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Evaluation of the Undrained Shear Strength in Preconsolidated Cohesive Soils Based on the Seismic Dilatometer Test

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Abstract: The undrained shear strength in cohesive soils can be evaluated based on measurements obtained from the standard dilatometer test (DMT) using single- and multi-factor empirical relationships. However, the empirical relationships presented in the literature may sometimes show relatively high values of the maximum relative error. The add-on seismic module to the seismic dilatometer test (SDMT) extends parameters measurable in a standard dilatometer test by the shear wave velocity V_s as an independent variable. Therefore, a method for evaluating the undrained shear strength in cohesive soils based on data obtained from the seismic dilatometer test is presented in this study. In the method proposed, the two-factor empirical relationship for evaluating the normalized undrained shear strength τ_{fu}/σ'_v is used based on independent variables: The normalized difference between the corrected second pressure reading and the corrected first pressure reading $(p_1 - p_o)/\sigma'_v$ and the normalized shear wave velocity $V_s/100$. The proposed two-factor empirical relationship provides a more reliable evaluation of the undrained shear strength in the tested Pleistocene and Pliocene clays in comparison to the empirical relationships presented in the literature, with a maximum relative error max *RE* at about ±20% and the mean relative error *RE* at about 8%.

Keywords: cohesive soils; undrained shear strength; seismic dilatometer test; statistical analysis

1. Introduction

In order to determine the geotechnical parameters the cone penetration tests and the standard penetration test (SPT) have wide applications. From in situ tests the cone penetration tests (cone penetration test CPT, cone penetration test with pore pressure measurement CPTU, seismic cone penetration test SCPT) have the widest use for estimating geotechnical parameters over a wide range of materials from very soft soils to weak rock. The flat dilatometer tests (DMT, SDMT) have become popular in many countries for estimating geotechnical parameters in non-cohesive and cohesive soils, however, their application is low in gravels and soft rocks [1,2].

The flat dilatometer was developed by Marchetti [3,4]. The detailed procedure for conducting the standard dilatometer test (DMT) and methodology of its interpretation were presented by Marchetti [5,6] and Marchetti et al. [7]. Comprehensive studies have been performed to improve some of the original correlations proposed by Marchetti. Numerous investigations have been made to expand the application of DMT in geotechnical engineering [8–17]. In most cases, the relationships used to evaluate geotechnical parameters from the dilatometer test such as the corrected first pressure reading p_o or the corrected second pressure reading p_1, or index parameters such as the material index I_D, horizontal stress index K_D, and dilatometer modulus E_D, are commonly applied.

Supplementing the instrumentation used to perform the dilatometer test with two geophones in the seismic dilatometer test (SDMT) has expanded possibilities of interpreting the test results [18,19]. The use of geophones enabled making measurements of an additional parameter in the form of shear wave velocity V_s. In this way, the SDMT, similarly to the SCPT, provides an additional measurement enabling the seismic assessment of the studied area and a more precise determination of many geotechnical parameters. Although the seismic dilatometer is used just for slightly more than twenty years, there is already extensive literature discussing the use of SDMT in soil characterization [20–25], assessment of geotechnical parameters [26–31], and design of geotechnical structures [32,33]. The shear wave velocity V_s with dilatometer measurements have been used so far to evaluate the initial shear modulus G_0 [19,26,28], decay of the shear modulus G with the shear strain γ [19,29], interrelationship between G_0 and operative modulus [27], detecting the presence of cementation [5], and cyclic resistance ratio CRR [30].

Undrained shear strength τ_{fu} is the basic parameter in the geotechnical design of different structures [34–47]. Evaluation of this parameter using dilatometer tests is usually based on empirical relationships. In single-factor relationship proposed by Marchetti [3] the horizontal stress index K_D is used as the independent variable. Roque et al. [38] proposed an approach for estimating undrained shear strength using the corrected second pressure reading p_1. A different relationship was proposed by Smith and Houlsby [39], in which undrained shear strength was a function of the corrected first pressure reading p_o. For normally consolidated marine clays, Iwasaki and Kamei [40] proposed a relationship, in which the dilatometer modulus E_D is as independent variable.

A multi-factor relationship was proposed by Rabarijoely [41] to evaluate undrained shear strength τ_{fu}, in which as independent variables were the in situ effective vertical stress σ'_v, as well as the net value of the corrected first pressure reading $(p_o - u_o)$ and the net value of a corrected second pressure reading $(p_1 - u_o)$. Galas [42] proposed a multi-factor relationship, in which the normalized net value of the corrected first pressure reading $(p_o - u_o)/\sigma'_v$ and the normalized net value of the corrected second pressure reading $(p_1 - u_o)/\sigma'_v$ were used as independent variables.

It should be noted that the regional geotechnical conditions could have substantial influence on the empirical relationships and the values of empirical coefficients [43–48]. It is important to point out, that the shear wave velocity V_s as an independent variable was not used so far to evaluate the undrained shear strength from the dilatometer test.

This paper presents the results of seismic dilatometer tests SDMT and laboratory tests of preconsolidated cohesive soils (Pleistocene clays and Pliocene clays). The undrained shear strength in cohesive soils is evaluated based on measurements obtained from the dilatometer test using single- and multi-factor empirical relationships presented in the literature. However, the empirical relationships sometimes show relatively high values of the maximum relative error max RE. In this study a method for evaluating the undrained shear strength of preconsolidated cohesive soils using statistical analysis based on data obtained from a seismic dilatometer test is presented. The presented multi-factor empirical relationship predicts the normalized undrained shear strength based on two independent variables: The normalized difference between the corrected second pressure reading and the corrected first pressure reading $(p_1 - p_o)/\sigma'_v$ and the normalized shear wave velocity $V_s/100$. The proposed multi-factor empirical relationship provides a more reliable prediction of the undrained shear strength in comparison to the empirical relationships presented in the literature, with a max RE at about ±20% and the mean relative error RE at about 8%.

2. Methods and Materials

2.1. Dilatometer Test Procedure

Standard DMT test procedure involves pushing the blade vertically into the ground with readings at selected test depths. The readings are generally made at every 0.2 m of depth. The first *A*-reading pressure occurs at membrane "lift-off" and the second *B*-reading pressure after 1.1 mm movement [5–7,49,50].

After appropriate corrections described by Marchetti, the values of *A* and *B* pressures yield the values of the corrected first pressure reading p_o (0.00 mm expansion) and the corrected second pressure reading p_1 [4]. Based on the corrected pressure readings p_o and p_1, as well as in situ hydrostatic pore pressure u_o and in situ effective vertical stress σ'_v, the following index parameters were proposed by Marchetti [3]:

Material index

$$I_D = (p_1 - p_o)/(p_o - u_o) \tag{1}$$

Horizontal stress index

$$K_D = (p_o - u_o)/\sigma'_v \tag{2}$$

Dilatometer modulus

$$E_D = 34.7 \cdot (p_1 - p_o) \tag{3}$$

The use of a seismic dilatometer SDMT with two geophones enabled making additional measurements of shear wave velocity *Vs* every 0.5 m of depth (Figure 1).

Figure 1. Dilatometer blade, measuring unit and seismic module of the seismic dilatometer test (SDMT).

2.2. Evaluation of Undrained Shear Strength From the Dilatometer Test

2.2.1. Single-Factor Relationships

Marchetti [3] proposed the following basic correlation between the normalized undrained shear strength and the horizontal stress index K_D for cohesive soils (for material index I_D < 1.2):

$$\frac{\tau_{fu}}{\sigma'_v} = 0.22 \cdot (0.5 \cdot K_D)^{1.25} \tag{4}$$

Analysis of the DMT results presented in the literature [10,43–45,47] indicates that for particular soils, the relationship between the overconsolidation ratio (OCR) and the horizontal stress index K_D, as well as the normalized undrained shear strength and the horizontal stress index K_D differ from that proposed by Marchetti [3] and can be modified as follows:

$$\frac{\tau_{fu}}{\sigma'_v} = S \cdot (n \cdot K_D)^m \tag{5}$$

where $S = (\tau_{fu}/\sigma'_v)_{nc}$ is the normalized undrained shear strength for normally consolidated soil, and n and m are empirical coefficients. It has long been recognized that normalized undrained shear strength in normally consolidated soil depends on the mode of testing, boundary conditions, strain rate, and other variables [34,35,37,44]. The different values of the S, n, and m parameters are shown in Table 1.

Table 1. Empirical coefficients in single- and multi-factor relationships to evaluate undrained shear strength τ_{fu} from the dilatometer test.

Author	Equation	Independent Variable	Empirical Coefficients
Single-Factor Empirical Relationships			
Marchetti [3]	(4)	K_D	Marchetti [3] cohesive soils for $I_D < 1.2$ $S = 0.22$, $n = 0.5$, $m = 1.25$
Modified Marchetti	(5)	K_D	Kamei and Iwasaki [44] marine clays $S = 0.35$, $n = 0.47$, $m = 1.14$
Roque et al. [38]	(6)	$(p_1 - \sigma_{ho})$	Roque et al. [38] clays $N_C = 5$–9 Galas [42] Pleistocene and Pliocene clays $N_C = 9.9$
Smith and Houlsby [39]	(7)	$(p_o - \sigma_{ho})$	Smith and Houlsby [39] clays $N_D = 4$–7 Galas [42] Pleistocene and Pliocene clays $N_D = 4.9$
Multi-Factor Empirical Relationships			
Rabarijoely [41]	(9)	σ'_v $(p_o - \sigma_{ho})$ $(p_1 - \sigma_{ho})$	Lechowicz et al. [51] Pleistocene and Pliocene clays $\alpha_o = 0.18$, $\alpha_1 = 0.14$, $\alpha_2 = 0.20$, $\alpha_3 = 0.15$
Galas [42]	(10)	$(p_o - \sigma_{ho})/\sigma'_v$ $(p_1 - \sigma_{ho})/\sigma'_v$	Galas [42] Pleistocene and Pliocene clays $a_o = 0.164$, $a_1 = 0.345$, $a_2 = 0.544$

Roque et al. [38] proposed a relationship for estimating the undrained shear strength based on the corrected second pressure reading p_1:

$$\tau_{fu} = (p_1 - \sigma_{ho})/N_C \tag{6}$$

where σ_{ho} is the in situ horizontal total stress, and N_C is the dilatometer factor for clays that varies from about 5 to 9. The research carried out by Roque et al. [38] and Galas [42] indicates the values of N_C factor as shown in Table 1.

A different relationship has been proposed by Smith and Houlsby [39], in which the undrained shear strength was a function of the corrected first pressure reading p_o:

$$\tau_{fu} = (p_o - \sigma_{ho})/N_D \tag{7}$$

where N_D is the dilatometer bearing capacity factor, whose values are shown in Table 1.

For normally consolidated marine clays Iwasaki and Kamei [40] proposed a relationship, in which the dilatometer modulus E_D is the independent variable:

$$\tau_{fu} = 0.018 \cdot E_D \tag{8}$$

2.2.2. Multi-Factor Relationships

A multi-factor relationship was proposed by Rabarijoely [41] to evaluate the undrained shear strength τ_{fu} of organic soils (organic mud, gyttja, and peat):

$$\tau_{fu} = \alpha_o \cdot \sigma_v'^{\alpha_1} \cdot (p_o - u_o)^{\alpha_2} \cdot (p_1 - u_o)^{\alpha_3} \tag{9}$$

where α_o, α_1, α_2, α_3 are the empirical coefficients. In this relationship, three factors are taken into account: The in situ effective vertical stress σ'_v, the net value of the corrected first pressure reading $(p_o - u_o)$ and the net value of the corrected second pressure reading $(p_1 - u_o)$. The net values of the corrected first and second pressure readings were obtained by subtracting the in situ hydrostatic pore pressure u_o from p_o and p_1. The empirical coefficients to Equation (9) for Pleistocene clays and Pliocene clays evaluated by Lechowicz et al. [51] are shown in Table 1.

A two-factor relationship was proposed by Galas [42] to evaluate the normalized undrained shear strength τ_{fu}/σ'_v of preconsolidated cohesive soils:

$$\frac{\tau_{fu}}{\sigma_v'} = a_o \cdot \left(\frac{p_o - u_o}{\sigma_v'}\right)^{a_1} \cdot \left(\frac{p_1 - u_o}{\sigma_v'}\right)^{a_2} \tag{10}$$

where a_o, a_1, a_2 are the empirical coefficients. In this relationship, two factors are taken into account: The net value of the corrected first pressure reading normalized by in situ effective vertical stress σ'_v and the net value of the corrected second pressure reading normalized by in situ effective vertical stress σ'_v. The empirical coefficients to Equation (10) for Pleistocene clays and Pliocene clays evaluated by Galas [42] are shown in Table 1.

2.3. Characteristics of the Tested Cohesive Soils

Field tests coupled with laboratory tests were performed on three test sites located in Warsaw, Poland (Figure 2). The tested cohesive soils are Pleistocene moraine and lacustrine clays and Pliocene clays.

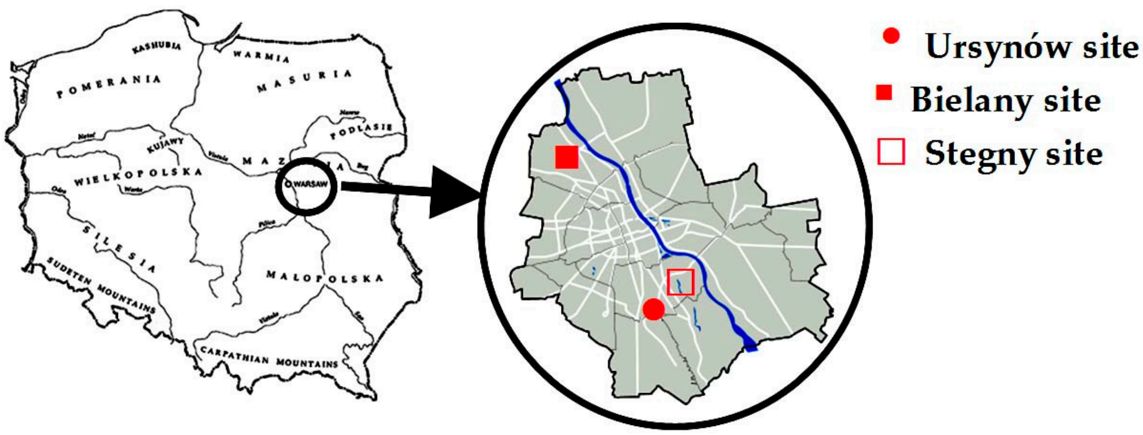


Figure 2. Location of test sites in Warsaw, Poland.

2.3.1. Ursynów Site

The Ursynów site is located on the Warsaw University of Life Sciences—SGGW campus in the southern part of Warsaw. Three locations were selected here: The Conference Auditorium, Building No. 37, and Building No. 34. The tested subsoil consists of anthropogenic fill with thickness 3.5–4.0 m below Pleistocene moraine deposits of the Riss Glaciation classified as stiff brown moraine clays of the Warta Glaciation with thickness from 2 m to 4 m and grey moraine clays of the Odra Glaciation with a thickness of 2.5–3.0 m.

The cohesive soils are underlain by Quaternary sands of the Mazovian Interglacial whose top lies at 9.5 m to 10.0 m below ground level. There is one aquifer on the Ursynów site; a free groundwater table is found in sands at a depth of 10 m to 12 m below ground level.

Glacial moraine deposits of the Warta and Odra Glaciations are classified as low plasticity clays. Grain size distribution of the tested soils (Figure 3) shows that according to EN ISO 14688-1 and 2 Standards [52,53], the glacial moraine clays are classified as silty sandy clays (sasiCl) and clayey sands (clSa). The index properties of the tested soils for nine undisturbed soil samples (No. 1–9) are shown in Table 2. The glacial moraine clays are preconsolidated with an overconsolidation ratio *OCR* determined from SDMT decreasing with depth from 20 to 12 for brown moraine clays of the Warta Glaciation and from 15 to 8 for grey moraine clays of the Odra Glaciation.

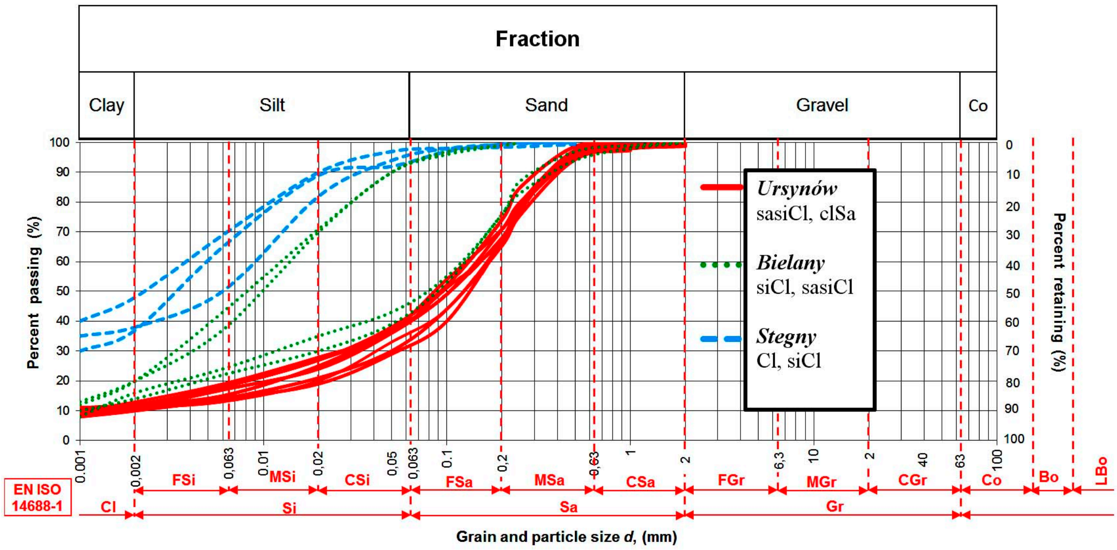


Figure 3. Grain size distribution of the tested cohesive soils.

Table 2. Index properties of the tested cohesive soils.

No.	Soil Type [53]	Depth (m)	w_n (%)	w_L (%)	w_P (%)	I_P (%)	I_C (-)	Fraction [52] (%)				Activity A (-)
								Gr	Sa	Si	Cl	
Ursynów site–Auditorium												
1	clSa/sasiCl	5.0–5.3	10.3	25.2	12.8	12.4	1.20	1	59	27	13	0.95
2	clSa	5.3–5.7	8.6	24.1	11.9	12.2	1.27	1	63	25	11	1.11
3	clSa	7.0–7.4	10.4	24.9	11.9	13.0	1.11	0	61	28	11	1.18
Ursynów site–Building No. 37												
4	clSa	6.5–6.9	9.0	25.1	11.7	13.4	1.20	1	68	20	11	1.22
5	sasiCl	7.0–7.4	10.4	24.6	11.9	12.7	1.12	1	58	30	11	1.15
6	sasiCl	8.5–8.9	9.9	26.5	12.4	14.1	1.18	2	58	29	11	1.28
Ursynów site–Building No. 34												
7	clSa	6.5–6.9	10.3	24.9	12.9	12.0	1.21	1	66	23	10	1.20
8	clSa/sasiCl	7.0–7.4	10.5	23.1	12.0	11.1	1.13	1	60	28	11	1.01
9	sasiCl	8.0–8.4	9.4	26.6	13.1	13.5	1.27	1	59	28	12	1.12
Bielany site												
10	sasiCl	15.0–15.4	10.4	30.9	13.1	17.8	1.15	1	52	31	16	1.11
11	siCl	7.1–7.6	19.2	38.0	18.8	19.2	0.98	0	7	73	20	0.96
12	siCl	8.5–8.9	21.1	34.6	17.5	17.1	0.79	0	7	73	20	0.86
13	sasiCl	12.0–12.5	11.4	27.7	12.3	15.4	1.05	1	57	29	14	1.10
Stegny site												
14	Cl	6.0–6.4	26.0	78.4	25.9	52.5	1.00	0	2	50	48	1.09
15	Cl/siCl	9.0–9.4	28.5	88.1	31.2	56.9	1.05	0	6	57	37	1.54
16	Cl/siCl	12.0–12.4	19.8	67.6	25.5	42.1	1.13	0	3	59	38	1.11

Notes: w_n—water content, w_L—liquid limit, w_P—plastic limit, I_P = w_L−w_P—plasticity index, I_C = (w_L−w_P)/I_P—consistency index, Gr—gravel, Sa—sand, Si—silt, Cl—clay, $A = I_P/f_{Cl}$—activity, f_{Cl}—percent of clay fraction, 1–16—number of tested cohesive soil.

2.3.2. Bielany Site

The Bielany site is located in the northern part of Warsaw. The cohesive soils studied are Pleistocene moraine and lacustrine deposits of the Warta Glaciation with a thickness of about 10–12 m and the top occurring at 5.5 m below ground level. The groundwater table was found at the depth of 5.4 m, or in the form of sifting in sandy interbeddings of cohesive soils. The main aquifer is drilled at the depth of 15.5–17.5 m; it is tight and stabilizes at the depth of about 5 m below ground level.

Moraine and lacustrine deposits of the Warta Glaciation are low plasticity clays. Grain size distribution of the tested soils (Figure 3) shows that according to EN ISO 14688-1 and 2 Standards [52,53], the cohesive soils are classified as silty sandy clays (sasiCl) and silty clays (siCl). The index properties of the tested soils for four undisturbed soil samples (No. 10–13) are shown in Table 2. The moraine and lacustrine clays are preconsolidated with overconsolidation ratio, *OCR*, increasing with depth from 3 to 7.

2.3.3. Stegny Site

The Stegny test site is located in the Vistula valley, on a floodplain terrace in the southern part of Warsaw. The cohesive soils tested occur in a glaciotectonically disturbed area and represent stiff Pliocene clays covered with fine and medium sands from the Würm-Vistula Glaciation period, with a thickness of 4.0–5.0 m. Pliocene clays are classified as high plasticity clays. The groundwater table was found at the depth of 3 m. Grain size distribution of the tested soils (Figure 3) shows that according to EN ISO 14688-1 and 2 Standards [52,53], the cohesive soils are classified as silty clays (siCl) and clays (Cl). The index properties of the tested soils for three undisturbed soil samples (No. 14–16) are shown in Table 2. The glacitectonically disturbed Pliocene clays are preconsolidated with the overconsolidation ratio, *OCR*, increasing with depth from 5 to 8.

3. Results

3.1. Results of Seismic Dilatometer Tests

Field investigations on test sites included seismic dilatometer tests and drillings with collecting of undisturbed soil samples by a Shelby sampler with an inner diameter of 89 mm. As part of the research work, 16 seismic dilatometer tests were made up to a depth of 9.5–16.0 m below ground level. The tests were carried out in nodes consisting of three to four SDMT, located in the vicinity of the borehole. The exceptions included studies carried out at the Bielany site, where the two drillings were completed with two SDMT. The seismic dilatometer test was performed in accordance with Marchetti's guidelines [7,8,19]. In order to generate the shear wave, a steel beam with a hammer construction that allows the hammer head to be easily hit against the beam's forehead was used. A hammer with a weight of approx. 15 kg and a beam with dimensions of 0.2 m × 0.7 m × 0.1 m were used. The instrument used to generate the waves in a ground was pressed by means of the probe's foot (Figure 4).

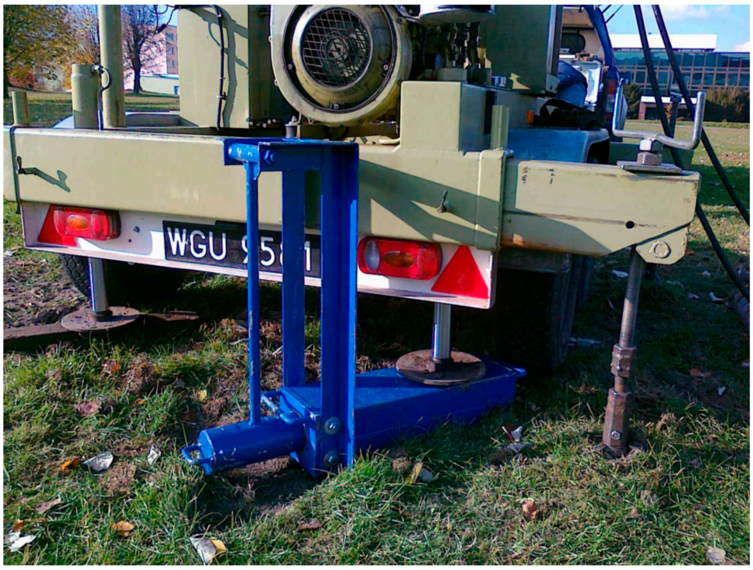


Figure 4. Steel beam with a hammer used to generate shear waves in SDMT.

SDMT data profiles including the corrected pressure readings p_o and p_1, shear wave velocity V_s as well as the index parameters I_D, K_D, and E_D from dilatometer tests obtained at the Ursynów, Bielany and Stegny sites, respectively, are presented in Figures 5 and 6. The cohesive soils from the Ursynów site showed that the values of the index parameters K_D and E_D decreased with depth. A greater variation of K_D and E_D values was obtained in brown moraine clays of the Warta Glaciation than in grey moraine clays of the Odra Glaciation. In the brown moraine clays of the Warta Glaciation the presence of cementation was detected. The index parameters K_D and E_D were increasing with depth in the moraine and lacustrine deposits of Warta Glaciation from the Bielany site and Pliocene clays from the Stegny site. The mean values, standard deviations and coefficients of variation of these readings and indexes are presented in Table 3. The average coefficients of variation for V_s, I_D, K_D, and E_D were 0.13, 0.43, 0.20, and 0.41, respectively. The dilatometer modulus E_D showed quite a high coefficient of variation. As an independent variable the shear wave velocity V_s showed quite a small coefficient of variation.

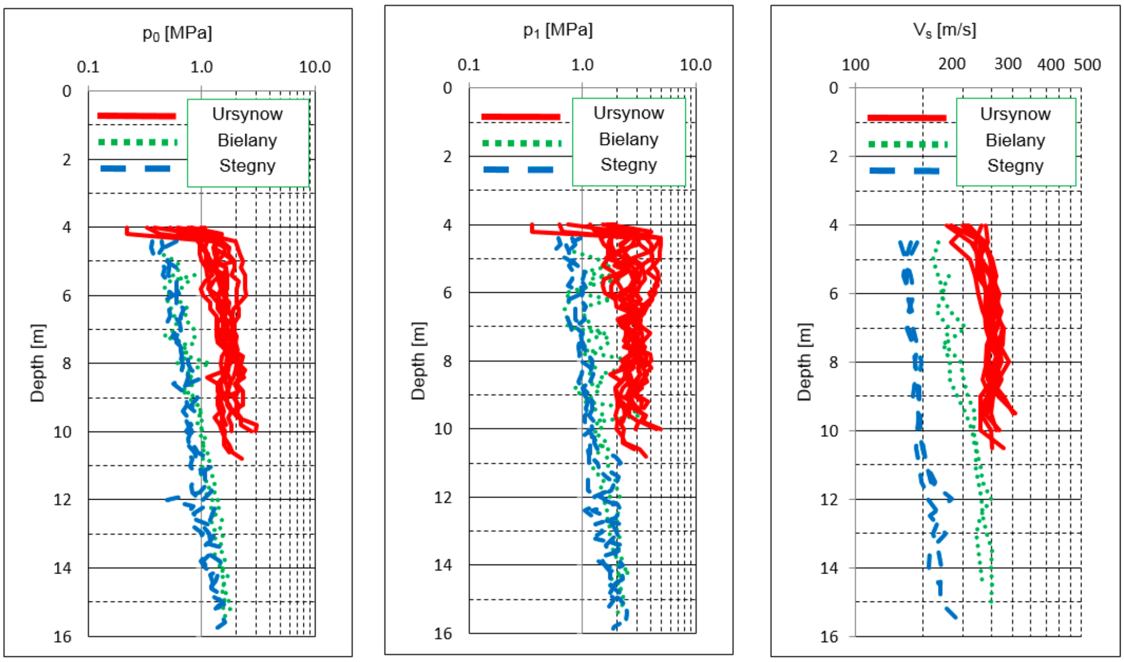


Figure 5. Profiles of the corrected pressure readings p_0 and p_1, and shear wave velocity V_s from seismic dilatometer tests obtained at the Ursynów, Bielany, and Stegny sites.

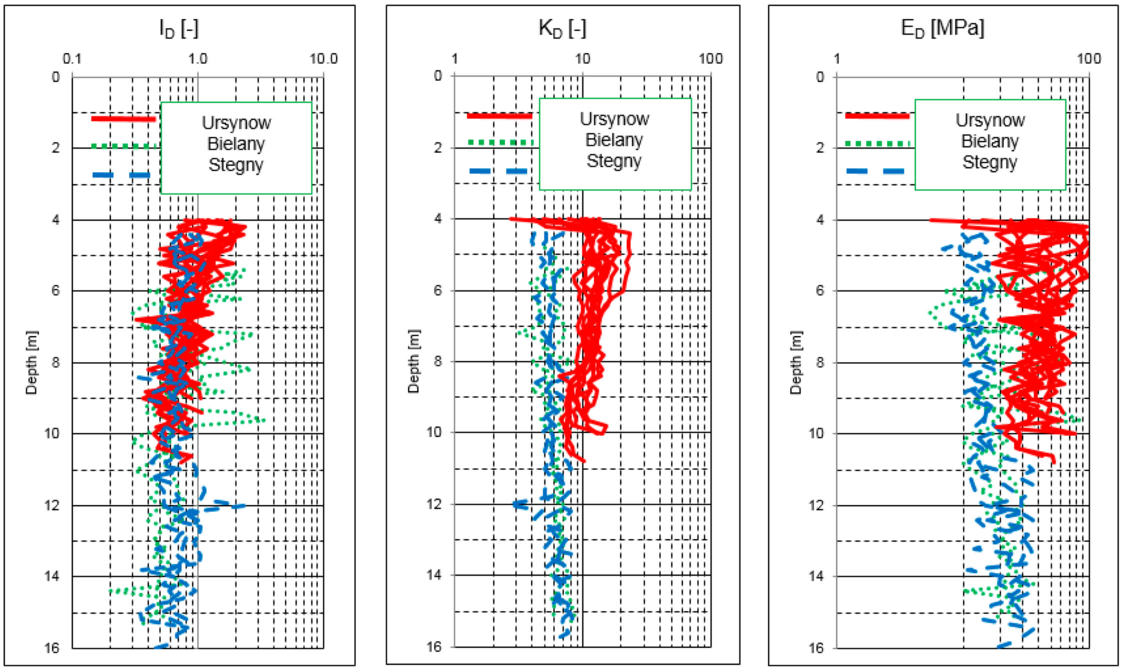


Figure 6. Profiles of the index parameters I_D, K_D, and E_D from seismic dilatometer tests obtained at the Ursynów, Bielany, and Stegny sites.

Table 3. Mean value, standard deviation, and coefficient of variation of: The corrected first pressure reading p_o or the corrected second pressure reading p_1, shear wave velocity V_s, material index I_D, horizontal stress index K_D, and dilatometer modulus E_D.

Site	Values	p_o (MPa)	p_1 (MPa)	V_s (m/s)	I_D (-)	K_D (-)	E_D (MPa)
Ursynów Auditorium	Mean value	1.363	2.569	395	0.93	11.1	42
	Standard Deviation	0.263	0.509	47.2	0.38	2.64	14.0
	Coefficient of Variation	0.19	0.20	0.12	0.41	0.24	0.33
Ursynów Building No. 37	Mean value	1.617	3.160	377	0.99	12.9	54
	Standard Deviation	0.420	0.924	40.5	0.46	4.04	23.7
	Coefficient of Variation	0.26	0.29	0.11	0.46	0.31	0.44
Ursynów Building No. 34	Mean value	1.370	2.503	397	0.84	11.3	39
	Standard Deviation	0.397	0.716	41.9	0.21	1.41	13.2
	Coefficient of Variation	0.29	0.29	0.10	0.24	0.12	0.33
Bielany	Mean value	0.972	1.610	320	0.78	6.1	22
	Standard Deviation	0.343	0.509	57.0	0.62	1.04	12.8
	Coefficient of Variation	0.35	0.32	0.19	0.78	0.17	0.58
Stegny	Mean value	0.810	1.337	196	0.71	5.9	18
	Standard Deviation	0.286	0.473	23.9	0.19	0.96	7.2
	Coefficient of Variation	0.36	0.36	0.12	0.28	0.16	0.38

3.2. Results of Triaxial Tests

The undrained shear strength τ_{fu} in cohesive soils from the Ursynów, Bielany, and Stegny sites was determined in CIU triaxial tests on isotropically consolidated samples with shearing in undrained conditions. The CIU triaxial tests were performed in four consecutive stages: Flushing, saturation, consolidation, and shearing in undrained conditions. Saturation of soil samples was performed using the back pressure method. Figure 7 presents the characteristics of the deviator stress q depending on the vertical strain ε_1 from CIU triaxial tests for the tested cohesive soils. Based on the deviator stress at failure q_f the undrained shear strength was determined as $\tau_{fu} = q_f/2$ which is shown in Table 4 and was used in statistical analysis.

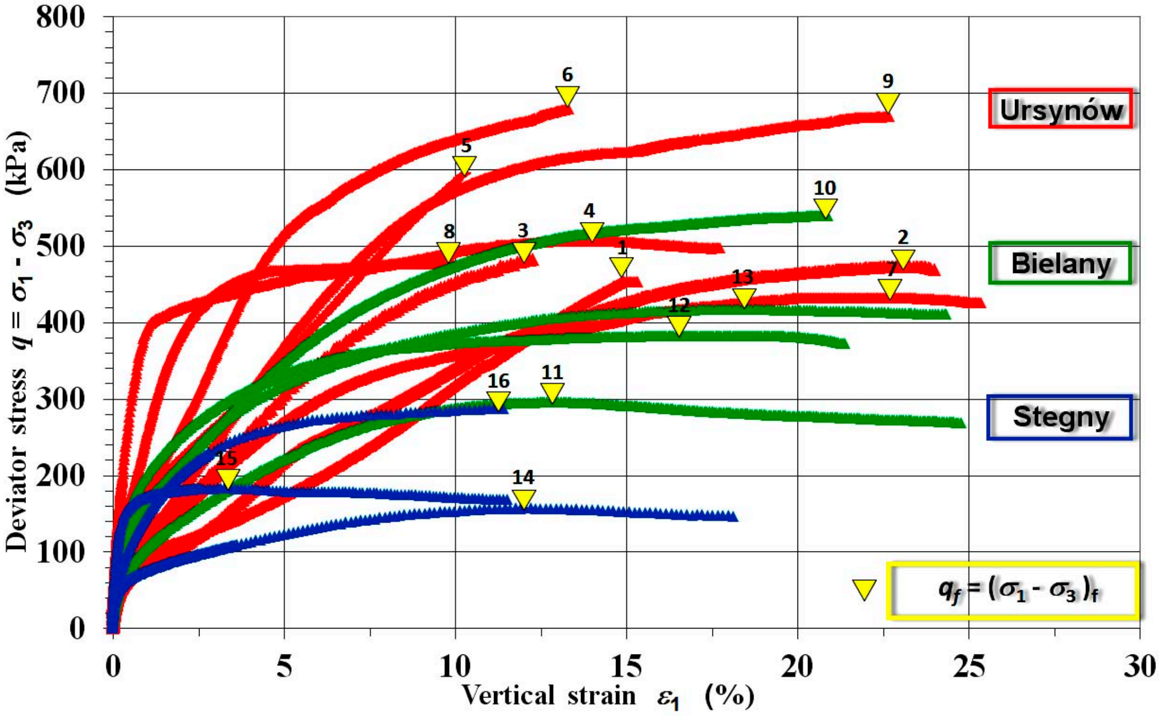


Figure 7. Characteristics of the deviator stress q depending on the vertical strain ε_1 from CIU triaxial tests for sixteen tested cohesive soils.

Table 4. Input and output values for the statistical analysis.

Site	Soil Type	σ'_v (MPa)	u_o (MPa)	p_o (MPa)	$(p_o - u_o)$ (MPa)	$(p_o - u_o)/\sigma'_v$ (-) X_1	p_1 (MPa)	$(p_1 - u_o)$ (MPa)	$(p_1 - u_o)/\sigma'_v$ (-) X_2	$(p_1 - p_o)/\sigma'_v$ (-) X_3	V_s (m/s)	$V_s/100$ (-) X_4	τ_{fu} * (MPa)	τ_{fu}/σ'_v * (-) Y
Ursynów Auditorium	clSa/sasiCl	0.094	0.0	1.286	1.286	13.739	2.544	2.544	27.179	13.440	388	3.880	0.223	2.382
	clSa	0.102	0.0	1.399	1.399	13.743	2.802	2.802	27.525	13.782	407	4.070	0.237	2.323
	clSa	0.135	0.0	1.545	1.545	11.461	2.999	2.999	22.248	10.786	424	4.240	0.240	1.780
Ursynów Building No. 37	clSa	0.124	0.0	1.662	1.662	13.361	3.074	3.074	24.707	11.346	384	3.840	0.254	2.038
	sasiCl	0.137	0.0	1.583	1.583	11.580	2.829	2.829	20.695	9.115	390	3.900	0.298	2.176
	sasiCl	0.170	0.0	1.631	1.631	9.594	2.604	2.604	15.318	5.724	398	3.980	0.340	1.997
Ursynów Building No. 34	clSa	0.120	0.0	1.322	1.322	11.017	2.501	2.501	20.842	9.825	421	4.210	0.216	1.800
	clSa/sasiCl	0.134	0.0	1.409	1.409	10.546	2.686	2.686	20.105	9.558	420	4.200	0.239	1.789
	sasiCl	0.155	0.0	1.945	1.945	12.548	3.592	3.592	23.174	10.626	395	3.950	0.335	2.158
Bielany	sasiCl	0.185	0.097	1.610	1.513	8.1960	2.267	2.170	11.755	3.559	398	3.980	0.270	1.463
	siCl	0.122	0.020	0.546	0.526	4.301	1.166	1.146	9.370	5.070	247	2.470	0.148	1.209
	siCl	0.143	0.034	0.746	0.712	4.989	1.350	1.316	9.222	4.233	283	2.830	0.192	1.342
	sasiCl	0.189	0.068	1.267	1.199	6.341	2.075	2.007	10.613	4.273	277	2.773	0.209	1.103
Stegny	Cl	0.092	0.020	0.536	0.516	5.609	0.930	0.910	9.891	4.283	175	1.750	0.079	0.853
	Cl/siCl	0.117	0.050	0.724	0.674	5.761	1.147	1.097	9.381	3.615	188	1.880	0.094	0.803
	Cl/siCl	0.142	0.080	0.873	0.793	5.585	1.528	1.448	10.197	4.613	229	2.290	0.144	1.011

Notes: * CIU TT—Triaxial Test on sample consolidated isotropically with shearing in undrained conditions.

4. Evaluation of the Undrained Shear Strength Based on Empirical Relationships

The variables from seismic dilatometer tests used in the evaluation of the undrained shear strength τ_{fu} are presented in Table 4.

In order to compare the undrained shear strength evaluated on the basis of empirical relationships with undrained shear strength from CIU triaxial tests obtained for a given depth, the measurements from seismic dilatometer tests were subjected to the averaging procedure. The applied procedure consisted of calculating the mean value of pressures p_o and p_1, shear wave velocity V_s, and index parameter K_D. The mean value for each selected depth came from three to four dilatometer tests (Bielany site is an exception) located in a given test node. Because CIU triaxial tests were made by using core samples of about 0.4 m in length, the mean was taken from three, nine, or twelve readings at a given depth and ±0.20 m.

Evaluation of the undrained shear strength τ_{fu} in preconsolidated cohesive soils from seismic dilatometer tests was performed using single- and multi-factor empirical relationships proposed by: Marchetti (4) [3], Roque et al. (6) [38], and Smith and Houlsby (7) [39], as well as the three-factor relationship of Rabarijoely [41] using Equation (9), the empirical coefficients determined by Lechowicz et al. [48] and the two-factor relationship of Galas [42] using Equation (10).

For each relationship, the maximum relative error max RE (Table 5) for the particular test site was selected from the values of relative errors RE calculated according to the formula:

$$RE = \left|\left(d^{(p)} - y^{(p)}\right)/d^{(p)}\right| \cdot 100\% \tag{11}$$

where p is the case number, $p \in \{1, \ldots, P\}$, P is the number of cases, $d^{(p)}$ is the measured value, and $y^{(p)}$ is the calculated value.

Table 5. The calculated maximum relative error max RE of the evaluated undrained shear strength τ_{fu} in cohesive soils from the Ursynów, Bielany, and Stegny sites.

		Single-Factor Relationships by:			Multi-Factor Relationship by:	
		Marchetti [3] Equation (4) max RE (%)	Roque et al. [38] Equation (6) max RE (%)	Smith and Houlsby [39] Equation (7) max RE (%)	Rabarijoely [41,51] Equation (9) max RE (%)	Galas [42] Equation (10) max RE (%)
Ursynów	A	34.3	14.7	11.7	30.9	16.4
	37	36.7	30.3	21.0	47.7	21.1
	34	22.8	6.0	2.8	42.5	8.6
Bielany		43.9	47.6	43.2	35.9	33.9
Stegny		21.3	7.4	24.5	40.5	28.7
Mean max RE		31.8	21.2	20.6	39.5	21.7
Mean RE (from 16 tests)		20.4	14.1	12.0	30.6	12.9

Using the max RE obtained for five locations the mean value of max RE was calculated for each relationship. Based on all test sites the mean value of the RE was also calculated for a particular relationship (Table 5).

Analysis of the calculation results indicated quite high values of max RE obtained for the undrained shear strength evaluated on the basis of the single-factor empirical relationship of Marchetti ranging between 21.3–43.9%. The max RE values calculated for τ_{fu} evaluated from the Roque et al. relationship with the bearing capacity factor N_C determined for Pleistocene and Pliocene clays ranged between 6.0–47.6%. The max RE values calculated for undrained shear strength evaluated from the Smith and Houlsby relationship with the bearing capacity factor N_D determined for Pleistocene and Pliocene clays were in the range of 2.8–43.2%. The mean values of the max RE for the Marchetti, Roque et al., and Smith and Houlsby relationships were 31.8, 21.2, and 20.6%, respectively. The mean values of the relative error *RE* (from 16 tests) for the Marchetti, Roque et al., and Smith and Houlsby relationships were 20.4, 14.1, and 12.0%, respectively.

The comparison carried out for the three-factor relationship of Rabarijoely, for which the empirical coefficients were determined for Pleistocene and Pliocene clays, indicated quite high max *RE* values of the τ_{fu} in the range of 30.9–47.7%. The mean value of max *RE* was 39.5% and the mean values of relative error *RE* (from 16 tests) was 30.6%. Using the two-factor relationship of Galas, the max *RE* values of the τ_{fu} ranged between 8.6–33.9%, the mean value of max *RE* was 21.7% with the mean value of relative error equal to 12.9%. This showed that the two-factor relationship of Galas gives a better accuracy compared to the Rabarijoely relationship.

This evidences that the evaluation of the undrained shear strength from dilatometer tests for the Pleistocene and Pliocene clays from the analyzed relationships resulted in quite high max *RE* values exceeding 34%. However, in the case of the Roque et al., Smith and Houlsby, and Galas relationships, the mean values of the max *RE* showed better accuracy and were at about 21%. From among the single-factor empirical relationships the Smith and Houlsby relationship and from among the multi-factor relationships the Galas relationship gave better accuracy with the mean value of the relative error *RE* at about 12% and 13%, respectively.

5. Statistical Analysis

Statistical analysis of the SDMT results was carried out using the Statistica software version 12 [54–56]. Models of linear and nonlinear regression were analyzed [57]. The analyzed data set comprised the P = 16 pattern described by one dependent variable the normalized undrained shear strength $Y = \tau_{fu}/\sigma'_v \in \{0.803–2.382\}$, and four independent variables: The normalized net value of the corrected first pressure reading $X_1 = (p_o - u_o)/\sigma'_v \in \{4.301–13.743\}$; the normalized net value of the corrected second pressure reading $X_2 = (p_1 - u_o)/\sigma'_v \in \{9.222–27.525\}$; the normalized difference between corrected second pressure reading and the corrected first pressure reading $X_3 = (p_1 - p_o)/\sigma'_v \in \{3.559–13.782\}$; and the normalized shear wave velocity $X_4 = V_s/100 \in \{1.750–4.240\}$, selected from the variables presented in Table 4.

Simple regression models were considered:

$$y = a + bx \pm SEE \tag{12}$$

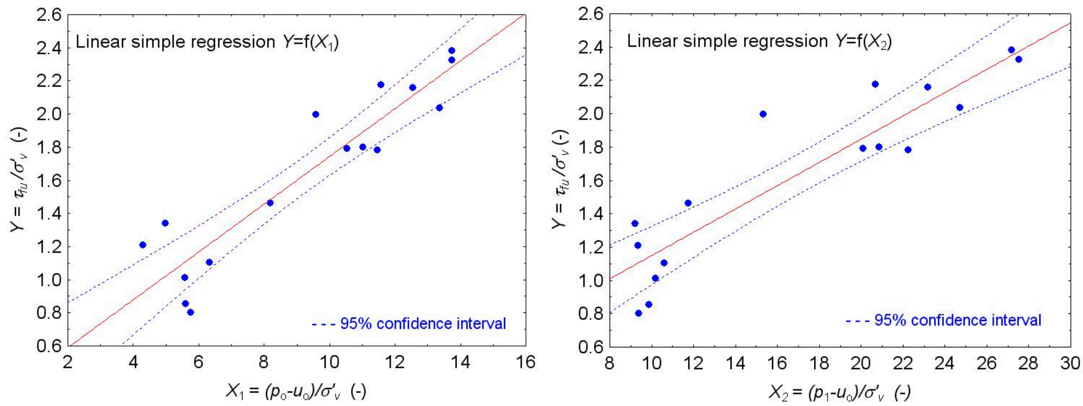
where a is intercept of line, b is slope of line, and SEE is the standard error of estimation.

In the case when measurement of the shear wave velocity V_s was not carried out, the τ_{fu}/σ'_v can be evaluated using simple linear regressions shown as Equation (13) or Equation (14), which explain 85.5% and 83.1% of the variability of $Y = \tau_{fu}/\sigma'_v$, respectively (Table 6). Plots of regression lines (13) and (14) with 95% confidence intervals are given in Figure 8. The simple linear regression models (15) and (16) shown in Table 6 gave worse accuracy.

Table 6. Comparison of the analyzed simple linear regression models.

Model	Equation Formula/No.	Determination Coefficient R^2 (-)	*SEE* (MPa)	Max *RE* (%)	Mean *RE* (%)
Simple linear regressions	$\tau_{fu}/\sigma'_v = 0.3023 + 0.1442((p_o - u_o)/\sigma'_v)$ (13)*	0.855	±0.2089	41.0	12.4
	$\tau_{fu}/\sigma'_v = 0.4513 + 0.0698((p_1 - u_o)/\sigma'_v)$ (14)**	0.831	±0.2262	37.7	13.0
	$\tau_{fu}/\sigma'_v = 0.6652 + 0.1258((p_1 - p_o)/\sigma'_v)$ (15)**	0.751	±0.2747	39.4	15.0
	$\tau_{fu}/\sigma'_v = -0.1159 + 0.5178(Vs/100)$ (16)*	0.757	±0.2714	33.0	12.0

Notes: *—coefficient a is statistically non-significant, **—all coefficients are statistically significant.



(**a**) (**b**)

Figure 8. Linear simple regression models with 95% confidence intervals: (**a**) Y = f(X1), (**b**) Y = f(X2).

The linear two-factor regression model was in the form of:

$$y = a_o + a_1 x_1 + a_2 x_2 \pm SEE \tag{17}$$

where a_0, a_1, and a_2 are regression coefficients.

The non-linear two-factor regression model was adopted based on the analysis of the curve shape and scatterplots of variables. Among the linearized models, i.e., those that can be reduced to a linear form by appropriate transformation of variables or parameters, the power function of the form was selected:

$$y = \beta_o \cdot {x_1}^{\beta_1} \cdot {x_2}^{\beta_2} \tag{18}$$

Logarithmic transformation was applied and a transformed regression model was obtained:

$$log\ y = log\ \beta_o + \beta_1 \cdot log\ x_1 + \beta_2 \cdot log x_2 \tag{19}$$

The non-linear two-factor regression was considered as the best model. The fulfilment of statistical assumptions (e.g., normality of residual distribution and residuals analysis) was checked.

A non-linear two-factor relationship to evaluate the normalized undrained shear strength τ_{fu}/σ'_v of preconsolidated cohesive soils based on seismic dilatometer test in a general form is proposed as follows:

$$\frac{\tau_{fu}}{\sigma'_v} = \beta_o \cdot \left(\frac{p_1 - p_o}{\sigma'_v}\right)^{\beta_1} \cdot \left(\frac{Vs}{100}\right)^{\beta_2} \tag{20}$$

where β_o, β_1, and β_2 are the empirical coefficients. The empirical coefficients to Equation (20) for Pleistocene clays and Pliocene clays were evaluated by statistical analysis using the input and output data shown in Table 4. The empirical coefficients of the linear two-factor relationship (Equation (21)) and non-linear two-factor relationship (Equation (22)) are presented in Table 7. Graphic presentation of the proposed relationship is shown in Figure 9.

Table 7. Comparison of the analyzed two-factor regression models.

Model	Equation Formula/No.	Determination Coefficient R^2 (-)	*SEE* (MPa)	Max *RE* (%)	Mean *RE* (%)
Linear multiple two-factor regression	$\tau_{fu}/\sigma'_v = 0.0403 + 0.0728((p_1 - p_o)/\sigma'_v) + 0.3055(Vs/100)$ (21)*	0.881	±0.1972	19.1	8.5
Non-linear multiple two-factor regression	$\boldsymbol{\tau_{fu}/\sigma'_v = 0.3676 \cdot ((p_1 - p_o)/\sigma'_v)^{0.2846} \cdot \cdot (Vs/100)^{0.7525}}$ (22)**	0.919	±0.0480	20.4	8.5

Notes: *—coefficient *a* is statistically non-significant, **—all coefficients are statistically significant.

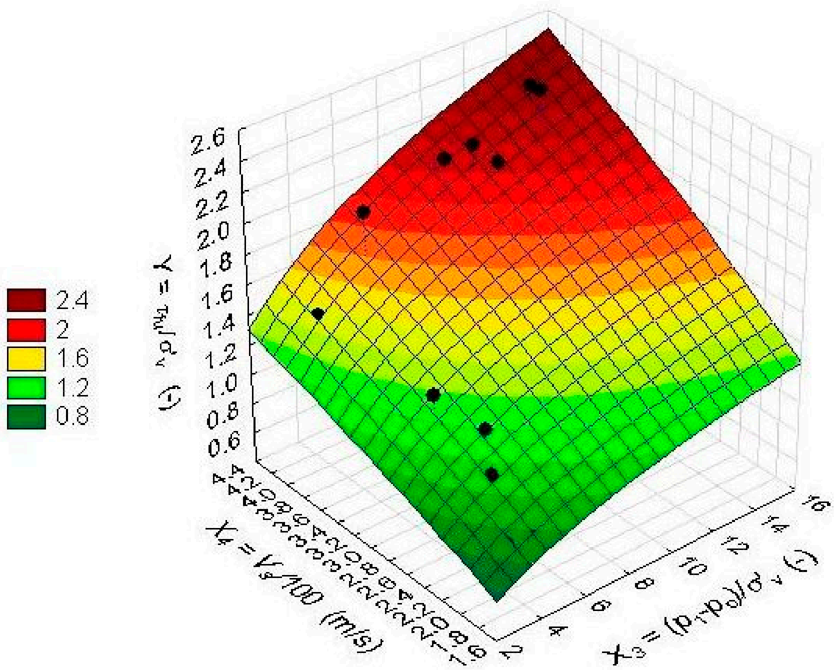


Figure 9. Graphic presentation of the proposed relationship (Equation (22)).

The proposed non-linear two-factor relationship (Equation (22), Table 7) was considered the best of all analyzed relationships. The accuracy of the prediction using the proposed relationship (Equation (22)) is illustrated by a graph of the dependence of the measured τ_{fu}/σ'_v values and τ_{fu}/σ'_v values predicted by the proposed relationship (Equation (22), Figure 10). The maximum relative prediction error max *RE* of the proposed relationship for τ_{fu}/σ'_v based on all the data was ±20.4%. A comparison between measured values of τ_{fu}/σ'_v obtained from the CIU triaxial tests and values of τ_{fu}/σ'_v evaluated using the proposed relationship (Equation (22)) are shown in Table 8. The mean value of the max *RE* was about 13.2% and the mean value of the relative error *RE* for all data was about 8.5%.

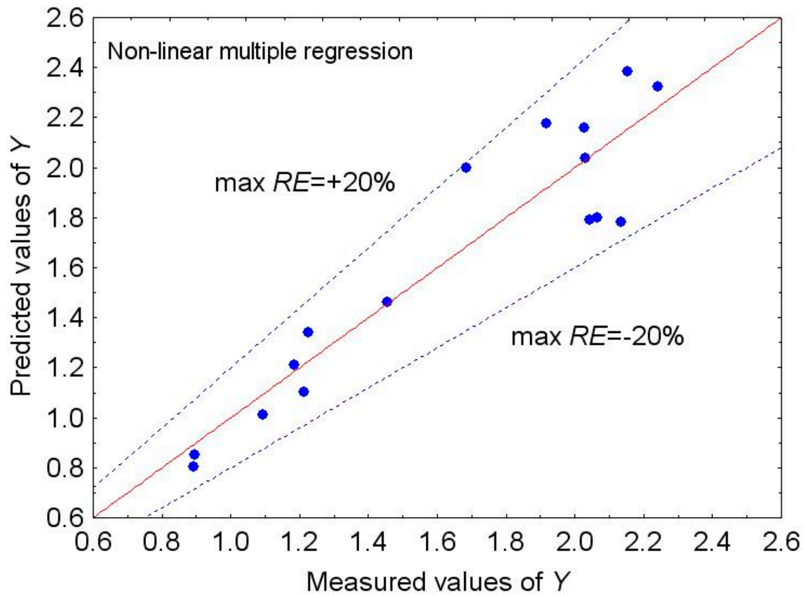


Figure 10. Comparison between measured values of the normalized undrained shear strength τ_{fu}/σ'_v and values predicted by the proposed relationship (Equation (22)).

Table 8. Comparison between measured values of τ_{fu}/σ'_v obtained from the CIU triaxial tests and values of τ_{fu}/σ'_v predicted using the proposed relationship (Equation (22)). Bold number shows the maximum value of *RE* for given site

Sites		Measured Values τ_{fu}/σ'_v from CIU TT (-)	Predicted Values τ_{fu}/σ'_v Using Equation (22) (-)	Relative Errors of Individual Case RE_p (%)
Ursynów	A	2.382	2.134	10.4
		2.323	2.228	4.1
		1.780	2.143	20.4
	37	2.038	2.018	1.0
		2.176	1.918	11.9
		1.997	1.706	14.6
	34	1.800	2.076	15.3
		1.789	2.056	14.9
		2.158	2.023	6.2
Bielany		1.463	1.490	1.9
		1.209	1.149	4.9
		1.342	1.210	9.8
		1.103	1.196	8.5
Stegny		0.853	0.847	0.8
		0.803	0.851	6.0
		1.011	1.059	4.7
				Max RE_p = 20.4%
			The mean of the marked values	Mean Value of Max *RE* = 13.2%
				Mean Value of *RE* = 8.5%

6. Conclusions

The analysis carried out for Pleistocene and Pliocene clays indicates that the evaluated undrained shear strength τ_{fu} from the dilatometer test on the basis of single-factor empirical relationships: Marchetti, Roque et al., and Smith and Houlsby gives quite high values of the maximum relative error max *RE* from 43% up to 48%. The two-factor relationship of Galas presented somewhat smaller values of the max *RE* up to 34%. From the single-factor empirical relationships the Smith and Houlsby relationship and from the multi-factor relationships the Galas relationship gave better accuracy with the mean value of relative error *RE* at about 12% and 13%, respectively. So far the shear wave velocity V_s as an independent variable was not used to evaluate the undrained shear strength from the dilatometer test.

The normalized undrained shear strength τ_{fu}/σ'_v in Pleistocene and Pliocene clays from the seismic dilatometer test (SDMT) can be evaluated using the proposed two-factor relationship based on the normalized difference between the corrected second pressure reading and the corrected first pressure reading $(p_1 - p_o)/\sigma'_v$, and the normalized shear wave velocity $V_s/100$. The proposed relationship provides the evaluation of the normalized undrained shear strength τ_{fu}/σ'_v in Pleistocene and Pliocene clays from the seismic dilatometer test with a maximum relative error at about ±20% and with the mean value of relative error *RE* at about 8%. Further research is needed for different types of cohesive soils with a higher variability of the plasticity index I_P, consistency index I_C, and stress history to verify empirical coefficients determined in this study.

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