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Soil shear strength as a function of N_{SPT} via Van der Veen extrapolation

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Abstract

In this study, a ground anchored wall located in Belo Horizonte containing 295 anchorages in a sandy silt soil was analyzed. The load capacity of all the anchors was calculated by extrapolation of the receipt tests by the Van der Veen method through the CsAGeo web application. The shear strength in the soil-anchor interface was obtained from a semi-empirical method. Two criteria to analyze the extrapolated curves of Van der Veen were used. In the first criteria, the shear strengths of all the extrapolated curves were calculated. In the second criteria, only those curves from the extrapolations considered as reliable were used to calculate the shear strengths. The main objective of this work is to set up the value of shear strength at the soil-anchor interface through extrapolation of the mathematical and semi-empirical methods. The semi-empirical method was carried out in soil and executive methodology equal to the work analysed, which increases the accuracy of these values contributing to containments using anchors. The results were grouped according to the ranges of the penetration resistance (N_{SPT}). An increasing tendency was identified in the shear strength as the value of the resistance to penetration increased. In addition, the linear tendency was observed for the reliable curves.

Keywords: anchor, load capacity, shear strength.

1. Introduction

In the execution of pre-stressed and re-injectible anchors, a load of pre-defined magnitude is applied, generating a shear strength between the soil and the anchor as shown in Figure 1. The shear strength developed at the soil-anchor interface contributes to the soil stability in which it is anchored. The value of this parameter is influenced by the properties and characteristics of the anchor set and the used methodology, making it non-trivial and difficult to determine parameters.

Figure 1
Mechanical representation of applied load transfer and development of shear strength at the soil-anchor interface (Souza, 2001).

The shear strength (q_s) developed at the soil-anchor interface can be estimated from experimental observations of the soil area with similar geomechanical characteristics. Some engineers, in the absence of experience, estimate this value through empirical correlations from literature. Consequently, the correlations may be elaborated under specific conditions that are not representative. As a result, they provide results that do not reproduce the reality for the soil type at the analyzed region, which produces imprecise results.

One of the best ways to determine the shear strength is through the field load tests, and recalculating the design. However, in practice, the value of this parameter is not determined by the execution of load tests during the construction. Due to the variability of the shear strength as a function of the soil type, conditions, and used methodology, it is not possible to establish a simple correlation to determine the value of this parameter. The shear strength determination from the NBR 5629 tests (ABNT, 2006) - qualification, receipt and fluency - is indispensable for the evaluation of the shear strength at the soil-anchor interface.

Several studies have been carried out to obtain correlations between the value of shear strength and its variables. Liang e Feng (2002) claim that understanding the anchor-soil interface behavior, and

consequently the shear strenght, is essential in order to predict the deformation of the prestressed anchor system in working load terms. Hanna (1982), Barley (1997), Woods and Barkhordari (1997) studied the physical-mathematical relationship between the shear strength developed at the interface and the bulb displacement. Ortigão (1997) presented results of pullout tests correlated with Nspt values. Thus, the performance of nails as a function of shear strength can be evaluated in the initial stages of a work, before performing in situ tests. Skrabl (2004) evaluated the relationship between the amount of steel and the shear strength. Barley (1997) and Alonso (2015) analyzed the influence of the number of bulbs. Proto Silva (2005), studied the shear strength of nails in residual gneiss soil. From direct laboratory shear tests, the

author proposed a semi-empirical relationship for the evaluation of this parameter. Springer (2006) correlated the tensile-strain behavior and shear strength as a function of the method of execution, considering the number of injections, the wash of the hole, the cure time of the cement slurry and the soil. Erlich and Silva (2012) analyzed the results of pull-out tests and found correlations between shear strength, Nspt and number of injections. Porto (2015) and Porto *et al.* (2017) evaluated the relationship with specific load capacity, injection pressure, and injection number.

In this scenario, this study analyzed a work located in Belo Horizonte containing 295 anchorages in a sandy silt soil to calculate the load capacity of all the anchors through the extrapolation of the tests of

receipt by the Van der Veen (1953) method and then estimate the value of shear strength to the work in study.

The shear strength values at the soil-anchor interface were calculated from the methodology proposed by Porto (2015). This methodology was chosen because it was elaborated for conditions closer to those evaluated by the case study. The values obtained were grouped by N_{SPT} range in two distinct groups by the extrapolation types considered. In the first group, all the extrapolations of the curves from the 295 receiving tests were evaluated. In the second group, only the curves considered reliable according to Aoki *et al.* (2013), with 59 extrapolations, were evaluated. Figure 2 summarizes the analysis stages.

| 295 receipt tests | → | Data storage (CsAGeo) | → | Van der Veen extrapolation | → | Aoki et. al. criteria (2013) | → | Calculation of q_s Porto (2015) |

Figure 2
Sequence of the analysis performed.

All calculations (load capacity by Van der Veen extrapolation and determination of shear strength) were performed through the web application CsAGeo proposed by Martini *et al.* (2016). The application allows to *(i)* catalogue the data from the received test bulletins of

all analyzed anchors; *(ii)* calculate the extrapolated load capacity; and (iii) issue individual reports containing the curves and specific information for each anchor.

The use of the web application in the calculations contributed to the organization, data logging security,

information quality, agility and productivity. In addition, it prevented loss or alteration as mentioned by Gouveia *et al.* (2017), since the data were stored directly in the software (offline and then transmitted and assured through the generation of reports).

Porto's Methodology

The methodology proposed by Porto (2015) was based on pre-stress tests developed at the end of the 1970s in the construction of the subway in the city of São Paulo and in other works

located at the states of São Paulo and Minas Gerais in the year of 2015, using different methodologies. From the results found and the extrapolation of Van der Veen, Equations 1, 2 and 3 were

proposed to calculate the geotechnical load capacity, average bulb diameter, and shear strength at the soil-anchor interface, respectively:

$$T_L = \pi D_S L_a q_s \qquad (1)$$

$$D_S = \beta \cdot D_P \qquad (2)$$

$$q_s = 10k \left(\frac{N_{SPT}}{3} + 1 \right) \qquad (3)$$

Where: T_L =load capacity (kN); D_S = average bulb diameter (m); L_a =length of the anchored parts of the tie rod (m); q_s =shear strength at

the interface soil-anchor (kN/m²); D_P =hole diameter (m); β =bulb increase coefficient due to injection, show in Table 1; k = coefficient of anchorage

(kN/m²), show in Table 1, N_{SPT} =number of blows obtained in the Standard Penetration test (SPT).

Soil Type	β	k
Silty clay	2.1	1.25
Sandy clay	2.1	0.95
Clayey silt	1.97	2.57
Silt	2.11	2.16
Sandy silt	2.25	1.74
Clayey sandy	2.2	2.67
Silty sand	2.2	2.24

Table 1
Values of the increase coefficient β and anchor coefficient k (Porto, 2015).

Extrapolation of Van der Veen (1953)

In this work, the loading line of the received tests were used for the application of Van der Veen (1953) extrapolation and the determination of the load capacity. Similarly, the load-displacement curves obtained by the load tests on the piles can be divided into three regions (Figure 3) and described as follows:

a) The first region (I) presents high proportionality between the loads and displacements. It is characterized as the plastic deformation stretch of the load-displacement curve.

b) The second region (II) has viscoelas-tic deformation. In this section, studies affirm that the application speed of the load significantly alters the presented displacements.

c) The third region (III) corresponds to the region of rupture in which is defined the maximum load supported by the anchorage (the load capacity).
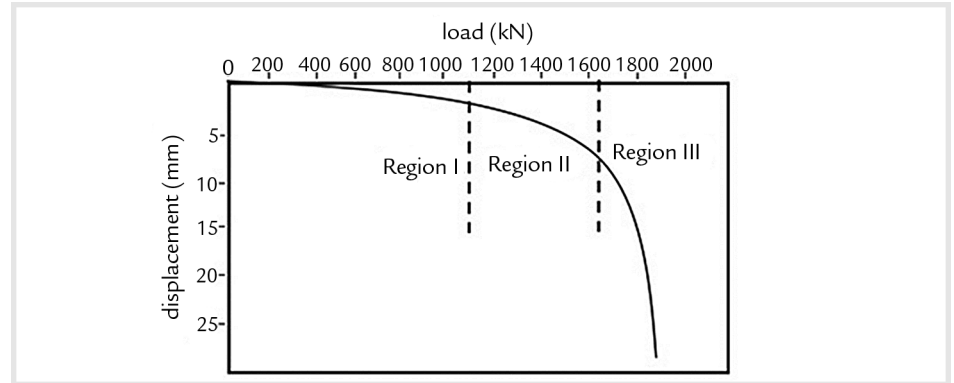

Figure 3
Load test x displacement extrapolated by Van der Veen (1953).

The proposed extrapolation method uses an exponential form, adjusting the points of the load-displacement curve to a mathematical function, analyzing the load capacity at theoretically infinite displacements.

$$ad = - \ln \left(1 - \frac{F_r}{F} \right)$$ (4)

where F is the load applied at the top of the anchor (kN); F_R is the last load corresponding to the vertical asymptote of the curve (kN) and is equal to the T_L referred in Equation 1; a is the coefficient that defines the shape of the curve (mm^{-1}); and d is the displacement corresponding to the load (mm).

Equation 4 corresponds to the equation of a line in which a and F_R are two constants determined, in a process of attempts, adopting values for F and

Equation 4 obtains the association of the load versus displacement curve:

plotting the corresponding graphs of $-\ln(1-F/F_R)$ versus d. The curve closest to a straight line will indicate the value of F_R (unknown value from the problem). The angular coefficient of this straight line is a as shown in Figure 4.
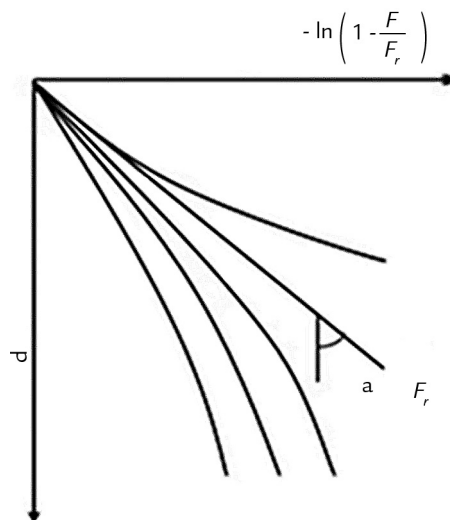

Figure 4
Van der Veen graphic solution (1953).

$$F = F_R \left(1 - e^{-adb} \right)$$ (5)

where b is the intercept in the axis of the displacements of the straight obtained in the semi logarithmic scale.

The coefficient of determination r^2 measures the conformity of the extrapolated section with the experimental

section. Thus, when its value is close to 1, the proposed model is adjusted to the experimental curve. When adapting to the Van der Veen's equation (1953), the author aimed at a value closer to 1, contributing to a better adjustment of the load-

displacement curve, with the intermediate and final points of the loading.

Although the extrapolation proposed by Van der Veen (1953) is a very practical alternative, it should be used with attention. According to Aoki *et*

al. (2013), if the determination coefficient is not sufficiently close to 1, the model proposed by Van der Veen (1953) should not be applied to the curve, otherwise the extrapolation will not be valid. Based on this interpretation, the author proposes reliability levels to the extrapolation. Aoki *et al.* (2013) relates the value of the load capacity obtained by extrapolation (F_R) with the maximum load applied in the test ($F_{máx}$), according to Table 1.

$[(F/F_R)-1]*100$	Extrapolation
≤ 25%	Reliable
25% - 50%	Acceptable
50% - 75%	Tolerable
≥ 75%	Unacceptable

Table 2
Validity of the extrapolation proposed by Van der Veen (1953) according to Aoki et al. (2013).

2. Material and methods

The case study refers to a construction of an automobile parking lot, located in the central region of the city of Belo Horizonte. The work is composed of seven anchored sections, denominated as sections *A, B, C, D, J, K, L*. The geological-geotechnical profile of each area was determined by selecting the closest and available N_{SPT} of each section under analyses. From this information, it was possible to determine the average N_{SPT} in the bulb of the anchor, relating it to the type of soil in which it is located. It was noticed that the majority of the bulbs where located at residual soil of a silt sandy medium, which was compact in the case of the first ones closest to the surface and compact for the deep rows. and a minority of them where located in gnaissic rock.

The six stretches of the anchored wall are composed of 475 permanent and temporary anchors, the distribution is shown in Table 2. The values of free and anchored length vary from 4 and 13 meters and the slope of the tie rod varies from 10 to 25 degrees, according to the type of tie rod and the section under which it is installed. Since the in situ tests were not followed up by the authors, a complete evaluation from the pre-stress bulletins was made. Among the 475 anchors selected for analysis, 140 were discarded. Therefore, 335 anchors were used in this study (245 temporary and 90 permanent). The reasons for the discard were *(i)* bulletins whose loading and displacement data were not informed; *(ii)* bulletins were missing; and *(iii)* tests in which the incorporation data of the tie rod were not informed. In addition, due to the restricted number of load capacity calculation methodologies applicable to bulbs located in rocks, 40 anchors (all permanent), whose bulbs were anchored in rocks, were also discarded, leaving 295 anchors. Figure 5 presents the sections, and the Table 3 presents the number of anchors in each of them and the life cycle type of the anchors (*i.e.*, temporary and permanent).

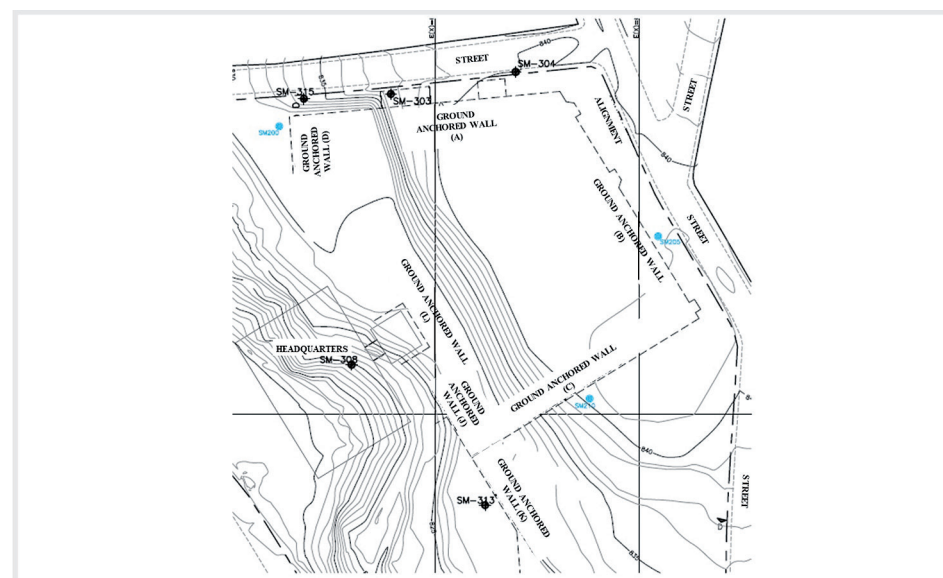| Stretch	n° of anchors	Life cycle	
		Temporary	Permanent
A	71	71	0
B	94	94	0
C	80	80	5
D	27	0	27
L	23	0	23
Total	295	245	50

Table 3
Characteristics of the anchors.

Figure 5
Overview of the ground anchored wall analyzed.

The tests were performed using a hydraulic jack assembly, a pump, and a manometer. In some tests, a Rudloff MP5-7C hydraulic jack - N ° 19 was used and in others, an Incotep Hm-04.

3. Results

The shear strength in the soil-anchor interface was calculated from the rearrangement of the load capacity equation proposed by Porto (2015). The values of shear strength found were organized according to their average values of N_{SPT} (Table 4) for all the tests and according to the tests considered reliable by Aoki *et al.* (2013). Due to the reduction of the sample space caused by the adoption of the exclusion criterion, there were no extrapolations of the tests whose N_{SPT} values of the anchors were in the 10-14 range. Therefore, the average value for this range is interpolated, as shown by the Table 4.

N_{SPT}	q_s (kPa) (all tests)	q_s (kPa) (Aoki et al. (2013) criteria)
5-9	81.11	80.17
10-14	86.06	84.17*
15-19	116.06	88.18
20-24	146.33	100.07
25-29	150.39	97.56
30-34	207.15	132.37
35-40	175.65	122.46

Table 4
Average values of N_{SPT} ranges.

The curve with the average values of shear strength for each N_{SPT} range for the two analyses (Aoki's criteria and all tests) are shown in Figure 6.
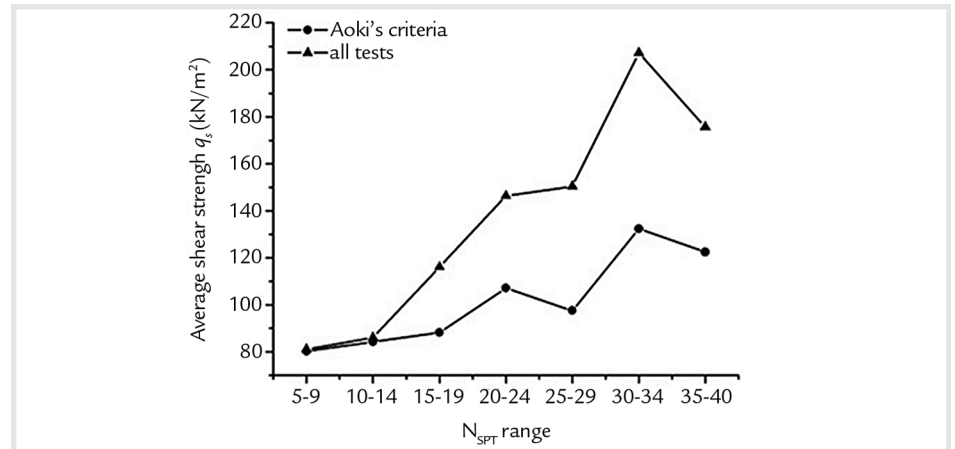
Figure 6
Average q_sversus N_{SPT}: all tests and Aoki et al. (2013) criteria.

Figures 7 and 8 present the shear strength values developed at the soil-anchor interface for each anchor and its respective value of N_{SPT}. Figure 7 presents a graph with shear strength values obtained based on the extrapolation of all the tests evaluated. Figure 8 only shows the extrapolations considered reliable according to the criterion of Aoki *et al.* (2013).
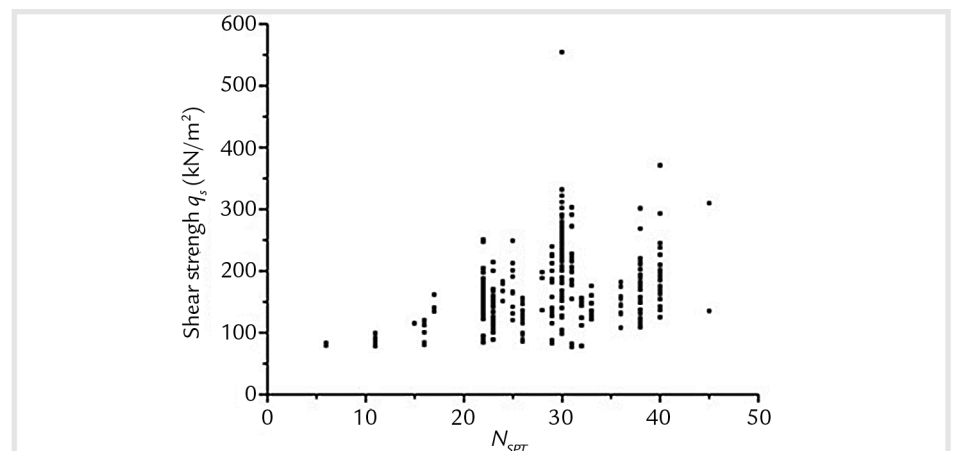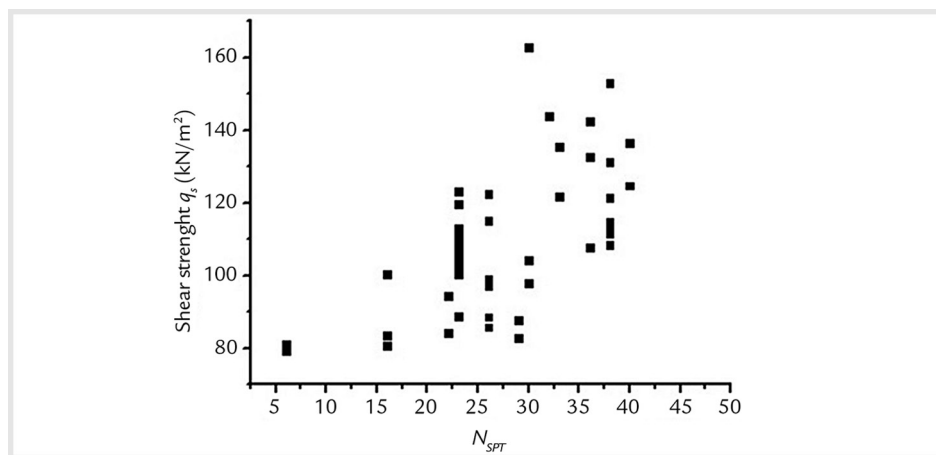
Figure 7
q_s versus N_{SPT}: all tests.

Figure 8

q_s versus N_{SPT}: tests considered reliable according to Aoki et al. (2013).

4. Discussion

The analyses of the average values obtained by resistance at the penetration range allows to conclude that there is a tendency of increase for the values of q_s as the resistance of the soil increases (Figure 6). For both analyses, it was observed that for the soils whose value are in the ranges of 25-29 and 35-40, there is an average value of q_s below the increasing behavior observed for the others N_{SPT} ranges.

Similarly, in the analyses performed for the extrapolations considered reliable, the shear strength values considering the N_{SPT} ranges were lower than in the evaluation of all the extrapolations. Although the average values have shown a tendency of linearity (excluding the N_{SPT} range of 25-29 and 35-40) between the values of N_{SPT} and shear strength, all the anchors analyzed (Figure 7 and 8) show very scattered results, which do not allow the identification of any behavioral curve. However, it was possible propose an initial estimation for the shear strength value at the soil-anchor interface of sandy silts (Table 5).

N_{SPT}	q_s (kPa)
5-9	75
10-14	85
15-19	95
20-24	105
25-29	115
30-34	125
35-40	135

Table 5

Proposal of initial estimation of q_s for sandy silts of the Belo Horizonte region.

5. Conclusions

The graphical analyses of the results (for all the analyzed criteria), demonstrates a tendency of a linear relationship for the increase between the shear strength value proposed by Porto (2015), and the penetration resistance of the studied soil. The observed dispersion can be related to the different vertical confinement tensions to which the analyzed bulbs are subjected. It is concluded that the restriction proposed by Aoki *et al.* (2013) presented results that are more conservative. The results of shear strength at the soil-anchor interface showed that it is not possible to establish a simple correlation with the value of N_{SPT} However, it was possible propose an initial estimation for the shear strength value at the soil-anchor interface of sandy silts.

Acknowledgments

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