Numerical analysis of large-diameter bored piles in sandy soil

Naloan C. Sampa[1], Gabriel F. Costa[1], Fabiano A. Nienov[2], João A. G. P. Paiva[1]

[1]*Dept. of Civil Engineering, Federal University of Santa Catarina*
Dept. of Civil Engineering, Federal University of Santa Catarina
João Pio Duarte da Silva - 205, 88040-900, Santa Catarina, Brazil
naloan.sampa@ufsc.br, gabrielf3@gmail.com, joaugusto.paiva@gmail.com

Abstract. The paper presents a numerical model to analyze the load transfer mechanisms and load-displacement patterns of large-diameter bored piles in sandy soil. The model is calibrated using a seven-layer soil profile, encompassing dense to loose sand layers and a silty-sandy clay layer located between depths of 18 m to 22 m. The pile and soil materials are represented with elastic model and Mohr-Coulomb elasto-plastic model, respectively. Discrepancies between experimental and numerical load-displacement curves are evident, particularly when calibrating the model with reference experimental and lab-derived soil parameters. Adjusting interface friction angles and reducing soil elastic moduli aligns the pile's behavior with numerical predictions for bentonite cases. However, polymer-modified cases display higher values and require more extensive parameter modifications. Discussing the effects of polymers and bentonite on lateral resistance, coupled with a comparison of the β coefficient, coefficient of lateral earth pressure (K), and frictional coefficient (μ) between numerical and experimental data, deepens the comprehension of bored pile behavior in sandy soils.

Keywords: numerical modeling, bored pile, bentonite, polymer, experimental field of Araquari.

1 Introduction

Large-diameter bored piles exceeding 0.60m are often essential in high-load engineering projects. When executed in regions with a high-water table and unstable soils, stabilizing fluids like bentonite or polymers are utilized to maintain excavation integrity (Nienov [1]). Bentonite slurry, a blend of bentonite and water, has been a preferred choice due to its accessibility, cost-effectiveness, and easy application (O'Neill and Reese [2]; O'Neill and Hassan [3]). However, environmental concerns, processing and mixing time, and reusability have led to the growing use of polymers in recent years (O'Neill and Reese [2]; O'Neill and Hassan [3]).

Stabilizing fluids permeate borehole pore walls during pile execution due to fluid-groundwater pressure differences, forming a "cake" layer whose thickness depends on fluid characteristics and soil permeability. Cake formation and its influence on lateral pile resistance have been investigated by Brown [4] and Lam et al. [5]. An impermeable and thin cake is crucial to maintain wall stability and prevent significant fluid seepage into the soil (O'Neill and Reese [2]; Hachich et al. [6]).

Load tests are an efficient method for investigating piles' behavior. Numerous studies have explored cake formation's dynamics and its effect on pile behavior using different stabilizing fluids, soils, and exposure times (Holden [7], O'Neill and Hassan [3], Lam et al. [5]). Unlike bentonite, polymer-based slurries generally do not form a thick cake on the soil surface (Lam and Jefferis [8]). Cake thickness removal impact was discussed by Hachich et al. [6], Tucker and Reese [9]. Hachich et al. [6] stated that excavation tools and placing concrete can easily remove a thick, weak cake; complete removal is more likely in low-permeability soil. Tucker and Reese [9], Majano et al. [10] highlighted key considerations regarding stabilizing fluids and their impact on pile performance. Lam et al [5] noted a noteworthy reduction in lateral pile resistance due to prolonged exposure to bentonite slurry and increased cake thickness. Majano et al. [10], Brown [4], Lam et al. [5], Nienov [1] reported higher lateral resistances in polymer-assisted piles compared to bentonite ones. Frizzi et al. [11] noted reverse behavior in rock and sand. Hachich et al. [6] emphasized that borehole wall constraint duration influences lateral friction more than bentonite-induced cakes. Despite polymer's superior performance, proper dosing and removal of the coarse particles accumulated from the bottom of the excavation before placing concrete are crucial (Jefferis and Lam [12]). In essence, factors such as viscosity, concentration, type, mixing time, and exposure duration of stabilizing fluid collectively influence cake formation, impacting pile performance.

The bearing capacity of piles relies on lateral friction and tip resistance. Hachich et al. [6] stated that lateral resistance maximizes at small deformations (5 mm to 10 mm), whereas more significant deformations (around 10% to 15% of pile diameter) are needed for the full mobilization of tip resistance. Soil, pile, and construction characteristics influence lateral pile resistance, underscoring the importance of understanding stabilizing fluids' impact through load tests, shear tests and numerical modeling. Tip resistance depends on soil and pile characteristics, as well as the excavation cleanliness. In bored piles, NBR 6122 [13] suggested limiting the tip resistance to 20% of working load.

Given the incomplete comprehension of cakes' influence on bored pile behavior, load capacity assessment rests on load tests. Traditional methods like those of Van der Veen [14], NBR 6122 [13], and Salgado [15] are employed to establish ultimate load capacity through load-displacement curves.

Equation 1 is a conventional expression for estimating the bearing capacity (R) of piles in sandy soils.

$$R = R_b + R_l = \sigma'_v N_q A_b + K\sigma'_v \tan\delta \, A_l \, , \tag{1}$$

where R_l and R_b are lateral resistance and tip resistance, respectively, σ'_v is the effective vertical stress, N_q is the bearing capacity factor, A_l and A_b are the base and lateral areas, respectively, K is the coefficient of lateral earth pressure, δ is the interface friction angle.

Bearing capacity of bored piles is commonly assessed via semi-empirical methods relying on Standard Penetration Test and Cone Penetration Test data. In Brazil, widely used methods include those proposed by Aoki and Velloso [16] (Eq. 4), Décourt and Quaresma [17] (Eq. 5), and Teixeira [18] (Eq. 6). Internationally, Eurocode 7 [19] and Bustamante and Frank [20] methods find extensive use for predicting pile load capacity.

$$R = R_b + R_l = \frac{KN_b}{F_1} A_b + \frac{U}{F_2} \sum_{1}^{n} (\alpha K N_l \Delta_l). \tag{4}$$

$$R = R_b + R_l = \alpha C N_b A_b + 10\beta \left(\frac{N_l}{3} + 1\right) UL. \tag{5}$$

$$R = R_b + R_l = \alpha N_b A_b + \beta N_l UL, \tag{6}$$

where U is the pile perimeter, while α, β, K, F_1, F_2, and C are coefficients or factors derived from tables that consider distinct soil and pile characteristics.

This paper seeks to enhance insights into the interaction mechanism between soil and bored piles. This is achieved by employing accurately calibrated numerical simulations. Additionally, the paper contrasts bearing capacity findings from semi-empirical methods, load tests, and numerical modeling to grasp the inherent conservatism and in each approach.

2 Material and Methods

2.1 Experimental field of Araquari

Instrumented static load tests were conducted on piles utilizing bentonite and polymer at the Araquari Experimental (area of 0.3 km²) Field in Santa Catarina, Brazil. Two geotechnical investigation campaigns were carried out to characterize the subsurface. The first campaign included 7 CPTu (Piezocone) tests, while the second involved 7 more CPTu tests, 3 SPT (Standard Penetration Test) tests, and 1 SDMT (Seismic Dilatometer Test) test. Additionally, physical characterization and triaxial tests were conducted (Nienov [1] and Lavalle [21]).

The geotechnical profile comprises compact to loose sand with a relative density ranging from 27% to 75%. An intermediate layer of silty-sandy clay is found at depths of 18 m to 22 m. Moisture content ranges from 10% to 70%, penetration resistance index (N_{spt}) spans 5 to 30, corrected cone tip resistance (q_t) varies between 2.5

MPa and 10 MPa, peak friction angle ranges from 23.3° to 38°, critical friction angle varies between 22.8° and 35.9°, and void ratio varies between 0.61 and 0.95 (Lavalle [21]). Groundwater table during excavation averaged about 3 m below the ground surface (Nienov [1]).

Static load tests were conducted on six piles, with two reference piles instrumented throughout their depth (polymer - ET4 and bentonite - ET5). Both piles had a diameter of 1 m and lengths around 24.1 m. Load tests were performed 143 days after pile ET4 execution and 100 days after pile ET5 execution (Nienov [1]). Further details about the experimental field and load tests are available in Nienov's work [1].

2.2 Estimating ultimate load and bearing capacity

Three distinct criteria were utilized to ascertain the ultimate load of piles ET4 and ET5, based on the load-displacement curves. The first criterion, following Salgado's [15] recommendation, sets the ultimate load at a displacement equivalent to 10% of the pile diameter, connecting this relative displacement to operational loss or structural collapse of the element. The second and third criteria are in accordance with NBR 6122 [13] and Van der Veen [14], respectively.

Five traditional semi-empirical techniques were employed to predict the bearing capacity of a bored pile. Within these methods, three - Aoki-Velloso [16], Décourt-Quaresma [17], and Teixeira [18] - evaluate the bearing capacity based on the N_{spt}, while the remaining two - Bustamante and Frank [20] and Eurocode 7 [19] - consider q_c.

2.3 Numerical modeling

Vertical load tests were simulated through two-dimensional Finite Element (FE) model, using Abaqus software. To account for the symmetry, we modeled a half-soil domain with dimensions of 12.5 m in length (12.5 times pile diameter), and 36.15 m in depth (1.5 times the pile length). The domain size was deliberately enlarged to minimize significant boundary effects on computed displacement, deformation, and stress. Refined meshing was applied around the pile tip (Fig. 1).

For boundary conditions, both vertical and horizontal displacement components were constrained at the base. On the right side and along the symmetry axis, only horizontal displacement was restricted. To replicate groundwater and saturated soil conditions, a pore pressure of 0 was imposed on the surface of the second layer.

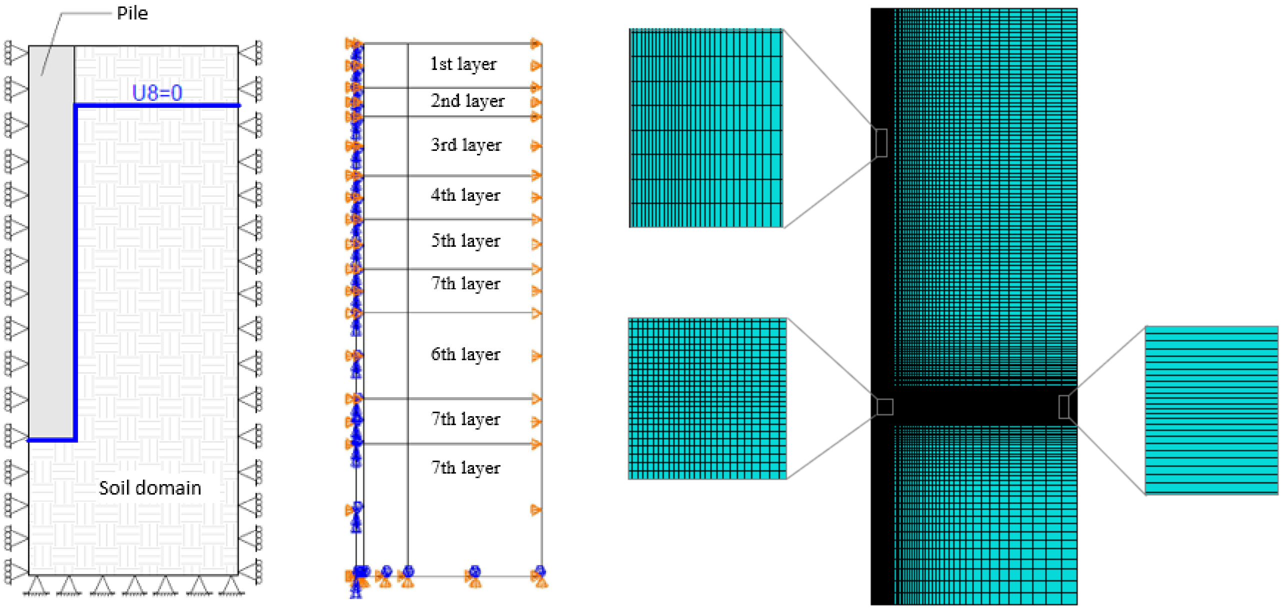

Figure 1. Boundary conditions and mesh of the discretized model.

Twenty-nine parametric analyses were conducted to calibrate numerical model, aiming to fit the experimental and numerical results. These analyses solely varied soil parameters. Reference parameter values across the seven layers were established from Nienov's [1] and Lavalle's [21] lab and field tests. Further specifics about the parametric study and the seven soil layer parameters are detailed in Costa [22].

The geometry and mechanical properties of the pile were similar to those of pile ET4 and ET5. The pile material was treated as elastic, characterized by a Young's modulus of 40.33 GPa and a Poisson's ratio of 0.3. Soil was defined as an elastoplastic material governed by the Mohr-Coulomb failure criterion. Solid homogeneous elements with pore pressure (CAX4RP) were employed in saturated layers, whereas CAX4R elements were utilized in the first layer and the pile.

To model pile-soil interactions, it was employed a surface-to-surface contact approach using the master-slave contact algorithm. Contacts were treated as hard and normal to interfaces, with shear contact between soil and pile simulated through the Coulomb friction model. Friction coefficient at the interface related to the critical friction angle ($\mu = \tan \phi_{cv}$).

The vertical load test simulation consisted of three steps: geostatic analysis, pile installation, and the load test. The initial step established the self-weight stress field using a body force option and a K_0 derived from $1 - \sin \phi$. The third step applied a displacement boundary condition, allowing a maximum 10% pile diameter displacement (0.1 m), aligned with experimental load test.

3 Results and Discussion

Figure 2a presents a comparison between load-displacement curves obtained from load tests and the numerical model. The conservative behavior of Analysis 1 (A-1), utilizing reference soil parameters, contrasts with load test curves. Adjusting soil parameters yielded curves (A-21 and some red curves) close in magnitude to the bentonite pile curve, albeit with differences in shape due to the cake effect observed in experimental results. In Figure 2b, load variations through depth, estimated via Teixeira, LCPC, Eurocode 7 and numerical approaches, emulate the bentonite pile's behavior. Notably, the Décourt-Quaresma and Aoki-Velloso curves are more conservative, while polymer pile loads remain substantially higher.

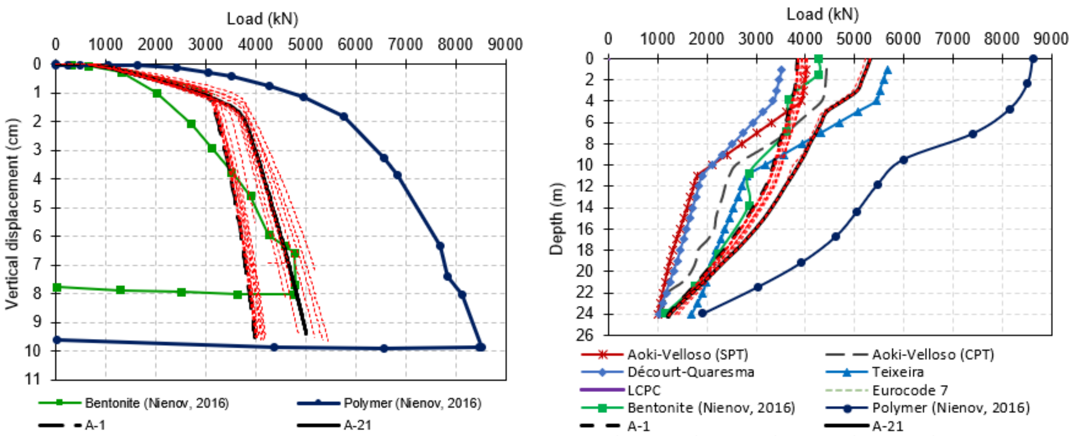

Figure 2. a) load – displacement curves; b) variation of the load capacity along the depth.

Figure 3a compares total load estimates using different methods with those from load tests on polymer and bentonite piles. The first criterion (Salgado [15]) generally yields higher loads, assuming a 10% pile diameter displacement. LCPC method significantly overestimates loads compared to the second and third criteria applied to polymer pile curve. Teixeira, Eurocode 7, and the 1st criterion applied to the bentonite pile curve exhibit comparable values (5500 kN to 5739 kN), while Aoki-Velloso and the 3rd criterion show similar magnitudes (4024 kN to 4690 kN). Décourt-Quaresma and criterion 2 applied to the bentonite pile curve align closely (3500 kN to 3519 kN). LCPC and criteria 2 and 3 applied to the polymer pile curve fall within the same range (6833 kN to 7024 kN).

In Figure 3b, a comparison of lateral and tip resistances reveals LCPC and Eurocode 7 estimate higher tip resistances of 3013 kN and 2561 kN, respectively. Conversely, LCPC and Teixeira methods estimate higher lateral resistances of 4011 kN and 4097 kN, respectively.

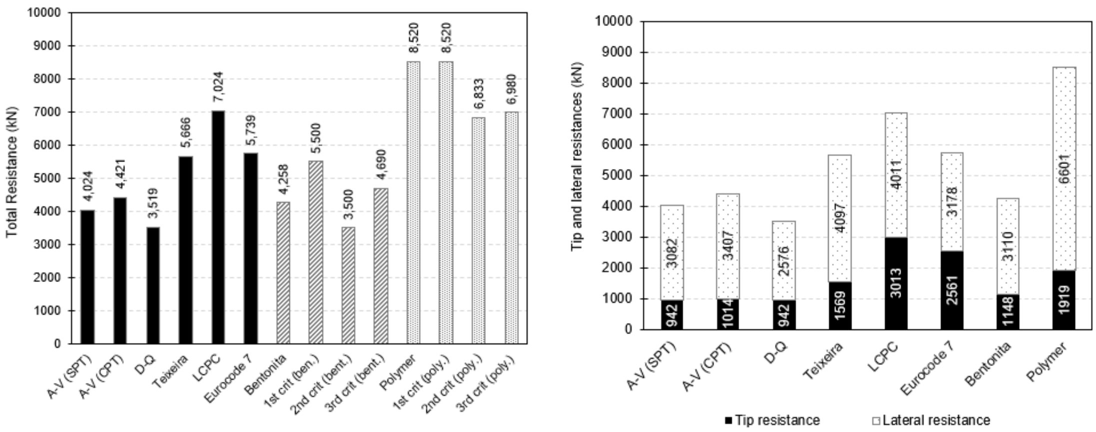

Figure 3. a) total resistance b) lateral and tip resistances for different methods.

Considering the application of distinct safety factors, Figure 4a compares allowable tip and lateral resistances using various partial and tip safety factors: Aoki-Velloso and Teixeira (2 and 2), Décourt-Quaresma (4 and 1.3), Bustamante and Frank, and Eurocode 7 (3 and 2). The allowable lateral resistance ranges from 1541 kN to 2048 kN. However, the Décourt-Quaresma method notably underestimates allowable tip resistance (236 kN), while the LCPC method overestimates it (1004 kN). Allowable tip resistance values estimated by other methods fall within 471 kN to 854 kN range.

Using the reference lateral resistance value (1148 kN) and tip resistance value (3110 kN) of the bentonite pile, ratios were calculated by comparing other methods' resistances to the bentonite pile values ($R_{\text{method}}/R_{\text{bentonite}}$). Confirming previous findings, the Aoki-Velloso and Décourt-Quaresma methods tend to yield lower lateral and tip resistances compared to the bentonite pile. Conversely, other methods exhibit the opposite behavior.

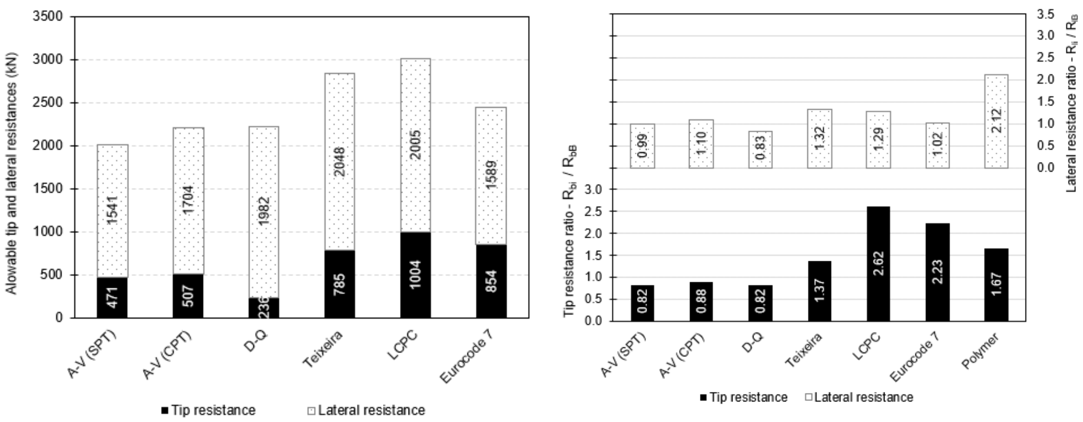

Figure 4. a) allowable tip and lateral resistances, b) ratios of lateral and tip resistances for different methods.

Due to page constraints, the paper only presents the findings of Analysis 1 and 21. Figure 5a illustrates β coefficient ($= K \tan\phi = q_l/\sigma'_v$) changes along the pile shaft for various displacement stages. In Analysis 1, pile displacement has no impact on β coefficient, while in Analysis 21, β values are influenced during initial displacement but stabilize later. Excluding peaks, β coefficients ranges around 0.2 to 0.5 in Analysis 1 and roughly 0.25 to 0.8 in Analysis 21. These fall within Fellenius' [23] range (0.3 to 0.9) for sandy soil bored piles. Analysis 1 values also align with Geo 1's [24] proposal (0.2 to 0.6) for the same soil type.

Figure 5b depicts interface friction coefficient ($\mu = \tan\delta = q_l/\sigma'_h$) variation, which depends on both pile roughness and soil friction angle. Both analyses show reduced friction coefficient across layers from the initial value. However, Analysis 1 finds pile displacement magnitude doesn't impact this change. The reduction in the friction coefficient is related to the increase in horizontal stress and consequently the compaction of the surrounding soil.

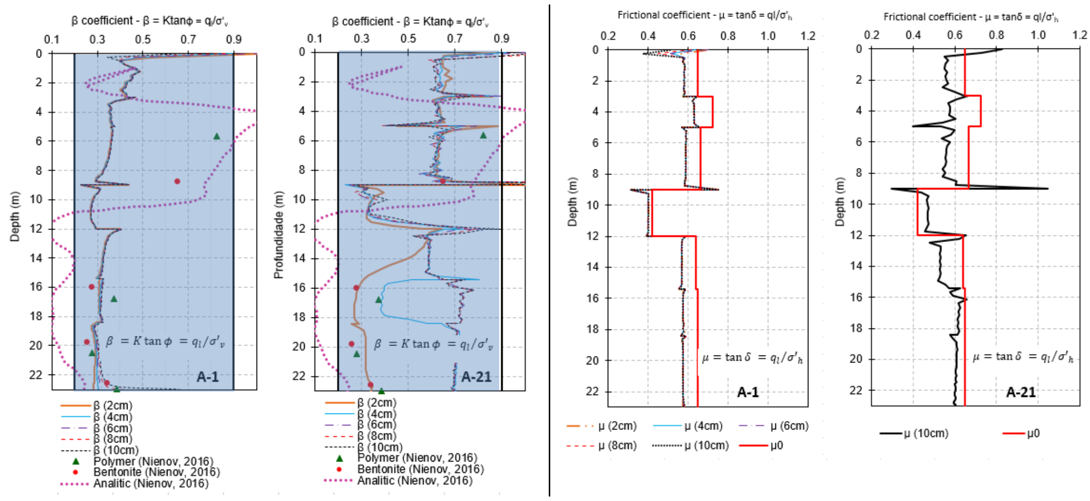

Figure 5. a) variation of β coefficient (A-1 and A-21), b) variation of frictional coefficient (A-1 and A-21).

For soil stress analysis around the pile, Figure 6a illustrates varying current actual coefficient of lateral earth pressure (K) at different load stages, comparing coefficients of lateral earth pressure: at-rest (K_0), active (K_a), and passive (K_p). Both sets show K values exceeding K_0 at depths less than 18 m. In Analysis 1, pile displacements don't influence K (0.44 to 0.74 range). Conversely, Analysis 21 sees K rise with pile displacement (0.45 to 1.1), more pronounced when friction coefficient and elasticity modulus increase together.

For an in-depth K analysis, Figure 6b offers K-to-K_0 ratio insights. In Analysis 1, this ratio decreases with depth (0.95 to 1.66). Analysis 21's K/K_0 ratio ranges 0.94 to 2.5, growing with pile displacement. Numeric simulation results match Nienov's [1] load tests, but slightly surpass Tomlinson and Woodward's [25] range (0.7 to 1) for piles in sands.

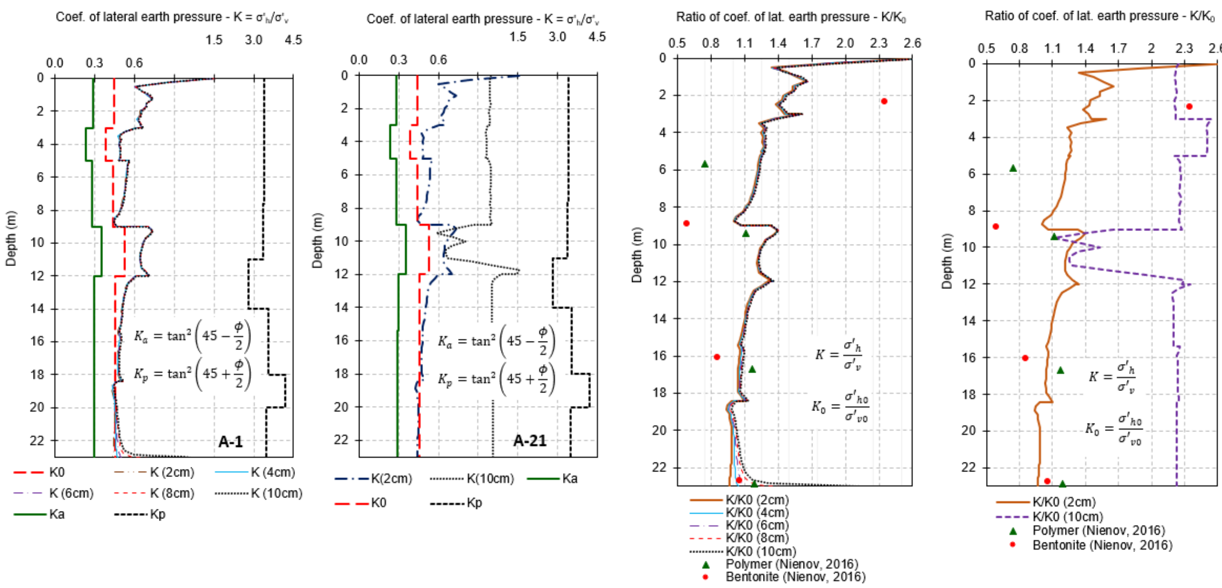

Figure 6. a) coefficient of lateral earth pressure b) ratio of coefficient of lateral earth pressure.

4 Conclusions

The paper introduces a numerical model to analyze load transfer mechanisms and load-displacement patterns of large-diameter bored piles in sandy soil. Results demonstrate the efficacy of both Brazilian and European semi-empirical methods in predicting pile load-bearing capacity. Brazilian methods using SPT data are more conservative than European CPT-based approaches. Brazilian methods align better with bentonite-based piles, while European methods align more closely with the results of piles constructed using polymers.

Salgado's [15] criterion surpasses NBR 6122 [13] and Van der Veen's [14] criteria significantly. Numerical modeling precisely replicates load test behavior, especially for bentonite-constructed piles. As such, the comparisons between the load-bearing capacity values and the analysis of the coefficients β, K, and μ presented in this study can contribute to the understanding of the behavior of bored piles in sandy soils.

Authorship statement. The authors hereby confirm that they are the sole liable persons responsible for the authorship of this work, and that all material that has been herein included as part of the present paper is either the property (and authorship) of the authors, or has the permission of the owners to be included here.

References

[1] F. A, Nienov. Desempenho de Estacas Escavadas de Grande Diâmetro em Solo Arenoso sob Carregamento Vertical. 2016. 267 p. Tese (Doutorado) - Programa de Pós-Graduação em Engenharia Civil, UFRGS – Porto Alegre,2016.
[2] M.W. O'neill, L.C. Reese. (1999). Drilled Shaft: Construction Procedures and Design Methods. Federal Highway Administration, United States, 790 p.
[3] M. W. O'neill, and K. H. Hassan. (1994). Drilled shafts: Effects of construction on performance and design criteria. Proc., Int. Conf. on Design and Construction of Deep Foundations, U.S. Federal Highway Administration, Washington, D.C., Vol. 1, 137–187.
[4] D. Brown. Effect of Construction on Axial Capacity of Drilled Foundations in Piedmont Soils. Journal of Geotechnical and Geoenvironmental Engineering, Vol. 128, No. 12, December 2002, pp. 967-973.
[5] C. Lam, V. Troughton, S. Jefferis, T. Suckling (2010). Effect of support fluids on pile performance – a field trial in east London. Ground Engng 43, No. 10, 28–31.
[6] W. Hachich et al. Fundações: teoria e prática. 2. ed. São Paulo: Pini, 1998.
[7] J. C. Holden (1984). Construction of bored piles in weathered rock,Part 4: Bentonite construction procedures. Tech. Rep. No. 6, Road Construction Authority of Victoria, Australia.
[8] C, Lam, S.A. Jefferis, and C.M. Martin (2013). Effects of polymer and bentonite support fluids on concrete–sand interface shear strength. Géotechnique 64.1 (2013): 28-39.
[9] K. L. Tucker, and L. C. Reese (1984). The effect of bentonitic slurry on drilled shafts. Res. Rep. 351-1F, Center for Transportation Research, University of Texas, Austin, Tex.
[10] R. E. Majano, M. W. O'neill, and K. M, Hassan. (1994). Perimeter Load Transfer in Model Drilled Shafts Formed Under Slurry. Journal of Geotechnical Engineering, ASCE, Vol. 120, No. 12, pp. 2136-2154.
[11] R. Frizzi, M. Meyer, and L. Zhou (2004) Full Scale Field Performance of Drilled Shafts Constructed Utilizing Bentonite and Polymer Slurries. GeoSupport 2004: pp. 573-586.
[12] S. A. Jefferis, C. Lam (2013). Polymer support fluids: use and misuse of innovative fluids in geotechnical works. Proceedings of the 18th international conference on soil mechanics and geotechnical engineering, Paris, vol. 4, pp. 3219–3222
[13] ASSOCIAÇÃO BRASILEIRA DE NORMAS TÉCNICAS. NBR 6122: Projeto e Execução de Fundações. Rio de Janeiro, 1993.
[14] C. Van der Veen (1953) The bearing capacity of a pile. In: Procedures of Third International Conference of Soil Mechanics and Foundation Engineering. Zurich Proceedings...v.2, p 84-90.
[15] R. Salgado. The Engineering of Foundations. McGraw-Hill, 2008.
[16] N. Aoki, D.A. Velloso. An approximate method to estimate the bearing capacity of piles. Proceedings, 5th. Pan American CSMFE, Buenos Aires, 1975, vol.1, p. 367-376.
[17] L. Décourt, A. R. Quaresma. Capacidade de carga de estacas a partir de valores de SPT. In: COBRAMSEF, 6., 1978, Rio de Janeiro. Anais... Rio de Janeiro: ABMS, 1978. v. 1, p. 45-54.
[18] A. H. Teixeira. Projeto e Execução de Fundações. In: Seminário de Engenharia de Fundações Especiais e Geotecnia. São Paulo, 1996.
[19] Eurocode 7. Geotechnical Design – Part 1: General rules, European Committee for Standardization, 2004.
[20] M. Bustamante, R. Frank (1999). Current French design practice for axially loaded piles. Ground Engineering, 32 (3), 38–44.
[21] L. V. A. Lavalle. Estudo da interação solo-concreto das estacas escavadas do campo experimental de Araquari. Dissertação (Mestrado em Engenharia Civil). Universidade Federal do Rio Grande do Sul, 2017.
[22] G. F. Costa. Análise da Capacidade de Carga de Estacas Escavadas Submetidas ao Carregamento Vertical no Campo Experimental de Araquari: Métodos Semiempíricos e Modelagem Numérica. Trabalho de Conclusão de Curso (Graduação em Engenharia Civil). Universidade Federal de Santa Catarina. Florianópolis, 2023.
[23] B. H. Fellenius. Basic of Foundation Desing. Electronic Edition. Sidney, 2023
[24] GEO (2006). Pile design and construction. GEO Publication 1/2006, Geotechnical Engineering Office, Civil Department, Hong Kong Government.
[25] M. Tomlinson, J. Woodward. Pile design and construction practice. 5ª ed. Londres: Taylor & Francis, 2008.