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Shaft resistance of piles in lateritic soils: a reevaluation based on Brazilian experience

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Article

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Abstract

Lateritic soils are widely distributed in tropical regions and are characterized by a cemented structure due to iron and aluminum oxides. Although the Standard Penetration Test (SPT) is commonly used in Brazil, it does not adequately capture the stiffness of these cemented layers, leading to overly conservative pile design and increased costs. To address this limitation, this study presents a reevaluation based on 43 pile load tests in lateritic soils across various Brazilian sites. Results confirm that the traditional method by Décourt & Quaresma (1978), as later adapted by Décourt (1996), underestimates shaft resistance, with measured values averaging 2.1 times higher. This study proposes an adjustment methodology introducing a correction factor for lateritic soils ($\beta = 2$), which produced satisfactory predictions in 76.75% of cases, with 39.5% within a ± 20% margin. However, 23.25% of cases exceeded this range, mainly for manually excavated small-diameter piles, indicating sensitivity to construction methods and the need for site-specific calibration.

1. Introduction

Lateritic soils are predominant in tropical regions located between latitudes 30° North and 30° South, where high temperatures and intense rainfall occur for at least part of the year (McNeil, 1964). These soils are estimated to cover approximately 8% of the Earth's land surface and are found across nearly the entire Brazilian territory (Bernucci, 1995). They also occur across various regions of Africa, South America, Australia, India, and Southeast Asia (Winterkorn & Chandrasekharan, 1951; Remillon, 1955; McNeil, 1964; Maignien, 1966; Townsend, 1970; Saunders & Fookes, 1970; Varghese, 1987). In Brazil, lateritic soils cover about 65% of the country's total area (Villibor & Alves, 2019). The characteristics of these soils vary significantly across different locations due to variations in geological and climatic conditions (Miguel & Bonder, 2012).

Lateritic soils originate from the process of laterization, which involves leaching and chemical weathering, resulting in layers enriched with iron and aluminum oxides. The presence of iron is one of the most important factors influencing their engineering properties (Winterkorn & Chandrasekharan, 1951). These soils often exhibit increased stiffness due to natural cementation caused by the precipitation of iron and aluminum minerals within soil pores and fissures. Acting as natural bonds between soil particles, cementation enhances both the stiffness and strength of the material, positively impacting the performance of structures built on such soils. Therefore, accounting for this characteristic is essential in the design and construction of foundations.

Estimating bearing capacity is a key step in foundation engineering projects. Traditionally, practitioners rely on semi-empirical methods, such as the Décourt & Quaresma method (1978, 1996), based on Standard Penetration Test (SPT) results. Décourt (1994) demonstrated that lateritic clays exhibit significantly higher stiffness than non-lateritic clays, even when presenting the same SPT values. Although the traditional SPT resistance index (N_{SPT}) is a valuable indicator of soil strength, it may fail to capture soil stiffness, particularly when cementation influences behavior under low-stress and low-deformation conditions. This limitation arises from the destructive nature of the SPT, which can cause significant soil disturbance and hinder detection of weak bonding (Décourt, 2001).

Ignoring the natural cementation of lateritic soils can lead to an underestimation of bearing capacity, resulting in oversized foundation designs and unnecessary costs. Décourt (2022) emphasized that relying solely on SPT data for foundation design in lateritic soils can cause serious issues, including construction difficulties that jeopardize structural integrity and foundation costs up to three to four times higher

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than necessary. In fact, in soils with lateritic characteristics, the shaft resistance of piles can be two to three times greater than estimates from conventional methods.

The actual performance of foundations is best demonstrated through load tests. Studies by Albuquerque & Carvalho (1999), Nogueira (2004), Albuquerque et al. (2005), Abreu (2013), Schulze (2013), Abreu et al. (2015), Polido et al. (2016), Cerqueira & Souza (2020), and others, have documented the superior performance of foundations in soils with lateritic behavior, exceeding the estimates provided by semi-empirical methods. In contrast, in collapsible lateritic soils subjected to inundation, capacity can drop dramatically. Scallet (2011) reported a 40% reduction in total load capacity (30% loss in shaft friction and 48.2% loss in end bearing), and Falcão (2021) observed a 31% decrease after 96 hours of flooding, underscoring how saturation and loss of suction severely impair pile resistance.

These contrasting results highlight the limitations of conventional semi-empirical methods in accurately representing the behavior of lateritic soils, particularly regarding the influence of natural cementation on shaft resistance. Given these limitations, an adjustment was made to the method originally proposed by Décourt & Quaresma (1978), as later adapted by Décourt (1996), to better capture the influence of natural cementation characteristic of lateritic soils. The proposed modification was evaluated using a database of 43 pile load tests conducted across different Brazilian regions. The results indicate that the adjusted approach provides more consistent estimates of shaft resistance, reducing the underestimation commonly observed in conventional methods. On average, the revised formulation improved the agreement between predicted and measured shaft resistance, with satisfactory or enhanced predictions obtained for nearly 77% of the analyzed cases.

2. Identifying lateritic soils

The initial step in a foundation engineering project involving lateritic soils is the detection and identification of lateritic behavior through field reconnaissance and geotechnical testing. Traditional classification methods, such as those of the Highway Research Board and the Unified Soil Classification System, can lead to misinterpretations. Consequently, several researchers have proposed alternative classification methods for soils exhibiting lateritic behavior. These include the MCT Classification – Miniature Compact Tropical (Nogami & Villibor, 1981), the Laterization Index (Ignatius, 1991), the Expedient Method of Disk (Fortes et al., 2002; Villibor & Alves, 2019), the G-MCT Classification for granular soils (Villibor & Alves, 2019), and the UCT Classification System – Unified Compact Tropical (Ignatius et al., 2020).

Beyond these classification approaches, lateritic soils can also be identified through their intrinsic physical and mineralogical characteristics. Formed through laterization in hot and humid tropical regions, they typically contain high concentrations of iron and aluminum oxides and hydroxides, may be either residual or transported, and often display reddish or yellowish coloration. Their clay fraction is mainly composed of kaolinite, and oxide coatings create natural cementation between grains, resulting in greater stiffness compared with non-lateritic soils (Décourt et al., 2016).

Considering the complexities of soil behavior, as highlighted by Mayne (2001), it is inadvisable to rely solely on the N_{SPT} value for the majority of geotechnical data necessary for engineering analyses. An important auxiliary parameter to N_{SPT} is the shear modulus, G, which is essential for characterizing the behavior of geotechnical materials under static and dynamic loading (Stokoe et al., 1989). The shear modulus is defined as the ratio of applied shear stress to the corresponding deformation in a material. Generally, the stress-deformation relationship of geomaterials is neither elastic nor linear. However, at very small shear strains (typically less than 0.001%), the stress-strain relationship can be approximated as linear elastic. Under these conditions, G can be considered constant and reaches its maximum, known as the small-strain shear modulus or maximum shear modulus (G_0).

The determination of G_0 can be performed by measuring the propagation velocity of shear waves (V_s) either in situ or through laboratory tests on undisturbed samples (Lambe & Whitman, 1969). Field tests offer advantages, as the soil is evaluated in its natural environment, under anisotropic stress conditions, and with minimal disturbance compared to laboratory tests (Mayne & Rix, 1993). In the field, this parameter can be obtained through seismic geophysical tests, which may be invasive or non-invasive. In situ seismic tests typically result in shear strains on the order of 10^{-4}% (Robertson et al., 1986). Furthermore, Gandolfo (2022) emphasizes that, particularly in tropical soils such as lateritic soils, determining V_s through seismic tests can reveal variations in cementation levels, weathering degree, and soil structure that often go undetected by traditional tests.

In Figure 1a, the values of G_0 for lateritic soils are presented based on the correlation proposed by Barros & Pinto (1997), which relates G_0, obtained from crosshole tests, to the N_{SPT} value. This correlation was developed based on data from red clays in São Paulo, including Moema, Bela Vista, Vila Madalena, and Paraíso, as well as lateritic soils from the interior of the state. Conversely, Figure 1b illustrates the analyses conducted by Barros & Pinto (1997) on saprolitic soils. Their study concluded that, even for equal N_{SPT} values, lateritic soils exhibit higher G_0 values due to a naturally cemented structure. In contrast, saprolitic soils behave more similarly to soils from temperate zones. This characteristic can be used to assess potential lateritic behavior, as highlighted by Décourt (2002): comparing G_0 values obtained from geophysical tests with those derived from the equation by Barros & Pinto (1997) effectively serves to investigate whether a soil exhibits characteristics

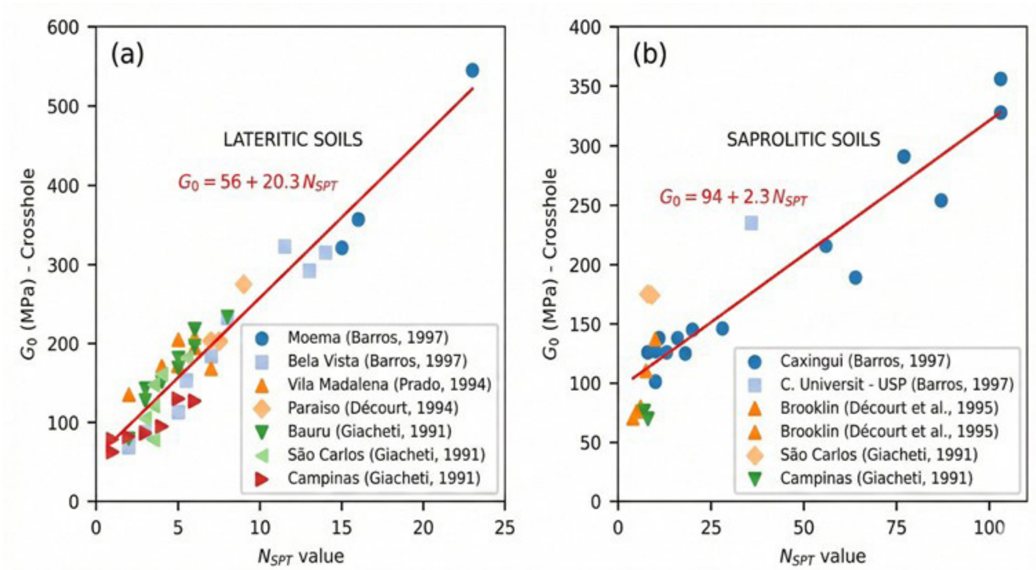

Figure 1. Correlation between G_0 and N_{SPT}: (a) lateritic soils; (b) saprolitic soils (adapted from Barros & Pinto, 1997).

associated with laterization. According to Décourt (2022), the method by Barros & Pinto (1997) generally allows clear identification of lateritic clays, although in some cases this identification may be less obvious.

Similarly, Schnaid et al. (2004) demonstrated that charts correlating seismic data with SPT tests, such as G_0 / N_{60} versus $(N_1)_{60}$, are effective tools for identifying soil microstructural characteristics like cementation and aging. Here, $(N_1)_{60}$ refers to the normalized N_{60}, which represents the SPT value adjusted to a reference efficiency of 60% of the SPT hammer, and accounting for effective overburden pressure. Their analysis revealed that lateritic soils exhibit distinct behavior due to microstructural features such as cementation or bonding, which most influence soil stiffness. Furthermore, they showed that lateritic soils typically have cemented structures, whereas clayey saprolitic soils generally lack such features. However, sandy saprolitic soils also showed evidence of cementation, indicating that saprolitic soils can exhibit varied behavior depending on their composition and microstructure.

3. Current design method and previous works

One of the most commonly employed semi-empirical methods in the Brazilian context is the approach developed by Décourt & Quaresma (1978). To adapt this method to various pile types, including excavated piles with bentonite slurry, general excavated piles (including open caissons), continuous flight auger piles, and high-pressure injected piles, Décourt (1996) introduced two correction factors, known as α and $\beta =$, which account for tip resistance and shaft resistance, respectively. According to the method proposed by Décourt & Quaresma (1978), with correction factors introduced by Décourt (1996), the estimated total load-bearing capacity of a pile ($R_{,est}$) is calculated as:

$$R_{,est} = \alpha K N_P A_P + 10\beta\left(\frac{N_s}{3}+1\right)u\,L \tag{1}$$

where:

$\alpha =$ correction factor for tip resistance;

$K =$ coefficient based on the type of soil (clay = 120 kN/m^2; clayey silt - residual soil = 200 kN/m^2; sandy silt - residual soil = 250 kN/m^2; and sand = 400 kN/m^2);

$N_P =$ average N_{SPT} for the tip using values above, at, and below the tip;

$A_P =$ cross-sectional area of the pile tip;

$\beta =$ correction factor for shaft resistance;

$N_S =$ average of the N_{SPT} values along the entire shaft of the pile, excluding values used for the pile tip;

$u =$ cross-sectional perimeter of the pile shaft;

L = length of the pile shaft.

The recommended values for α and $\beta =$, depending on the type of pile and the characteristics of the soil, are presented in Tables 1 and 2, respectively. It is important to note that these values remain unchanged (equal to 1) for precast, metal, and Franki-type piles.

Tables 1 and 2 show that certain factors significantly reduce shaft and tip resistance, depending on the type of pile and the nature of the soil. In particular, for soils exhibiting lateritic behavior, researchers, including Décourt, have focused on establishing specific adjustments to correct the conservatism of the method proposed by Décourt & Quaresma

Table 1. Values of factor α for the tip, based on the type of pile and soil (Décourt, 1996).

Soil type	Bored (general)	Bored (bentonite)	Continuous flight auger	Root pile	Injected under high pressure
Clays	0.85	0.85	0.3*	0.85*	1.0*
Intermediate	0.6	0.6	0.3*	0.6*	1.0*
Sands	0.5	0.5	0.3*	0.5*	1.0*

*Values are indicative only, due to the limited amount of available data.

Table 2. Values of factor $\beta =$ for the shaft, based on the type of pile and soil (Décourt, 1996).

Soil type	Bored (general)	Bored (bentonite)	Continuous flight auger	Root pile	Injected under high pressure
Clays	0.8	0.9*	1.0*	1.5*	3.0*
Intermediate	0.65	0.75*	1.0*	1.5*	3.0*
Sands	0.5	0.6*	1.0*	1.5*	3.0*

*Values are indicative only, due to the limited amount of available data.

Table 3. Values of factor β_l for soils with lateritic behavior.

Pile type	Reference	Location	β_l
Continuous flight auger	Polido et al. (2016)	Vitória/ES	1.94 – 4.39
	Cerqueira & Souza (2020)	Itaquera/SP	2.30 – 3.60 (2.9*)
	Albuquerque et al. (2005)	Campinas/SP	1.58 – 2.05 (1.86*)
	Abreu (2013)	Distrito Federal	2.39*
	Abreu et al. (2015)	Distrito Federal	1.74 – 6.25 (3.64*)
Precast	Abreu (2013)	Distrito Federal	2.60*
	Abreu et al. (2015)	Distrito Federal	1.80 – 4.41 (3.11*)
Bored	Albuquerque et al. (2005)	Campinas/SP	1.87*
	Schulze (2013)	Campinas/SP	3.27*
---------------------	Décourt (2018, 2022)	---------------------	2.00 – 3.00

*Average values.

(1978), with correction factors introduced by Décourt (1996). Table 3 presents a summary of recent literature in which β_l (where the subscript l refers to "lateritic") is back-calculated from shaft resistance measured in load tests.

Values in Table 3 can serve as references for preliminary evaluations of the bearing capacity of pile foundations in soils with lateritic behavior. However, it is important to emphasize that they are indicative and should not be considered conclusive. To ensure both safety and efficiency in foundation design, particularly in lateritic soils, conducting load tests is essential. These tests allow for an accurate assessment of the actual bearing capacity and help determine the appropriate enhancement coefficients for the specific foundation.

4. Reevaluation

This reevaluation seeks to establish a refined value of β for predicting shaft resistance of pile foundations in lateritic soils. To support this objective, several historical cases are reviewed and the experimental results are summarized herein, forming the database for the present analysis.

The database comprises 10 studies in which piles were installed and subjected to load tests in soils exhibiting lateritic behavior. These studies include both fully embedded and partially embedded piles in lateritic soils.

4.1 Experimental field of Unicamp – Campinas/SP

The soil at the Experimental Field for Soil Mechanics and Foundations of the State University of Campinas (Unicamp), in Campinas, São Paulo, originates from colluvial diabase. The surface layer consists of a highly porous silty-sandy clay, classified as both lateritic and collapsible. This layer has an average thickness of 6.5 meters and is categorized as lateritic clay (LG') according to the MCT classification. Beneath it lies a layer of residual silty clayey sand derived from diabase, extending to a depth of 19 meters. The water table is located approximately 17.7 meters below the surface. According to the MCT classification, the second layer is identified as non-lateritic clay (NG') and is characterized by a predominance of weathered materials (Albuquerque, 1996).

Research conducted by Albuquerque (2001), Albuquerque & Carvalho (1999), Albuquerque et al. (2005), Nogueira (2004), and Schulze (2013) was carried out at the Unicamp experimental field. The experiments included precast piles (Pile 2), excavated piles (E01, E02, E03, and SC1), continuous flight auger piles (HC1, HC2, and HC3), and root piles (C1, C2, C3).

Table 4 summarizes the load test results for the analyzed piles, including shaft resistance (R_L), tip resistance (R_P), and total failure load (R_T). The table also presents the estimated shaft resistance ($R_{L,est}$) calculated from borehole data using

Table 4. Compiled data from the Unicamp Experimental Field.

Reference	Pile	D (m)	L (m)	L_L (m)	L_L / L (%)	Estimated	Test Load		
						$R_{L,est}$ (kN)	R_L (kN)	R_P (kN)	R_T (kN)
Albuquerque & Carvalho (1999) and Albuquerque (2001)	Pile2	0.18	14.00	6.00	42.86	226.00	219.00	43.00	262.00
Albuquerque (2001) and Albuquerque et al. (2005)	E01	0.45*	12.00	6.00	50.00	358.00	680.60	3.40	684.00
	E02	0.45*	12.00	6.00	50.00	358.00	656.70	13.30	670.00
	E03	0.45*	12.00	6.00	50.00	358.00	667.90	25.10	693.00
	HC1	0.40	12.00	6.00	50.00	440.00	858.20	101.80	960.00
	HC2	0.40	12.00	6.00	50.00	440.00	904.00	71.00	975.00
	HC3	0.40	12.00	6.00	50.00	440.00	697.00	23.00	720.00
Nogueira (2004)	C1	0.40	12.00	6.00	50.00	555.00	950.00	30.00	980.00
	C2	0.40	12.00	6.00	50.00	534.30	958.30	21.70	980.00
	C3	0.40	12.00	6.00	50.00	541.00	937.90	42.10	980.00
Schulze (2013)	SC1	0.25	5.00	5.00	100.00	56.00	183.00	0.00	183.00

*Diameter measured after the pile installation.

the Décourt & Quaresma (1978), method, with correction factors introduced by Décourt (1996). Additionally, it reports the pile length in contact with lateritic soil (L_L) and the corresponding percentage of the total pile length (L_L / L).

All piles were instrumented with electrical strain gauges, enabling assessment of load transfer along the shaft and at the pile tip.

4.2 Condominium – Itaquera/SP

This case is located in the Resende Formation of the São Paulo Basin, composed of weathered clay deposits that predominantly exhibit lateritic behavior. The geotechnical profile of these sediments in the region is outlined according to Takiya (1997), apud Cerqueira & Souza (2020). The lateritic nature of the soil was further confirmed based on the correlations proposed by Barros & Pinto (1997), as analyzed by Cerqueira & Souza (2020). Considering the average N_{SPT} value of 24 from the boreholes, the ratio between the G_0 of lateritic and saprolitic soils varies between 2.72 and 3.64, with an average value very close to 3.0, as cited by Décourt (1996).

Table 5 presents the measured values of R_L, R_P, and R_T from the load tests, along with the pile dimensions and the ratio L_L / L. The table also includes the $R_{L,est}$, calculated using the Décourt & Quaresma (1978) method based on the nearest borehole data.

According to Cerqueira & Souza (2020), R_T corresponds entirely to R_L, since the maximum settlement of 15 mm recorded during the test was insufficient to significantly mobilize R_P.

4.3 UEL experimental field – Londrina/PR

The Professor Saburo Morimoto Experimental Field for Geotechnical Engineering (CEEG), at the State University of Londrina (CEEG/UEL), features a soil layer extending from the surface to a maximum depth of 12 meters. This layer consists of porous silty clay, with consistency ranging from very soft to medium, exhibiting lateritic and collapsible characteristics. As reported by Campos et al. (2008), laboratory geotechnical tests on samples from this soil were conducted by Décourt in 2002. The results indicated a laterization index of 1.54 (Ignatius, 1991) where values above 0.3 indicate lateritic soil, while below 0.3 do not. The soil was also classified as LG', according to the MCT classification.

Branco (2006) performed analyses on various types of piles, including excavated piles using mechanical augers, specifically A1(O) and A4(O), while Campos et al. (2008) investigated the behavior of manually excavated piles (ETM) and driven concrete piles (ACA). Table 6 summarizes pile dimensions, the ratio L_L / L, measured R_L, R_P, and R_T, and $R_{L,est}$ calculated using the method proposed by Décourt & Quaresma (1978), with correction factors introduced by Décourt (1996).

At UEL, Branco (2006) applied CAPWAP analysis to estimate R_L for piles A1(O) and A4(O). For the remaining piles (ETM and ACA), R_L was determined using the approach proposed by Décourt (1998, 2002), which employs the Modified Brierley Method (MBM) and the stiffness method (apud Campos et al., 2008).

4.4 Construction projects in the administrative regions of the Federal District

The climate in the Federal District is characterized by two distinctly different seasons: a rainy season and a dry season. Over time, this pattern has facilitated the leaching of salts and other soluble compounds from the upper layers, followed by their deposition in the deeper soil layers (Araki,

Table 5. Data from the tested pile in the São Paulo Basin – Itaquera/SP.

Reference	Pile	D (m)	L (m)	L_L (m)	L_L / L (%)	Estimated	Test Load		
						$R_{L,est}$ (kN)	R_L (kN)	R_P (kN)	R_T (kN)
Cerqueira & Souza (2020)	Tested	0.70	18.00	18.00	100.00	2853.74	7690.37	0.00	7960.37

Table 6. Compiled data from the UEL Experimental Field.

Reference	Pile	D (m)	L (m)	L_L (m)	L_L / L (%)	Estimated	Test Load		
						$R_{L,est}$ (kN)	R_L (kN)	R_P (kN)	R_T (kN)
Branco (2006)	A1 (O)	0.25	12.00	10.00	83.33	207.00	199.00	139.00	338.00
	A4 (O)	0.25	12.00	10.00	83.33	207.00	305.00	110.00	415.00
Campos et al. (2008)	ETM1L3	0.20	3.00	3.00	100.00	25.10	34.00	6.10	40.10
	ETM2L3	0.20	3.00	3.00	100.00	25.10	32.00	6.10	38.10
	ETM3L3	0.20	3.00	3.00	100.00	25.10	29.10	9.20	38.30
	ETM1L6	0.20	6.00	6.00	100.00	62.80	76.50	31.50	108.00
	ETM2L6	0.20	6.00	6.00	100.00	62.80	88.10	20.40	108.50
	ETM3L6	0.20	6.00	6.00	100.00	62.80	88.50	21.70	110.20
	ACA3(1)	0.20	3.00	3.00	100.00	25.40	75.00	30.50	105.60
	ACA3(2)	0.20	3.00	3.00	100.00	25.40	97.50	66.50	164.00
	ACA3(3)	0.20	3.00	3.00	100.00	25.40	130.70	12.70	143.40
	ACA6(1)	0.20	6.00	6.00	100.00	62.90	185.20	12.70	198.00
	ACA6(2)	0.20	6.00	6.00	100.00	62.90	162.40	99.70	262.10

1997 *apud* Abreu, 2013). The cumulative effect of these seasonal changes contributes to the formation of lateritic soils.

Studies by Silva (2011), Abreu (2013), and Abreu et al. (2015) were conducted in the cities of Guará, Ceilândia, Gama, and Samambaia, respectively. In these cases, the soils were classified as LG' using the disk method, except for the study by Silva (2011), where the classification was based on X-ray diffraction. The analyzed piles included continuous flight auger piles (PCCI, E1 to E7), while the remaining piles were vibrated precast circular reinforced concrete piles with a hollow section.

Table 7 summarizes the load test results, including R_L, R_P, R_T, and estimated $R_{L,est}$ using the Décourt & Quaresma (1978) method, with correction factors introduced by Décourt (1996), along with pile length L and ratio L_L / L.

The PCCI pile was instrumented with strain gauges to determine the R_L component. Abreu (2013) estimated R_L through CAPWAP analysis for some piles (E104a, E208B, E43D, and E53B), while for the remaining piles, the shaft load percentage indicated by CAPWAP was applied to the load tests to estimate R_L. Finally, R_L in continuous flight auger piles (E1–E7) was determined based on load tests using the stiffness method proposed by Décourt (1996, 2008), as reported by Abreu et al. (2015).

4.5 Comparison of estimated vs. measured shaft resistance

In the analyzed cases, most piles had a large portion of their length in contact with the lateritic soil layer, and load tests revealed predominantly frictional behavior, with most of the applied load absorbed by the shaft. Therefore, this study focused on analyzing the relationship between estimated shaft resistance ($R_{L,est}$) and the measured shaft resistance (R_L) from load tests.

Figure 2 presents a bar graph where the y-axis represents the ratio between $R_{L,est}$, based on the Décourt & Quaresma method (1978, 1996), and the measured R_L obtained from load tests. The piles are plotted along the x-axis. The analysis includes the scenarios presented in Tables 4 through 7. The graph indicates that, in general, the Décourt & Quaresma method tends to underestimate shaft resistance, with the exception of Pile 2 and A1(O).

To conduct a better comparative analysis, a scatter plot (Figure 3) was created to correlate the measured and estimated shaft resistance values. In the plot, measured values are shown on the y-axis, and estimated values on the x-axis. Most data points lie above the 45° line (x = y), indicating a tendency for underestimation, meaning the estimated values are lower than the actual measured values.

The identified discrepancy suggests that the Décourt & Quaresma (1978) method, with correction factors introduced by Décourt (1996), does not adequately account for the cementation resulting from laterization in the lateritic soil layer when assessing shaft resistance. It is important to highlight that even when using other established estimation methodologies, some authors, for example, Schulze (2013) and Albuquerque (2001), reported even lower load capacity values. This consistent underestimation of shaft resistance has significant implications

Table 7. Compiled data from the cities of the Federal District.

Reference	Pile	D (m)	L (m)	L_L (m)	L_L / L (%)	Estimated	Test Load		
						$R_{L,est}$ (kN)	R_L (kN)	R_P (kN)	R_T (kN)
Silva (2011)	PCCI	0.40	15.12	7.00	46.27	360.00	860.00	120.70	980.70
Abreu (2013)	E208B	0.42	18.64	15.00	80.47	701.00	1536.00	414.00	1950.00
	E212B	0.42	20.00	15.00	75.00	766.00	1610.00	433.00	2043.00
	E207	0.42	21.90	15.00	68.49	880.00	1412.00	380.00	1792.00
	E101	0.42	22.10	15.00	67.87	894.00	1491.00	401.00	1892.00
	E203b	0.33	16.20	15.00	92.59	478.00	543.00	244.00	787.00
	E104a	0.33	16.50	15.00	90.91	490.00	814.00	366.00	1180.00
	E54B	0.42	15.10	14.50	96.03	402.00	1800.00	510.00	2310.00
	E43C	0.42	16.75	14.50	86.57	463.00	1971.00	559.00	2530.00
	E53B	0.42	18.10	14.50	80.11	532.00	1869.00	581.00	2450.00
	E43D	0.42	18.50	14.50	78.38	552.00	1631.00	419.00	2050.00
Abreu et al. (2015)	E1	0.80	13.12	12.00	91.46	1413.00	3290.00	-	-
	E2	0.80	14.24	12.00	84.27	1351.00	2120.00	-	-
	E3	0.80	18.64	12.00	64.43	1860.00	5880.00	-	-
	E4	0.80	17.68	12.00	67.93	3369.00	4680.00	-	-
	E5	0.80	14.00	12.00	85.71	2596.00	3930.00	-	-
	E6	0.60	18.40	12.00	65.22	1859.00	4950.00	-	-
	E7	0.60	16.16	12.00	74.26	1468.00	4150.00	-	-

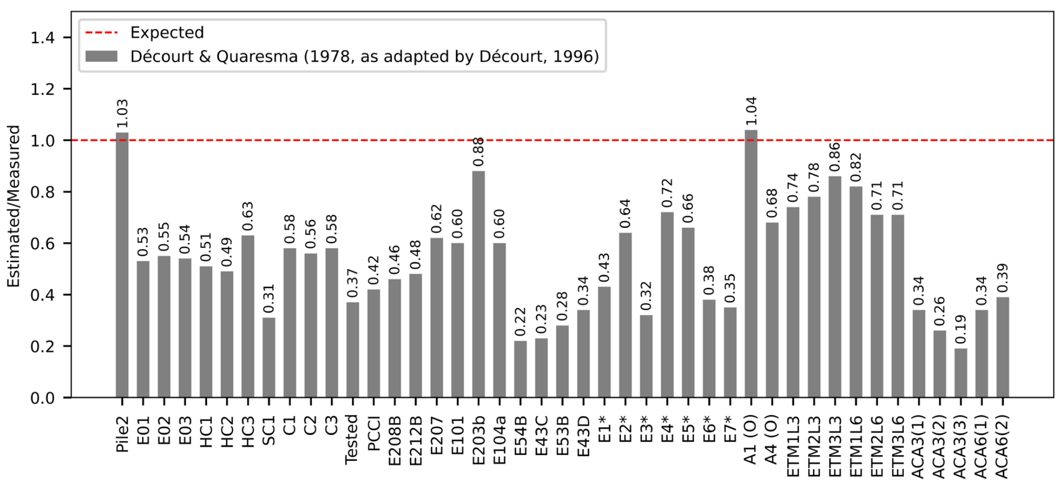

Figure 2. Bar graph showing the ratio between estimated shaft resistance $R_{L,est}$, according to the Décourt & Quaresma (1978) method, with correction factors introduced by Décourt (1996), and measured R_L from load tests.

for the design of foundations in lateritic soils, potentially leading to oversized structures and unnecessary costs.

A detailed analysis of the trend graph reveals that the measured values are approximately double the estimated ones. This observation corroborates Décourt's (2018, 2022) indications, suggesting that the shaft resistance of piles in lateritic soils is two to three times greater than that calculated by conventional methods. However, it is relevant to note that most piles are only partially embedded in lateritic soils. Additionally, these values may vary slightly depending on geometric factors and the degree of laterization present.

This observed discrepancy points to a potential limitation in the current method when applied to lateritic soils, motivating the development of an adjustment to account for these effects. Therefore, the next section presents a proposed adjustment to the Décourt & Quaresma (1978) method, with correction factors introduced by Décourt (1996), aimed at increasing the accuracy of shaft resistance estimates in these soils.

4.6 Proposed Adjustment to the Décourt & Quaresma Method (1978, as adapted by Décourt, 1996)

To improve the estimation of shaft resistance in piles embedded in soils with lateritic behavior, an adjustment to the Décourt & Quaresma method (1978, 1996) is proposed. The proposed adjustment aims to address the limitations of

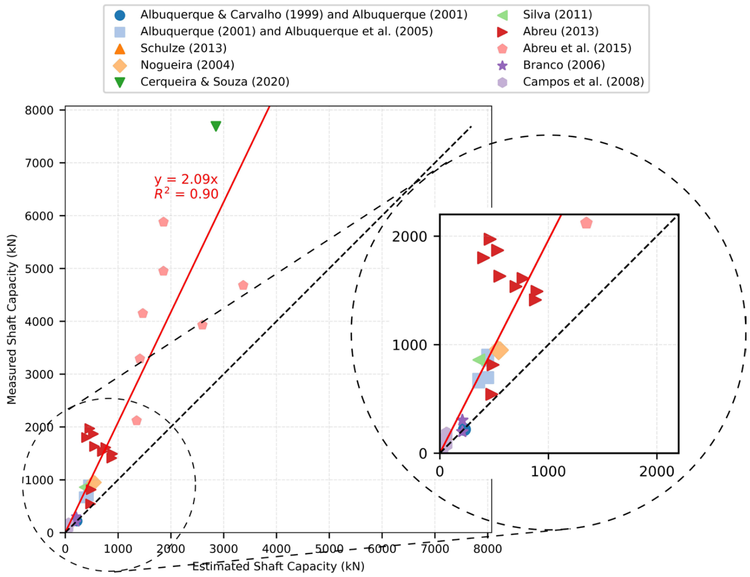

Figure 3. Comparison between measured shaft resistance from load tests and estimated values using the Décourt & Quaresma (1978) method, as modified by Décourt (1996).

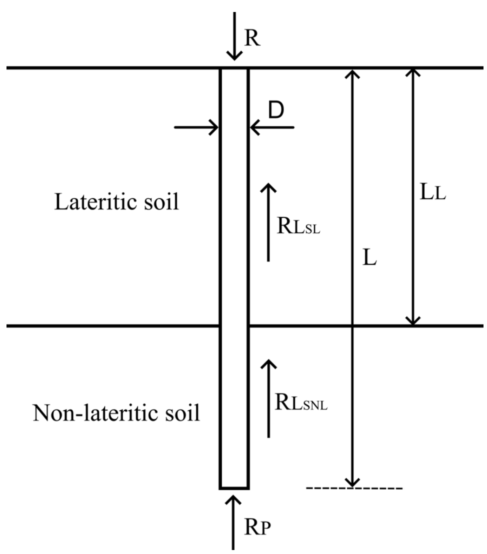

Figure 4. Schematic representation of lateral resistance calculation in lateritic soils.

the original method, which does not consider the distinct characteristics of lateritic soils. Notably, most of the analyzed piles are in contact with soil layers exhibiting both lateritic and non-lateritic behaviors. The proposed adjustment introduces a separate assessment of shaft friction for each layer type, as illustrated in Figure 4.

In Equation 1, originally proposed by Décourt & Quaresma (1978) and later adapted by Décourt (1996), the factor β adjusts the shaft resistance according to pile type and soil conditions, as shown in Table 2. Most β values are indicative and less than or equal to 1, except those related to root piles and piles injected under high pressure, which have values of 1.5 and 3. For soils exhibiting lateritic behavior, the parameter β_l, is used instead.

As presented in Equation 2, the shaft resistance within the lateritic soil layer is calculated as:

$$R_{L_{LS,est}} = 10\,\beta_l\left(\frac{N_{SPT\,SL}}{3}+1\right)u\,L_L \qquad (2)$$

where:

$N_{SPT\,LS}$ = average N_{SPT} along the pile shaft in contact with lateritic soil;

L_L = length of the pile in contact with lateritic soil;

As noted by Décourt (2018, 2022) and supported by the data presented in Figure 3, the shaft resistance of piles in soils with lateritic characteristics can be two to three times greater than that estimated by conventional methods. Therefore, β_l values may range from 2 to 3. However, these values are indicative only (as are those presented in Table 3) and can vary depending on the specific characteristics of the lateritic soils, as well as the type of pile used. Thus, the ideal value should be determined through more in-depth studies and, above all, load tests.

For soils that do not exhibit lateritic behavior, the original equation of the method remains unchanged, as shown below:

$$R_{L_{NLS,est}} = 10\,\beta\left(\frac{N_{SPT_{SNL}}}{3}+1\right)u\left(L-L_L\right) \qquad (3)$$

where:

$N_{SPT_{NLS}}$ = average N_{SPT} along the shaft segment in contact with non-lateritic soil.

The total lateral load capacity ($R_{L,est}$) is estimated by applying Equation 4. This expression accounts for the shaft resistance in both the lateritic and non-lateritic soil layers. Subsequently, the sum of these two terms provides the value of $R_{L,est}$:

$$R_{L,est} = R_{L_{LS,est}} + R_{L_{NLS,est}} \tag{4}$$

5. Discussion

The focus of this analysis is to evaluate the effectiveness of the proposed adjustment in estimating the load capacity of piles in lateritic soils through a comparative analysis between the results obtained by applying the proposed formula and those measured in load tests.

To this end, the methodology established by Schulze (2013), based on the work of Fellenius (1980), was adopted. It incorporates a fundamental criterion to ensure the reliability of results in the study of foundations in lateritic soils. This criterion defines a margin of error of ± 20% relative to the failure load obtained from the load test, accounting for natural variability and methodological uncertainty. This practice is widely recognized in national and international publications as a reference for assessing the accuracy of load capacity estimates.

Figure 5 presents a bar chart in which the y-axis represents the ratio between estimated and measured values for the Décourt & Quaresma (1978) method, as modified by Décourt (1996), and the proposed adjustment. The x-axis shows the tested piles. In the proposed adjustment, a β_l value of 2 was applied to the lateritic soil layer to account for the increased shaft resistance due to natural cementation. For the non-lateritic soil layer, a β value of 1 was used as a neutral reference, aiming to isolate the effect of laterization and focus the adjustment on the enhanced behavior of lateritic soils.

Nevertheless, for preliminary studies, engineers may choose to use the original β values from Table 2 for the non-lateritic layer if specific site conditions or pile installation methods require it. It should be noted that most of these original values are indicative only and generally lower than 1, except for root piles and piles injected under high pressure.

From the graph (Figure 5), approximately 39.5% of the piles had estimated shaft resistance values within the ± 20% margin of the measured load test results. Although this proportion may seem modest at first glance, it represents a significant improvement compared to the traditional method, which typically underestimated shaft resistance by an average factor of 2.1. Furthermore, many piles that fell outside this ± 20% margin still showed considerable improvement relative to the original approach, even though they did not fully meet the ± 20% criterion. Overall, the adjustment methodology provided estimates that were either within the ± 20% margin or closer to the measured values compared to the original method in 76.75% of the cases. Regarding the root piles (C1, C2, and C3), the estimated loads remained below the acceptable limit despite the observed improvements. However, by applying the factor β = 1.5, as recommended by Décourt (1996) for this type of pile, regardless of the soil type in which they were installed, the estimated values approached the measured results more closely.

Among the evaluated piles, 10 (23.25%) exceeded the upper limit, nine of which had diameters less than or equal to 0.25 m. Eight of these piles (18.60%) were located in the Londrina region and consisted exclusively of excavated piles. For piles A1 (O), A4 (O), and the ETM group (1L3, 2L3, 3L3, 1L6, 2L6, 3L6), the estimated values significantly exceeded the measured results. Overestimation reached approximately 153% for pile A1 (O), about 65% for A4 (O), and ranged from 65% to 115% for the ETM piles.

Another noteworthy case is the precast pile Pile2, which showed an overestimation of approximately 33%. Field observations during driving indicated that vibrations in

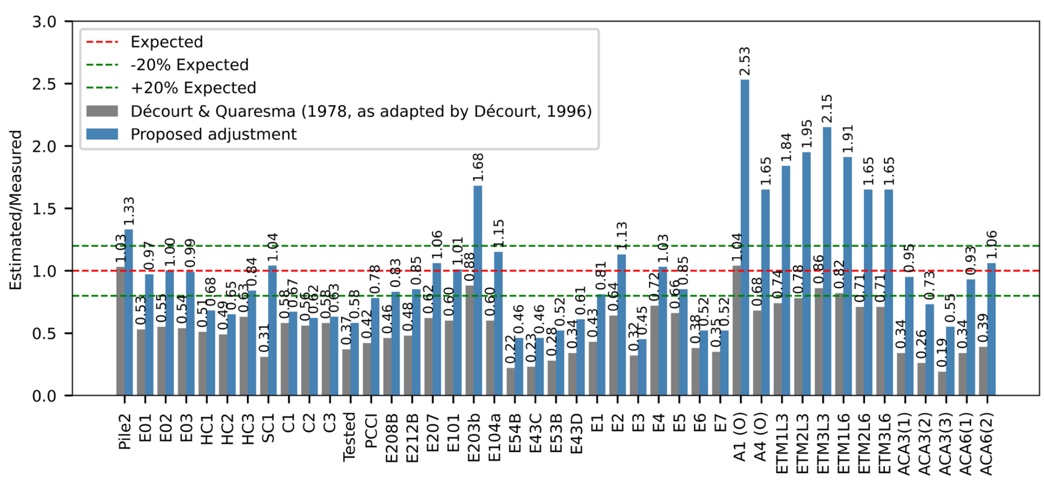

Figure 5. Bar graph showing the ratio between estimated and measured shaft resistance for the analyzed piles.

the upper, highly porous soil layers led to the formation of a gap between the pile and the surrounding soil, extending to a depth of about 1.5 m (Albuquerque & Carvalho, 1999). This phenomenon should be considered when driving piles in such soils, especially friction piles, as it can significantly reduce the mobilized shaft resistance. The partial loss of contact likely explains why the estimated value exceeded the measured result.

In these cases, the soil-shaft interaction did not fully mobilize cementation effects, making $\beta_l = 2$ excessive. Using this value without adjustment poses a risk of unsafe designs, as the overestimation may result in calculated capacities exceeding the soil's actual bearing capacity. It is therefore recommended to apply safety factors or calibrate β_l using site-specific load tests prior to design.

Interestingly, although both ETM and ACA piles in Londrina have the same diameter (0.20 m) and length (3.00 m or 6.00 m), the measured values for the ACA piles were more consistent with the estimates, whereas the ETM piles exhibited significant overestimations. This difference is attributed to the construction process of the Rammed Concrete piles (ACA), where the free fall of the ram reduces the void ratio around the shaft and beneath the pile tip, thereby increasing soil resistance. Furthermore, the gradual placement of the concrete promotes the formation of lateral bulbs and tip enlargement, thereby increasing the contact area between the soil and the pile and enhancing the load-bearing capacity (Campos et al., 2008). In contrast, ETM piles, which are executed by direct excavation, do not induce these effects, resulting in lower local resistance and, consequently, an overestimation of the predicted load capacity. These results demonstrate that the mobilization of shaft resistance is strongly influenced by both construction method and soil response, as illustrated by the contrasting behavior of ETM and ACA piles.

6. Conclusion

This study demonstrated that the method proposed by Décourt & Quaresma (1978), with correction factors introduced by Décourt (1996), tends to underestimate the shaft lateral resistance of piles in lateritic soils, due to the omission of the cementation effects that are characteristic of this type of soil. The analysis of 43 tested piles revealed that the measured lateral resistances were on average 2.1 times higher than the estimated values, corroborating findings in the literature, such as those by Décourt (2018, 2022), which indicate resistances two to three times greater than conventional predictions.

The proposed adjustment, which includes a specific correction factor for lateritic sections ($\beta_l = 2$), resulted in a significant improvement in prediction accuracy: 76.75% of the piles exhibited satisfactory performance, with 39.5% falling within the ± 20% range, a margin widely accepted in geotechnical practice. Underestimations were significantly reduced, highlighting the potential for project optimization and resource savings.

However, 23.25% of the piles showed overestimations beyond the acceptable range, reaching up to 153% in cases involving small-diameter piles (≤ 0.25 m) and manually augered piles. Conversely, rammed concrete piles presented results more consistent with the estimates, indicating that the construction method directly influences the mobilization of lateral resistance. These variations reinforce the need to calibrate the β_l factor based on local conditions and the adopted construction technique.

To enhance the reliability of the methodology, its application should be complemented by load tests and the acquisition of geophysical parameters, such as shear wave velocity and maximum shear modulus, that allow for spatial mapping of the degree of cementation.

The limitations of this study include the heterogeneity of the database, particularly regarding pile types, diameters, and construction techniques, as well as the empirical nature of the correction factor. Moreover, the degree of laterization and the execution procedures were not uniformly controlled. All cases studied were conducted under non-inundated, constant moisture conditions. In saturated conditions, the loss of suction can reduce resistance.

Future research should explore correlations between β_l and G_0, and develop protocols that integrate in-situ stiffness assessments with semi-empirical methods. Expanding the database, particularly for small-diameter piles, is also essential. The proposed methodology represents an important advancement; however, its application requires careful technical judgment and should be accompanied by field testing and detailed geotechnical characterizations to ensure structural safety, economic efficiency, and compatibility with site-specific design conditions.

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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

Alan Henrique Carneiro Brito: conceptualization, data curation, visualization, formal analysis, investigation, methodology, writing – original draft. Alessandro Cirone:

conceptualization, methodology, supervision, validation, writing – review and editing. Luciano Décourt: conceptualization, supervision, validation.

Data availability

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

Declaration of use of generative artificial intelligence

This work was prepared without the assistance of anygenerative artificial intelligence (GenAI) tools or services.All aspects of the manuscript were developed solely by theauthors, who take full responsibility for the content of thispublication.

List of symbols and abbreviations

u	Perimeter of the pile cross-section
ACA	Driven Concrete Piles
A_P	Cross-sectional area of the pile tip
CAPES	Higher Education Personnel Improvement Coordination (Brazil)
CAPWAP	Case Pile Wave Analysis Program
CEEG/UEL	Geotechnical Engineering Experimental Field / State University of Londrina
D	Pile diameter
ETM	Manually excavated piles
G	Shear modulus
G_0	Small-strain (maximum) shear modulus
G-MCT	MCT for granular soils (classification system)
K	(Décourt & Quaresma, 1978) Coefficient based on the type of soil
L	Length of the pile shaft
LG’	Lateritic Soil (MCT classification)
L_L	Length of the pile in contact with lateritic soil
L_L / L	Percentage of total length in lateritic soil
MBM	Modified Brierley Method
MCT	Miniature Compact Tropical (soil classification system)
NG’	Non-Lateritic Soil (MCT classification)
N_P	Average N_{SPT} for the tip using values above, at, and below the tip;
N_S	Average of the N_{SPT} values along the entire shaft of the pile, excluding values used for the pile tip
N_{SPT}	Standard Penetration Test resistance index
$N_{SPT_{LS}}$	Average N_{SPT} in the lateritic soil layer
$N_{SPT_{NSL}}$	Average N_{SPT} in the non-lateritic soil layer
N_{60}	SPT value adjusted to a reference efficiency of 60% of the SPT hammer
PUC-Rio	Pontifical Catholic University of Rio de Janeiro
$R_{,est}$	Estimated total load capacity of the pile
R_L	Measured shaft resistance
$R_{L,est}$	Estimated shaft resistance
$R_{L_{LS,est}}$	Estimated shaft resistance in the lateritic soil layer
$R_{L_{NLS,est}}$	Estimated shaft resistance in the non-lateritic soil layer
R_P	Measured tip resistance
R_T	Measured total failure load
SPT	Standard Penetration Test
UCT	Unified Compact Tropical (soil classification system)
V_s	Shear wave velocity
α	(Décourt, 1996) Correction factor for tip resistance
β	(Décourt, 1996) Correction factor for shaft resistance
β_l	Correction factor for lateritic soils

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