Proposal of a failure envelope for artificially cemented sand

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Abstract

Soils are particulate materials, which often stand out for their low shear strength. In most cases, this aspect justifies adapting the geotechnical project for this characteristic, which can be even more problematic in the case of soft sands, since it often occurs in superficial layers in various parts of the metropolitan region of Fortaleza. Therefore, the soil improvement procedure with the addition of small cement fractions can be crucial. The content to be adopted should be such as to bring a more economically viable alternative to the geotechnical design of a shallow foundation, for example. The present research aims to propose a modified Mohr-Coulomb envelope for artificially cemented soils. The proposal is based on the development of two functions that relate shear strength parameters (cohesion and friction angle) with the cement content. In order to assess the proposed functions, triaxial shear tests were performed on the sand with different cement contents. The adjustments presented consistent results. In addition, the proposed envelope was validated using published results encountered in literature.

Keywords: shear strength, envelope, artificially cemented sand, laboratory tests.

1. Introduction

The formation of residual soils occurs due to the rock matrix weathering, whereby the bonds between minerals are broken, forming new minerals. In transported soils, the formation involves erosion, transport, deposition and densification processes under the action of their own weight. In the case of residual soils, the transformation of rock into soil has no individualized separation of particles as occurs in transported soils. However, residual soils present a weakening of their bonds, whereas in transported soils, the tendency is opposite, that is, post-depositional variations may occur, which induces the formation of bonds between the particles. Thus, regardless of the origin of soils, a remarkable feature in their structural skeleton and thereforein their mechanical behavior, is the bonds between the particles, which can form naturally cemented soils.

Cementation occurs due to the various geological mechanisms that create bonds between soil particles, such as aging, chemical reactions, carbonates, silicates, iron oxides, and natural cementing agents. Due to the difficulties of *in situ* sampling, the mechanical characteristics of cemented soils are consistently studied using artificial samples prepared in the laboratory and cured by different cementing agents (Amini and Hamidi, 2014).

The mechanical behavior of cemented soils has been studied by many researchers (Clough *et al.*, 1981; Leroueil and Vaughan, 1990; Airey, 1993; Coop and Atkinson, 1993; Cuccovillo and Coop, 1997; Cuccovillo and Coop, 1999; Huang and Airey, 1998; Consoli *et al.*, 2000; Consoli *et al.*, 2009; Consoli *et al.*, 2010; Consoli *et al.*, 2011, Consoli *et al.*, 2012; Hamidi and Hooresfand, 2013; Shahnazari and Rezvani, 2013; Amini and Hamidi, 2014; Cook *et al.*, 2015). The mechanical behavior of cemented soils is influenced by diverse parameters, such as cement content, cement type, density, confining stress, grain size, and stress-strain history. However, even though the influence of the amount of cement on the shear strength of artificially cemented soils is well established, there is no failure criterion that considers them explicitly (Consoli *et al.*, 2012). With regard to cemented soils, the literature infers that the increase in cement content considerably increases characteristics, such as stiffness and shear strength of the soil.

The present research aims to analyze the soil behavior of artificially cemented soils for different cement contents and propose a modified Mohr-Coulomb envelope for artificially cemented soils. The proposed linear envelope is based on the development of two functions that relate shear strength parameters (cohesion and friction angle) with the cement content. In order to assess the proposed functions, triaxial shear tests were performed on sand with various cement contents.

2. Material and methods

An experimental programme has been carried out in order to evaluate the mechanical behavior of an artificially cemented soil with different cement contents. Initially, the geotechnical properties of the studied soil were characterized. Then a number of compaction tests were carried out. Finally, triaxial compression tests were also performed in specimens under a relatively small range of effective confining pressures (50, 100 and 200 kPa) and different cement contents.

The sand soil used in the testing was obtained from the city of Fortaleza, in the Northeast of Brazil. The soil specific gravity (Gs) of the analyzed soil is 2.63, and from the particle size analysis, it is observed that it has approximately 8% fines, 64% fine sand, 28% medium sand, and uniformity and curvature coefficients are 2.5 and 1.1 respectively, as shown in Figure 1. Since this is a non-plastic soil, it can be classified as a poorly graded sand with silt (SP-SM) according to the Unified Soil Classification System.

Portland cement (Type II) was used as the cementing agent. A total of 28 days was adopted as the curing time. The specific gravity of the cement grains is 3.02. Tap water was used for the characterization tests and for molding specimens for the compaction and triaxial compression tests. The chemical composition of the Portland cement employed in this research was provided by the manufacturer, and is shown in Table 1.

Table 1 - Portland cement chemical composition.

Components	% In mass
SiO_2	22.82
Fe_2O_3	2.43
Al_2O_3	6.63
CaO	55.59
MgO	3.77
SO_3	1.87

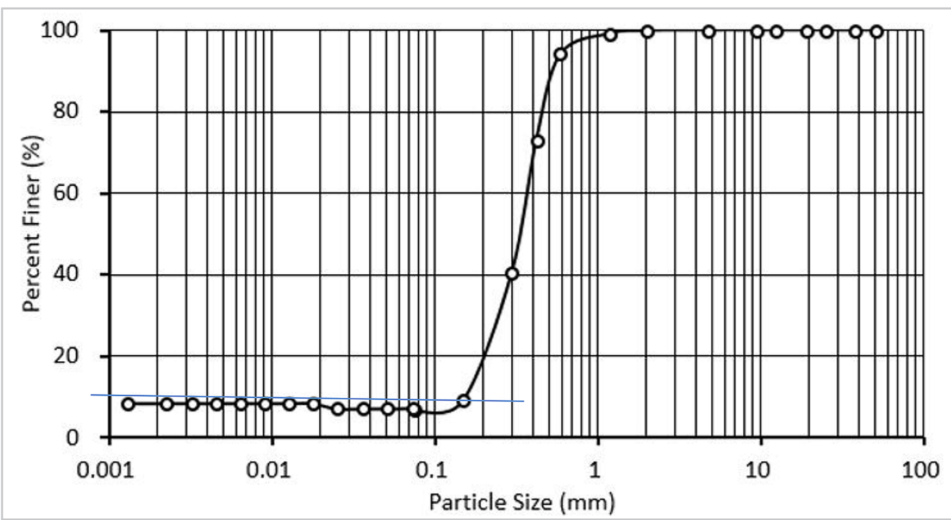

Figure 1 - Gradation curve of the studied soil.

Cylindrical specimens 50 mm in diameter and 100 mm high with different cement contents were used in the triaxial tests. For the preparation of specimens, the required quantities of soil, water and cement were determined in relation to the dry mass of soil. After weighing all materials, water was then added to the dry components and the resulting mass was placed in a plastic bag to prevent moisture loss. In the case of cemented soil mixtures, before the water addition, the soil and cement were mixed until complete homogenization. The specimen was then statically compacted in one layer inside a cylindrical split mould, which was lubricated, so the specific layer reached the specified dry unit weight, which varies from 16.7 to 17.1 kN/m^3, according to the cement content used. After the moulding process, the specimen was immediately extracted from the split mould and its weight, diameter and height measured with accuracies of about 0.01 g and 0.1 mm, respectively. The samples were considered appropriate for testing if they met the following tolerances: dry unit weight (γ_d): degree of compaction between 99% and 101% (the degree of compaction being defined as the value obtained in the moulding process divided by the target value of (γ_d). Dimensions: diameter to within ±0.5 mm and height ±1 mm. Figure 2 shows the specimens after the moulding process.

Figure 2 - Triaxial test specimens with 10% cement content.

Standard Proctor tests (ABNT NBR 7182, 2016) were performed on specimens with different cement contents. The water content used in triaxial compression tests was the optimum moisture content value obtained from the compaction tests with different cement contents.

Undrained triaxial compression tests were carried out under controlled deformation at a displacement rate of 0.1667 mm/min. Full drainage during shearing was verified by measuring the excess pore pressure at the end of the specimen opposite to the drainage. The execution of the triaxial tests followed the general procedures described by the British Standard methods of the test (British Standards Institution, 1990). A computer-controlled triaxial cell was used to test the samples at the confining pressures (CPs) of 50, 100, and 200 kPa. The outer surface of samples was soft enough to minimize the effect of membrane penetration. As a result, flexible membranes do not affect pore pressure generation in a saturated condition. Membranes with average thickness of 1 mm were used during the execution of the tests. All specimens were fully saturated in two steps prior to shearing. At the first step, de-aired water was flushed from the bottom of the sample under a very low pressure difference of 10 kPa for a period of 24 hours. After that, both cell and back pressure were increased simultaneously to 310 kPa and 300 kPa for complete saturation at the second step. The saturation procedure was considered to be completed when Skempton's B value (Skempton, 1954) of 0.9 was reached. The samples were consolidated up to the desired confining pressures. After that, the axial load was applied under undrained conditions until failure.

3. Results and discussions

A Standard Proctor test was performed on the uncemented sample. An optimum moisture content (w_o) of approximately 10% and a maximum dry bulk density (ρ_{dmax}) of 16.8 kN/m³ was obtained. In order to verify the influence of the cement content on the compaction curve, three Standard Proctor tests were carried out using cement contents of 2, 5 and 10%. The results show that the increase in cement content presented negligible changes of optimum moisture content and maximum dry unit weight, as shown in Figure 3. The specific gravity, optimum moisture content and maximum dry unit weight are summarized in Table 2. A summary of triaxial test results at failure is shown in Table 3. The triaxial test were carried out on specimens with no addition of cement and cement contents (Ci) of 2, 5 and 10%.

Table 2 - Compaction and characterization results.

Parameters	*Ci* (%)			
	0	2	5	10
w_o (%)	10	10.1	10.4	10.5
γ_{dmax} (kN/m^3)	16.7	16.9	16.9	17.1
G_s	2.63	2.64	2.65	2.67

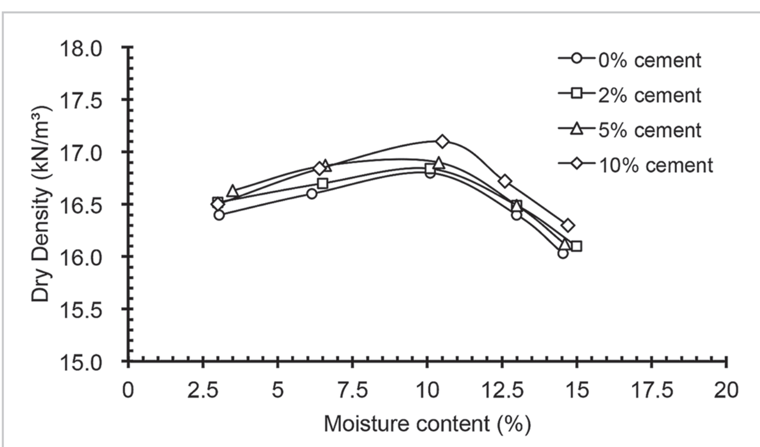

Figure 3 - Compaction curves of samples with different cement contents.

Table 3 - Summary of triaxial test results at failure.

Ci (%)	Confining pressure (kPa)	Axial strain (%)	Deviatoric stress (kPa)	ϕ'	*c*'
0	50	0.5	161	31.3	1
	100	0.5	283		
	200	0.5	506		
2	50	1.2	699	34.1	86
	100	1.1	1292		
	200	1.0	1978		
5	50	0.9	3462	37.4	203
	100	0.9	4631		
	200	0.8	5543		
10	50	0.8	6876	48.12	426
	100	0.8	9289		
	200	0.8	12105		

Deviatoric stress-axial strain curves is depicted in Figure 4 for the consolidated undrained conditions. It can be observed that the studied material presented a typical behavior of cemented soils, with increased peak resistance and initial stiffness as a function of the increased cement content and applied confining stress. The uncemented sample showed a seemingly linear initial stress-strain behavior followed by a well-defined plastification point from which the axial stresses did not change significantly. The plastification point is verified in an axial strain of about 0.5% for the uncemented sample.

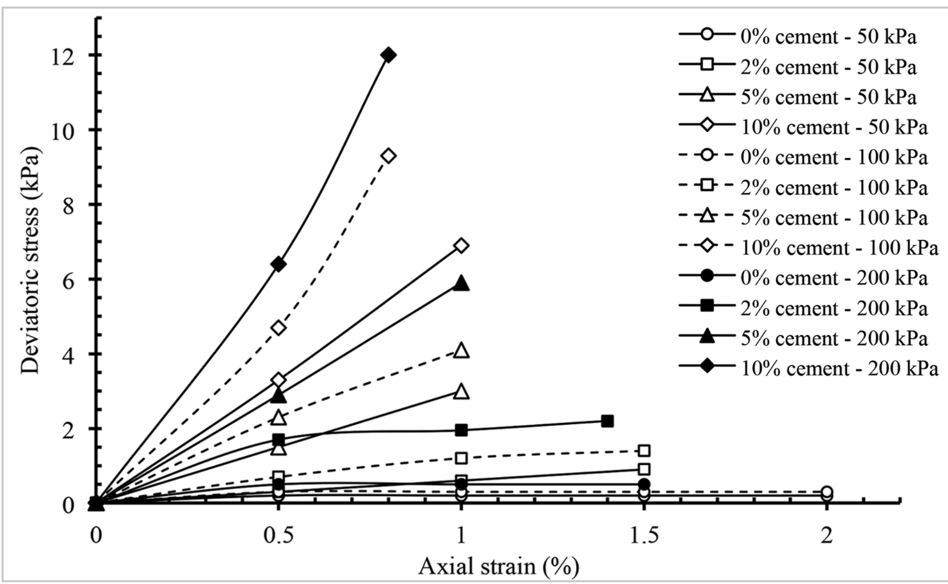

Figure 4 - Deviatoric stress-axial strain curves of samples with different cement contents.

The cemented samples with cement contents of 2, 5 and 10% presented an abrupt failure after the linear part of the deviatoric stress-axial strain curve. The maximum axial strain for these samples decreased due to the increase of cement content, varying between 0.8 and 1.2%. The cemented sample with a 2% cement content presented a plastification point at 1% strain before failure.

An axial strain of about 0.5% was adopted as the failure criterion for the uncemented samples. Cemented samples with cement content of 2% presented a nonlinear behavior up to an axial strain value of 1%, in which strain is increasing without shear strength mobilization. Therefore, an axial strain of about 1% was adopted as failure criterion for these samples. The cemented samples with cement content of 5 and 10% showed an apparent peak point associated with the failure. Thus, the peak stress was adopted as the failure criterion for these samples. For the cemented samples with 5 and 10% cement, the point of maximum stress deviation was adopted as the breaking point.

Figure 5 presents the classic failure modes of the uncemented and cemented samples. Although dilation arose at different confining pressures, the uncemented sample in an undrained test showed a barreling mode without shear plane formation (Figure 5a). While in the cemented sample with a cement content of 2%, the failure mode was a combination of barreling shape and shear plane, where barreling was the predominant mode (Figure 5b). An increase in cement content expanded the thickness of the shear band. Cemented samples with more cement content (5 and 10%) experimented a mode of brittle failure and underwent significant dilation with an apparent peak point in the stress-strain curve (Figure 5c and 5d). This behavior for both uncemented and cemented samples was also verified by Amini and Hamid (2014).

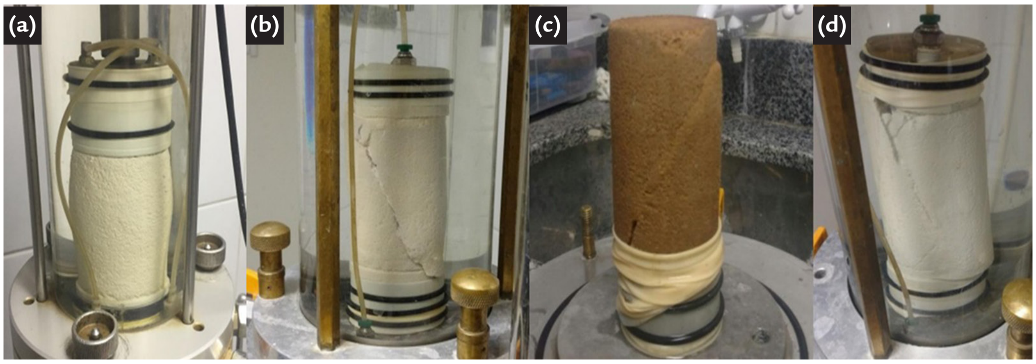

Figure 5 - Failure mode of tested samples. (a) uncemented sample; (b) 2% cement content sample; (c) 5% cement content sample; (d) 10% cement content sample.

Figure 6 indicates the variation of the excess pore pressure with the mean effective stress in the consolidated undrained tests. Positive pore pressure occurred at the beginning of loading, followed by negative pore pressure at the final state. Same as the volume change, increase in cement content and decrease in confining pressure increased negative suction at the end of loading process. It can be observed that increase in cement content altered the volumetric behavior of the material, indicating a dilatation tendency during shear, verified by the excess pore pressure reduction, presenting a typical behavior of very dense materials.

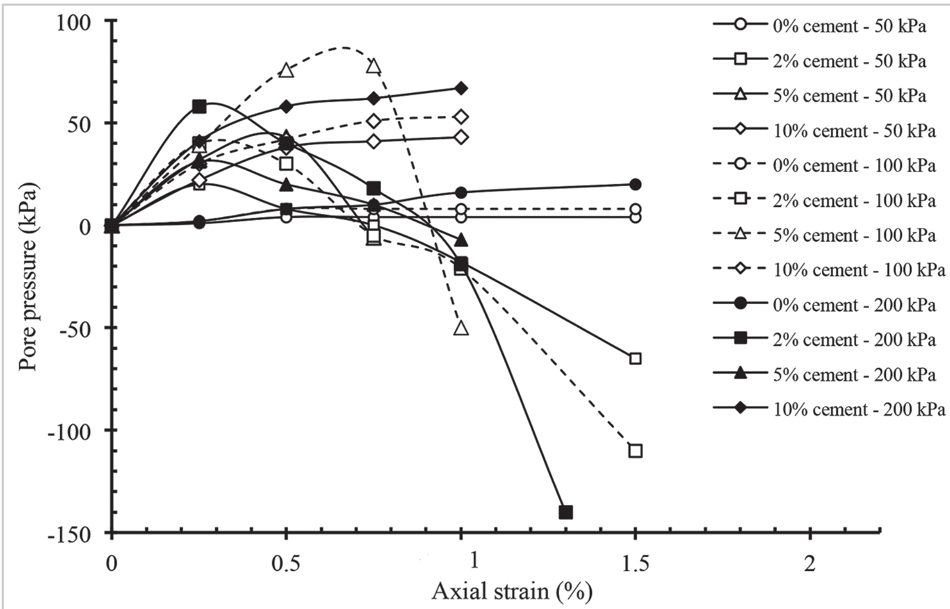

Figure 6 - Pore pressure curves of samples with different cement contents.

3.1 Proposal of a failure envelope for artificially cemented sand

The failure envelope of artificially cemented soils exhibits a nonlinear behavior. The curvature increases with the increase of cement content and the slope decreases with the increase of confining stress (e.g Lade and Overton, 1989; Leroueil and Vaughan, 1990; Cuccovillo and Coop, 1999, Marri, 2010). However, most current engineering applications, which use artificial cement improvement, are restricted to low cement contents. Although, this behavior varies with the confining stress, most of the civil works occur between small and medium depths, therefore, presenting a small variation of confining stresses. Thus, for these practical situations, the use of linear behavior allows a better application in geotechnical projects. Artificial cementation improves the compressibility and shear strength of soils. In unsaturated soils, suction has a similar effect, although this effect is not permanent. This an important aspect because it will be used to define a formulation for shear strength of artificially cemented sands in this article. Based on this observation, the present study proposes the use of the modified Mohr-Coulomb failure criterion by Fredlund *et al.* (1978), with some adjustments, to estimate the shear strength of artificially cemented sands.

In the unsaturated soil model, the shear strength parameter determination is made as a function of suction, whereas in the methodology proposed in this research, the determination will be made as a function of the cement content. The design of geotechnical works is strongly influenced by the strength and stability conditions of the soil massif. For this reason, the soil strength criterion has always been a focus of interest in geotechnical engineering, that is, the failure criteria are formulations that seek to simulate the stress conditions in which material failure occurs. Failure occurs when the shear stress acting on the shear plane reaches the material's maximum shear strength. In Mohr's criterion, the envelope is commonly curved, although it can be adjusted by a line in the range of normal stresses of interest, that is, by adopting the Mohr-Coulomb criterion, which is described by the following equation.

$$\tau = c' + \sigma' . \tan\varphi' \qquad (1)$$

Using the Mohr-Coulomb failure criterion, Fredlund *et al.* (1978) incorporated a suction effect. The shear strength is then described by the following equation.

$$\tau = c' + (\sigma' - u_a) . \tan\varphi' + (u_a - u_w) . \tan\varphi^b \qquad (2)$$

The Mohr-Coulomb criterion modified by Fredlund *et al.* (1978) assumes that the soil massif is isotropic and can be represented by an equivalent continuous medium, where the cohesion portion is influenced by the saturation degree. In case of artificially cemented sands, isotropy can also be assumed and the soil's shear strength increases as a cement content function. However, unlike that which occurs in unsaturated soils, where moistening increases the saturation degree, it causes a decrease of soil shear strength. While in cemented soils, it can be idealized that an increase in cement content generates not only a cohesion increase but also an increase of the friction angle. Thus, for the use of the proposed envelope, both cohesion and friction angle will be calculated as a function of the cementation degree to which the soil is subjected. Shear strength parameters were obtained from triaxial compression tests on specimens with different cement contents in order to develop the proposed failure envelope (Table 3). Thus, equation correlations were used to obtain shear strength parameters adapted to the modified Fredlund *et al.* (1978) model for artificially cemented sands, which can be written as:

$$\tau = C_t + \sigma . \tan\varphi_{Ci} \qquad (3)$$

where τ is the shear stress, C_t is the cement-soil mixture cohesion, σ is the normal stress, and Ci is the cement-soil mixture friction angle.

However, it is important to demonstrate that both the resulting friction portion and the existing cementation content of the studied mixture will be influenced by the initial friction angle (soil without cement addition), as the total cohesion will also be a combination of the soil cohesive intercept in its natural state and an increased cementitious bond will generate a significant increase in the total cohesion intercept. The cohesion and friction angles of the studied artificially cemented sands will be related by functions presented respectively as:

$$\varphi_{Ci} = \varphi_0 . e^{\alpha . Ci} \qquad (4)$$

where φ_{Ci} is the cement-soil mixture friction angle, φ_0 is the soil friction angle, α is the adjustment factor, and Ci is the cement content

$$C_t = c' + Ci . \tan\beta \qquad (5)$$

where is the C_t is the total cohesive intercept, c' is the uncemented soil cohesive intercept, Ci is the cement content, and β is the differential cohesion intercepts slope.

Considering cementation influence on the friction angle and the cohesive intercept, it is possible to correlate Equations 3, 4 and 5 in order to obtain a general failure envelope equation for artificially cemented soil.

$$\tau = c' + \sigma . \tan(\varphi_0 . e^{\alpha . Ci}) + \tan\beta \qquad (6)$$

Figure 7 and 8 present linear and exponencial correlations between the cement content, cohesion and friction angle. Values of the cohesion intercept and friction angle were obtained from triaxial compression tests on cemented samples with cement contents of 2, 5 and 10%. It is worth to mention that both correlations presented an excellent determination coefficient (R^2). Thus, obtaining adequate α and φ_0 values for the calculation of the friction portion and β values for the cohesion portion, as shown by the following equations.

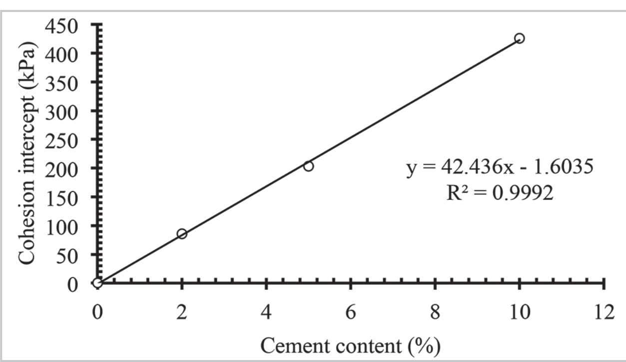

Figure 7 - Cohesion intercept x cement content correlation.

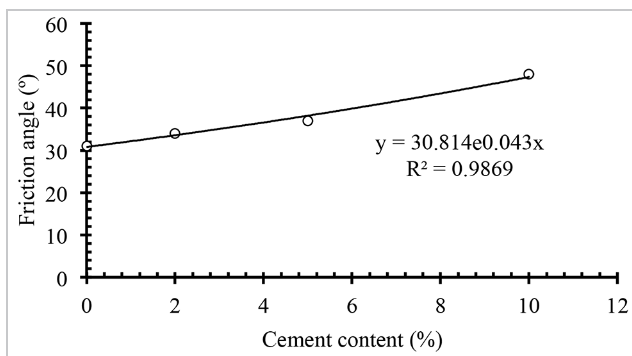

Figure 8 - Friction angle x cement content correlation.

$$c' = 42.436 \times Ci - 1.6035 \qquad (7)$$

$$\varphi_{Ci} = 30.814 \times e^{0.043Ci} \qquad (8)$$

3.2 Proposal validation

The proposed failure envelope was validated using results reported in literature (Rohlfes Jr, 1996; Cruz, 2008; Lopes, 2012). It is important to note that different soil types, cement contents and confining pressures were used by the researchers that were employed in the proposal validation (Table 4).

Table 4 - Literature test parameters.

Reference	Soil type	Cement contents (%)	Confining pressures (kPa)
Rohlfes Jr (1996)	Clayey sand	0; 5; 7	20; 60; 100
Cruz (2008)	Sand	1; 5; 10	20; 200; 400
Lopes (2012)	Sand	2; 5; 10	100; 200; 400

Experimental and estimated shear strength parameters (φ' and *c*') for researches used for validation are shown in Table 5. The proposed methodology application was able to estimate shear strength parameters (φ' and *c*') with a reasonable agreement when comparing the results obtained by Rohlfes Jr (1996), Cruz (2008) and Lopes (2012). It can also be observed that the cohesion portion adjustment overestimated the reference values when analyzing cemented samples with 5% cement content of Rohlfes Jr (1996) and Cruz (2008). Failure envelopes obtained by the researchers and estimated failure envelopes are depicted in Figure 9.

Table 5 - Estimated and experimental shear strength parameters.

Reference	*Ci* (%)	φ' (°)*	φ' (°)	*c*` (kPa)*	*c*` (kPa)
Rolfhes Jr (1996)	0	35	35	7	12
	5	44	44	140	123
	7	49	49	193	205
Cruz (2008)	1	34	36	23	19
	5	38	35	145	152
	10	44	46	296	293
Lopes (2012)	2	28	27	101	103
	5	32	33	193	190
	10	39	38	345	346

* Estimated values

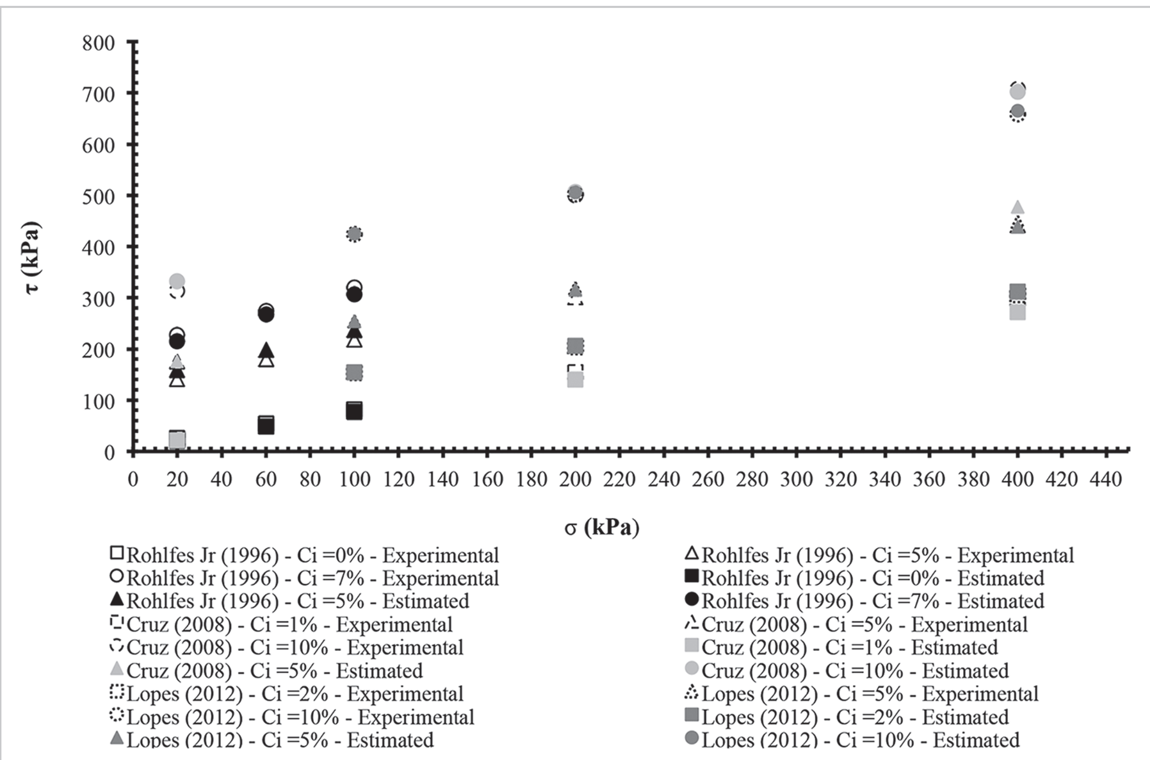

Figure 9 - Experimental and estimated failure envelopes.

The Mohr-coulomb criterion was chosen, based on the same principle used by Fredlund *et al*. (1978), who observed the soil shear strength variation as a function of suction variation. Similarly, the increase in cement content can lead to significant variation in soil strength, as observed in this study. Finally, it is noteworthy that the use of the modified Mohr-Coulomb criterion presented itself as a good methodology to estimate the mechanical behavior of artificially cemented sands for engineering applications. The proposed methodology was applied to triaxial test data on cemented samples reported in literature (Rohlfes Jr, 1996; Cruz, 2008; Lopes, 2012). The proposed methodology presented reasonable estimates for both shear strength parameters and failure envelopes of artificially cemented sands. It is important to note that the present research evaluated low levels of confining stress (values under 1 MPa). However, most applications of artificially cemented sands are within the confining stress range of this research.

3.3 Artificially cemented soil application

Artificially cemented soils can be used for different purposes. Bastos (2017) empirically analyzed the use of this material to improve the bearing capacity of soils by simulating a practical engineering situation, in which the bearing capacity of a square footing with a 2 m side was determined, placed at a depth of 0.5m, in a soil with characteristics of fine sand (without cementation) similar to the soil of the city of Fortaleza-CE, whose data are presented in Table 6.

Table 6 – Shallow foundation simulation data (Bastos, 2017).

B (Footing width)	**D (Embedment depth)**	**γ_n (Soil bulk unit weight)**	**φ' (Soil friction angle)**	**c' (Cohesive intercept)**
2 m	0.5 m	18.7 kN/ m³	30.8°	0 kPa

Equation 10, developed by Terzaghi (1943), corrected for the square footing foundation condition, was used to calculate the bearing capacity.

$$q_{ult} = 1.3\, c\, N_c + \gamma\, D\, N_q + 0.8\, \gamma\, B\, N_\gamma \qquad (10)$$

Where:

q_{ult}, ultimate gross bearing capacity;
c, cohesive intercept;
γ, soil bulk unit weight;
D, embedment depth;
B, footing width;
N_c, bearing capacity factor related to soil cohesion;
N_q, bearing capacity factor related to soil overburden;
N_γ, bearing capacity factor related to soil friction.

To calculate N_c, N_q e N_γ coefficients, equations are also required and are expressed as follows:

$$N_q = \frac{a_\theta^2}{2\cos^2\left(45 + \frac{\varphi}{2}\right)} \qquad (11)$$

$$N_c = cotg\,\varphi\,[N_q - 1] \qquad (12)$$

$$N_y = 2\,(N_q + 1)\,tg\,\phi \qquad (13)$$

$$a_\theta = e^{\left(\frac{3\pi}{4} - \frac{\varphi}{2}\right) tg\,\varphi} \qquad (14)$$

Where:

a_θ, coefficient related to the soil friction angle;
ϕ, soil friction angle.

It is possible to verify that except γ_n, B, D and the shape factors (square footing), all other factors vary their value significantly with the soil shear strength parameters (c and φ), which are not constant and change with the cement increment. Therefore, for the calculation to consider the cement influence, the soil shear strength parameters (friction angle and cohesion) used in Equations 5.2 to 5.5 were the same as those used in the envelope methodology proposed herein. A simulation was performed considering the variations of the friction angle and cohesion as a function of the cement content increment, as presented in this article, following the Mohr-Coulomb methodology modified for cemented soils. The contents varied from 0 % (no cement added) to hypothetically 15 % (with 15 % cement by weight). The obtained results are shown in Table 7.

Table 7 – Numerical application of Terzaghi's method for bearing capacity of square footing (2m x 2m).

% Ci	φ (°)	φ (rad)	α_θ	N_q	N_c	N_γ	c (kPa)	γ (kN/m³)	D (m)	B (m)	q_{ult} (kPa)	F.S.	σ_{adm} (MPa)
0	30.81	0.54	3.47	24.73	39.78	30.69	0.00	18.70	0.50	2.00	1149.43	3.00	0.38
1	32.17	0.56	3.69	29.10	44.68	37.87	42.43	18.70	0.50	2.00	3870.13	3.00	1.29
2	33.58	0.59	3.93	34.64	50.67	47.33	84.87	18.70	0.50	2.00	7329.99	3.00	2.44
3	35.06	0.61	4.21	41.74	58.06	59.98	127.30	18.70	0.50	2.00	11793.90	3.00	3.93
4	36.60	0.64	4.54	51.00	67.33	77.23	169.73	18.70	0.50	2.00	17644.84	3.00	5.88
5	38.21	0.67	4.91	63.29	79.14	101.20	212.17	18.70	0.50	2.00	25448.56	3.00	8.48
6	39.88	0.70	5.36	79.94	94.47	135.28	254.60	18.70	0.50	2.00	36061.38	3.00	12.02
7	41.64	0.73	5.88	103.03	114.77	184.96	297.03	18.70	0.50	2.00	50815.39	3.00	16.94
8	43.47	0.76	6.51	135.91	142.33	259.53	339.47	18.70	0.50	2.00	71848.46	3.00	23.95
9	45.38	0.79	7.29	184.19	180.81	375.27	381.90	18.70	0.50	2.00	102716.38	3.00	34.24
10	47.37	0.83	8.25	257.71	236.31	562.07	424.33	18.70	0.50	2.00	149582.74	3.00	49.86
11	49.45	0.86	9.48	374.48	319.54	877.72	466.77	18.70	0.50	2.00	223659.36	3.00	74.55
12	51.62	0.90	11.09	569.63	450.32	1441.10	509.20	18.70	0.50	2.00	346536.39	3.00	115.51
13	53.89	0.94	13.27	916.47	667.79	2515.54	551.64	18.70	0.50	2.00	562721.47	3.00	187.57
14	56.26	0.98	16.32	1581.55	1055.73	4738.56	594.07	18.70	0.50	2.00	971890.54	3.00	323.96
15	58.73	1.03	20.82	2984.71	1811.91	9833.28	636.50	18.70	0.50	2.00	1821386.99	3.00	607.13

Thus, it can be verified that even in mixtures with small cement content, it is possible to obtain a considerable level of mechanical capacity improvement, not only by increasing the foundation bearing capacity that is directly linked to the chosen foundation element, which can change entirely in a new analysis, but mainly by the increase of soil cohesion and friction properties that are directly influenced by the soil mechanical strength. Finally, an increase in bearing capacity can be observed due to cement content rise (Figure 10).

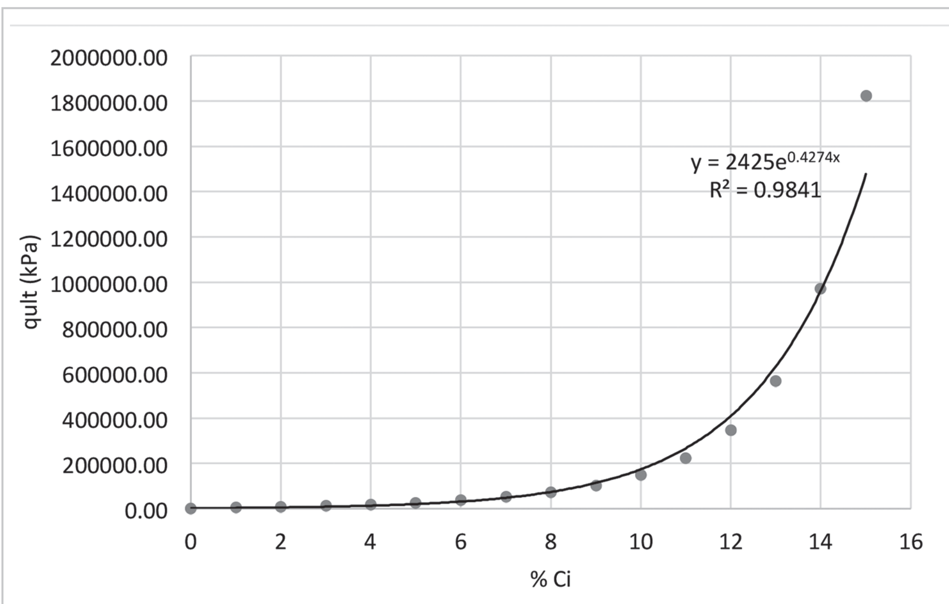

Figure 10 – Foundation bearing capacity (q_{ult}) versus cement content (% Ci).

4. Conclusions

The present study deals with the engineering characteristics of cemented sand mixtures. The following conclusions can be drawn based on the test results:

• The Mohr-Coulomb failure criterion amended with the Fredlund et al. (1978) proposal satisfactorily represents the linear failure envelopes of the studied sand–cement mixtures.

• Brittle behavior was verified in cemented sands, while a barreling failure was observed for uncemented sands.

• Correlations between the cement content, cohesion and friction angle presented an excellent determination coefficient (R^2)

• The proposed methodology presented adequate estimates for both shear strength parameters and failure envelopes of artificially cemented sands.

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