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Analysis of the Borehole Shear Test to obtain soil shear strength parameters in the field: a comparison with Direct Shear

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Abstract

With regard to obtaining soil resistance parameters, the Borehole Shear Test (BST) is a facilitating option, especially in cases of landslide studies, in which the speed in carrying out the tests is a great differential. Since the BST test is performed in situ, not requiring the collection of an undeformed sample, its agility is its greatest advantage, especially in emergency underground investigations. This research aims to compare the results presented by the Borehole Shear Test with the results of the Direct Shear test. In order to ensure equity in the performance of the tests, all the samples used were molded in the laboratory using a predetermined mixture of sand, lime and water. All the tests were performed with specimens of the same dry specific mass, in drained condition and consolidated with the same normal stress. Also, this study presents the characterization of the sand used and the final mixture with respect to its physical indexes and void indexes.

1 INTRODUCTION

The intensive use of land, mainly caused by the continuous population growth, has highlighted the need of faster and more efficient methods to study soil's resistance parameters, as well as its other characteristics. Such agility is necessary, for example, for situations where there is a risk of landslides due to bad weather or human interference.

Therefore, among the methods for determining soil shear strength, the direct shear test is the oldest (Das, 2007). According to Viecili (2003), in order to perform it, normal shear stress must be applied to the sample taken from the field, undeformed, taken to the laboratory for analysis, and the shear stress that generates the rupture is observed. The Borehole Shear Test has the advantage of allowing the generation of results in the field, which allows agility in the determination of resistance parameters (Domingues et al., 2019), eliminating the need to take an undeformed sample and take it to the laboratory. Moreover, according to Domingues et al. (2019), BST allows the analysis of a larger area in a shorter time. This factor makes BST an extremely interesting alternative for the generation of geotechnical maps of risk to landslides, since these maps tend to represent an entire watershed or an entire municipality, for example.

This article aims to analyze the reliability of the shear strength parameters, i.e., cohesion (c) and angle of friction (φ), obtained through the Borehole Shear Test, making a comparison with the results obtained through the Direct Shear Test on the same soil. Both tests were performed in the laboratory with specimens made from the compacted mixture of fine sand, lime and water.

2 TESTS TO OBTAIN THE SOIL SHEAR STRENGTH PARAMETERS

2.1 Direct Shear Test

The Direct Shear test can be broken down into two stages, the first being consolidation. The specimen is consolidated through the action of normal stress (σv), which is kept constant; and the second stage is the shear. With the support of a controlled deformation machine, a displacement to the lower section of a split box is employed, gradually imposing the shear stress (τ), which is extended until shear stress ruptures the specimen along the contact plane between the two sections of the box. According to the type of soil used, its respective speed is designated and according to the type of test (drained consolidated, non-drained consolidated), drainage can be provided in the sample.

It is possible to portray the result of the Direct Shear test in a specimen, through a pair of coordinates in the Cartesian plane, where the axis of the abscissas represents the horizontal displacement (εh) and the axis of the ordinates represents the shear stress (τ). However, by repeating the test from different normal stresses for other specimens, a set of pairs of values is obtained (σ, τ) for each stage, where the axis of the abscissas represents the normal stress (σn) and the axis of the ordinates represents the maximum shear stress (τmax). These pairs of values are plotted on a Cartesian system $\tau = f(\sigma)$, thus obtaining the resistance envelope. With this envelope, the shear strength parameters, c and φ, can be defined by means of the Mohr-Coulomb failure criterion.

2.2 Borehole Shear Test

For the execution of the test, the soil is first drilled with the aid of a one-piece Dutch auger, 75mm. After the translation, the shearing probe is inserted up to the depth at which the test is to be performed. According to the objective of the test, the depth of the hole may vary. At each stage, normal stresses are applied by means of a manometer with a manual pump. With this, the probe expands and transfers the load applied on the plates to the ground walls. In this way, the consolidation time should be awaited, which, according to the instructions of the equipment designers, varies between 5 to 15 minutes, according to the soil granulometry. Although this time is short, the equipment was developed with this consideration, and is thus used in several studies, such as Lutenegger and Timian (1987), which analyze the results of BST in a marine clay. We must also consider that Lutenegger and Hallberg (1981) clarify that the shear occurs directly on the face of the hole. According to the equipment developer, Handy Geotechnical Instruments: "Stage testing causes drainage times to be cumulative. Water squeezes out around edges of the shear plates, so this is a drained test."

Also, it is worth mentioning that in the Direct Shear test there is a decompression of the soil when the undeformed sample is prepared, being necessary to reapply these stresses during the test. This effect is extremely reduced in the case of BST since the soil is kept confined.

The shear strength of the soil is measured by extracting the probe at a controlled speed of 2 crank rotations (clockwise) per second. The shear stress is observed on the shear gauge and its maximum value is recorded manually. The maximum shear stress recorded indicates shear failure. To obtain the soil resistance parameters, c and φ, the Mohr-Coulomb method is used, plotting the results of each stage in one on a shear stress graph (τ) versus normal stress (σ). As with the Direct Shear Test, with the BST it is also necessary to have at least three different confining pressures and their corresponding shear stress at failure to plot the Mohr-Coulomb envelope.

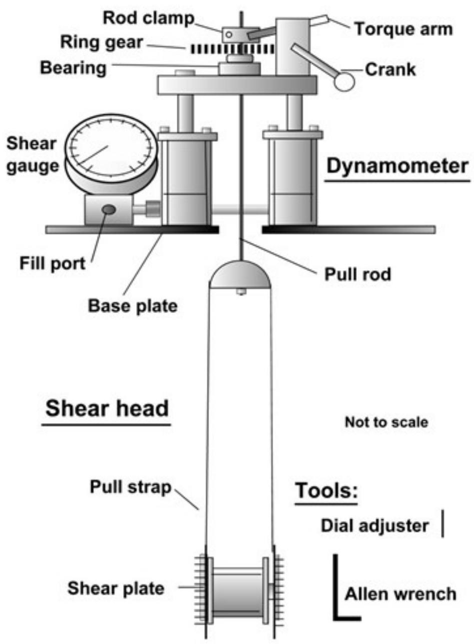

Figure 1. Illustration of the BST components (Handy Geotechnical Instruments, Inc.).

Although there is no maximum theoretical depth at which the test can be performed, there is a difficulty in reaching great depths; a consequent fact of the number of threadable rods that should be employed, besides the maximum depth that the Dutch auger is capable of digging. According to the equipment manual provided by Handy Geotechnical Instruments, if the soil in question used in the test is composed of more than 10% of gravel, there is a difficulty in adequately excavating the hole for the introduction of the shear sensor, besides that the boulder particles in contact with the shear plates end up interfering negatively on the quality of the test results.

2.3 Studies comparing the Direct Shear Test and Borehole Shear Test

The BST, acquired by the Geotechnical Mapping Laboratory (LAMGEO) of the Federal University of Santa Catarina (UFSC), is the only existing Brazilian equipment. Due to this fact and its recent use, there are few studies comparing the data obtained in the Direct Shear and BST tests in order to validate its use, especially in places that require a high number of tests.

In this perspective, Contessi (2016) had in his work the purpose of analyzing the relationship between the results of the two types of tests in laboratory-compacted specimens. The author performed the failure in a state of natural and flooded humidity, with normal stress ranges very close to those applied for each envelope and under the same drainage conditions. According to Contessi (2016), the study of the results expressed very similar values for the resistance parameters, with parallel envelopes and close cohesive intercept values. Therefore, the author concluded that there is no mistrust in the validity of BST results, since they expressed themselves very close and consistent with each other, with possible variations resulting from the procedures of both tests that could affect the soil structure.

Sakamoto et al. (2016) performed tests on the soils of the geotechnical units Podzólico Vermelho Amarelo of granite substrate and Gleissolo of alluvial quaternary sediments of the Itacorubi watershed located in Florianópolis. According to the authors, the field tests in flooded state presented similar values for the friction angle, however, there were significant differences for the natural state and for the cohesion values.

Sbroglia et al. (2018), in turn, analyzing the data obtained from the Direct Shear and Borehole Shear Test in samples of residual soils from the states of Rio de Janeiro and Santa Catarina, came to the conclusion that the failure envelopes, under the same drainage conditions and normal stresses, obtained satisfactory correlations and, as a result, achieved quite similar resistance parameters consistent with the values presented in the literature for residual soils. The authors found more significant variations in the cohesion parameter.

Table 1 presents the results obtained by Contessi (2016), Sakamoto et al. (2016) and Sbroglia et al. (2018) for the shear strength parameters of the soils by the Direct Shear and BST tests performed under natural and flooded drainage conditions.

Table 1. Variation (V) between cohesion and friction angle results for the Direct Shear Test (DST) and Borehole Shear Test (BST) under natural (N) and inundated (I) conditions.

Soil Types		Cohesion [kPa]			Friction Angle [°]		
		DST	BST	V	DST	BST	V
I	Granite Residual Soil[1]	9.4	13.1	3.7	35.7	46.5	10.8
N	Granite Residual Soil[1]	22.8	14.5	8.3	43.5	45.1	1.6
N	Granite Residual Soil[2]	7.4	21.0	13.6	38.0	43.1	5.1
I	Granite Residual Soil[2]	3.8	4.0	0.2	34.4	32.6	1.8
N	Granite Residual Soil[2]	18.4	9.0	9.4	42.1	26.6	15.5
I	Granite Residual Soil[2]	5.4	1.0	4.4	35.8	35.6	0.2
N	Granite Residual Soil[2]	51.3	13.0	38.3	54.2	42.6	11.6
I	Granite Residual Soil[2]	18.2	6.0	12.6	28.8	35.0	6.2
N	Sedimentary Soil[2]	28.3	9.0	19.3	27.0	36.3	9.3
I	Sedimentary Soil[2]	5.1	0.0	5.1	29.4	35.6	6.2
I	Granite Residual Soil[3]	1.0	3.6	2.6	41.5	39.3	2.2
I	Gneiss Residual Soil[3]	8.3	12.0	3.7	30.2	31.0	0.8
I	Migmatite Residual Soil[3]	15.4	11.0	4.4	29.8	32.0	2.2
I	Granite Residual Soil[3]	13.8	7.0	6.8	36.4	33.3	3.1
I	Granulite Residual Soil[3]	14.7	16.6	1.9	30.8	34.7	3.9
I	Riolite Residual Soil[3]	14.6	26.3	11.7	22.7	23.6	0.9

Source: 1Contessi, 2016; 2Sakamoto et al., 2016; 3Sbroglia et al., 2018.

3 MATERIALS AND METHOD

This experimental work was carried out to compare the resulting strength parameters of BST and DST.

To perform the proposed comparison, sand was purchased in a store specialized in construction in several bags. Therefore, the sand was mixed and completely homogenized to create a single batch. After that, it was sifted in a thick sieve to remove possible impurities and put back to the original bags.

3.1 Test Bodies

The samples were prepared in the laboratory using fine sand, lime, and water, with proportion 10:1.5:1.5 in weight, respectively, previously reported by Domingues et al. (2019). The components were mixed until a homogeneous mixture was obtained.

With the material already prepared, it was placed into a PVC pipe, with 200 millimeters of external diameter and 192 millimeters of internal diameter and 600 millimeters of high, and compacted. The compacting process was carried out according to the following steps: the pipe was filled with 2488g of the mixture, compacted dynamically until this amount of mixture achieved 5 centimeters of high. After the compaction of the layer, the process was repeat until completing 5 layers of compacted mixture.

3.2 Particle Analysis Test

Particle size analysis is used to divide soil particles into groups by their dimensions (soil fractions) and determine their relative proportions to the total weight of the sample, since this property influences the interaction between soil particles.

The result of the particle size analysis, which is used in this study, (Table 2) was obtained by the previous study (Domingues et al., 2019), according to NBR 7181 (Soil - Particle Size Analysis). Briefly, the test consists of sieving a sample of the sand directly by the method of fine sieving (aperture sieves 2.0-1.2 - 0.6 - 0.42 - 0.30 - 0.15 and 0.074 mm in diameter) and all the material retained in each was weighed. The weight retained in each one was plotted and the granulometric curve of the sand was obtained (Figure 2).

Table 2. Results of characterization tests of the sand (Domingues et al., 2019).

Granulometry [%]	
Clay and Silt	0.13
Fine sand	94.05
Medium sand	4.48
Thick sand	1.35
Gravel	0.00

From the previous study (Domingues et al., 2019), the soil was characterized as poorly graded, predominantly composed of sand (99.9%), and in its largest composition, fine sand (94%). The Unified Soil Classification System (SUCS) was used, which resulted in a poorly graded sand with little or no fine, SP acronym.

Also, to better characterize the sand, the determination of the minimum and maximum voids indexes according to MB-3388 and MB-3324, respectively, was performed. The specific mass of sand was obtained from a previous study by Domingues et al. (2019), is 2.65 g/cm³.

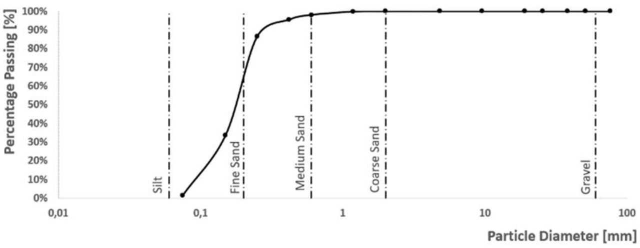

Figure 2. Granulometric curve of the sand (Domingues et al., 2019).

With the results provided by the results of characterization tests of the sand (Domingues et al., 2019), others relevant descriptors of the granulometric size distribution were obtained (Das, 2007): the particle size distribution, which is the value of the particle diameter at 50% in the cumulative distribution, (D50), the coefficient of uniformity (Cu) and coefficient of curvature (Cc). The values 1.667 and 0.6 were obtained for Cu and Cc, respectively. D50 is 0.15 millimeters.

3.3 Calculation of Physical Indices

Physical indices quantify the proportions of the three phases of the soil, between weights, volumes, and are used to assess the conditions under which the soil is at the time of their determination.

For the determination of all Physical Indices the already obtained data is used: the total weight of sand (W_t), the total volume of sand (V_t), moisture content (w) and the actual density of particles (δ). In this analysis, it was used the sand after being left for 24 hours in the drying oven, in this case the moisture content for the calculation of the physical indices is zero.

Using the relations between the weights and volumes presented below:

$$W_t = W_s + W_w \tag{1}$$

$$V_t = V_s + V_w + V_{ar} \tag{2}$$

Where, W_s is the weight of solids; W_w is the weight of water; Vs is the volume of solids; V_w is the volume of water and Var air volume. From the mentioned relations, one can calculate the physical indices presented in this article.

3.4 Specific sample weights

According to Das (2007), the natural unit weight (γ) of a sample is 'the weight of the soil per unit volume', i.e. the way the soil naturally presents itself. Also, according to the author, "in many situations, to solve problems of earthworks, it is necessary to know the weight per unit volume of soil, excluding all water", this is the apparent dry specific weight (γ_d).

In this study, the specimens were prepared as specified by Domingues et al. (2019), presenting an average natural specific weight (γ) of the samples of 16.80 kN/m³ and a moisture content of approximately 12.12%. The mean values of the dry apparent specific weight (γ_d), in turn, obtained from the relationship between the weight of solids and total volume, was 14.94 kN/m³.

3.5 Blend void index

Thus, the sample void index was obtained from Domingues et al. (2019) through the relationship between the volume of voids and the volume of solids in the specimens, resulting in a value of 1.001. According to Bowles (1979), this value is expected for a cohesive soil, since it is in the range between 0.7 and 1.1. Considering that the pozzolanic reaction of lime confers cohesion to the mixture, this behavior is in accordance with what is expected for the sample.

3.6 Direct Shear Test

The Direct Shear test was performed according to the American Society for Testing and Materials (ASTM) D3080/1998, at the UFSC Soil Mechanics Laboratory.

The specimens were placed in the Direct Shear press and brought to failure in the partially saturated condition. The moment of stabilization of the pore pressure was first admitted as the moment when there were no more variations in the vertical deformation, waiting at least 1 hour for the shear execution. From then on, by applying the normal stresses of 20, 40, 60 and 80 kPa, we determined the consolidation of the specimen.

After consolidation, the shear stage occurred. With the movement of the lower case of the Direct Shear press at a standard speed of 0.307 mm/min, shearing was performed.

3.7 Borehole Shear Test

The Borehole Shear Test was performed in accordance with the Handy Geotechnical Instruments procedure (2017) and performed at the UFSC Soil Mechanics Laboratory. The used soil samples were obtained as previously described in this work.

At first, the soil was drilled through the 75 millimeters diameter Dutch auger. The ratio between the PVC Pipe diameter and the Dutch auger is 0.390. For the positioning of the probe, the depth of the hole was approximately 400 millimeters, being 0.666 of the total depth of the PVC Pipe. In this experiment, the border effects caused by the PVC pipe were not considered.

After assembling the equipment, with the aid of a manometer with a manual pump, the normal tensions of 20, 40, 60 and 80 kPa were applied. Previously, the test specimen consolidation time of 15 minutes was considered for the application of the first normal stress and 5 minutes for the subsequent normal stresses.

The shear strength of the soil was measured by pulling the probe at a controlled speed of 2 crank's rotations per second, which represented a linear speed of 0.360 mm/min.

4 RESULTS AND DISCUSSIONS

4.1 Void ratio

To obtain values for the minimum and maximum void ratios, the specific masses of three specimens were calculated from the mass and volume values presented in Table 3 and Table 4 below.

Table 3. Minimum void ratio.

Test Bodies	Mould + Soil mass [g]	Soil mass [g]	Dry specific mass [g/cm³]
1	6165.6	1553.6	1.5536
2	6152.9	1540.9	1.5409
3	6144.4	1532.4	1.5324
	Max. dry specific mass [g/cm³]		1.542
	emin		0.718

Table 4. Maximum void ratio.

Test Bodies	Mould + Soil mass [g]	Soil mass [g]	Dry specific mass [g/cm³]
1	6065.9	1453.9	1.4539
2	6068.9	1456.9	1.4569
3	6062.9	1450.0	1.4500
	Min. dry specific mass [g/cm³]		1.454
	emax		0.823

As explained in Table 3 and Table 4, the fifth column shows the result of the minimum and maximum voids ratios, which are 0.718 and 0.823, respectively. In addition, the fourth column shows the values of the apparent dry specific mass of the specimens and the maximums and minimums, of 1.542 and 1.454, respectively.

According to Bowles (1979), the void index of sandy soil varies from 0.5 to 0.8, so the values are within the expected. Still, the high value of the minimum index can be related to the granulometry of the sand used, which did not have any fine material in its composition. This fine material could have better filled the spaces between the grains of sand.

4.2 Physical indices

The physical indexes were calculated for the mixture of sand, lime and water. Also, the physical indexes were calculated for the sand used, in the dry state, in the maximum and minimum void indexes. The results are p presented in the tables 5-7.

Table 5. Physical indexes of the mixture (sand, lime and water).

Physical indices		
Humidity Content (h)	12.12	%
Void Index (e)	1.001	
Aeration Grade (A)	64.51	%
Porosity (n)	50.03	%
Saturation Degree (Sr)	35.49	%
Apparent Natural Specific Weight (γnat)	16.80	kN/m³
Apparent Specific Dry Weight (γd)	14.94	kN/m³
Saturated Specific Weight (γsat)	19.85	kN/m³
Real Specific Weight of Solids (γs)	29.89	kN/m³
Specific Submerged Weight (γsub)	10.04	kN/m³
Real Solids Density (Gs)	3.05	

Table 6. Physical indexes of the sand with emax.

Physical indices		
Humidity Content (h)	0.00	%
Void Index (e)	0.823	
Aeration Grade (A)	100.00	%
Porosity (n)	45.15	%
Apparent Specific Dry Weight (γd)	14.26	kN/m³
Real Specific Weight of Solids (γs)	26.00	kN/m³
Real Solids Density (Gs)	2.65	

Table 7. Physical indexes of the sand with emin.

Physical indices		
Humidity Content (h)	0.00	%
Void Index (e)	0.718	
Aeration Grade (A)	100.00	%
Porosity (n)	41.80	%
Apparent Specific Dry Weight (γd)	15.13	kN/m³
Real Specific Weight of Solids (γs)	26.00	kN/m³
Real Solids Density (Gs)	2.65	

It can be highlighted that the addition of lime, as well as its pozzolanic action, has an effect on the physical indices of the sand. Both with respect to sand in its loosest state and in its most compacted state, the mixture with lime has increased the void ratio, which suggests the formation of a microstructure in the sample, resulting from the hydration of lime. Still, we can notice that the mixture presented a dry specific weight with intermediate value, being in the interval between the values of maximum and minimum dry specific weight presented by the sand. This suggests that the sample has an average compaction state.

4.3 Comparison of BST Results x Direct Shear

Four Direct Shear and three BST tests were performed. Analyzing graphically all the results, a trend line was generated for each of the tests.

The tests with the highest linear regression coefficient (R^2) were chosen for comparison, as illustrated in Figure 3.

Through the failure envelopes presented above, the soil resistance parameters were determined by the two tests, presented in Table 8.

Table 8. Soil resistance parameters.

	Direct Shear	BST
Cohesion [kPa]	8.57	9.50
Friction Angle [°]	37.38	31.38

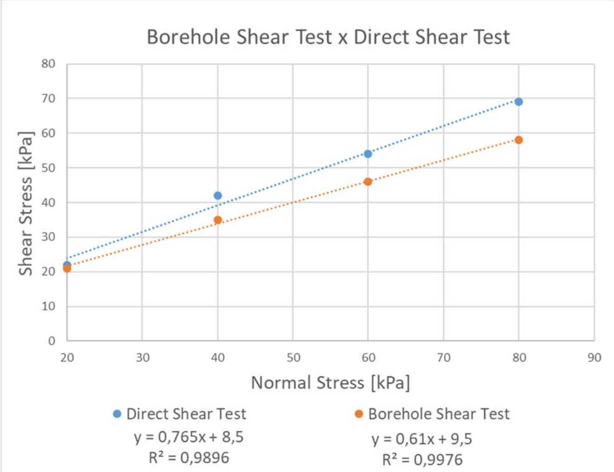

Figure 3. BST x Direct Shear Test.

We can analyze that there is a variation around 11% in the value of cohesion and 16% in the value of the friction angle, but still presenting a great similarity, especially in the value of cohesion (difference of only 0.93 kPa). According to Domingues et al. (2019), it can be observed that possible variations in the results resulting from the test procedures are acceptable, especially in the BST test, where the rupture speed is obtained manually.

5 CONCLUSION

The Borehole Shear Test presents itself as an alternative to the traditional Direct Shear Test, especially for situations where there is a need for various tests, such as in geotechnical surveys. Furthermore, BST is suitable for cases where speed is essential, such as in emergency situations of possible landslides. This study aimed to evaluate the similarity between the resistance parameters obtained through the Direct Shear and the Borehole Shear Test, concluding that the values founded are similar under controlled conditions. Especially regarding the cohesion value, the results appear to be reliable. However, it is necessary to proceed with the research to assess the need to establish a mathematical correlation between the two tests.

Furthermore, this study analyzed the characteristics of the material used for the preparation of specimens in the laboratory and its behavior, concluding that the sand used presents a variation between the maximum and minimum void index and that the hydration reaction of lime provides cohesion to the mixture.

Finally, it is recommended that other comparative studies be carried out between BST results with Direct Shear and Triaxial Compression tests, especially for clayey soils. It is also recommended that the border effect should be evaluated in the studied cases, which could be done by finite elements methods.

6 REFERENCES

Bowles, J.E., (1979). “Physical and Geotechnical Properties of Soils.” International Student Edition, McGraw-Hill, Tokio, Japan.

Brazilian Association of Technical Standards, (1984). NBR 7181: Soil - Particle Size Analysis.

Brazilian Association of Technical Standards, (1991). MB-3388: Determination of the Minimum index void ratio of cohesionless soils.

Brazilian Association of Technical Standards, (1990). MB-3324: Determination of the Maximum index void ratio of cohesionless soils.

Contessi, R.J., (2016). “Borehole Shear Test: A comparison with the Direct Shear Test.” Graduation work, Federal University of Santa Catarina, Florianópolis, Brazil.

Das, B.M., (2007). “Fundamentals of Geotechnical Engineering”. 6th edition, Ed. Thomson Learning, São Paulo, Brazil.

Domingues, R.T., Sbroglia, R. M., Rigotti, J.A.N., Huller, J., Christ, C. E., Higashi, R.A.R., (2019). “Comparison between the Direct Shear Test and the Borehole Shear Test in obtaining soil shear strength parameters”, XII Geosul, Joinville, Brazil.

Lutenegger, A. J., Hallberg, G. R., (1981). “Borehole Shear Test in Geotechnical Investigations.” U.S.A.: Laboratory Shear Strength of Soil. ASTM STP 740. R. N. Yong and F. C. Townsend, Eds., American Society for Testing and Materials.

Lutenegger, A.J., Timian, D.A., (1987). “Reproducibility of Borehole Shear Test Results in Marine Clay”. Geotechnical Testing Journal, GTJODJ, Vol.10, No. 1, pp. 13-18.

Sakamoto, M.Y. et al. (2016). “Use of a borehole shear test method for geotechnical mapping of landslide risk areas.” Landslides and Engineered Slopes. Experience, Theory and Practice – Aversa et al. (Eds). Associazione Geotecnica Italiana, Rome, Italy. p.1783-1790.

Sbroglia, R.M. et al., (2018). “Use of Borehole Shear Test to Obtain Shear Strength Data: Comparison to Direct Shear Test.” In: XIII IAEG Congress, 2018, San Francisco. IAEG/AEG Annual Meeting Proceedings, San Francisco, California, v.6. Switzerland: Springer Nature. p.145-151.

Viecili, C., (2003). “Determination of Ijuí Soil Resistance Parameters from the Direct Shear Test”. Graduation work, Northwestern Regional University of Rio Grande do Sul State, Ijuí, Brazil.