Methodological contribution to predict the preliminary bearing capacity of root piles embedded in soft rocks located in Northeastern Brazil

Contribuição metodológica para previsão da capacidade de carga de estacas raiz em rochas brandas localizadas no Nordeste do Brasil

Contribución metodológica a la predicción de la capacidad de carga de pilotes de raíz en rocas blandas ubicadas en el Noreste de Brasil

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ABSTRACT
The bearing capacity of a pile installed in a rock is usually predicted through the recommenda-tions of technical standards, empirical correlations, theoretical analyses, and/or full-scale loading tests. These methods use strength and deformability parameters, as well as parameters of the structural elements in contact with the rock. In general, geotechnical investigations carried out on rocks are far more complex, requiring longer periods and at higher operational costs. In this sense, the concepts applied in Rock Mechanics, such as geomechanical classifications, can contribute to better predictions on the behavior of piles embedded in soft rocks. This research analyzed data from piles whose stratigraphic profile had sections embedded in soft rocks that had been submitted to static load tests. After soil characterization was performed, multiple methods available in literature to estimate the bearing capacity of rock piles were evaluated and a method to estimate the failure load of root piles with sections embedded in soft rock was proposed. The presented formulation agreed fairly well with experimental values and can be easily applied in daily practice, being especially appropriate for preliminary design and dimensioning of piles embedded in soft rocks.

Keywords: Foundations on Rocks. Bearing Capacity. Root Piles. Geomechanical Classification.

RESUMO
A capacidade de carga de uma estaca instalada em rocha geralmente é prevista por meio de recomendações de normas técnicas, correlações empíricas, análises teóricas e/ou ensaios de carregamento em escala real. Esses métodos utilizam parâmetros de resistência e deformabilidade, bem como características dos elementos estruturais em contato com a rocha. Em geral, as investigações geotécnicas realizadas em rochas são consideravelmente mais complexas, exigindo

períodos mais longos e custos operacionais mais elevados. Nesse sentido, os conceitos aplicados na Mecânica das Rochas, como as classificações geomecânicas, podem contribuir para previsões mais precisas sobre o comportamento de estacas embutidas em rochas brandas. Esta pesquisa analisou dados de estacas cujo perfil estratigráfico apresentava trechos embutidos em rochas brandas que foram submetidas a ensaios de carga estática. Após a caracterização do solo, diversos métodos disponíveis na literatura para estimar a capacidade de carga de estacas em rocha foram avaliados, e foi proposto um método para estimar a carga de ruptura de estacas raiz com trechos embutidos em rocha branda. A formulação apresentada mostrou boa concordância com os valores experimentais e pode ser facilmente aplicada na prática cotidiana, sendo especialmente adequada para o projeto preliminar e o dimensionamento de estacas embutidas em rochas brandas.

Palavras-chave: Fundações em Rochas. Estacas Raíz. Capacidade de Carga. Classificação Geomecânica.

RESUMEN

La capacidad de carga de una pila instalada en roca suele predecirse mediante recomendaciones de normas técnicas, correlaciones empíricas, análisis teóricos y/o ensayos de carga a escala real. Estos métodos utilizan parámetros de resistencia y deformabilidad, así como características de los elementos estructurales en contacto con la roca. En general, las investigaciones geotécnicas realizadas en rocas son considerablemente más complejas, requieren períodos más prolongados y presentan costos operativos más elevados. En este sentido, los conceptos aplicados en la Mecánica de Rocas, como las clasificaciones geomecánicas, pueden contribuir a obtener predicciones más precisas sobre el comportamiento de pilas empotradas en rocas blandas. Esta investigación analizó datos de pilas cuyo perfil estratigráfico presentaba secciones empotradas en rocas blandas que fueron sometidas a ensayos de carga estática. Tras la caracterización del suelo, se evaluaron diversos métodos disponibles en la literatura para estimar la capacidad de carga de pilas en roca, y se propuso un método para estimar la carga de rotura de pilotes raíz con secciones empotradas en roca blanda. La formulación presentada mostró una buena concordancia con los valores experimentales y puede aplicarse fácilmente en la práctica cotidiana, siendo especialmente adecuada para el diseño preliminar y el dimensionamiento de pilas empotradas en rocas blandas.

Palabras clave: Fundaciones en Rocas. Pilotes Raíz. Capacidad Portante. Clasificación Geomecânica.

1 INTRODUCTION

The stresses imposed by modern engineering structures on the soils have required better solutions so that they can safely bear the diverse loads. Bored, root, or continuous flight auger piles embedded in soft rocks are quite common in everyday practice. However, they demand more representative methodologies to predict the behavior of soft rocks and estimate the piles bearing capacity.

In general, the prediction methods to determine the ultimate bearing capacity of piles consider that the shear strength will come from: (a) the pile shaft only; (b) the pile toe only; and (c) the pile shaft and toe together. According to the rock substrate, the best hypothesis can be selected.

Several empirical proposals are available in literature to design piles embedded in rocks, such as: Ladanyi (1977), Pells & Turner (1979), Poulos & Davis (1980), Williams et al. (1980), Horvath et al. (1983), Kulhawy & Goodman (1987), Rowe & Armitage (1987), Goodman (1989), Juvêncio et al. (2017), Rezazadeh & Eslami (2017), Gharsallaoui et al. (2017), and Mascarenhas (2018).

Literature on design of piles embedded in rocks displays a great variety of long-established proposals. More recently, Juvêncio et al. (2017) presented an empirical method to predict the bearing capacity of piles embedded in rocks using the Rock Quality Index (RQD). However, Pells et al. (2017) highlighted the difficulty in obtaining representative values of RQD and recommended the use of geomechanical classifications, such as the Geotechnical Strength In-dex, GSI, developed by Hoek (1994).

In this context, following the example of Rezazadeh & Eslami (2017), Gharsallaoui et al. (2017), and Mascarenhas (2018), the present study proposes a simplified empirical formulation to predict the bearing capacity of root piles embedded in soft rocks, using geomechanical classifications. The proposed expression is a modification of the method proposed by Mascarenhas (2018), in which the uniaxial strength of the rock was replaced by GSI. This change made the method simpler, more practical, and quite appropriate for the preliminary design of root piles embedded in soft rocks.

2 MATERIALS AND METHODS

The present study searched for engineering works in Brazil that had piles embedded in rocks, along with available data from geotechnical investigations, particularly results from rotary drillings and static load tests. Four root piles were selected from the dataset provided by the geotechnical company Tecnord Transfor Fundações LTDA.

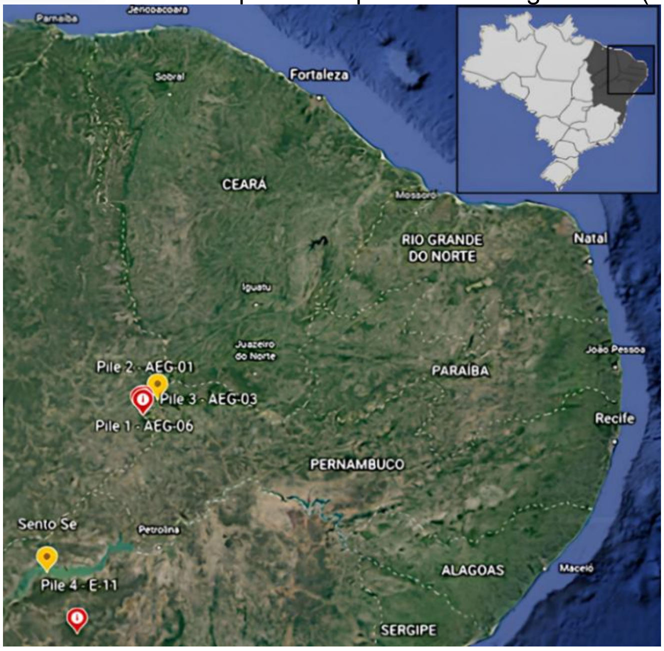
Three of the root piles (AEG-06, AEG-01, AEG-03) were installed in a wind farm complex called Chapada do Piauí III, located in the town of Simões, State of Piauí. The fourth pile (E-11) was executed in Campo Largo Wind Farm, located in the town of Sento Sé, State of Bahia. The Table 1 presents more details about the selected piles and The Figure 1 shows the location of the sites.

Table 1. Characteristics of selected piles

Site (Wind Farm)	UTM Coordinates (m)	Pile Code	Pile Diameter (m)	Pile Total Length (m)	Pile Length in Rock (m)	Workload (kN)
Ventos de Santo Augusto VII	-315,000W, -9,146,000S	AEG-06	0.410	12.00	8.00	1,168.93
Ventos de Santo Augusto II	-315,000W, - 9,140,000S	AEG-01	0.410	11.00	4.40	1,128.73
Ventos de São Virgílio III	-317,000W, -9,137,000S	AEG-03	0.410	12.00	9.00	1,128.73
Campo Largo IV	-229,000W, -8,842,000S	E-11	0.410	17.00	16.50	1,304.26

Source: Santos Neto (2022)

Figure 1. Location of selected piles. Adapted from Google Earth (7th edition)



Source: Santos Neto (2022)

Three of the root piles (AEG-06, AEG-01, AEG-03) were installed in a wind farm complex called Chapada do Piauí III, located in the town of Simões, State of Piauí. The fourth pile (E-11) was executed in Campo Largo Wind Farm, located in the town of Sento Sé, State of Bahia. The Table 1 presents more details about the selected piles and The Figure 1 shows the location of the sites.

The bearing capacity of the studied piles was estimated through expressions proposed by several authors. Pile behavior was simulated using numerical and analytical methods, available in three softwares developed by Rocscience Inc.: (i) RSPile (ii) RS2; and (iii) RS3.

Later, a method was developed and validated to predict the ultimate bearing capacity (Q_{ult}) of root piles installed in soft rocks. The data used in the validation step corresponded to two different piles, which were not part of the development dataset.

3 RESULTS AND DISCUSSIONS

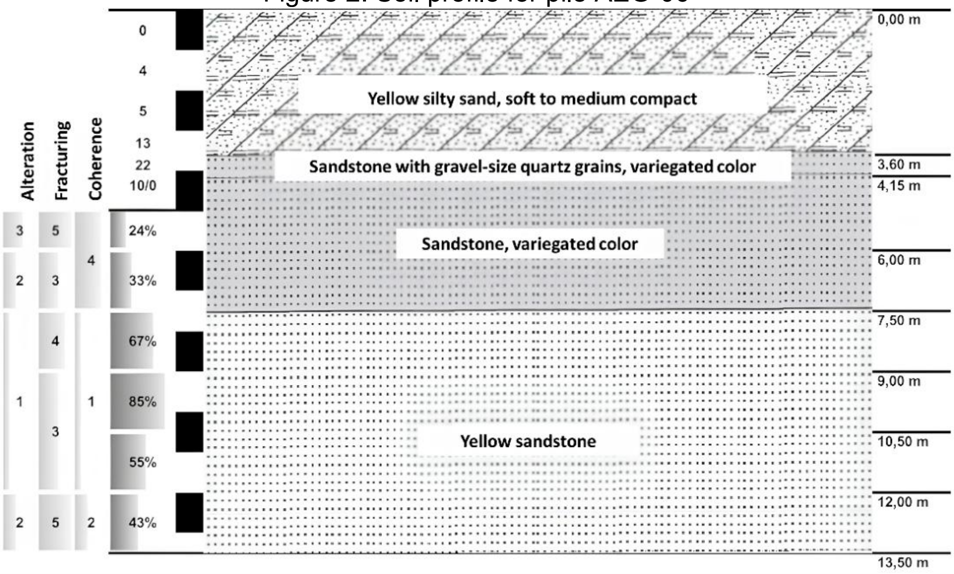
3.1 SOIL CHARACTERIZATION

Soil characterization was carried out considering results of SPT and rotary drillings, in which degree of rock alteration varied from unaltered rock to extremely altered rock, the degree of fracturing varied from slightly fractured to fragmented, and the degree of coherence ranges from very coherent to friable.

Figure 2 shows the stratigraphic profile of the area where pile AEG-06 was installed, where a 3.60-m-thick upper layer of yellow silty, loose to moderately compact. The second layer was a sandstone with gravel-size grains of quartz, variegated color, going down to -4.15m.

The third layer was a sandstone of variegated color that reached the depth of -7.5m. The fourth layer had a very rocky substrate (a yellow sandstone). Because of that, the SPT N-value was replaced by the Rock Quality Designation (RQD), whose average value was 29%. When the depth of -13.50m was reached, the test was stopped, with an average RQD of 63%. The presence of water in the soil was not detected in any of the sites where the analyzed piles were installed.

Figure 2. Soil profile for pile AEG-06



Source: Santos Neto (2022)

The stratigraphy of the area where pile AEG-01 was installed had a 6.60-m-thick layer of sandy silt with variegated lateritic concretions, soft to very compact, with SPT N-value of 55. The second layer corresponded to a sandstone of variegated color, reaching -10.50 m deep and a null RQD. The third layer was a red sandstone, reaching -17m, with a very high SPT N-value.

The area where pile AEG-03 was installed featured a 0.42-m-thick layer of gray silty sand. The second layer was a yellow silty sand, poorly to moderately compacted, that went down to a depth of -3.60m. The next layer was a sandy silt with lateritic concretions of variegated color, poorly to very compacted, and with increasing SPT N-values to a depth of -5.12m. Variations of friable sandstones were observed in the fourth layer, with RQD ranging from 0 to 60%.

As to pile E-11, the soil stratigraphy had a 0.50-m-thick layer of a fine, dark gray silty sand, with organic matter and sandstone pebbles. Below, there was a 1.00-m-thick sandstone layer, with intercalations of siltstone and gray and pink lateritic concretions, and RQD of 30%. The third layer was a silicified sandstone, intercalated with gray and pink siltstone, and RQD of 27% to a depth of -3.00m. Below, a sandstone with gray and red silicified layers, and RQD of 89% to a depth of -4.50m. The lower portion of this profile corresponded to a silicified sandstone, with an intercalation of gray and red siltstone, and RQD around 90%.

3.2 GEOMECHANICAL CLASSIFICATION OF STUDIED SOILS

The geotechnical characterization was complemented by determining the GSI for each rock layer in the profiles. The GSI System (MARINOS & HOEK, 2000) was chosen due to its already well-established correlations with the deformability modulus, as well as with the parameters of the generalized Hoek-Brown failure criterion.

These correlations are promptly determined from simple geological observations in the field or from analysis of field information, using adapted versions of classifications like the Rock Mass Rating – RMR (BIENIAWSKI, 1989) or the index Q (BARTON et al., 1974) and without dealing with the presence of water or orientation of discontinuities in relation to the pile. In this study, GSI values for each rock layer were analytically determined.

In this study, the propositions of the Brazilian standard DNER-PRO 102 (1997) regarding rock mass quality and Marinos & Hoek (2000) regarding uniaxial strength of the intact rock were merged. With the GSI values calculated for each rock layer, the strength parameters were obtained through the generalized Hoek and Brown failure criterion. In Table 2, for all layers assessed in this study, it was possible to obtain the GSI directly, just considering Bieniawski's overall RMR, with no further need to determine Q.

Table 2. Results of rock characterization for the assessed piles using the GSI classification

Depth (m)	GSI			
	Pile AEG-06	Pile AEG-01	Pile AEG-03	Pile E-11
1.00				50
2.00				55
3.00				60
4.00				64
4.15	35			64
5.00	35			67
5.10	35		48	67
6.00	50		47	67
7.00	50	35	47	67
7.50	50	35	35	67
8.00	58	35	35	64
9.00	64	35	35	69
10.00	64	35	35	69
10.50	64	35	35	69
11.00	60		35	69
12.00	48		35	69
13.00	48		35	69
13.50	48		35	69
14.00			48	69
15.00			48	69
16.00			48	69
17.00			53	69
$GSI_{average}$	53	35	65	39

Source: Santos Neto (2022)

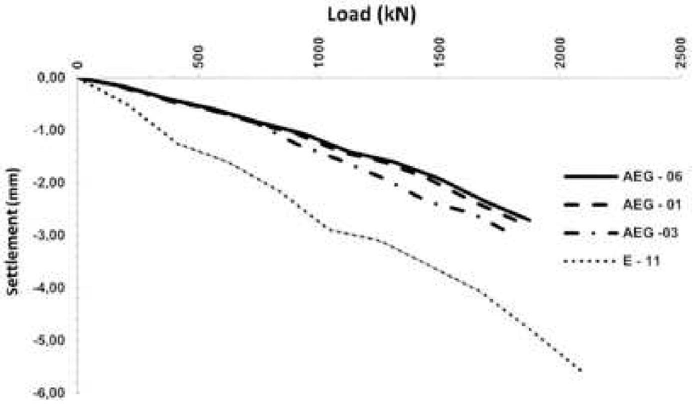
With all the necessary data available, the analytical methods to design piles embedded in rocks could be utilized. For the soil profile of pile AEG-06, the GSI values ranged from 35 to 64, with rock mass uniaxial compressive strength reaching up to 8.59MPa. As for the profile adjacent to pile AEG-01, lower GSI values (around 35) were obtained, with uniaxial compressive strength of

0.84MPa. Pile AEG-03 showed GSI values varying between 35 and 48, with rock mass uniaxial strength of 2.57MPa. Pile E-11 was installed in a really solid foundation, with a GSI of 69 and the highest uniaxial com-pressive strength of all soil stratigraphies, 12.27MPa.

3.3 RESULTS OF STATIC LOAD TESTS

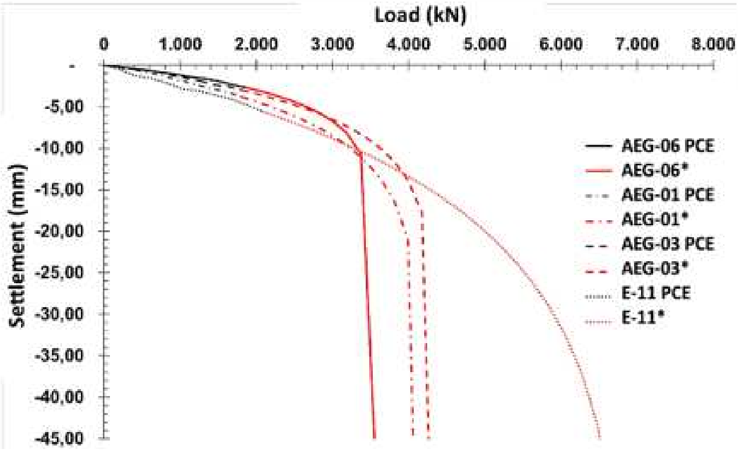
Figure 3 shows the load-settlement curves for the static load tests (SLT) analyzed in this study. As mentioned, all piles had sections embedded in rock layers. As the test goal was to guarantee the admissible service load considering a factor of safety (FS) of 1.6, and due to a limitation of the SLT reaction system, none of the piles reached physical failure. Thus, the failure loads were obtained through Van der Veen's extrapolation (1953), whose results are shown in Figure 4 and Table 3.

Figure 3. Curves load-settlement for piles AEG-06, AEG-01, AEG-03 and E-11



Source: Santos Neto (2022)

Figure 4. Failure loads obtained through Van der Veen's extrapolation for piles AEG-06, AEG-01, AEG-03 and E-11



Source: Santos Neto (2022)

In Table 3, it is possible to notice that the estimated failure load for piles AEG-06 and AEG-01 was around de 3,600kN. The residual settlement for both piles was very small, below 1.67mm. For piles AEG-03 and E-11, the obtained failure loads were 4,354.1kN and 6,893.0kN, respectively, with a residual settlement smaller than 1.17mm for both piles.

Table 3. Failure loads and Factors of safety (FS) obtained through Van der Veen's extrapolation

Pile	Failure Load (kN)	Workload (kN)	Max. Tested Settlement (mm)	Residual Settlement (mm)	FS
AEG-06	3,558.8	1,168.9	2.71	0.40	3.0
AEG-01	3,628.4	1,128.7	3.82	1.67	3.1
AEG-03	4,354.1	1,128.7	3.01	1.17	3.9
E-11	6,893.0	1,304.3	5.58	0.98	5.3

Source: Santos Neto (2022)

4 PREDICTIONS OF FAILURE LOAD THROUGH LITERATURE PROPOSALS

Bearing capacity estimates were made for the four studied piles (see Table 1). Methods used and values obtained are shown in Figure 5, where the estimated ultimate bearing capacity (Q_{ult}) was divid-ed by the correspondent results from the static load tests performed. Thus, in Figure 5, "100%" on the ordinate (y) axis means that the predicted value coincides with the reference (experimental) value. Here, it is worth mentioning that, for soil sections, the method proposed by Décourt & Quaresma (1982) was used.

As to pile AEG-06, out of the 16 evaluated methods, only 3 showed predictions lower than the ref-erence value. The methods that provided the most concordant predictions were Mascarenhas (2018), 20% above the reference value, and Décourt & Quaresma (1982), 8% below. Interestingly, the latter was not developed for rocks but for soils, and yet, the results were very good.

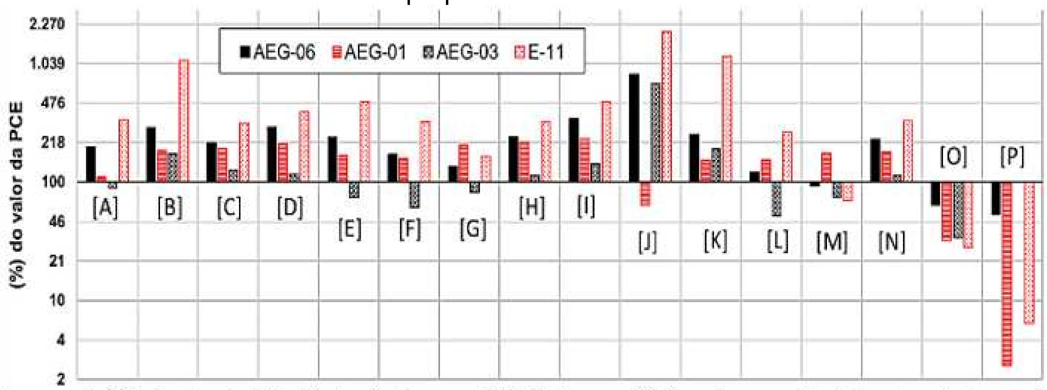
For pile AEG-01, the most concordant predictions were obtained by Ladanyi (1977), 10% above reference value, and Williams et al. (1980), 37% below. Numerical simulations performed by the softwares RS2 and RS3 provided predictions much lower than reference value: 68% and 97% respec-tively.

For pile AEG-03, the methods that had the best performance were software RSPile, 13% above, and Ladanyi (1977), 12% below.

As to pile E-11, the methods Décourt & Quaresma (1982) and softwares RS2 and RS3 had results below reference value (31%, 73%, and 94%, respectively), similar to what happened with pile AEG-06. The methods that provided the better predictions were Goodman (1989), 34% above, for a less rough bore hole scenario, and Décourt & Quaresma (1982), 31% below.

In general, Goodman (1989), Mascarenhas (2018), and Décourt & Quaresma (1982) were the methods that provided the closest results to reference values.

Figure 5. Predictions for ultimate bearing capacity (Qult) from different softwares and methods proposed in literature



Legend: [A]: Ladanyi; [B]: Pells & Turner; [C]: Kulhawy & Goodman with lateral resistance by Horvath & Kenney (1979); [D]: Kulhawy & Goodman with lateral resistance by Williams et al.; [E]: Kulhawy & Goodman with lateral resistance by Rowe & Armitage (1984); [F]: Rowe & Armitage; [G]: Goodman - Pessimistic; [H]: Goodman - Average; [I]: Goodman - Optimistic; [J]: Williams et al.; [K]: Horvath et al.; [L]: Mascarenhas; [M]: Décourt & Quaresma; [N]: Software RSPile; [O]: Software RS2; and [P]: Software RS3.

Source: Santos Neto (2022)

5 DEVELOPMENT OF A PROPOSAL TO PREDICT ULTIMATE BEARING CAPACITY OF PILES INSTALLED IN SOFT ROCKS

Subsequently, a proposal to predict the bearing capacity of piles embedded in soft rocks was developed, based on the equation proposed by Mascarenhas (2018). Initially, the method by Décourt & Quaresma (1982) was used to determine the load absorbed in the soil sections. For the rocky layers, the resisted load was defined considering the value obtained in the static load tests. The results for the evaluated piles are shown in Table 4.

Table 4. Distribution of load between rock and soil sub-layers

Pile	Q_{ult} (kN)	Load Absorbed by Soil Sub-layers (kN)	Load Absorbed by Rocky Sub-layers (kN)
AEG-06	3,558.80	1,021.61	2,537.19
AEG-01	3,628.40	3,346.09	282.31
AEG-03	4,355.65	413.69	3,941.96
E-11	6,893.00	314.80	6,578.20

Source: Santos Neto (2022)

Seeking to establish a better-fit relationship regarding Rock load/$GSI_{average}$ versus L/D (i.e., pile length embedded in rock divided by pile diameter), the uniaxial compressive strength of the intact rock was replaced by the $GSI_{average}$.

It is worth mentioning that the original proposal by Mascarenhas (2018) was developed from numerical simulations correlating the failure stress (normalized in relation to the uniaxial compressive strength of the intact rock) and the L/D ratio of the pile embedded in rock. In this context, the incorporation of GSI proposed in this research aimed at including characteristics of rock discontinuities in the estimation. Furthermore, using $GSI_{average}$ eliminated the need to use different correlations to obtain the deformability parameters and rock strength.

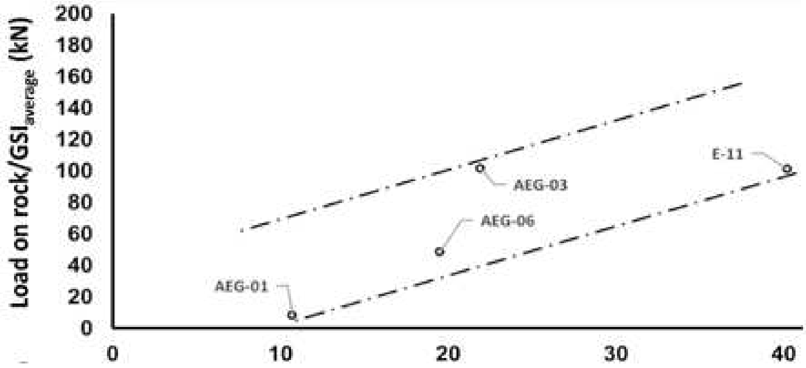
Figure 6 shows the relationship between the load resisted by rocky sub-layers (normalized by $GSI_{average}$) and the L/D ratio for the surveyed piles, obtained from results of static load tests. The trends in Figure 6 were used to find the best fit between $Q_{ult}/GSI_{average}$ and L/D, in order to correlate Q_{ult} (normalized by GSI) and L/D ratio.

Figure 7 shows some of the multiple equations obtained in this process. The best fit was obtained using a polynomial curve, presented in Equation 1, which had a good coefficient of determination (R2 > 0.86):

$$\frac{Q_{ult}}{GSI} = -0.1986.\left(\frac{L}{D}\right)^2 + \frac{13.409L}{D} - 116.26 \qquad (1)$$

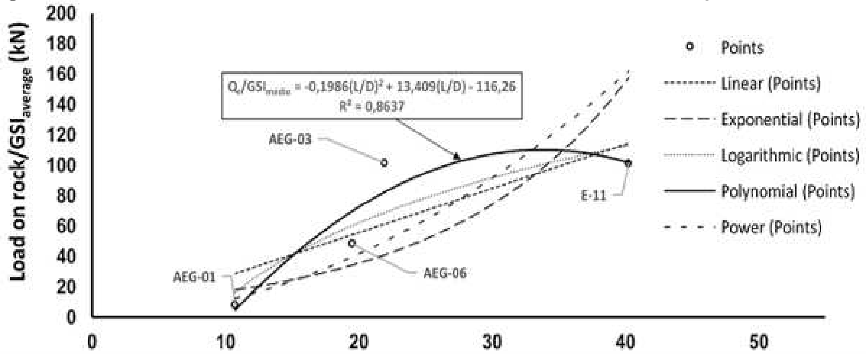
Thus, the proposal here presented modification of what was proposed by Mascarenhas (2018), by replacing the uniaxial strength by GSI, as well as the calculation of the failure stress by failure load of the pile section embedded in rock. An advantage of this method is the possibility of predicting the bearing capacity of the pile in soft rock without carrying out uniaxial compression tests, but consider-ing the geomechanical characteristics of the rock mass according to GSI, developed by Hoek (1994).

Figure 6. Correlation between load resisted by rocky sub-layers and L/D ratio



Source: Santos Neto (2022)

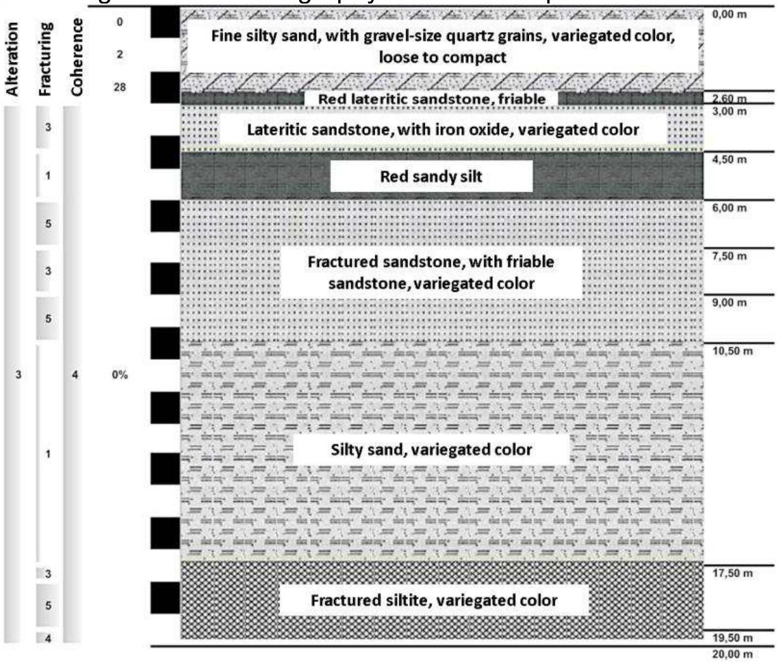
Figure 7. Different correlations obtained between $Q_{ult}/GSI_{average}$ and L/D ratio



Source: Santos Neto (2022)

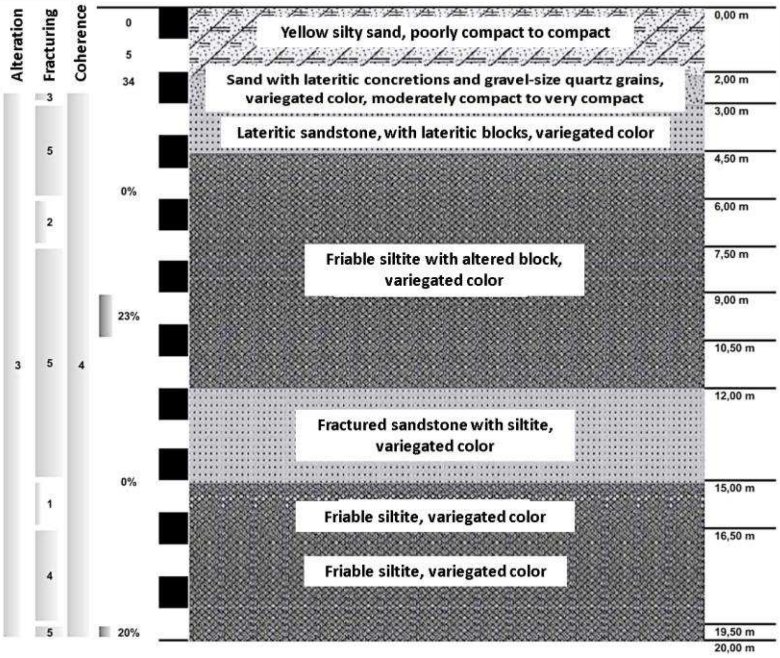
Two different root piles with a section embedded in rock – not used in the development phase – were selected to validate the proposed method: AEG-13 and V-AEG-06, installed in two wind farms (Ventos de Santo Estevão III and Ventos de Santo Estevão V) located in the town of Araripina, State of Pernambuco, Brazil. Table 5 presents their geometric characteristics and Figures 8 and 9, the soil stratigraphies obtained through SPT and rotary drillings carried out when they were installed.

Figure 8. Soil stratigraphy for validation pile AEG-13



Source: Santos Neto (2022)

Figure 9. Soil stratigraphy for validation pile V-AEG-06



Source: Santos Neto (2022)

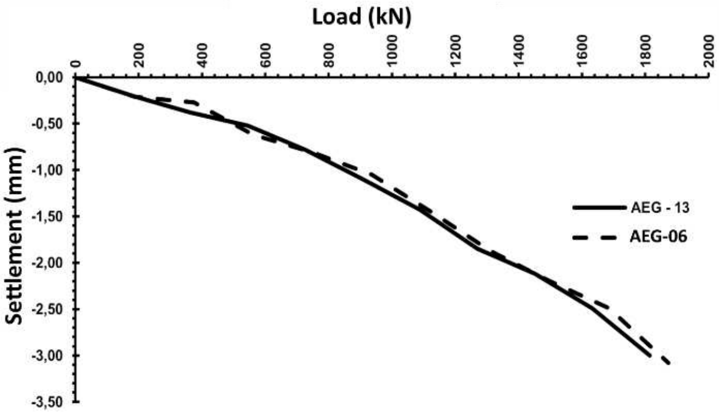
Table 5. Characteristics of piles used in the validation stage

Pile	Reinforcements	D (m)	L_{Total} (m)	L_{embed} (m)
AEG-13 (Ventos de Santo Estevão III)	7 Ø 20 with stirrups Ø 6.3 every 20cm	0.41	10.00	7.00
V-AEG-06 (Ventos de Santo Estevão V)	7 Ø 20 with stirrups Ø 6.3 every 20cm	0.41	11.00	8.84

Source: Santos Neto (2022)

Failure loads of the validation piles were determined using Van der Veen's extrapolation (1953). Results of the static load tests performed are shown in Figure 10 and summarized in Table 6, where maximum loads, maximum settlement, and residual settlement are also presented.

Figure 10. Curves load versus settlement for the validation piles



Source: Santos Neto (2022)

Table 6. Failure loads and settlements for the validation piles

Pile	Failure Load (kN)	Work Load (kN)	Max. Tested Settlement (mm)	Residual Settlement (mm)
AEG-13 (Ventos de Santo Estevão III)	7 Ø 20 with stirrups Ø 6.3 every 20cm	0.41	10.00	7.00
V-AEG-06 (Ventos de Santo Estevão V)	7 Ø 20 with stirrups Ø 6.3 every 20cm	0.41	11.00	8.84

Source: Santos Neto (2022)

In order to determine the loads absorbed by soil sub-layers, the method proposed by Décourt & Quaresma (1982) was again used. The loads absorbed by rocky sections are presented in Table 7.

Table 8 presents the GSI for each rock sublayer of the validation piles (AEG-13 and V-AEG-06), as well as their average values. Figure 11 shows the agreement between the curve of the proposed equation and the points corresponding to the validation piles, AEG-13 and V-AEG-06.

Table 9 and Figure 12 summarize the characteristics of validation piles, as well as the estimated and reference loads absorbed by section embedded in rock.

Table 7. Load resisted by soil and rocky sub-layers for the validation piles

Pile	Ultimate Bearing Capacity (kN)	Load Absorbed by Soil Sub-layers (kN)	Load Absorbed by Rocky Layers (mm)
AEG-13	3,084.14	754.78	2,329.36
V-AEG-06	2,996.87	623.52	2,373.34

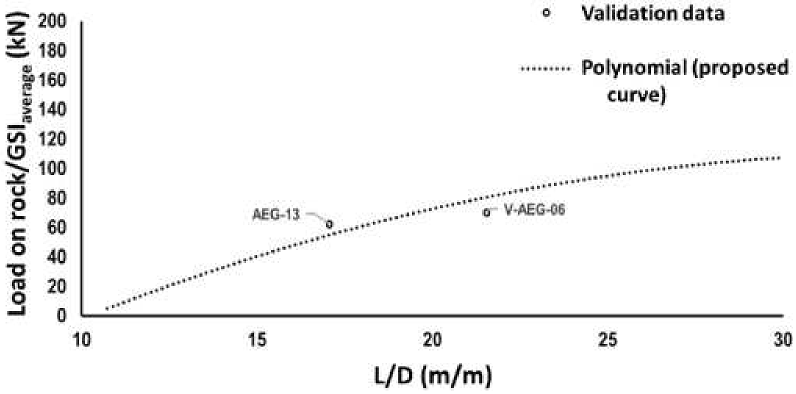
Source: Santos Neto (2022)

Table 8. Results of rock characterization for the assessed piles using GSI classification

Depth (m)	GSI	
	Pile AEG-13	Pile V-AEG-06
2.16	37	37
3.00	37	35
4.00	37	35
4.50	44	35
5.00	44	32
6.00	35	39
7.00	35	39
7.50	37	32
8.00	37	32
9.00	35	32
10.00	35	32
11.00		32
$GSI_{average}$	37.60	34.08

Source: Santos Neto (2022)

Figure 11. Validation of the correlation between load on rock and L/D ratio of the piles.



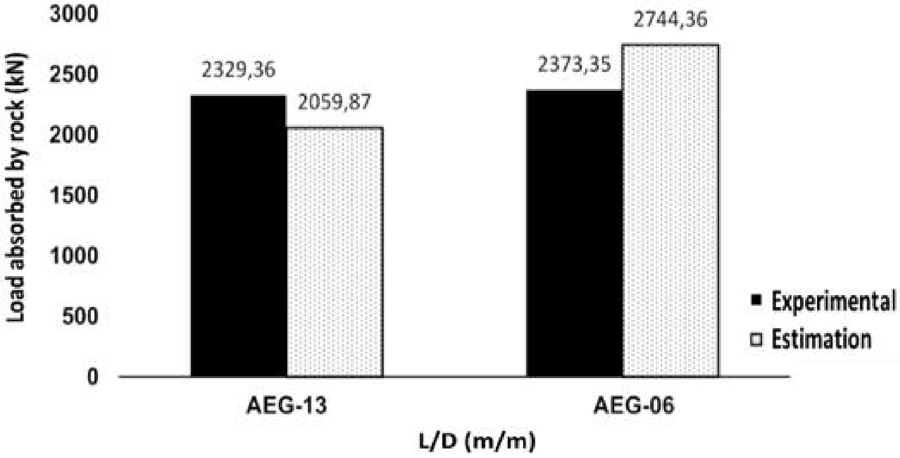
Source: Santos Neto (2022)

Table 9. Summary of characteristics of validation piles

Parameter	Pile AEG-13	Pile V-AEG-06
Total length C (m)	10.00	11.00
Length embedded in rock L (m)	7.00	8.84
L/D ratio	17.07	21.56
GSI	37.60	34.08
Rock failure load – experimental value (kN) - $Q_{ult,reference}$	2,329.36	2,373.35
Rock failure load – estimated value (kN) - $Q_{ult,estimated}$	2,059.87	2,744.36
$Q_{ult,estimated}$ / $Q_{ult,reference}$ (%)	88.43%	115.63%

Source: SANTOS NETO (2022)

Figure 12. Comparison between loads absorbed by rock for validation piles



Source: Santos Neto (2022)

Figure 12 shows a good agreement between the proposal here presented and the reference values obtained for the validation piles, with a maximum percent difference (the absolute value of the change in value, divided by the mean of the 2 numbers, multiplied by 100) of 15.63%.

6 CONCLUSIONS

This research aimed to propose a simplified empirical formulation to predict the bearing capacity of root piles embedded in soft rocks, using geomechanical classifications. The proposed model offers a simplified yet robust predictive tool, particularly suitable for infrastructure developments in Northeastern Brazil, where soft rock formations are common and geotechnical investigations are often constrained by cost and logistics.

As to the data analyzed in this study, predictions from numerical simulations based on empirical correlations were about 70% below the experimental results (reference values), whereas predictions made through semi-empirical methods mostly overestimated pile bearing capacity.

Among the rock-specific methods, the ones that better agreed with experimental values were Goodman (1989) and Mascarenhas (2018), both with an average error of 48% above reference value. The method proposed by Décourt & Quaresma (1982), despite not being specifically for rocks, had an average error of only 4% above experimental values, proving to be the best performance among the evaluated literature methods.

For all piles assessed in the validation stage, predictions obtained through the method proposed in this study had a fairly good agreement with experimental values. The method proposed here has the advantage of correlating the estimated bearing capacity of the evaluated pile with the geomechanical characteristics of the rock mass, through the Geotechnical Strength Index (GSI), presenting itself as a viable alternative in the preliminary prediction of the behavior of piles embedded in soft rocks, providing a cost-effective and operationally efficient alternative to conventional approaches that rely on extensive laboratory testing and complex numerical modeling.

The main limitations of this study are the limited dataset, restricted to four instrumented piles. Furthermore, as the static load tests did not reach physical failure, ultimate loads were derived through theoretical extrapolation, introducing a degree of uncertainty. Future research should include additional full-scale load tests and further calibration of the proposed equation with more instrumented piles, aiming to enhance its accuracy and extend its applicability to a broader range of pile geometries and rock mass conditions.

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