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Statistical Evaluation of Sleeve Friction to Cone Resistance Ratio in Coarse-Grained Soils

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Abstract: The investigation of soil is a particularly important stage of structural design. Cone penetration tests (CPTs) are the most common soil investigation techniques. The results of these tests provide information about the values of cone resistance (q_c) and sleeve friction (f_s), which correspond to depth. Previous studies have shown that the ratio of sleeve friction to cone resistance depends on the particle size distribution in soil and its use for soil classification. Unfortunately, as an analysis of the literature shows, there is no such classification for coarse-grained soils. This paper presents statistically significant differences in the ratio of f_s to q_c in coarse-grained soils. Based on the research performed, the proposed coefficients depend on the classification of coarse-grained soils with respect to the size of the soil particles. The data investigated were obtained from study reports on 35 sites (5934 tests) at which the main type of soil was coarse-grained and contained different sizes of particles. Following a statistical analysis, five groups of tested coarse-grained soils, silty fine sand, clayey fine sand, fine sand, medium sand and gravelly coarse sand together with gravel, are derived. The analysed data show statistically significant differences in the ratio of f_s to q_c considering this particular type of soil. A ratio of f_s to q_c with a probability of 95% is proposed for sandy soils. The values for silty fine sand, clayey fine sand, fine sand, medium sand and gravelly coarse sand mixed with gravel are 0.009459, 0.010982, 0.009268, 0.008001 and 0.006741, respectively. A linear relationship between the f_s and q_c indexes is also suggested.

Keywords: cone penetration test (CPT); cone resistance (q_c); sleeve friction (f_s); statistically significant dependence; sandy soil characteristics

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1. Introduction

The modern construction industry is focused on the rational design of sustainable buildings. The effectiveness of a structural solution depends on the design procedure and the reliability of the initial data. Foundation structures are crucial structural elements. Research data on soil and the interpretation of the information obtained directly affect the accuracy of solutions for foundation structures. Therefore, soil investigation is an especially important stage and must be conducted with high precision.

The cone penetration test (CPT) is one of the most popular in situ testing methods used in modern geotechnics. The technique is fast (20 mm/s), has an excellent price/performance ratio and provides reliable derived soil indicators [1–3]. Over the course of this test, direct measurements are obtained to develop the main indicators—cone resistance (q_c) and local sleeve friction (f_s). The CPT is applied to all types of soil. Although it is hardly possible to accurately determine the granulometric composition of soil using this particular testing method, fairly clear differences in soil behaviour can be found. Cone resistance (q_c) is higher in sand than in clay of the same strength, and the ratio of f_s to q_c is lower in sand than in clay [4].

The very first tests conducted for soil classification, in agreement with the CPT, included a mechanical cone. Begemann (1965) noticed a linear relationship between q_c and f_s [5]. Based on CPT studies of Dutch soils, Begemann proposed and graphically presented a relationship between sleeve friction, cone resistance and soil type.

Charts presented later used the qc and friction ratio (Rf = (fs/qc) × 100%) (Douglas and Olsen, 1981) [6]. The ratio was widely employed by Robertson to classify soils in line with their different behaviours [7–11]. The resulting diagrams show that the friction ratio Rf of the tested types of soil varies within wide limits.

The CPT is important for classifying soils and designing foundation structures; it is also widely used for calculating the bearing capacity of pile foundations. There are two approaches to such calculations. The load-bearing capacity of friction is calculated directly using f_s (sleeve friction) values, or the cone resistance (q_c) values determined via the CPT are multiplied by an empirical coefficient α_s, which indicates the ratio between f_s and q_c (Vukicevic et al., 2017) [12]. In the second case, a problem originates because the empirical coefficient barely reflects real ratios and does not adequately assess the soil's behaviour based on its granulometry.

Both soil classification and the calculation of the bearing capacity of pile foundations clearly show that the friction ratio of cone resistance reflects the type of soil behaviour, considering the evaluation of its type in consonance with its granulometric composition [3,4].

This study aimed to analyse data obtained using the CPT on coarse soils and to examine the possibility of dividing similar soils into smaller groups (for example, fine sand, medium sand and gravel), considering their friction to cone resistance ratios. The introduced classification of soil is important from a practical point of view for solving design tasks [4,13,14]. Regulatory documents do not provide the above classification, although it significantly affects the results of design tasks. A more precise classification of soils leads to more reliable and thus more accurate solutions.

2. Theoretical Basis for the Analysis of Soil Behaviour in Line with Data from the CPT

The use of q_c and f_s values obtained from the CPT is integral to various pile design methods, including empirical, limit state design, load and resistance factor design and advanced numerical analyses. These methods leverage the information provided by the CPT to accurately assess pile bearing capacity and ensure the safety and efficiency of foundation structures. As already mentioned in Section 1, there two methods used to determine load shaft resistance [12]. The first method directly uses f_s (sleeve friction) values from the CPT. In the second method, empirical coefficients are used to evaluate the behaviour of different soils with different soil granulometric compositions. This method is used because experience has shown that the CPT sleeve friction (f_s) value is less repeatable (±0.5% of the full-scale output) than the cone resistance (q_c) value due to differences in cone design and tolerances.

The second method is widely used in Lithuania when calculating the bearing capacity of piles.

The characteristic pile ultimate compressive resistance, determined from ground test results according to the widely used method in Lithuania proposed by Furmonavičius [15,16], is as follows:

$$R_k = \frac{R_s + R_b}{\xi} \tag{1}$$

where R_s is the ultimate shaft resistance of a pile, calculated using ground parameters from test results; R_b is the compressive resistance of the ground against a pile in the ultimate limit state, calculated using ground parameters from test results; and ξ represents correlation factors used to derive characteristic values from ground test results, depending on the number of profiles of tests (Table 1 [17]).

Table 1. Correlation factors used to derive characteristic values from ground test results, depending on the number of profiles of tests according to EU 7 [17].

ξ for n=	1	2	3	4	5	7	10
ξ_3	1.40	1.35	1.33	1.31	1.29	1.27	1.25
ξ_4	1.40	1.27	1.23	1.20	1.15	1.12	1.08

ξ_3—used for average values; ξ_4—used for minimal values.

The ultimate shaft resistance of a pile may be obtained by calculating

$$R_s = \frac{\sum_{i=1}^{n} A_{s;i} \cdot q_{s;i}}{\gamma_{R;s}} \tag{2}$$

where $A_{s;i}$ is the pile shaft surface area in layer I; $\gamma_{R;s}$ is the modelling coefficient, which evaluates the type of pile, as shown in Table 2; and $q_{s;i}$ is the unit shaft resistance in layer I, according to Furmonavičius [15,16]:

$$q_{s;i} = \alpha_{s;i} \cdot q_{c;i} \tag{3}$$

where $q_{c;i}$ is the cone resistance in layer i from the cone penetration test; and $\alpha_{s;i}$ is the correlation coefficient between the pile shaft resistance and the cone resistance in layer i [15,16], which depends on the soil type (see Table 3).

Table 2. Modelling coefficient values according to Furmonavičius [15,16].

Type of Pile	$\gamma_{R;s}$	$\gamma_{R;b}$
Driven	1.10	1.10
Bored displacement	1.10	1.35
Continuous flight auger (CFA), bored	2.00	1.50

Table 3. α_s and α_b, according to Furmonavičius [15,16].

Soil Type	q_c, MPa	α_s, kPa	α_b, MPa	$q_{s;max}$, kPa	$q_{b;max}$, MPa
Clay (moraine)	<5	0.050	1.0 *	200	6.5
	≥5		0.8 *		
Clay		0.035	1.0	150	
Silt		0.025	0.6	150	
Sand [16]	≤10	0.010 *	0.5	170	
	≥25	0.008 *			
Sand [15]	≤10	0.010			
	>10	$q_s = 110 + 4 \cdot (q_c - 10)$			

* intermediate values are linearly interpolated.

The compressive resistance of the ground against a pile, in the ultimate limit state, is

$$R_b = \frac{A_b \cdot q_b}{\gamma_{R,b}} \tag{4}$$

where A_b is the area of the pile base, q_b is the average base resistance obtained from the cone resistance q_c from the cone penetration test and $\gamma_{R;b}$ is the coefficient of modelling, which evaluates the type of pile (see Table 2).

The average base resistance q_b is calculated as the average value of the base resistance in the interval between one width of pile up to the pile base and from four widths of pile to below the pile base:

$$q_b = \frac{\sum_{j=1}^{m} \alpha_{bj} \cdot q_{cj} \cdot h_j}{\sum_{j=1}^{m} h_j} \quad (5)$$

where $\alpha_{b;j}$ is the correlation coefficient between the base resistance $q_{b;j}$ and the cone resistance $q_{c;j}$ in layer j [15,16], which depends on the type of soil (see Table 3), and h_j is the thickness of soil layer j in the analysed interval.

From the long-term experience of the authors, it was noticed that in coarse and mixed soils (from gravel to clayey fine sand), the unit shaft resistance q_s is often lower than the sleeve friction resistance f_s measured via the CPT. Particularly large differences are observed in coarse soils.

According to the calculation method presented above, the shaft resistance q_s from the cone penetration test is calculated according to the cone resistance q_c. Table 3 presents the obtained values of shaft resistance q_s and their corresponding α_s depending on q_c.

Statistical research methods were used to investigate differences in the f_s/q_c ratio depending on soil coarseness. Subsequent to the formation of samples with respect to granulometry, descriptive statistics were used to describe statistical samples. Thus, statistical values, including the mean, mode, median, standard deviation, standard error, sample variance, kurtosis, skewness, range, minimum and maximum values observed in the sample, coefficients of determination and correlation, count or sample size and a confidence level of 95%, were determined.

The determination coefficient R^2 (R-Squared) is a statistical measure in a regression model determining the proportion of variance in a dependent variable explained by an independent variable. R-Squared is an indicator of how properly the collected data fit the regression model, and it is equal to the square of the correlation coefficient R:

$$R\text{Squared} = R^2 = \left(\frac{n(\sum x \cdot y) - (\sum x)(\sum y)}{\sqrt{\left[n \sum x^2 - (\sum x)^2\right]\left[n \sum y^2 - (\sum y)^2\right]}} \right)^2 \quad (6)$$

where x and y are two variables determining the linear correlation.

The formula for calculating R-Squared indicates that in the case of a correlation between two variables, a change in the independent variable will likely result in a change in the dependent variable. Interpretations of the correlation coefficient R are shown in Table 4.

Table 4. Interpretations of the correlation coefficient [18].

Interpretation	Correlation Coefficient R
Weak correlational relationship or no correlation	$0 \ldots \pm 0.3$
Moderate correlational relationship	$0.3 \ldots 0.7$ $-0.3 \ldots -0.7$
Strong correlational relationship	$0.7 \ldots 0.9$ $-0.7 \ldots -0.9$
Very strong correlational relationship	$0.9 \ldots 1.0$ $-0.9 \ldots -1.0$

3. Analysis and Evaluation of Statistical Data

To conduct this study, data from reports on engineering geological surveys carried out in the territory of the Republic of Lithuania were obtained. The figures of 35 different objects containing static probes and boreholes were analysed. Regarding the boreholes, soil layers were identified; thus, only cases of sandy soils were selected. The choice of sandy soils was due to the fact that pore pressure was not measured. Therefore, an uncorrected cone resistance was used for the analysis. CPTs recorded q_c and f_s every 0.2 m. In order for the recorded values of q_c and f_s to be assigned to an appropriate layer of soil, the values at the top and bottom of the layer were excluded, i.e., the upper and lower 0.2 m of the layer were not accepted. The completed data formed a sample of 5934 positions (q_c and a corresponding f_s).

A tensometric CPT was used for the tests, with cone compression force measurement limits from 0 to 100 kN and an area of 10 cm^2. Therefore, 100 kN corresponds to 100 MPa. The cone resistance accuracy is ±0.1% of the cone capacity (full-scale output). The sleeve friction measurement force ranged from 0 to 15 kN, and the area was 150 cm^2. A 15 kN force corresponds to 1 MPa. The sleeve friction accuracy is ± 0.5% of the full-scale output.

For each measured pair of q_c and f_s, an empirical coefficient α_{sd} equal to the q_c/f_s ratio was calculated:

$$a_{sd} = \frac{f_s}{q_c} \tag{7}$$

where f_s is the local sleeve friction, kPa, and q_c is the cone resistance, kPa.

Figure 1 shows the distribution of the values of the empirical coefficient α_s.

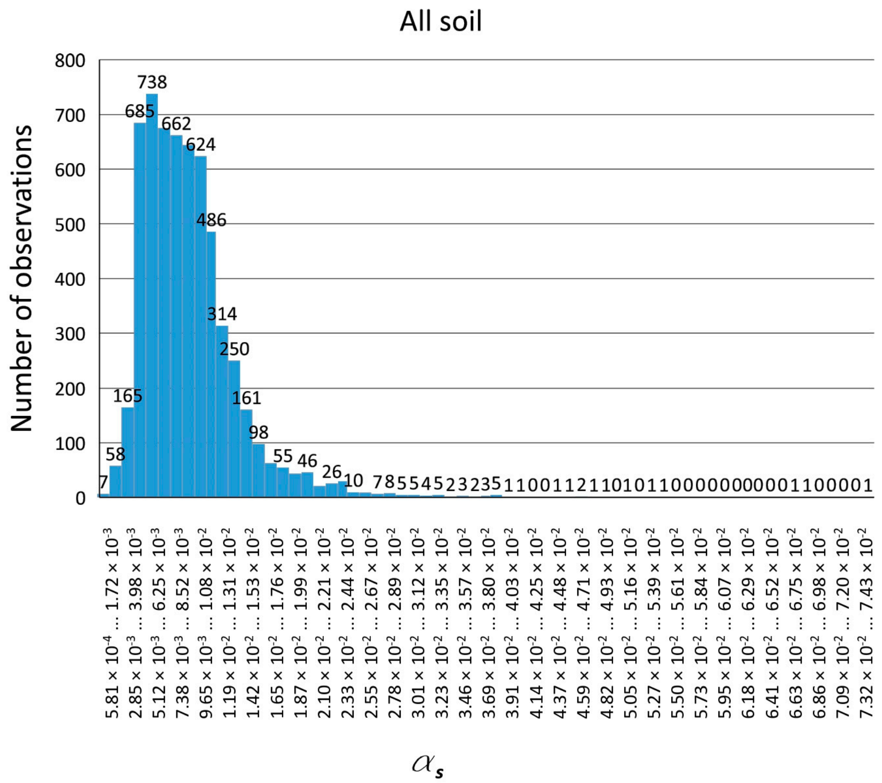


Figure 1. The distribution of all values of $\alpha_{sd} = f_s/q_c$.

This stage of the research is schematically described by the algorithm presented in Figure 2.

Next, as stated in the diagram of soil behaviour proposed by Roberson, Figure 3 presents the distribution of the friction ratio R_f (R_f = (f_s/q_c)100%) values of all the conducted tests depending on the cone resistance q_c.

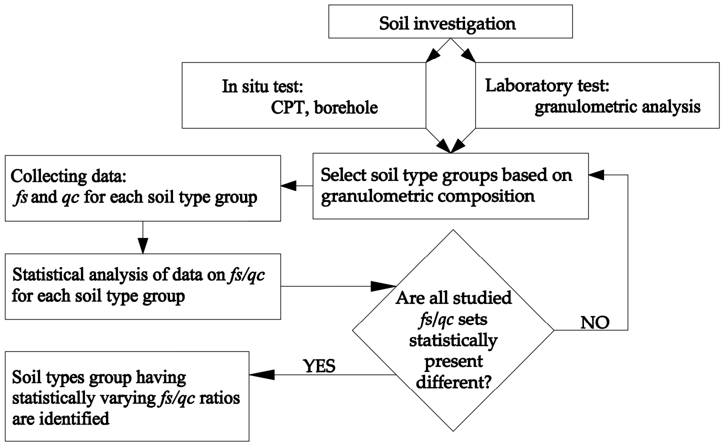


Figure 2. The algorithm of the conducted research.

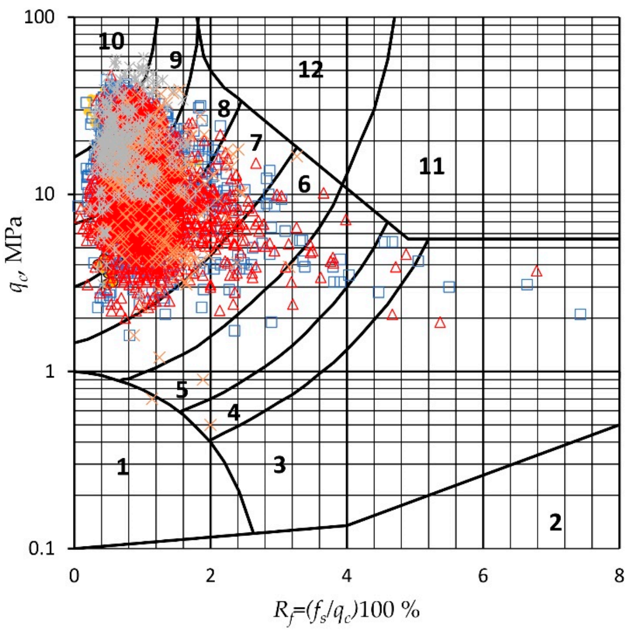


Figure 3. The dependence of R_f distribution on q_c (Robertson, 2010) [12]. The numbered zones correspond to the following: 1—sensitive fine-grained; 2—organic material; 3—clay; 4—silty clay to clay; 5—clayey silt to silty clay; 6—sandy silt to clayey silt; 7—silty sand to sandy silt; 8—sand to silty sand; 9—sand; 10—gravelly sand to sand; 11—very stiff fine-grained (overconsolidated or cemented); 12—sand to clayey sand (overconsolidated or cemented).

The majority of the surveyed points fall into zones 6–10, which correspond to the soil names determined during the survey. Also, some points strongly deviate from the majority of observations. Due to the testing specificity, the findings of the soil tests vary widely. Thus, not all the results reflect the characteristics of the studied set. Excessively deviant values must be eliminated from further evaluations.

Therefore, having found the mean values of f_s/q_c and their standard deviations, excessively deviant values were eliminated from the sample in line with the rule of three standard deviations.

Having excluded the values deviating from the average value of α_s by three standard deviations, corresponding to a confidence level of P = 99.7% in the studied case, for sandy soils, a strong linear relationship between cone resistance and the corresponding local sleeve friction was observed.

Table 5 presents the statistical characteristics of the soil tests conducted on the entire set (prior to exclusion). Table 5 also shows the statistical characteristics of the set when, in agreement with the assumption of three standard deviations, excessively deviant values (following exclusion) were eliminated.

Table 5. Statistical indicators for the sets of soil prior to and following exclusion.

	All (Following Exclusion)	All (Prior to Exclusion)
Mean	0.008606	0.009320
Standard Error	4.41×10^{-5}	6.48×10^{-5}
Median	0.008257	0.008479
Standard Deviation	0.00332	0.00499
Sample Variance	1.11×10^{-5}	2.49×10^{-5}
Kurtosis	−0.233	20.518
Skewness	0.526	3.045
Range	0.017990	0.073704
Minimum	0.000581	0.000581
Maximum	0.018571	0.074286
Count	5678	5934
Confidence Level (95.0%)	8.6625×10^{-5}	1.2698×10^{-4}

Figure 4 reflects the results in Table 5 and shows the part of the results falling within the area of three standard deviations (results that were not excluded) and the part that was rejected from the area of three standard deviations and was therefore excluded (blue dots).

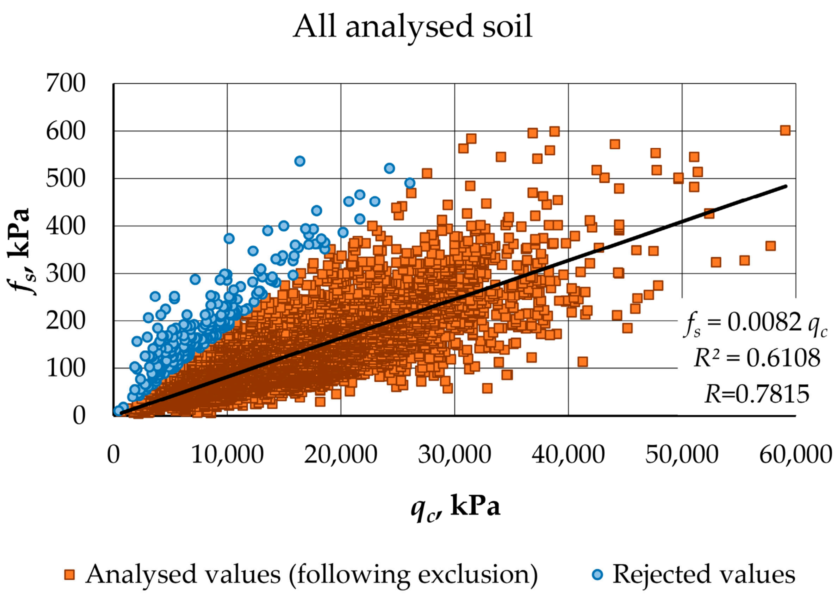


Figure 4. The relationship between cone resistance and sleeve friction in sandy soils.

Having eliminated the values that were three standard deviations from the mean, the equation of the line passing through the origin of the coordinates was derived by applying the least squares method:

$$f_s = \alpha_{sd} q_c \tag{8}$$

where α_{sd} = 0.0082 and the strength of the relationship between the variables in the linear model is R = 0.7815.

The value of R shows a strong correlation. Having processed the statistical data, a linear relationship between cone resistance q_c and local sleeve friction f_s in sandy soils was obtained.

$$f_s = 0.0082q_c \tag{9}$$

Having calculated the friction ratio R_f of the remaining pairs, the latter corresponds to the distribution of sandy soils, as proposed by Roberson in the diagram of soil behaviour (Figure 5). The figure provides data on the exclusion of excessively deviant values (see Table 5).

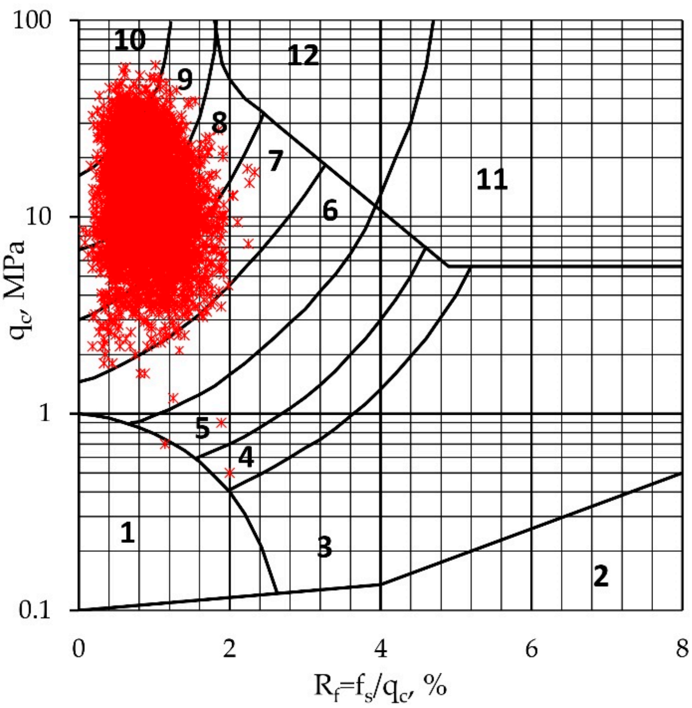


Figure 5. The relationship between R_f and q_c. (1) Sensitive fine-grained; (2) clay–organic soil; (3) clays—clay to silty clay; (4) silt mixtures—clayey silt and silty clay; (5) sand mixtures—silty sand to sandy silt; (6) sands—clean sands to silty sands; (7) dense sand to gravelly sand; (8) stiff sand to clayey sand (overconsolidated or cemented); (9) stiff fine-grained (overconsolidated or cemented); (10) gravelly sand to sand; (11) very stiff fine-grained (overconsolidated or cemented soil); (12) sand to clayey sand (overconsolidated or cemented soil).

Figure 5 shows that having excluded excessively deviant values, most of the observed values fall within zones 7 to 10, which confirms that sandy soil is under consideration.

In line with the granulometric composition (conforming to the data on the boreholes made next to the probes), the total sample suggests six types of soil:

- Clayey fine sand (c f S);
- Silty fine sand (sl f S);
- Fine sand (f S);
- Medium sand (m S);
- Gravelly coarse sand (g c S);
- Gravel (G).

Having found the average values and standard deviations of the obtained samples, excessively deviant values were eliminated from the samples in accordance with the rule of three standard deviations. The resulting samples had the statistical characteristics presented in Table 6.

Table 6. Statistical indicators for the sets of soils grouped according to granulometric composition.

	Clayey Fine Sand	Silty Fine Sand	Fine Sand	Medium Sand	Gravelly Coarse Sand	Gravel
Mean	0.011689	0.010096	0.009395	0.008155	0.006940	0.006699
Standard Error	3.57×10^{-4}	3.22×10^{-4}	6.51×10^{-5}	7.85×10^{-5}	9.61×10^{-5}	1.16×10^{-4}
Median	0.011897	0.009661	0.009379	0.007697	0.006174	0.006342
Standard Deviation	4.09×10^{-3}	4.15×10^{-3}	3.29×10^{-3}	3.13×10^{-3}	2.66×10^{-3}	2.29×10^{-3}
Sample Variance	1.67×10^{-5}	1.72×10^{-5}	1.08×10^{-5}	9.82×10^{-6}	7.07×10^{-6}	5.25×10^{-6}
Kurtosis	−0.825	0.685	−0.106	−0.099	−0.115	−0.295
Skewness	0.248	0.927	0.341	0.663	0.730	0.556
Range	0.019161	0.021171	0.018405	0.016965	0.013201	0.010998
Minimum	0.004153	0.001446	0.000811	0.000581	0.001593	0.001973
Maximum	0.023314	0.022617	0.019216	0.017546	0.014795	0.012971
Count	131	165	2550	1593	765	388
Confidence Level (95.0%)	0.000707	0.000637	0.000128	0.000154	0.000189	0.000229

Next, using Student's *t*-test, the samples of the groups identified in accordance with the granulometric composition were checked in order to determine whether the samples were statistically different. The calculation results are presented in Table 7. Each cell of the table contains a correlation coefficient indicating the relationship significance between the individual samples. An intercomparison of the samples (Table 7) indicates that certain samples are interdependent.

Table 7. The values of correlation coefficients (Pearson's correlation) among 6 identified samples.

	Clayey Fine Sand	Silty Fine Sand	Fine Sand	Medium Sand	Gravelly Coarse Sand	Gravel
Clayey fine sand	1.00×10^{0}	1.06×10^{-3}	3.42×10^{-9}	1069×10^{-17}	7.00×10^{-26}	1.59×10^{-27}
Silty fine sand		1.00×10^{0}	3.46×10^{-2}	2.27×10^{-8}	1.86×10^{-17}	3.46×10^{-19}
Fine sand			1.00×10^{0}	2.50×10^{-33}	2.68×10^{-87}	4.23×10^{-71}
Medium sand				1.00×10^{0}	4.43×10^{-22}	1.02×10^{-23}
Gravelly coarse sand					1.00×10^{0}	1.11×10^{-1}
Gravel						1.00×10^{0}

Above, 1.00×10^{0} indicates no statistical difference between the corresponding samples, 1.06×10^{-3} shows a statistically very low similarity between the samples and 1.10×10^{-1} demonstrates statistically similar samples. For this reason, the last two samples, including gravelly coarse sand (c g S) and gravel (G), are combined into a single sample.

The procedure for statistical processing was repeated after finding the average value (c g S + G) of the new sample and the standard deviation. Excessively deviant values were eliminated from the sample in line with the rule of three standard deviations. The statistical characteristics of the samples were obtained (Table 8).

Table 8. Statistical indicators for soils in line with granulometric composition after combining g c S and G sets.

	Clayey Fine Sand	Silty Fine Sand	Fine Sand	Medium Sand	Gravelly Coarse Sand + Gravel
Mean	0.011689	0.010096	0.009395	0.008155	0.006889
Standard Error	3.57×10^{-4}	3.22×10^{-4}	6.51×10^{-5}	7.85×10^{-5}	7.58×10^{-5}
Median	0.011897	0.009661	0.009379	0.007697	0.006290
Standard Deviation	4.09×10^{-3}	4.15×10^{-3}	3.29×10^{-3}	3.13×10^{-3}	2.58×10^{-3}
Sample Variance	1.67×10^{-5}	1.72×10^{-5}	1.08×10^{-5}	9.82×10^{-6}	6.65×10^{-6}
Kurtosis	−0.825	0.685	−0.106	−0.099	−0.041
Skewness	0.248	0.927	0.341	0.663	0.727
Range	0.019161	0.021171	0.018405	0.016965	0.013026
Minimum	0.004153	0.001446	0.000811	0.000581	0.001593
Maximum	0.023314	0.022617	0.019216	0.017546	0.014519
Count	131	165	2550	1593	1158
Confidence Level (95.0%)	0.000707	0.000637	0.000128	0.000154	0.000149

Next, the samples of the groups newly identified with respect to granulometric composition were checked to determine whether the samples were statistically different by applying Student's *t*-test. The calculation results are presented in Table 9.

Table 9. The values of correlation coefficients (Pearson's correlation) among 5 identified samples.

	Clayey Fine Sand	Silty Fine Sand	Fine Sand	Medium Sand	Gravelly Coarse Sand + Gravel
Clayey fine sand	1.00×10^{0}	1.06×10^{-3}	3.42×10^{-9}	2.69×10^{-17}	2.64×10^{-26}
Silty fine sand		1.00×10^{0}	3.46×10^{-2}	2.27×10^{-8}	3.93×10^{-18}
Fine sand			1.00×10^{0}	2.50×10^{-33}	1.96×10^{-125}
Medium sand				1.00×10^{0}	2.06×10^{-30}
Gravelly coarse sand + gravel					1.00×10^{0}

The remaining five samples correspond to soils categorised as clayey fine sand, silty fine sand, fine sand, medium sand and gravelly coarse sand mixed with gravel and are described as statistically different.

Figure 6 presents the mean values of all the statistically different samples $R_f/100\%$ and intervals with a confidence level of 95%. The figure also shows the relationship between the granulometric composition of the soil; the finer a soil is or the more it is composed of finer particles, the higher its $R_f/100\%$. Additionally, a greater dispersion of the values of fine soil is observed.

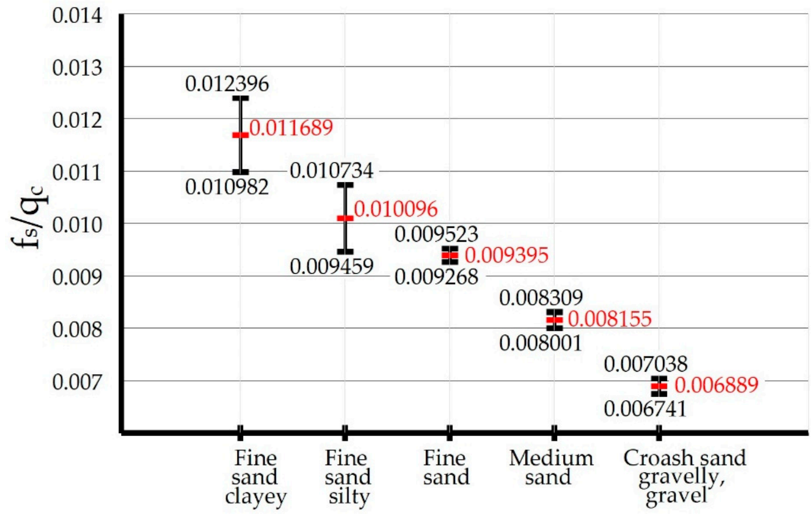


Figure 6. f_s/q_c with respect to the granulometric composition of soil, with a confidence level of 95%.

In consonance with $R_f/100\%$, for five identified statistically different samples, the linear relationships of the lines passing through the origin of the coordinates were recorded by employing the least squares method:

$$f_s = \alpha_{sd} q_c \tag{10}$$

The strength of the relationship between the linear model and R^2 (Figure 7) was determined.

Figure 7 shows blue points, the values of which deviate within a margin of three standard deviations (3σ). In the search for a linear relationship, these values were rejected.

α_{sd} and R^2 are given in Table 10 together with the mean values of the samples $R_f/100\%$ and the confidence intervals of those values.

Table 10. Statistical parameters of different samples.

	Clayey Fine Sand (c f S)	**Silty Fine Sand (st f S)**	**Fine Sand (f S)**	**Medium Sand (m S)**	**Gravelly Coarse Sand, Gravel (c g S + G)**	**All (Following Exclusion)**
Parameters for linear equations						
α_{sd}	0.0109	0.0101	0.0090	0.0077	0.0073	0.0082
R^2	0.6928	0.5605	0.7242	0.5118	0.6184	0.6108
R	0.8323	0.7487	0.8510	0.7154	0.7863	0.7815
Mean values of $R_f/100\% = f_s/q_c$ with a confidence level of 95%						
f_s/q_c	0.011689	0.010096	0.009395	0.008155	0.006889	0.008258
Mean values of f_s/q_c with a confidence level of 95%						
from	0.010982	0.009459	0.009268	0.008001	0.006741	0.007740
to	0.012396	0.010734	0.009523	0.008309	0.007038	0.009472

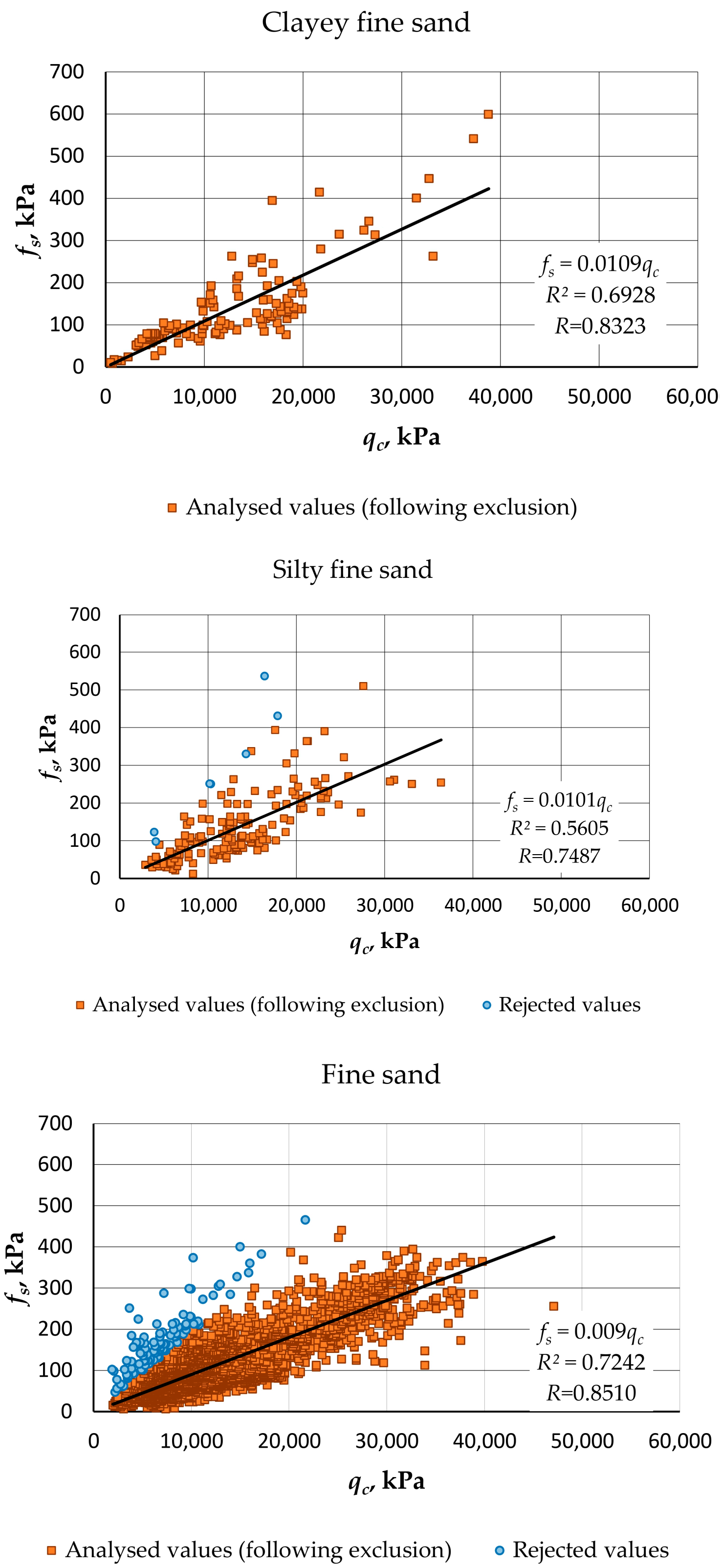


Figure 7. *Cont.*

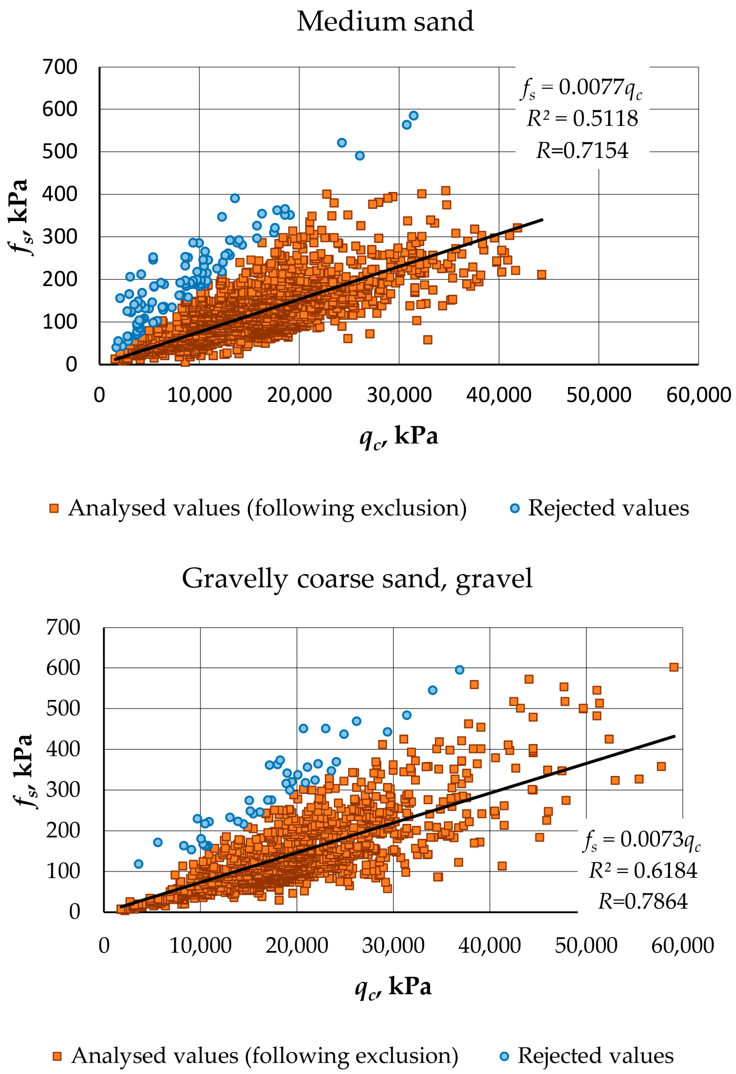
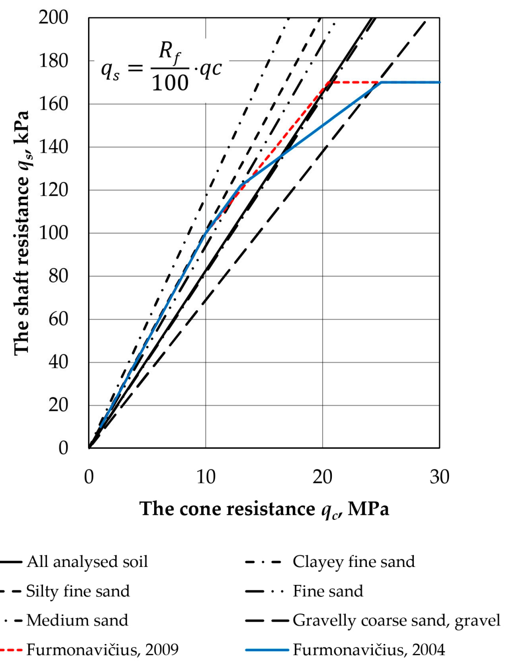


Figure 7. Linear relationship of the identified statistically different samples.

The strength of the relationship between the variables in the linear model, R, shows a strong correlation between f_s and q_c in all samples without separating the soils into the groups determined according to granulometry; regarding α_{sd} and $R_f/100\%$ (or f_s/q_c), the values for the medium sand and gravelly coarse sand + gravel groups are too high and for the remaining groups, they are too low.

Comparing the unit shaft resistance q_s used in the pile calculations with the results of this study, it can be seen that only the calculated value of the unit shaft resistance q_s for clayey fine sand and silty fine sand is higher than that currently used for pile design. Meanwhile, for the rest of the soils with low q_c values, the calculated values of shaft resistance q_s are lower than those currently used for pile design (Figure 8).

Thus, a finer division provides more accurate results. Following more detailed research, the α_{sd} value obtained could be applied for calculating the bearing capacity of a pile foundation.



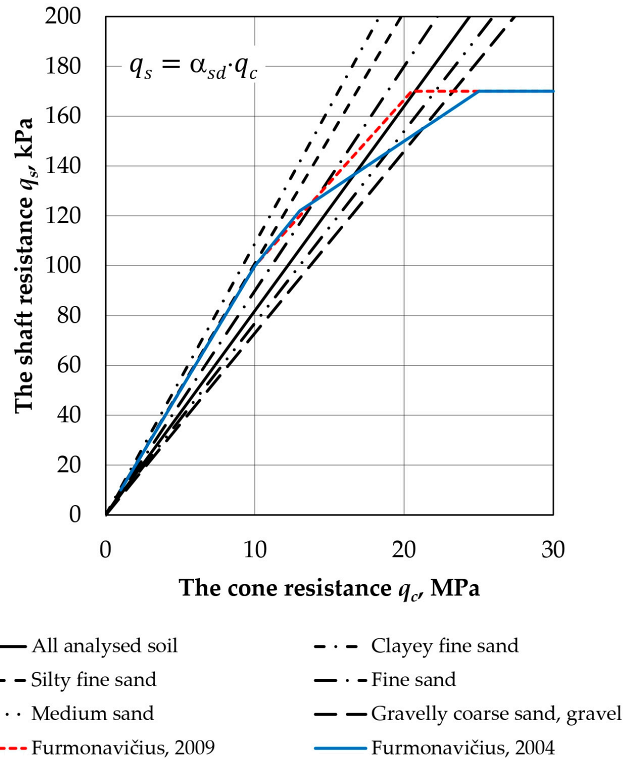


Figure 8. The relationship between q_s and q_c for the analysed groups of soils and croase soils according Furmonavičius [15,16].

4. Conclusions and Suggestions

1. A statistical analysis of the tested soil showed that the correlational relationship of the tested coarse-grained soil between the sleeve friction and cone resistance in the studied sample (5634 observations) is strong and reaches R = 0.7815. The obtained relationship of the sample is f_s = 0.0082q_c.
2. Regarding the boreholes, six samples of different soils were identified: silty fine sand, clayey fine sand, fine sand, medium sand, gravelly coarse sand and gravel.

The difference between the samples of gravelly coarse sand and gravel was found to be statistically insignificant (0.11049). The other samples showed statistically significant differences.

3. Statistically different samples like silty fine sand, clayey fine sand, fine sand, medium sand and gravelly coarse sand mixed with gravel were identified. The soils of these isolated groups have a strong correlation between the local sleeve friction and cone resistance (R = 0.7154. . .0.8510).
4. The statistical analysis of the tested soil showed a confidence level of 95% and determined that the f_s/q_c ratio is not lower than the ratio calculated for silty fine sand, reaching 0.009459; for clayey fine sand, this ratio is equal to 0.010982, for fine sand, it reaches 0.009268, for medium sand, it is 0.008001 and for gravelly coarse sand and gravel, it is 0.006741. An increase in the particle size of sandy soil leads to a decrease in the ratio between the local sleeve friction and cone resistance.
5. The determined f_s/q_c values are applicable only to the tested types of soil. In order to apply f_s/q_c ratios to the classification of coarse-grained soils, performing a statistical analysis of CPT data in a specific area is required. The studied relationships between f_s and q_c in five identified statistically different groups of soil demonstrated a strong relationship between the above-mentioned indicators, thus providing linear equations for the established relationships. The relationships found are valid only for the tested soils. For a broader application, additional research is needed.
6. The study showed that the shaft resistance q_s values currently used in pile load-bearing capacity calculations can be more accurately estimated using α_{sd} or $R_f/100\%$ for finer coarse-grained soil types, depending on the granulometric composition. However, to determine exact values, more extensive studies are needed, including larger soil samples and evaluating additional properties of the gravel (origin, moisture, etc.).

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