kN/m³)	Model	ϕ (º)	c (kPa)
Soil	22	0.3	0.6	17	Elastoplastic	23	10
failure material	2.2	0.40	-		Elastoplastic	10	3
Concrete	30.000	0.20	-	25	Elastic	35	442

Note: E is the elastic modulus; ν is the Poisson's ratio; K_0 is the coefficient of earth pressure at rest; γ is the unit weight; ϕ is the soil friction angle; c is the cohesion.

6. Analysis and discussion of the results

The load versus settlement curves obtained with the aid of (see Figure 3) was used to obtain the ultimate load for the displacement of 80 mm (10% of the diameter of the shaft of the analyzed foundations. With the readings of the top and tip of the shaft, the friction and tip resistance plots were determined, as well as the respective load vs. settlement curves (Figures 4 and 5). In these, this resistance increases linearly with the increase in shaft length. The same does not occur with the tip resistance for the cases of isolated pile and defective caisson, which remained constant with increasing shaft length in depth. On the other hand, for the intact caisson case, tip resistance increases with depth, presenting a significant increase when comparing the 10 m shaft with the 15 and 20 m lengths.

Comparing the piles and caisson with defective induced at the top of the widened base, they presented similar behavior to each other, i.e., they were close to the behavior of a single pile. And, considering the specific conditions of this paper, these foundations have a predominance of skin friction resistance in their bearing capacity, as previously mentioned by Das (2010). Such hypothesis is proven when we observe the results from the numerical analysis of the caisson with the low strength material placed at the top of the widened base, since this material prevents the load transfer to the base due to its higher deformability and lower strength. The values for ultimate load, allowable load and respective settlement were extracted from the load vs. settlement curves and are presented in Table 3.

The presence of the widened base directly influences the load transfer of the caisson foundation because the presence of the defect above the top of the caisson base caused the load transfer by skin frictional resistance to present similar behavior to the pile (Figure 6).

In the case of the foundation in an intact caisson consisting of shaft and widened base, it is verified that the tip resistance predominates in the bearing capacity (Table 3), as observed by Murthy (2002). However, this predominance of the tip resistance is more significant for loading stages greater than 60% of the ultimate bearing capacity of the caisson (20% d) and tends to reduce with increasing depth of settlement of the caisson, i.e., with increasing length "L" of the shaft (Figure 6b, 6e and 6h). On the other hand, skin frictional resistance shows predominance in the early stages of loading, between 10% and 60% of the caisson bearing capacity, as well as an increasing tendency with increasing caisson settlement depth (Figure 6).

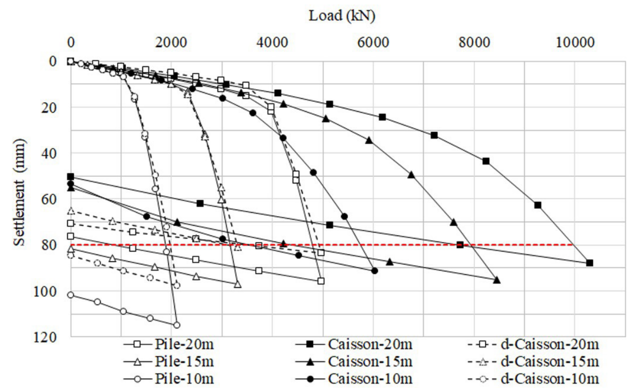


Figure 3. Load-settlement curves of the analyzed cases.

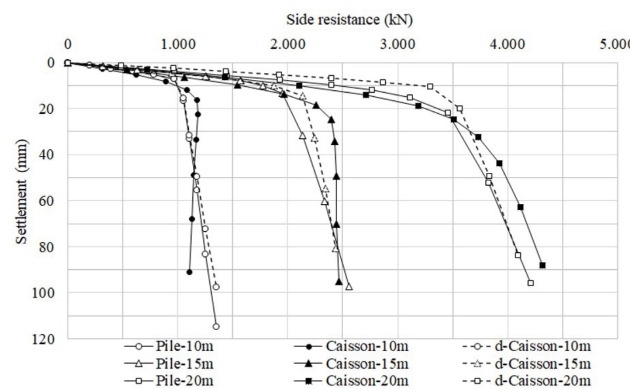


Figure 4. Skin friction resistance *vs.* settlement curves of the analyzed cases.

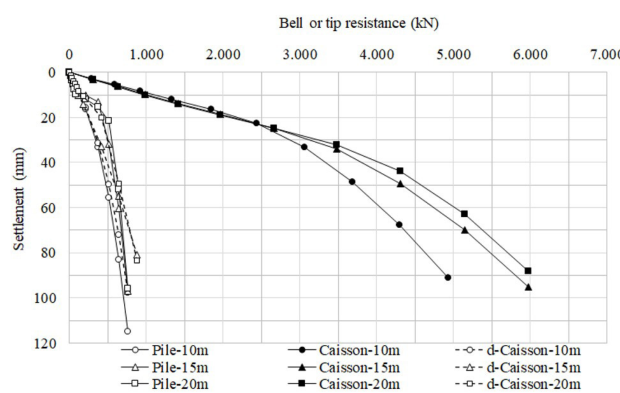
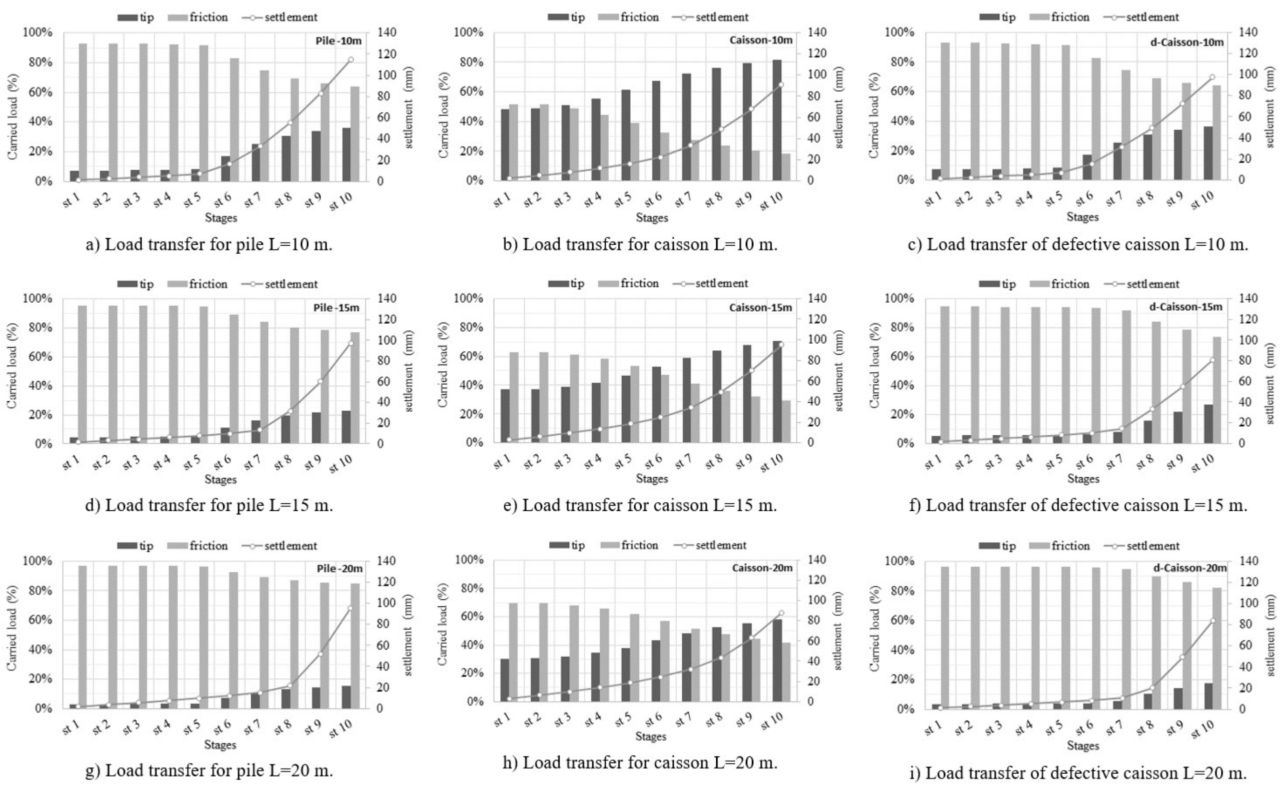


Figure 5. Tip resistance *vs.* settlement curves of the analyzed cases.

Table 3. Summary of ultimate and allowable loads.

Analyse cases	Ultimate Load - for 80 mm			Allowable Load			
	Q_{ult}	Q_{su}	Q_{tip}	Q_{all}	s_{all}	Q_{su}	Q_{tip}
	[kN]	[kN]	[kN]	[kN]	[mm]	[kN]	[kN]
Pile-10m	1.875	1.242	633	938	6.07	621	317
Caisson-10m	5.743	1.112	4.631	2.871	15.29	556	2.316
d-Caisson-10m	1.965	1.282	683	982	6.45	641	341
Pile-15m	3.163	2.338	708	1.582	7.52	1.169	354
Caisson-15m	7.936	2.452	5.484	3.968	17.14	1.226	2.742
d-Caisson-15m	3.306	2.431	875	1.653	7.89	1.215	438
Pile-20m	4.797	4.077	720	2.398	9.38	2.038	360
Caisson-20m	9.973	4.257	5.716	4.987	18.11	2.129	2.858
d-Caisson-20m	4.924	4.066	858	2.462	6.62	2.033	429



a) Load transfer for pile L=10 m.

b) Load transfer for caisson L=10 m.

c) Load transfer of defective caisson L=10 m.

d) Load transfer for pile L=15 m.

e) Load transfer for caisson L=15 m.

f) Load transfer of defective caisson L=15 m.

g) Load transfer for pile L=20 m.

h) Load transfer for caisson L=20 m.

i) Load transfer of defective caisson L=20 m.

Figure 6. Load Transfer for piles, caisson and defective caisson for different shaft lengths.

From the cases analyzed, even for caisson foundations, side resistance becomes significant at greater depths. The opposite is observed for the resistance of the base, which is more significant when the caisson rests on shallower depths, and therefore, the potential for skin friction to develop is smaller, i.e., with less potential for skin friction development (Figure 6).

The presence of low strength material, as a way to represent a “constructive defective”, occurred in the top of the widened base modified the load distribution in the foundation, resulting in a situation similar to an single pile, except for the fact that in the pile, there is some tip resistance, while in the caisson with defective, there is a predominance of skin frictional resistance in the bearing capacity of the foundation (Figure 6).

In all cases analyzed, the tip or widened base resistance tends to decrease concomitantly with the increase of the shaft length. For the depths of 10, 15 and 20 m, this reduction is 19%, 24% and 18%, respectively for pile, intact caisson and defective caisson (Figure 7). On the other hand, it was observed that skin frictional resistance for the same depths increases in the same proportions.

The percentage of load transfer through the widened base decreases linearly with increasing shaft-base area ratio, since the area of the shaft increases with the depth of placement of the caisson. While the load transfer through the shaft increases with the same ratio (Figure 8).

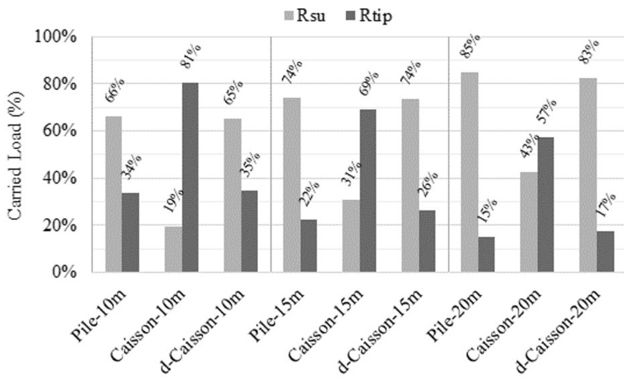


Figure 7. Skin friction and tip resistance of foundation types at different depths.

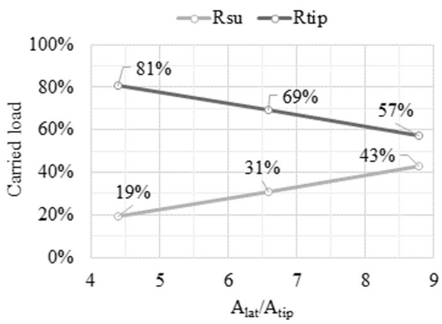


Figure 8. Load transfer to bell and shaft for different area ratios.

At greater depths, the lateral resistance increases and equals 43% of the caisson bearing capacity, against 57% for the base resistance. The obtained results agree results agree with those observed by Teixeira (1997) e Carneiro (1999).

7. Conclusion

This paper analyzed the behavior of caisson with and without a widened base, laid at 10, 15 and 20 m of depth, three of which had deformable material at the top of the widened base. Thus, it was found that with increasing depth, the frictional resistance of the surrounding soil contributes significantly to the bearing capacity of the caissons at hand.

For situations in which the caissons rest in deep layers, the base widening can be removed, since the foundation behaves similarly to a pile, as occurred in the case of the caisson with a defective low strength material at the top of the widened base that modified the load distribution in the foundation.

The widened base caisson may represent only an economic expense of material, labor and time, since in all cases analyzed, the strength of the tip or widened base tends to decrease concomitantly with the increase in length of the shaft. On the other hand, for situations in which the foundation rest at shallow depths, one can choose to enlarged the base or not, leaving to the designer's discretion the decision of sizing it as a pile or caisson with a belled geometry, since in this case the base resistance is preponderant in the bearing capacity of the caisson.

The non-widening of the base may be an important consideration in foundation designs that use caisson, as it would reduce risks to human lives, besides providing a lower consumption of materials and reducing the generation of carbon released into the atmosphere.

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Declaration of interest

The authors have no conflicts of interest to declare. All co-authors have observed and affirmed the contents of the paper and there is no financial interest to report.

Authors' contributions

Bárbara Estéfany Pereira: conceptualization, methodology, validation, writing – original draft. Jean Rodrigo Garcia: data analysis, supervision, validation, writing – review & editing, funding acquisition.

Data availability

The datasets generated analyzed in the course of the current study are available from the corresponding author upon request.

List of symbols

c	intercept cohesion
d	shaft diameter
h_B	base height
q	surcharge
B	footing width
D	base diameter
E	Young's modulus
H	base installation depth
K_0	coefficient of at-rest earth pressure
N_c, N_q, N_γ	bearing capacity factors
$\overline{N}_{SPT}$	blow count average
γ	unit weight
ν	Poisson's ratio
σ	normal stress
σ_{adm}	allowable stress
σ'_{vb}	vertical effective stress in the toe
σ_{rup}	bearing pressure
τ	shear stress
ϕ	soil friction angle

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