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Estimation of the undrained shear strength of Adapazari fine grained soils by cone penetration test

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ABSTRACT

A comprehensive study to determine the undrained shearing resistance of fine grained soils by the use of cone penetration test (CPTu) was carried out on the thick fluvial sediments of the plain of Adapazari, Turkey. The soundings were performed adjacent to geotechnical boreholes for comparison. Eighty-three undisturbed samples of low, intermediate and high plasticity were procured in the process, which were classified and tested in unconsolidated undrained triaxial (UU) conditions in the laboratory. The cone resistance, sleeve friction and pore water pressure values recorded along the length of undisturbed sampling were averaged to obtain characteristic values. It is known that the empirical formulae to interpret CPT readings use cone factors N_k, N_{kt} and N_{ke} which can be highly variable. This can lead to unacceptable error levels in the assessment of undrained shear strength (s_u). Nevertheless, it was noticed in this study that the undrained strengths estimated by employing the traditional relationships for CPT came out quite close to the values measured in the laboratory if soil classes are taken into account. Since soil sampling is not done in the cone penetration test by default, classification of the soil is attempted by the use of the soil type behavior index (I_c). This study showed that the error margins for s_u values measured in the laboratory and inferred from CPT are considerably reduced by using the soil type behavior index (I_c) to determine soil behavior type specific cone factors.

1. Introduction

There is considerable demand to obtain the undrained shear strength (s_u) of fine grained soils in many geotechnical problems, because it is a fundamental property. The bearing capacity of a shallow foundation, stability analyses of slopes and embankments, shaft and tip resistance of drilled shafts and driven piles are calculated using this parameter. The analyses compare the shearing resistance of the soil with the actual loads imposed on the system. Undrained shear strength can be measured by a variety of in situ testing devices as well as laboratory. The related field tests are the standard penetration test (SPT) (Sowers, 1954; Terzaghi and Peck, 1967; Stroud, 1974; Sowers, 1979), cone penetration test (CPT) (Lunne et al., 1997), flat dilatometer test (DMT) (Marchetti, 1980; Tanaka and Tanaka, 1996; Młynarek et al., 2018) and the pressuremeter test (PBT) (Gibson and Anderson, 1961; Marshland and Randolph, 1977). Strength is measured by various tests in the laboratory ranging from unconfined compression to the complicated hollow cylinder. However, the most popular laboratory tests to evaluate undrained shear strength of fine grained soil are the triaxial (TX) and unconfined compression (UC) tests.

The cone penetration test (CPT, CPTu), which was developed in the 30s, has found increasing use in recent years to determine the physical, mechanical and dynamic properties of soils, due to its rapid execution and ability to record the soil profiles continuously. With the aid of developing technology, the properties such as resistivity, chemistry of ground water, its pH, and the temperature can also be measured with this equipment.

The aim of this paper is to test and to optimize the formulation of the correlation of CPTu results and undrained shear strength values measured by means of unconsolidated undrained triaxial tests (UU). As mentioned by Lunne et al. (1997), there are theoretical solutions and empirical approaches to evaluate undrained shear strength (s_u) from cone penetration data in the literature. Theoretical solutions such as cavity expansion theory (Skempton, 1951; Vesic, 1972; Vesic, 1975), strain path theory (Baligh, 1985; Teh, 1987) and numerical approaches (Ladanyi, 1967) take the undrained modulus (E_u), and the stiffness index (I_r) into account. Other theoretical approaches are similar to the calculation method based on the theory of bearing capacity of footings (Terzaghi, 1943; de Beer, 1977). The general bearing capacity equation is expressed as

$$q_c = N_c s_u + \sigma_{vo} \tag{1}$$

where σ_{vo} is the in situ total stress, q_c bearing capacity of the cone and N_c a theoretical cone factor representing the bearing capacity factor.

There are several relationships developed to estimate undrained

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Table 1
Laboratory and in-situ test data.

ID	Depth (m)	w_L	w_P	w_n	FC%	CLASS	e_0	γ_n (kN/m^3)	S_r (%)	s_u (kPa)	GWT (m)	q_c (MPa)	f_s (MPa)	u_2 (MPa)
1	2.50	86	32	43	99	CH	1.17	17.20	97	69	2.00	0.946	0.033	0.011
2	2.50	82	27	30	99	CH	1.34	17.40	100	48	0.85	0.540	0.031	−0.007
3	10.00	82	26	46	100	CH	1.20	17.07	95	71	1.50	0.996	0.050	0.137
4	3.00	81	25	38	99	CH	1.09	17.82	100	70	1.80	1.049	0.053	0.043
5	2.50	79	33	46	100	MH	1.37	17.00	100	34	1.58	0.728	0.029	0.070
6	2.50	76	33	50	81	CH	1.37	17.00	100	40	2.30	0.657	0.026	0.008
7	2.50	74	32	58	99	CH	1.37	17.41	100	36	2.35	0.661	0.019	−0.001
8	2.50	74	25	45	98	CH	1.23	17.30	99	50	2.80	0.508	0.029	0.033
9	3.95	74	27	39	100	CH	1.23	17.81	100	76	1.70	0.788	0.062	0.031
10	9.00	74	29	41	100	CH	1.69	17.80	100	82	1.70	1.010	0.081	0.008
11	4.00	73	27	41	100	CH	1.13	17.38	97	77	0.80	0.901	0.051	−0.032
12	2.50	71	30	36	100	CH	1.20	17.70	100	47	1.80	0.804	0.033	−0.001
13	3.00	71	31	37	99	CH	1.02	18.20	100	84	0.80	0.978	0.064	0.079
14	2.40	70	27	44	100	CH	0.95	18.32	99	32	1.50	0.504	0.010	−0.010
15	7.64	70	27	47	99	CH	1.24	17.12	100	46	3.40	0.804	0.021	−0.045
16	4.00	69	24	36	98	CH	0.95	18.20	100	72	2.50	1.054	0.026	0.016
17	2.50	69	28	43	100	CH	1.24	17.30	99	48	2.00	0.822	0.032	0.000
18	2.50	69	29	54	99	CH	1.28	16.99	100	40	1.40	0.367	0.022	0.004
19	2.50	68	26	43	99	CH	1.18	17.70	100	51	1.80	0.683	0.034	0.005
20	3.00	68	25	46	99	CH	1.19	17.80	100	32	0.40	0.487	0.024	0.002
21	2.50	65	31	42	94	CH	1.17	17.20	98	50	0.50	0.630	0.054	0.058
22	10.00	64	22	36	97	CH	1.47	18.00	100	62	2.80	0.856	0.044	0.035
23	2.50	62	23	49	98	CH	1.57	17.06	97	22	2.25	0.520	0.015	0.040
24	2.50	61	23	38	100	CH	1.01	17.90	100	72	0.40	0.713	0.042	0.039
25	10.00	61	22	40	99	CH	1.05	18.30	99	67	2.10	0.967	0.050	0.099
26	2.50	59	28	41	100	CH	1.01	17.18	95	44	1.00	0.612	0.043	0.038
27	2.50	58	26	44	94	CH	1.09	17.57	100	44	0.65	0.687	0.033	−0.038
28	2.50	55	19	28	99	CH	0.85	18.25	91	74	3.60	0.841	0.047	0.032
29	8.50	52	23	39	100	CH	1.12	17.60	99	44	0.85	1.016	0.036	0.014
30	2.50	52	20	38	93	CH	1.17	18.00	99	41	2.70	0.450	0.020	0.018
31	4.50	52	26	43	100	CH	1.12	17.72	100	40	1.80	0.653	0.026	0.021
32	2.50	51	26	41	96	CH	1.15	17.40	98	31	1.00	0.569	0.012	−0.016
33	2.50	51	26	38	99	CH	1.31	17.06	99	23	0.60	0.519	0.022	0.053
34	4.00	50	23	39	96	CH	1.03	17.90	100	41	0.70	0.634	0.030	0.035
35	3.00	50	22	29	95	CH	0.98	18.11	100	55	2.30	0.691	0.036	0.039
36	2.50	49	22	39	99	CI	1.04	18.08	100	32	1.85	0.731	0.024	0.011
37	7.00	49	26	36	98	CI	1.11	17.15	97	38	1.00	0.849	0.022	−0.024
38	3.10	49	22	45	99	CI	1.23	16.78	93	39	1.18	0.637	0.018	−0.014
39	2.50	48	25	45	99	CI	1.10	17.59	95	35	0.60	0.645	0.020	−0.025
40	2.50	48	22	33	97	CI	1.02	17.91	99	46	1.65	0.829	0.036	0.009
41	6.00	47	22	36	90	CI	0.88	18.75	100	54	2.00	1.100	0.037	−0.077
42	2.50	47	20	36	89	CI	0.91	18.26	100	54	1.60	0.475	0.020	−0.026
43	7.85	46	22	40	100	CI	1.09	17.71	99	65	1.18	1.135	0.030	−0.019
44	6.60	45	27	42	100	MI	1.07	17.64	98	78	2.40	0.911	0.058	0.089
45	8.50	44	29	36	99	MI	0.90	18.60	99	140	1.45	2.084	0.082	−0.030
46	10.00	44	20	30	96	CI	0.81	18.86	99	100	0.50	1.037	0.036	0.088
47	8.45	44	20	36	98	CI	0.95	18.33	100	41	1.10	0.757	0.030	0.074
48	6.00	43	21	26	72	CI	1.01	17.68	95	51	1.65	1.081	0.046	0.237
49	2.50	42	17	39	95	CI	1.17	17.40	99	38	1.10	0.485	0.024	0.066
50	6.00	42	27	38	95	MI	0.94	18.10	98	87	2.50	1.017	0.052	0.062
51	2.50	42	19	36	99	CI	0.90	19.05	100	52	1.95	0.484	0.023	0.011
52	2.90	42	17	38	100	CI	1.00	17.73	94	46	1.10	0.737	0.016	0.002
53	2.50	41	18	35	92	CI	0.92	17.91	93	41	1.90	0.889	0.049	−0.008
54	2.50	41	18	35	95	CI	0.94	17.80	95	47	2.20	0.446	0.019	0.002
55	4.00	41	25	35	96	CI	1.11	17.83	100	31	2.20	0.535	0.027	−0.054
56	3.90	41	26	45	98	MI	1.09	17.26	92	44	2.40	0.565	0.026	−0.001
57	3.04	40	19	36	97	CI	0.96	18.29	100	38	1.10	0.641	0.025	−0.038
58	10.00	39	19	33	91	CI	0.96	18.59	100	95	1.70	1.012	0.039	−0.034
59	2.50	37	25	35	70	MI	0.95	18.14	98	40	2.00	0.811	0.031	0.057
60	2.50	37	18	36	97	CI	1.04	17.50	98	30	0.80	0.616	0.025	0.019
61	11.50	35	22	33	99	CI	1.03	17.76	100	76	0.60	1.772	0.078	0.013
62	2.50	35	26	37	93	MI	0.92	18.37	100	41	0.90	0.580	0.027	0.073
63	6.12	35	22	32	98	CI	0.82	18.99	99	63	1.10	1.386	0.045	−0.080
64	2.50	34	NP	35	97	ML	0.91	18.77	100	59	2.10	1.546	0.014	−0.035
65	4.00	34	NP	41	66	ML	0.77	19.72	100	87	1.90	2.236	0.042	0.051
66	3.50	34	22	28	74	ML	0.93	18.30	100	41	1.30	1.052	0.015	−0.022
67	3.00	34	21	36	99	CL	0.92	18.34	100	71	2.70	1.244	0.039	0.008
68	2.50	33	NP	34	96	ML	0.94	18.14	97	48	2.20	1.816	0.055	0.017
69	2.50	32	NP	36	80	ML	0.95	18.43	100	33	1.00	1.110	0.030	0.048
70	1.30	32	20	34	78	CL	0.84	18.85	100	44	1.10	1.665	0.008	−0.012
71	2.77	32	18	37	93	CL	0.92	18.46	100	26	0.50	0.813	0.006	−0.079
72	4.50	32	23	38	92	CL	1.17	17.49	100	23	0.50	0.802	0.004	−0.065
73	2.50	31	NP	35	98	ML	0.89	19.10	100	62	0.40	1.343	0.032	0.017
74	6.05	31	24	30	96	ML	0.98	18.45	100	77	1.18	2.418	0.048	−0.082

(continued on next page)

Table 1 (*continued*)

ID	Depth (m)	w_L	w_P	w_n	FC%	CLASS	e_0	γ_n (kN/m^3)	S_r (%)	s_u (kPa)	GWT (m)	q_c (MPa)	f_s (MPa)	u_2 (MPa)
75	2.05	31	21	35	94	CL	0.87	18.43	97	57	0.50	1.710	0.007	−0.077
76	2.50	30	NP	33	88	ML	0.82	19.50	100	38	0.70	1.459	0.030	0.023
77	10.00	30	20	26	94	CL	0.73	19.04	97	60	1.60	1.214	0.031	0.019
78	2.00	29	NP	28	82	ML	0.84	18.96	98	28	1.35	0.778	0.001	−0.012
79	2.50	28	NP	39	76	ML	0.90	18.50	99	30	1.80	0.591	0.011	−0.004
80	2.50	27	NP	34	88	ML	0.88	19.06	100	39	2.30	1.379	0.019	−0.011
81	2.50	NP	NP	37	76	ML	0.84	19.40	98	35	2.10	0.915	0.003	0.014
82	2.50	NP	NP	30	81	ML	0.68	19.59	100	35	2.00	1.016	0.009	−0.033
83	0.20	NP	NP	30	55	ML	0.73	19.42	100	49	5.00	1.440	0.030	0.001

w_L: liquid limit, w_P: plastic limit, w_n: natural water content, FC: fines content, CLASS: soil class according to TS1500 (2000), e_0: void ratio, γ_n: unit weight, S_r: saturation ratio, s_u: undrained shear strength, GWT: ground water table, q_c: cone resistance, f_s: sleeve friction, u_2: pore pressure behind cone.

Table 2
Variations in N_k values with reference to the type of laboratory test performed.

Researcher/s	Soil type	Reference test/s	N_k
Sanglerat (1972)	Normally consolidated clays		15–21
	Stiff clays		22–26
Anagnostopoulos (1974)	Soft silty clays	UU	17
Kjekstad et al. (1978)	Non-fissured overconsolidated clays	UU	17
Lunne and Kleven (1981)	Normally consolidated marine clays	FVT	11–19
Tumay et al. (1982)	Mississippi deltaic deposits		6.7
Meigh (1987)	Normally consolidated clays		15–21
	Stiff fissured overconsolidated clays		27 ± 3
Zervogiannis et al. (1987)	Alluvial deposits	UU	18
Nevels Jr (1989)	Alluvial soils, highly plastic, firm to stiff	UU	39
	Clays tends to be come soft with depth	PMT	34
Stark and Delashaw (1990)	Non-fissured normally to lightly OC clays	UU	8.5–16.5
Koukis et al. (1997)	Soft silty clays, in Patras, Greece	UU	19
Eid and Stark (1998)	Soft to stiff, saturated glacial clays	UCS	15.5
Jörß (1998)	Marine clays		20
	Boulder clays		15
Chen (2001)	Three test sites in Malaysia		5–12
Sabatini et al. (2002)	Beaumont Formation-Houston Area	UU, CU, DMT	19
	Montgomery Formation-Houston Area	UU, CU, DMT	23
Ricceri et al. (2002)	Venice lagoon soils	UU	11–25
Anagnostopoulos et al. (2003)	Alluvial deposits of Greece	UU vs electrical cone	17.2
	Alluvial deposits of Greece	UU vs mechanical cone	18.9
Gebreselassie (2003)	For different soil types	–	7.6–28.4
Önalp et al. (2006)	Adapazarı NL/lightly over consolidated clays	UU	14.55
Hong et al. (2010)	Busan clay, Korea (25% < I_p < 40%)	TX	7–20
Almeida et al. (2010)	High plasticity, soft clay, (42% < I_p < 400%)	FVT	4–16
Rémai (2013)	Soft Holocene clays, Hungary	UC, CU	18.6

FVT: field vane test, PMT: pressuremeter test, DMT: dilatometer test.

shear strength from cone penetration parameters. These relationships can be classified into four main categories based on total cone resistance, net cone resistance, effective cone resistance and excess pore water pressure. The equations resulting from these approaches are similar to the theoretical solution of Terzaghi (1943).

Investigations that have focused on the estimation of these empirical factors in order to define the undrained shear strength s_u are numerous (Keaveny and Mitchell, 1986; Konrad and Law, 1987; Yu and Mitchell, 1998). There is no consensus among the researchers on the value of the factors witnessed by large scatter. Kulhawy and Mayne (1990) studied the calibration procedures for cone factors in the laboratory and stated that there are several factors that affect the value of empirical cone factors such as testing and sampling methods, direction of loading, strain rate, boundary conditions, stress level as well as disturbance effects. On the other hand, there are several tests for measuring the undrained shear strength in the laboratory such as unconfined, triaxial and vane whose results can be employed for comparison with the CPT data (Jamiolkowski et al., 1985; Kulhawy and Mayne, 1990).

Undrained shear strength of normally consolidated clays is measured by using the unconsolidated undrained triaxial test (UU) (Bishop and Henkel, 1962), laboratory vane test (Blight, 1968; Matsui and Abe, 1981; Chandler, 1988), pocket penetrometer (ASTM WK27337, 2010) and falling cone (Tanaka et al., 2012). The UU test (quick) (Head and Epps, 2011), which is considered ideal for saturated normally consolidated clays, was employed in this study. At least two samples are tested at different confining pressures in this test. When the sample is removed from the UD tube, it is under an isotropic negative stress ($\sigma' = -u_e$) (Holtz et al., 2011). It retains its integrity after extraction under the effect of this negative pore water pressure, which enables it to 'stand'. Under the applied cell pressure, the isotropic total stress ($\sigma_{cell} = \sigma_3$) will be resisted by the pore water pressure generated: $\sigma_{cell} = \sigma_3 = \Delta u_w$. In the UU test, consolidation of the sample is not allowed and shearing is carried out immediately upon raising of the confining pressure in undrained condition and the deviator stress causing the yield of the sample is measured. The test is repeated at a higher cell pressure. Although the cell pressures in consecutive tests are increased, the effective stress remains constant at $-u_e$. Consequently, the undrained strength will be independent of the cell pressure since it is a function of the effective stress. Accordingly, if three specimens are tested at different cell pressures, three stress circles having with identical diameters will be obtained providing a horizontal failure envelope (Skepton, 1948). The expression for shearing resistance (s_u) expressed as;

$$s_u = c_u + \sigma \tan \phi_u \quad (2)$$

where c_u is the undrained shear strength, σ is the vertical total stress and ϕ_u is the angle of undrained shearing resistance. Total stress will

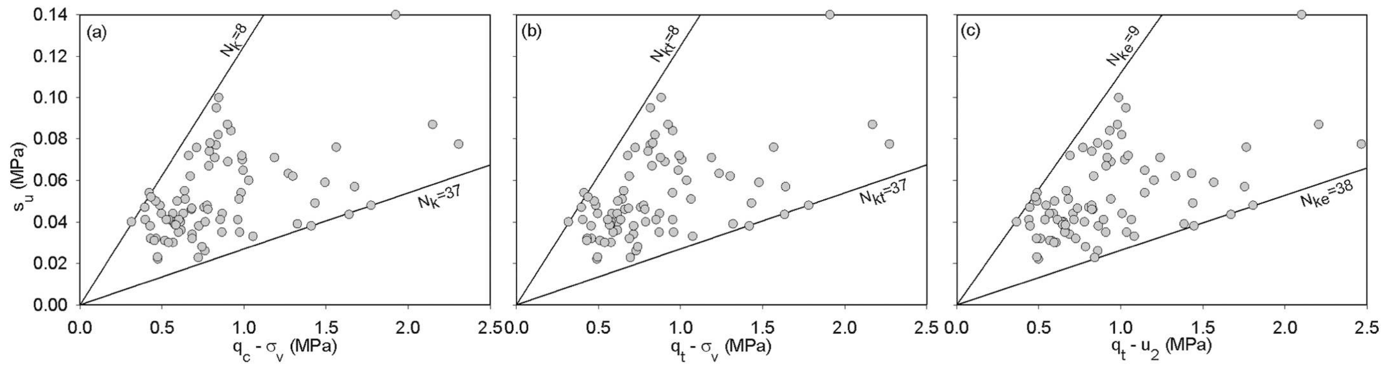

Fig. 1. (a) Cone factor N_k vs s_u; (b) Cone factor N_{kt} vs s_u; and (c) Effective cone factor N_{ke} vs s_u for Adapazari soils.

Table 3
Variations of N_{kt} values with type of laboratory test used for reference.

Researcher	Soil and test types	N_{kt}
Lunne et al. (1985)	North Sea soils, *CAUC* triaxial	9–20
Aas et al. (1986)	$s_{u(lab)} = (s_{u(c)} + s_{u(d)} + s_{u(e)})/3$[a], 3% < I_P < 50%	8–16
La Rochelle et al. (1988)	FVT, *no relationship with* I_p	11–18
Rad and Lunne (1986)	$s_{u(c)}$	8–29
Powell and Quertman (1988)	$s_{u(c)}$	10–20
	$s_{u(c)}$ (in fissured clays)	10–30
Senneset et al. (1989)	For normally consolidated clays	10–15
	For overconsolidated clays	15–19
Luke (1995)	Danish Clays, CAUC	8.5–12
Tanaka and Tanaka (1996)	UCS	8–16
	FVT	9–14
Fukasawa and Kusakabe (2001)	DST-1/UCT/FVT	10.8/13/ 11.6
Fukasawa et al. (2004)	UCS, DST, DST-1,2,3	12
Karakouzian et al. (2003)	Under consolidated marine clay, FVT	10–15
Lunne et al. (2005)	Soft to firm clays, $s_{u(c)}$	12
Önalp et al. (2006)	UU ($s_{u(c)}$)	7–29
Low et al. (2010)	3 onshore and 11 offshore clays, $s_{u(c)}$	11.9
Rémai (2013)	UC, CU	17–32
Mayne et al. (2015)	51 soft to firm intact clays, CAUC	11.8

[a] $s_{u(c)}$: triaxial compression, $s_{u(d)}$: triaxial extension, $s_{u(e)}$: direct simple shear, DST: direct shear test, FVT: field vane test, I_P: plasticity index, UCS: unconfined compression test, CAUC: anisotropically-consolidated-undrained test.

transform to $s_u = c_u$ because $\phi_u = 0$. This is known as the $\phi_u = 0$ condition and the parameter obtained is defined as the undrained shearing resistance, s_u.

A survey of the literature on the subject indicated that no significant study has been done to evaluate undrained shear strength parameter with reference to soil classification by the use of CPT data.

2. Properties of Adapazari soils and the database

The studied soils consist of fluvial deposits that were transported to the Adapazari plain (Turkey) by River Sakarya that flows along the eastern border of the city Adapazari nowadays. These soils were deposited during the Holocene and are normally consolidated to lightly overconsolidated ($OCR_{avg} \approx 3$) at shallow depth (Bol, 2013; Özocak et al., 2013) with sensitivities rising to 1.94 (Arel and Özocak, 2005). There are studies that investigated the distribution and origins of the soils on the Adapazari plain (Bol, 2003; Bol et al., 2010; Bol, 2012) which showed that all the sub-facies that could be found in a typical floodplain are present throughout the soil profile.

A rich database named "*Adapazarı Soil Database*" collected from test results from the plain, the site of the catastrophic earthquakes of 1999, has been used for this research. This database has been established with

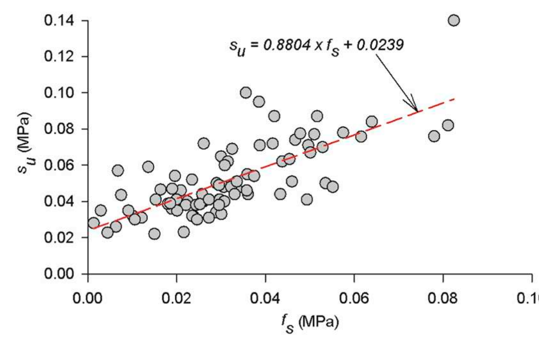

Fig. 2. Sleeve friction (f_s) vs undrained shear strength (s_u).

information collected from several research projects (Bray et al., 2001; Bol et al., 2013; Önalp et al., 2007; Önalp et al., 2010). Disturbed and undisturbed samples were taken at several depths of boreholes and tested in the laboratory. Information on soil classification tests, triaxial tests and cone penetration data have been taken from the database at levels where undrained shear strengths are available. Additional soundings by using a 200 kN acoustic CPTu equipment and boreholes have also been performed to complement the available data. Table 1 summarizes the results of the available laboratory and field work.

3. Methods

Undrained shear strength (s_u) using "total" cone resistance from cone penetration test data is calculated with the following equation.

$$s_u = \frac{(q_c - \sigma_{vo})}{N_k} \tag{3}$$

where N_k is an empirical cone factor and σ_{vo} is the total in-situ vertical stress. Table 2 presents the values for N_k, proposed by various investigators. The table indicates a wide variation in the values. Evaluating the data used in this study (Table 1) N_k was found to vary between 8 and 37 as depicted in Fig. 1a.

The corrected total cone resistance q_t is given by

$$q_t = q_c + (1 - a) \times u_2 \tag{4}$$

Here, a = area ratio of the cone (two different cones with area ratios (a) of 0.58 and 0.60 have been used in this study) and u_2 = pore pressure measured behind the cone. Thus, the cone factor is redefined as:

$$N_{kt} = \frac{(q_t - \sigma_{vo})}{s_u} \tag{5}$$

A summary of the findings of several researchers for N_{kt} is given in Table 3. This table shows variations of N_{kt} values with reference to the type of laboratory test used and it is clear that no definite value for N_{kt} can be extracted from the data. Similarly, total cone resistance was

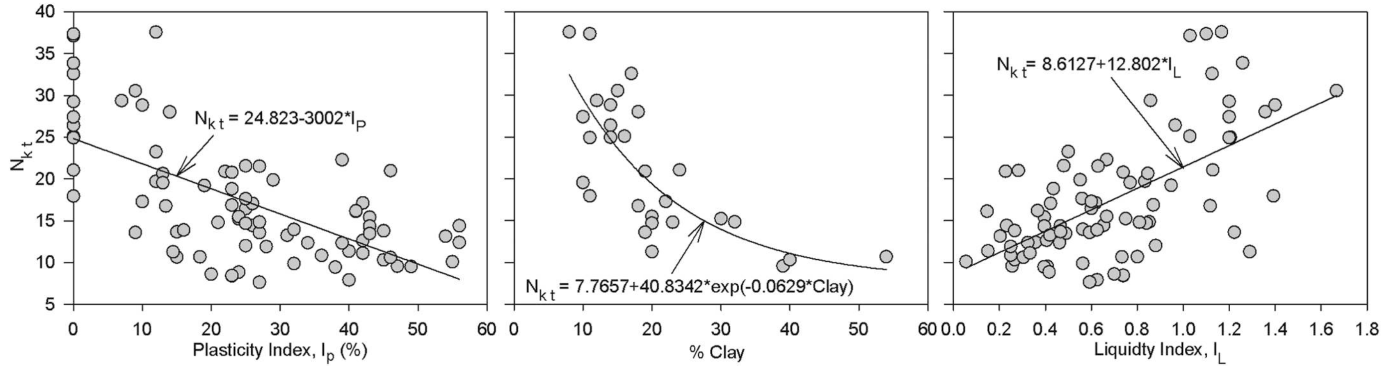

Fig. 3. The influence of physical properties on N_{kt}.

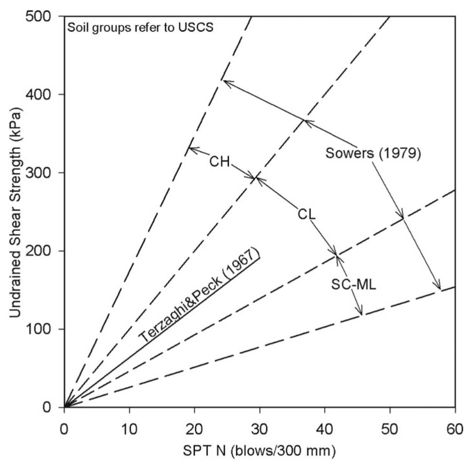

Fig. 4. Sowers (1979) chart to estimate undrained shear strength from SPTN.

found to vary between 8 and 37 (Fig. 1b) by the evaluation of the data in Table 1. Additionally, Fig. 1b indicates that cone factors calculated with corrected cone resistances (N_{kt}) were not noticeably different from the uncorrected values.

According to Campanella et al. (1982), s_u can be calculated by using effective cone resistance (q_e);

$$s_u = \frac{q_e}{N_{ke}} = \frac{(q_t - u_2)}{N_{ke}} \tag{6}$$

Lunne et al. (1985) showed that effective cone factor (N_{ke}) varies between 1 and 13 and developed a correlation for N_{ke} and B_q. Karlsrud et al. (2005) developed correlations for the estimation of N_{ke}, taking into account the sensitivity of the soil (S_t) and the pore pressure coefficient B_q and proposed

$$N_{ke} = 11.5 - 9.05B_q \quad for \quad S_t < 15 \tag{7}$$

$$N_{ke} = 12.5 - 11.0B_q \quad for \quad S_t > 15 \tag{8}$$

Pore pressure ratio (B_q) can be calculated as;

$$B_q = \frac{\Delta u}{q_t - \sigma_{vo}} = \frac{u_2 - u_0}{q_t - \sigma_{vo}} \tag{9}$$

where σ_{vo} = in-situ total vertical stress, u_0 = in-situ equilibrium water pressure, u_2 = pore water pressure measured behind the cone, Δu = excess pore water pressure measured during penetration (u_2-u_0). N_{ke} values were found to vary between 9 and 38 (Fig. 1c) for Adapazari soils.

Lunne et al. (1997) reported the relationships that have been proposed for the induced excess pore pressure Δu and the undrained strength. These relationships have a general form of

$$s_u = \frac{\Delta u}{N_{\Delta u}} \quad (\Delta u = u_2 - u_0) \tag{10}$$

$N_{\Delta u}$ is found to vary between 2 and 20 based on cavity expansion theory. Lunne et al. (1985) used triaxial compression (*CAUC*) test results and found a reliable correlation for pore water pressure ratio (B_q), with corresponding $N_{\Delta u}$ values varying between 4 and 10. Karlsrud et al. (1996) similarly used *CAUC* test results to obtain a relationship between s_u and $N_{\Delta u}$ where they showed that $N_{\Delta u}$ varied between 6 and 8. La Rochelle et al. (1988), using uncorrected field vane (*FVT*) as reference strength, found that $N_{\Delta u}$ varied between 7 and 9 for three Canadian clays, even though OCR ranged between 1.2 and 5.0. Ricceri et al. (2002) proposed Eq. (11) to express the relationship between $N_{\Delta u}$ and B_q for alluvial deposits of the Venice Lagoon.

$$N_{\Delta u} = 18.6 \times B_q + 0.13 \tag{11}$$

Begemann (1965) was the first to suggest that the undrained shear strength (s_u) in clayey soils can be estimated from sleeve friction (f_s). He demonstrated that the undrained shear strength measured in the vane

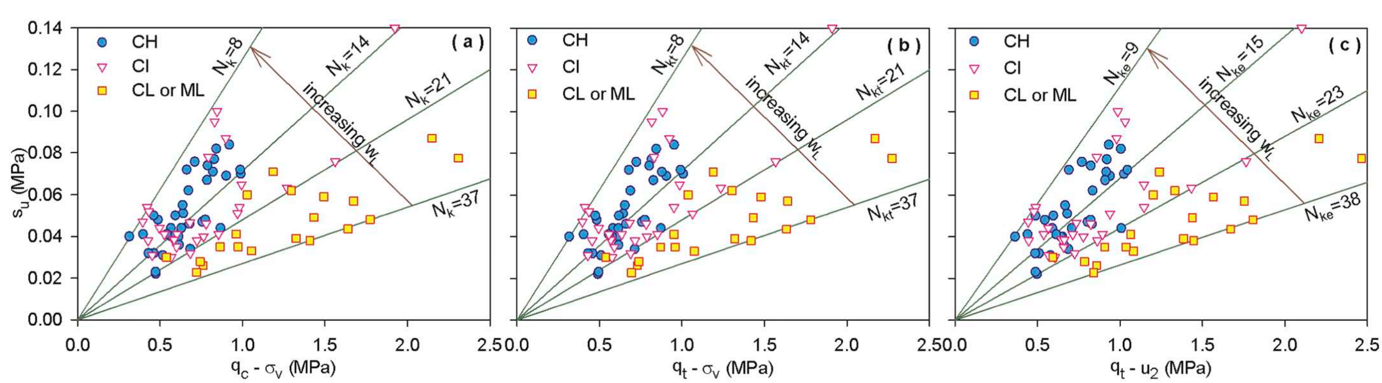

Fig. 5. Estimation of undrained strength based on class of soil.

Table 4
A summary of the proposed formulas for I_c.

Been and Jefferies (1992)	$I_c = \sqrt{\{3 - \log[Q(1 - B_q) + 1]\}^2 + [1.5 + 1.3(\log F)]^2}$	(15)
Jefferies and Davies (1993)	$I_c = \sqrt{\{3 - \log[Q(1 - B_q)]\}^2 + [1.5 + 1.3(\log F)]^2}$	(16)
Robertson and Wride (1998)	$I_c = \sqrt{[3.47 - \log Q]^2 + [1.22 + \log F]^2}$	(17)
Juang et al. (2003)	$I_c = \sqrt{[3.47 - \log q_{c1N}]^2 + [1.22 + \log F]^2}$	(18)
Li et al. (2007)	$I_{c,m} = \sqrt{\{3.25 - \log[Q(1 - B_q)]\}^2 + [1.5 + 1.3(1 + \log F)]^{2.25}}$	(19)
Robertson (2010)	$I_{SBT} = \sqrt{[3.47 - \log(q_c/p_a)]^2 + [1.22 + \log R_f]^2}$	(20)
Bol (2013)	$I_c = \sqrt{\{3.47 - 0.9\log[Q(1 - 0.01\,i)]\}^2 + \{1.4 + 2[\log F/(1 - 0.01\,i)]\}^2}$	(21)

test was 1/14th of the tip resistance measured by the mechanical cone while the sleeve resistance has almost the same value with the s_u measured in the *FVT*. According to Anagnostopoulos et al. (2003) the f_s/s_u ratio is 1.26 and 1.00 for the mechanical and the electrical cones, respectively. This ratio was determined to be 0.608 for Adapazari soils.

Fig. 2 shows the the relationship between sleeve friction (f_s) and undrained shear strength (s_u) from *TX-UU* test with a coefficient of determination $R^2 = 0.536$ for Adapazari soils. One can express the following relationship for Adapazari soils where s_u and f_s are in MPa.

$$s_u = 0.8804 \times f_s + 0.0239 \tag{12}$$

According to Powel and Lunne (2005) undrained shear strength obtained from the frictional resistance represents the remoulded undrained strength. Because of this, undrained shear strength from sleeve friction may have incorrect values.

Available evidence shows that estimating the undrained strength value from CPTu readings shows a high scatter, regardless of the method used to estimate it. The reason for this may possibly be that s_u is not the only parameter representing strength. It is likely that the type of soil, penetration rate and the effect of type of test used for reference may influence the results (Senneset et al., 1989). The index properties of the samples used to obtain their correlation with N_{kt} are shown in Fig. 3. The value of N_{kt} drops with increasing plasticity index and clay content and decreasing liquidity index. This finding suggests that the class of the soil may be directly influencing the cone factors. Accordingly, it was decided to place emphasis on determining the class of soil.

A study conducted by Sowers (1979) to analyze the undrained shear strength deduced from standard penetration test (Fig. 4) shows that each of the main soil groups such as CH, CL, SC and ML (ASTM D2487-17, 2017) should processed with different factors. The use of a soil type dependent factor reduces the error margin, instead of using an average trend line. This study uses a similar attempt to relate shear resistance to CPTu data.

The Turkish Standard for soil classification TS 1500/2000, separates fine grained soils into three regions as of low ($w_L < 35$), intermediate ($35 < w_L < 50$) and high plasticity ($w_L > 50$) on the plasticity chart. This enables the investigator to evaluate soils of intermediate plasticity as a different group whereby the wide band for soils of low plasticity (CL) increases the error margins in the Sowers Chart.

When the values of undrained shear strength measured in the laboratory are plotted against the CPT data for each soil group (Fig. 5), distinct zones arise similar to the Sowers Chart. By evaluating Fig. 5, it can be seen that as the liquid limit increases, cone factors decrease. It can thus be inferred that different soil classes are indeed clustered in separate regions. Accordingly the type of a soil can be determined through CPT measurements, despite the fact that no samples are obtained in this test.

Initial studies to classify soils with cone penetrometer were carried out by using cone resistance (q_c) and sleeve friction (f_s), without correcting them for the effect of porewater pressure or overburden pressure (Begemann, 1965). Sanglerat et al. (1974) argued that the friction ratio ($R_f = f_s/q_c$) and the cone resistance are functions of the class of soil. Friction ratios (R_f) are more commonly used in classification than sleeve friction (f_s) alone. The early classifications were mostly carried out with the aid of pre-prepared charts (Begemann, 1965; Sanglerat et al., 1974; Schmertmann, 1978; Douglas and Olsen, 1981; Jones and Rust, 1982; Senneset et al., 1989; Eslami and Fellenius, 1997). It appears that Robertson's charts (Robertson et al., 1986; Robertson, 1990) are the most favoured.

In addition to these, studies on soil classification in recent years have mainly been aimed at defining the “Soil Behavior Type Index” I_c, for which the normalized cone resistance (Q), normalized friction ratio (F) and a pore pressure coefficient (B_q) are used. A summary of the cited formulae for I_c is given in Table 4. In this table, Robertson and Wride (1998) formula appears again as the most popular. Robertson and Wride (1998) proposed the use of the normalized penetration resistance Q (dimensionless) and the normalized friction ratio F (in per cent) to calculate the I_c values as

$$Q = \left(\frac{q_c - \sigma_{vo}}{P_{a2}}\right)\left(\frac{P_a}{\sigma'_{vo}}\right)^n \tag{13}$$

$$F = \left[\frac{f_s}{q_c - \sigma_{vo}}\right]100 \tag{14}$$

where σ_{vo} and σ'_{vo} are the total and effective overburden stresses respectively. P_a is the reference pressure with the same units used in Eq. (13) (i.e. $P_a = 100$ kPa if σ'_{vo} is in kPa) and P_{a2} is the reference pressure with the same units as q_c and σ_{vo} used in Eq. (13) (i.e. $P_a = 0.1$ MPa if q_c and σ_{vo} are in MPa).

The soil behavior indices in this study were calculated using all the proposed formulas and each sample was grouped according to its soil behavior index. The CPT data and the undrained shear strengths measured in the laboratory were then plotted using these soil groups. Cone factors appeared to be more consistent for the groups with similar I_c, as shown in Fig. 6.

The average of the upper and lower limit values of each group of similar soil behavior indices (I_c) were selected as cone factors. The error margin emerged to be narrower this way, compared to the case if all data were evaluated and no distinction for the soil classes was made.

Fig. 6(A1) shows the s_u values measured in the laboratory against the total cone resistance (q_c-σ_{vo}). These points were grouped by their I_c values calculated by the formula of Jefferies and Davies (1993). The N_k values corresponding to these I_c values were determined drawing the boundaries between these groups. Thus, the average N_k value for the soils with $3.29 > I_{cJD} > 2.59$ can be taken as $N_k = (7.81 + 13.16)/2 = 10.49$. Similar evaluations are presented in Table 5 for both N_{kt} and N_{ke} according to the I_c formulae proposed by other researchers quoted.

3.1. Soil classification by using pore pressure gradients (i)

Bol (2013) showed that the changes in the pore water pressures (i) during a CPTu test is closely related to the type of soil and proposed that parameter i is a better alternative to the other methods used for classifying soils through the use of CPT measurements. The parameter i representing the pore water pressure gradients along the soil profile during penetration is a dimensionless number describing the changes in

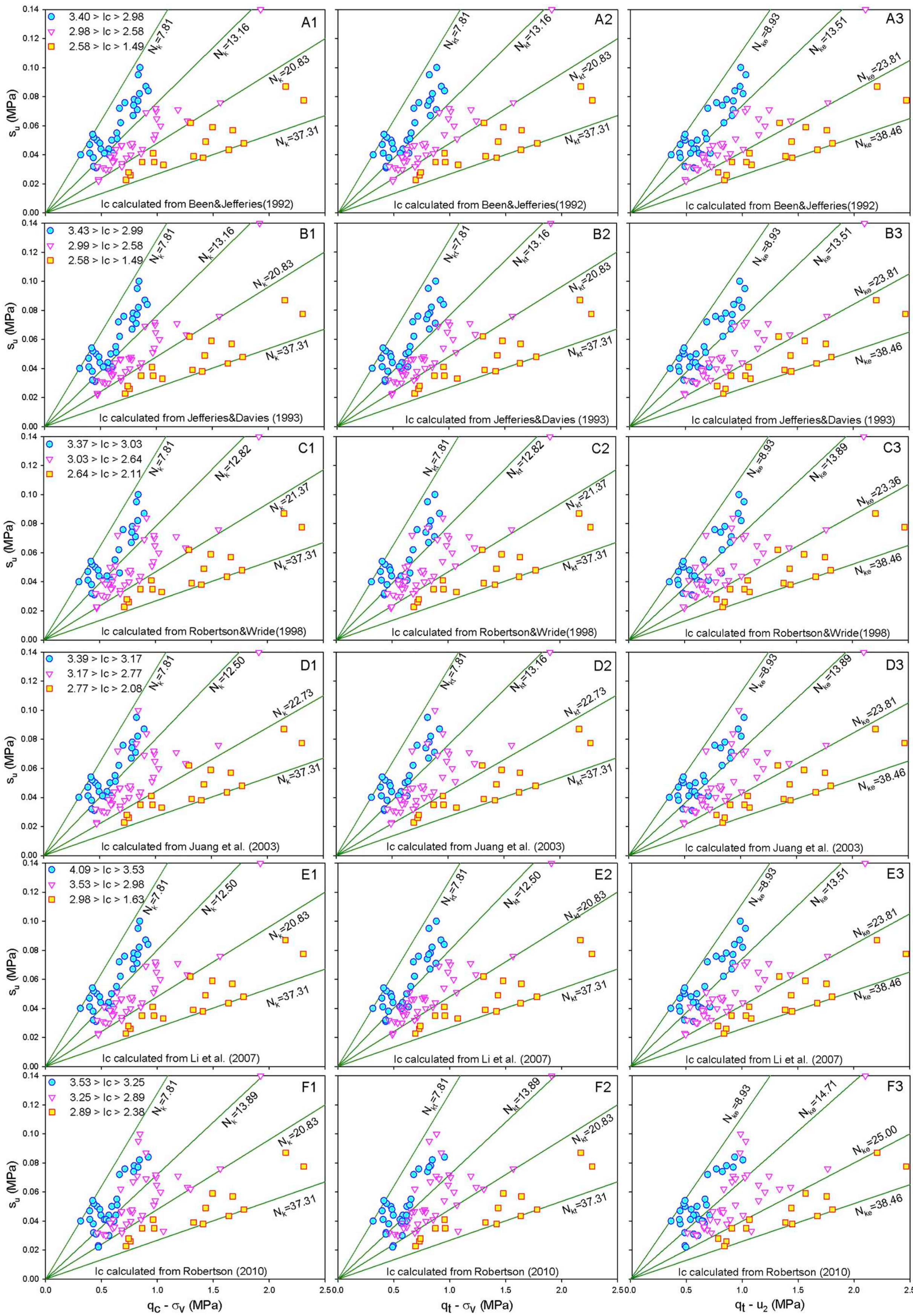

Fig. 6. Determination of undrained strength based on different soil type behavior index (I_c).

Table 5
The limits for N_k, N_{kt} and N_{ke} based on different soil type behavior index (I_c).

I_c calculated from	I_c intervals	N_k (1*)			N_{kt} (2*)			N_{ke} (3*)		
		Lower	Upper	Average	Lower	Upper	Average	Lower	Upper	Average
Been and Jefferies (1992) (A[a])	3.40–2.98	7.81	13.16	10.49	7.81	13.16	10.49	8.93	13.51	11.22
	2.98–2.58	13.16	20.83	17.00	13.16	20.83	17.00	13.51	23.81	18.66
	2.58–1.49	20.83	37.31	29.07	20.83	37.31	29.07	23.81	38.46	31.14
Jefferies and Davies (1993) (B[a])	3.43–2.99	7.81	13.16	10.49	7.81	13.16	10.49	8.93	13.51	11.22
	2.99–2.58	13.16	20.83	17.00	13.16	20.83	17.00	13.51	23.81	18.66
	2.58–1.49	20.83	37.31	29.07	20.83	37.31	29.07	23.81	38.46	31.14
Robertson and Wride (1998) (C[a])	3.37–3.03	7.81	12.82	10.32	7.81	12.82	10.32	8.93	13.89	11.41
	3.03–2.64	12.82	21.37	17.09	12.82	21.37	17.09	13.89	23.36	18.63
	2.64–2.11	21.37	37.31	29.34	21.37	37.31	29.34	23.36	38.46	30.91
Juang et al. (2003) (D[a])	3.39–3.17	7.81	12.50	10.16	7.81	13.16	10.49	8.93	13.89	11.41
	3.17–2.77	12.50	22.73	17.61	13.16	22.73	17.94	13.89	23.81	18.85
	2.77–2.08	22.73	37.31	30.02	22.73	37.31	30.02	23.81	38.46	31.14
Li et al. (2007) (E[a])	4.09–3.53	7.81	12.50	10.16	7.81	12.50	10.16	8.929	13.51	11.22
	3.53–2.98	12.50	20.83	16.67	12.50	20.83	16.67	13.51	23.81	18.66
	2.98–1.63	20.83	37.31	29.07	20.83	37.31	29.07	23.81	38.46	31.14
Robertson (2010) (F[a])	3.531–3.248	7.81	13.89	10.85	7.81	13.89	10.85	8.93	14.71	11.82
	3.248–2.886	13.89	20.83	17.36	13.89	20.83	17.36	14.71	25.00	19.85
	2.886–2.381	20.83	37.31	29.07	20.83	37.31	29.07	25.00	38.46	31.73

[a] These numbers and letters indicate that graph numbers in Fig. 6.

Table 6
Soil classes identified by I_c values (Bol, 2013).

Zone	I_c	Soil class
1	$I_c < 1.40$	SP or Gravels
2	$1.40 < I_c < 1.80$	SW-SM or SP-SM
3	$1.80 < I_c < 2.45$	SM or ML
4	$2.45 < I_c < 2.90$	CL-ML
5	$2.90 < I_c < 3.48$	CI-MI or CL
6	$3.48 < I_c < 4.00$	CH or CI
7	$I_c > 4.00$	CH

pore water pressures during flight.

$$i = \frac{\Delta u_2}{\Delta \sigma_{vo}} = \frac{u_{2z2} - u_{2z1}}{(\sigma_{voz2} - \sigma_{voz1})} \tag{22}$$

where z_1 and z_2 are the start and end depths of the studied layer in the soil profile, u_{2z2} and u_{2z1} are the pore water pressures corresponding to depths z_2 and z_1, and σ_{voz2} and σ_{voz1} are the calculated total overburden stresses at these depths.

It was found that the parameter i increased with increasing liquid limit, while decreasing with increasing grain size. In addition, i assumed negative values for soils of dilative character such as silts. Bol (2013) suggested intervals of soil type behaviour index (I_c, Eq. (21))

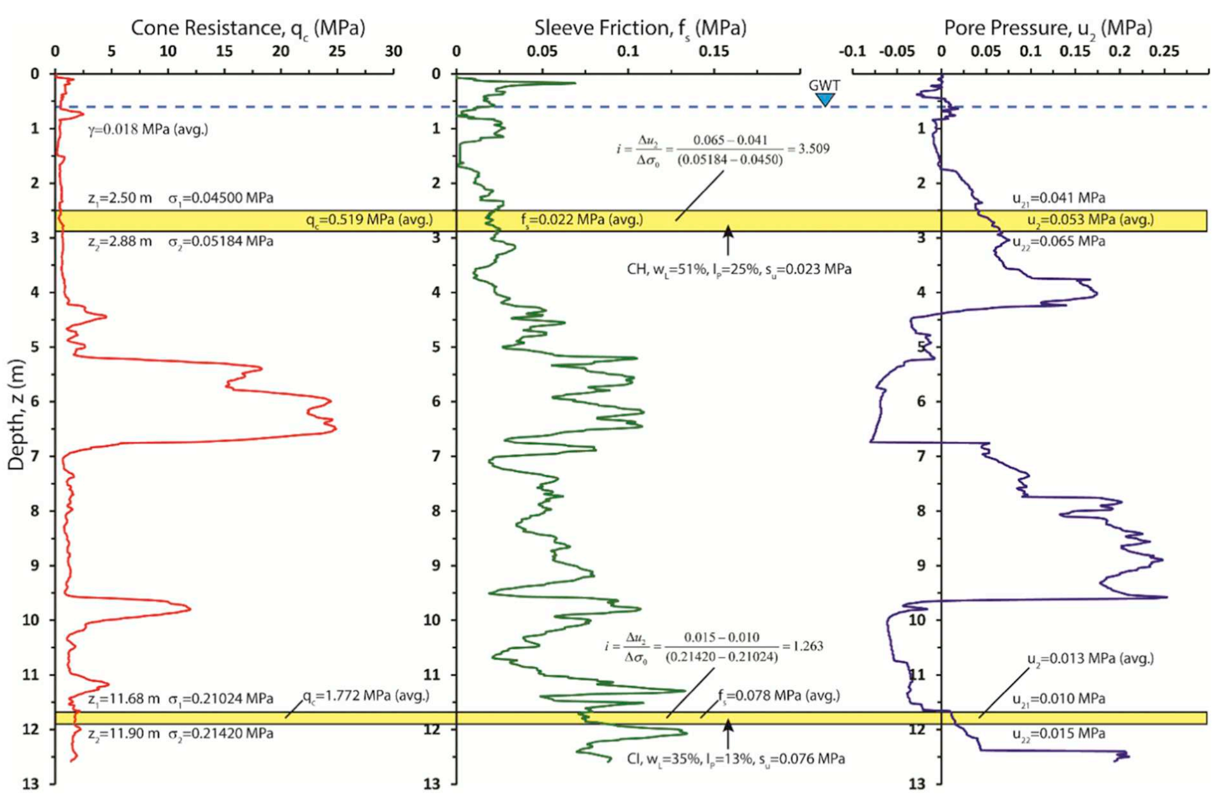

Fig. 7. CPT graph for ID = 33 and ID = 61 data.

Table 7
Example calculation of i by using ID:33 and ID:61 data.

ID	Depth (m)	q_c (MPa) (avg.)	f_s (MPa) (avg.)	u_2 (MPa) (avg.)	z_1 (m)	z_2 (m)	σ_{0z1} (MPa)	σ_{0z2} (MPa)	u_{2z1} (MPa)	u_{2z2} (MPa)	i	I_c
33	2.50	0.519	0.022	0.053	2.50	2.88	0.04500	0.05184	0.041	0.065	3.509	3.581
61	11.50	1.772	0.078	0.013	11.68	11.90	0.21024	0.21420	0.010	0.015	1.263	3.691

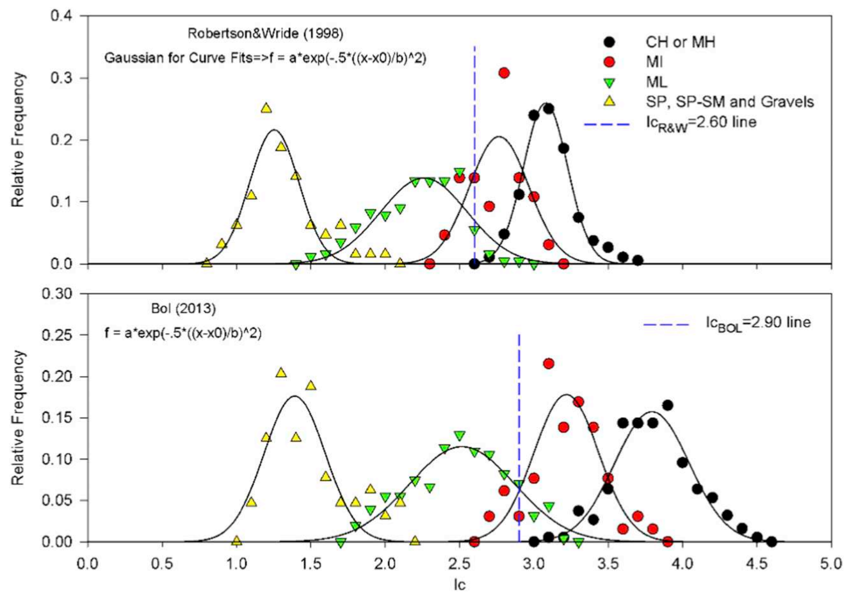

Fig. 8. Relative frequency diagrams of soil classes according to Robertson and Wride (1998) and Bol (2013) (*modified from*Bol (2013)).

corresponding to TS1500 (2000) soil classes (Table 6). The difference of this method from the others is to identify soil class directly from laboratory test results.

The procedure for calculating the parameter i is demonstrated in the example profile illustrated in Fig. 7. The CPT data taken from Table 1 for ID = 33 (2.50 m) and ID = 61 (11.50 m) were used. The data suggested that the samples denoted by 33 and 61 corresponded to depths 2.50–2.88 m and 11.68–11.90 m, and the excess porewater pressures measured in this range are listed in Table 7. According to Bol (2013), parameter i should not be used in transition zones because sudden drops may occur at these boundaries. The procedure for calculating i needs a careful choice of calculation boundaries.

A procedure similar to that described by Robertson and Wride (1998) was employed Table 7 to calculate the I_c values with Eq. (21). In this procedure, normalized penetration resistance Q is calculated by using the corrected total cone resistance q_t instead of q_c in Robertson and Wride (1998) formula.

$$Q = \left(\frac{q_t - \sigma_{vo}}{P_{a2}}\right)\left(\frac{P_a}{\sigma'_{vo}}\right)^n \tag{23}$$

They stated that it will be appropriate to adopt $n = 1$ if $I_{cR\&W(1998)} > 2.60$ for clays but n will be 0.5 for sandy soils with $I_{cR\&W(1998)} < 2.60$, in modifying the value of I_c. However, Bol (2013) showed that the border separating clays and sands is actually $I_c = 2.90$ as shown in Fig. 8.

The procedure is as follows: Calculate Q using Eq. (20) with $n = 1$. Use the calculated Q to calculate I_c with Eq. (18). This value is used for classification if $I_c > 2.90$. In case $I_c < 2.90$, take $n = 0.50$ to repeat the first step. If I_c comes out smaller than 2.90, this second estimate is used for classification. However, if $I_c > 2.90$ adopt $n = 0.75$ to get the final value of Q and the I_c related to it.

The flow chart illustrating the procedure (Bol, 2013) is presented in Fig. 9. The results of classification for the data listed in Table 1 are summarized in Table 8.

Following the above procedure, the I_c calculated by Eq. (18) were used to correlate the s_u measured in the laboratory with the CPTu data. The I_c values calculated according to Bol (2013) are divided into groups with lower and upper limits as shown in Fig. 10. The averages of the limits for each soil group have been selected as cone factors N_k, N_{kt} and N_{ke}. The maximum, minimum, and average cone factors for all ranges of I_c are shown in Table 9.

Tables 5 and 9 show that the values of N_k and N_{kt} are similar. The reason for this is that the measured pore pressures (u_2) are not so high and cone resistance (q_c) is always much higher than u_2 for soils studied. Thus, cone resistance correction for measured pore pressure has a negligible influence on the distribution of data points.

Fig. 11 shows the correlation of the undrained shear strength calculated using the average cone factors for each range of I_c listed in Table 9 with the undrained shear resistance. It is seen that all the points are clustered in the immediate vicinity of the x = y line with reasonably high R^2 values.

4. Sensitivity analysis

In order to confirm the robustness of the developed procedure, it is important to test the proposed models using performance evaluation criteria such as the correlation coefficient (R^2), the mean absolute relative error (*AARE*) and the mean square error (*MSE*).

Average absolute relative error (AARE) can be computed with the expression

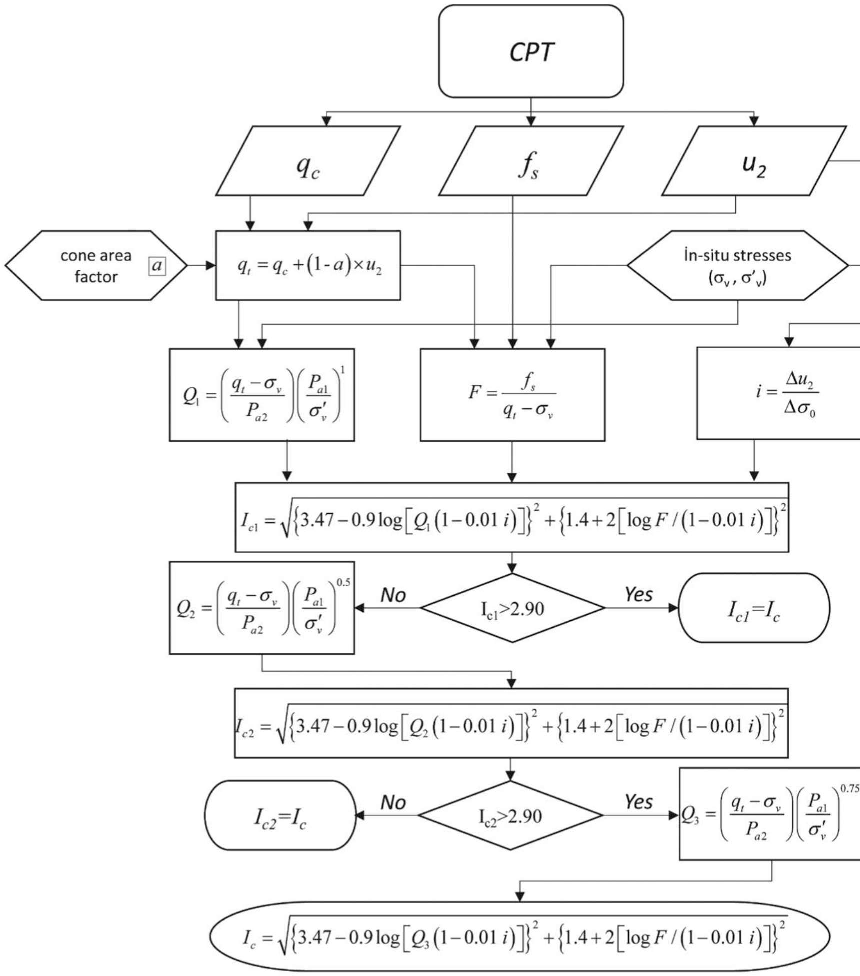

Fig. 9. Flow chart illustrating the application of proposed soil classification from CPT data by using Bol (2013) procedure.

$$AARE = \frac{1}{N}\sum_{p=1}^{N}|RE| \tag{24}$$

in which

$$RE = \frac{s_{um}-s_{ue}}{s_{um}}\times 100 \tag{25}$$

Here, RE is the relative error in percent, s_{um} is the measured undrained shear strength, s_{ue} is the estimated undrained shear strength and N is the total number of data points.

The mean square error (MSE) is defined as,

$$MSE = \frac{1}{N}\sum_{i=1}^{n}(s_{um}-s_{ue})^2 \tag{26}$$

The *AARE* enables not only the prediction of the undrained shear strength, but also the distribution of prediction errors. Ideally, the value of *AARE* and *MSE* should be zero and R^2 should be equal to unity.

Table 10 lists the intervals of I_c obtained by using the formulae of different researchers and the results of their statistical analyses. It is obvious that the estimated s_u values evaluated with the methods described above reflect the measured s_u values reasonably well. Robertson (2010) formula (Eq. (20)) gave results that did not conform with the general trend. Table 10 indicates that the formulae proposed by Jefferies and Davies (1993), Been and Jefferies (1992), Li et al. (2007) and Bol (2013) for calculating undrained strength using I_c from CPTu produced results which showed better agreement with the s_u measured in the laboratory. The common approach by the researchers above is the use of the pore water pressure values from CPTu in the expressions they developed for I_c.

The discrepancy ratio $D_r = s_{um}/s_{ue}$ was employed to evaluate the method proposed in this paper by an alternative approach. The discrepancy ratio (D_r) will be unity if the measured undrained shear strength (s_{um}) is equal to the estimated undrained shear strength (s_{ue}), as desired. Fig. 12 shows discrepancy ratios for all estimated s_u values obtained by all I_c intervals. Data points of Jefferies and Davies (1993), Been and Jefferies (1992), Li et al. (2007) and Bol (2013) methods have only a negligible number of data points being located outside the 30% error band. The proposed methodology to evaluate undrained shear strength from CPTu data is believed to provide accurate and reliable results.

Multiple linear regression analyses (*MLRA*) have been performed to show the relation between physical properties and the formulations of I_c

Table 8
Calculated i and I_c values.

ID	u_{2z1} (MPa)	u_{2z2} (MPa)	σ_{voz1} (MPa)	σ_{voz2} (MPa)	i	I_c	ID	u_{2z1} (MPa)	u_{2z2} (MPa)	σ_{voz1} (MPa)	σ_{voz2} (MPa)	i	I_c
1	0.012	0.010	0.0439	0.0508	−0.292	3.363	43	−0.021	−0.017	0.1411	0.1440	1.389	3.424
2	−0.018	0.001	0.0529	0.0576	4.060	3.883	44	0.083	0.102	0.1195	0.1206	17.593	4.167
3	0.116	0.138	0.1800	0.1890	2.444	3.904	45	−0.026	−0.003	0.1512	0.1591	2.904	3.535
4	0.035	0.054	0.0576	0.0630	3.519	3.657	46	0.076	0.117	0.1800	0.1858	7.118	3.755
5	0.068	0.071	0.0457	0.0482	1.190	3.483	47	0.073	0.075	0.1494	0.1516	0.926	3.835
6	0.008	0.007	0.0421	0.0504	−0.121	3.583	48	0.204	0.227	0.1102	0.1170	3.363	3.610
7	−0.004	0.000	0.0439	0.0479	1.010	3.403	49	0.053	0.074	0.0558	0.0616	3.646	3.814
8	0.025	0.039	0.0457	0.0522	2.160	3.946	50	0.062	0.052	0.1127	0.1217	−1.111	3.819
9	0.029	0.032	0.0767	0.0788	1.389	4.082	51	0.009	0.014	0.0450	0.0540	0.556	3.856
10	0.002	0.015	0.1692	0.1724	4.012	4.294	52	0.000	0.005	0.0551	0.0576	1.984	3.184
11	−0.032	−0.032	0.0731	0.0760	0.000	3.764	53	−0.012	−0.006	0.0313	0.0360	1.282	3.676
12	−0.005	0.002	0.0439	0.0511	0.972	3.553	54	0.001	0.003	0.0508	0.0522	1.389	3.816
13	0.045	0.117	0.0540	0.0612	10.000	3.848	55	−0.056	−0.052	0.0814	0.0853	1.010	4.042
14	−0.017	−0.007	0.0369	0.0432	1.587	3.232	56	−0.010	0.007	0.0702	0.0738	4.722	3.886
15	−0.045	−0.045	0.1278	0.1350	0.000	3.596	57	−0.040	−0.037	0.0522	0.0576	0.556	3.613
16	0.009	0.020	0.0637	0.0713	1.455	3.257	58	−0.040	−0.030	0.1710	0.1782	1.389	3.834
17	−0.004	0.003	0.0446	0.0522	0.926	3.514	59	0.045	0.068	0.0461	0.0482	10.648	3.573
18	−0.001	0.008	0.0544	0.0572	3.125	4.090	60	0.011	0.025	0.0457	0.0540	1.691	3.536
19	0.003	0.006	0.0385	0.0475	0.333	3.684	61	0.010	0.015	0.2102	0.2142	1.263	3.690
20	−0.001	0.004	0.0565	0.0590	1.984	3.784	62	0.075	0.071	0.0508	0.0544	−1.111	3.646
21	0.045	0.074	0.0342	0.0432	3.222	3.946	63	−0.080	−0.079	0.1102	0.1116	0.694	3.448
22	0.030	0.039	0.1807	0.1850	2.083	4.099	64	−0.013	−0.045	0.0454	0.0540	−3.704	2.598
23	0.029	0.043	0.0472	0.0511	3.535	3.483	65	0.060	0.045	0.0781	0.0810	−5.208	2.895
24	0.035	0.044	0.0479	0.0544	1.389	3.718	66	−0.028	−0.015	0.0846	0.0929	1.570	2.982
25	0.055	0.140	0.1764	0.1818	15.741	4.142	67	0.006	0.010	0.0558	0.0583	1.587	3.329
26	0.023	0.045	0.0450	0.0511	3.595	3.932	68	0.023	0.013	0.0403	0.0439	−2.778	3.083
27	−0.041	−0.038	0.0547	0.0619	0.417	3.682	69	0.044	0.050	0.0486	0.0590	0.575	3.141
28	0.003	0.036	0.0443	0.0504	5.392	3.841	70	−0.012	−0.026	0.0238	0.0248	−12.963	2.137
29	0.005	0.028	0.1490	0.1544	4.259	3.673	71	−0.085	−0.075	0.0482	0.0522	2.525	2.786
30	0.009	0.023	0.0504	0.0558	2.593	3.874	72	−0.068	−0.062	0.0810	0.0853	1.389	2.743
31	−0.001	0.040	0.0828	0.0900	5.694	3.769	73	0.028	0.010	0.0421	0.0439	−10.000	2.914
32	−0.021	−0.012	0.0513	0.0576	1.429	3.235	74	−0.082	−0.082	0.1062	0.1084	0.000	2.910
33	0.041	0.065	0.0450	0.0518	3.509	3.569	75	−0.074	−0.079	0.0338	0.0378	−1.263	2.168
34	0.031	0.039	0.0886	0.0972	0.926	3.813	76	0.022	0.023	0.0450	0.0472	0.463	2.929
35	0.038	0.036	0.0479	0.0580	−0.198	3.773	77	0.012	0.033	0.1717	0.1778	3.431	3.515
36	0.007	0.012	0.0457	0.0490	1.543	3.426	78	0.003	−0.018	0.0299	0.0353	−3.889	2.442
37	−0.055	0.022	0.1285	0.1336	15.278	3.614	79	−0.001	0.000	0.0490	0.0511	0.463	3.176
38	−0.018	−0.009	0.0619	0.0648	3.125	3.412	80	−0.011	−0.013	0.0486	0.0540	−0.370	2.875
39	−0.031	−0.019	0.0425	0.0475	2.381	3.351	81	0.029	0.003	0.0443	0.0486	−6.019	2.482
40	0.009	0.009	0.0450	0.0540	0.000	3.557	82	−0.027	−0.041	0.0385	0.0414	−4.861	2.735
41	−0.079	−0.077	0.1080	0.1170	0.222	3.588	83	0.000	−0.001	0.0054	0.0097	−0.231	2.806
42	−0.031	−0.025	0.0486	0.0540	1.111	3.769							

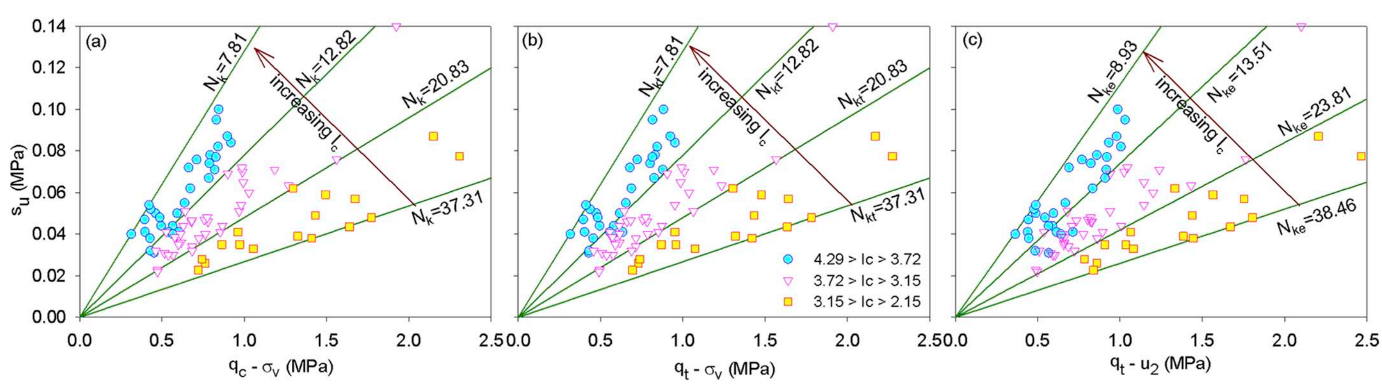

Fig. 10. Determination of undrained strength based on soil type behavior index (I_c defined by Bol (2013)).

Table 9
The limits for N_k, N_{kt} and N_{ke} based on soil type behavior index according to Bol (2013).

I_c intervals	N_k			N_{kt}			N_{ke}		
	Lower	Upper	Average	Lower	Upper	Average	Lower	Upper	Average
3.72–4.30	7.81	12.82	10.32	7.81	12.82	10.32	8.93	13.51	11.22
3.15–3.72	12.82	20.83	16.83	12.82	20.83	16.83	13.51	23.81	18.66
2.13–3.15	20.83	37.31	29.07	20.83	37.31	29.07	23.81	38.46	31.14

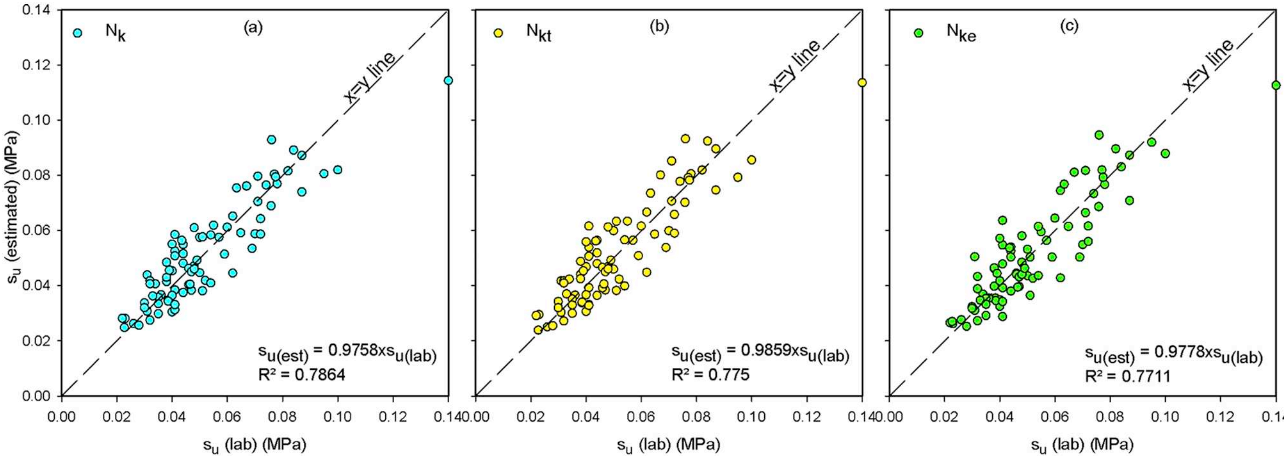

Fig. 11. Comparison of undrained shear strengths of measured in the laboratory and predicted from CPT.

Table 10
Statistical evaluation of the results.

I_c intervals calculated from	AARE (N_k)	AARE (N_{kt})	AARE (N_{ke})	MSE (N_k)	MSE (N_{kt})	MSE (N_{ke})	R^2 (N_k)	R^2 (N_{kt})	R^2 (N_{ke})
Been and Jefferies (1992)	13.725	14.620	14.109	7.41E-05	8.08E-05	8.28E-05	0.780	0.767	0.771
Jefferies and Davies (1993)	13.725	14.620	14.109	7.41E-05	8.08E-05	8.28E-05	0.780	0.767	0.771
Robertson and Wride (1998)	15.635	16.305	16.526	1.15E-04	1.18E-04	1.32E-04	0.673	0.669	0.646
Juang et al. (2003)	15.795	16.464	15.589	1.45E-04	1.51E-04	1.44E-04	0.571	0.564	0.581
Li et al. (2007)	14.259	15.139	14.380	7.58E-05	8.37E-05	8.55E-05	0.787	0.775	0.771
Robertson (2010)	19.286	19.573	19.234	2.70E-04	2.64E-04	2.79E-04	0.410	0.415	0.420
Bol (2013)	14.006	14.826	14.380	7.50E-05	8.17E-05	8.55E-05	0.786	0.775	0.771

proposed by different investigators. The dependent variable was the soil behaviour index ($I_{c\text{-}CPT}$) value proposed by each researcher whereas σ_{vo}, w_L, w_P, w_n and FC were the independent variables. Subsequently a comparison of the soil type behaviour indices $I_{c\text{-}CPT}$ obtained from CPTu measurements was made with those calculated by the model formulations resulting from regression analyses ($I_{c\text{-}MLRA}$). The relative error RE, to incur in the calculation of $AARE$ used in statistical analyses shall be as follows

$$RE = \frac{I_{c-CPT} - I_{c-MLRA}}{I_{c-CPT}} \times 100 \tag{27}$$

On the other hand, the mean square error (MSE) is defined as,

$$MSE = \frac{1}{N}\sum_{i=1}^{n}(I_{c-CPT} - I_{c-MLRA})^2 \tag{28}$$

Table 11 lists the results of the statistical analyses. Similar to the illustration in Table 10 it is observed that the correlation coefficients R^2 for the methods proposed by Jefferies and Davies (1993), Been and Jefferies (1992), Li et al. (2007) and Bol (2013) where excess porewater pressures are taken into account for the calculation of I_c are higher than other methods. The reason for low values of $AARE$ and MSE calculated by the Robertson and Wride (1998),

Juang et al. (2003) and Robertson (2010) methods is possibly the relatively low intervals of I_c adopted (see Table 5). By evaluating Tables 10 and 11 together; it can be seen that the methods, that establish the best relationship between physical properties and I_c, also estimate the undrained shear strength more accurately too.

5. Conclusion

This paper aimed at obtaining the undrained shear resistance (s_u) of fine grained soil through CPTu measurements with reference to its soil behavior type index. The s_u values of undisturbed samples were measured by UU triaxial testing in the laboratory. The CPT soundings were carried out in adjacent boreholes where parameters q_c, f_s and u_2 were recorded at identical depths with UD sampling.

The soil type behavior indices (I_c) proposed by different investigators were calculated.

The graphs defining the relationship s_u-I_c were plotted with zones of different soil classes that were assigned specific cone factors. It was observed that the s_u values for almost all proposed I_c, produced results are close to the values measured in the laboratory. In addition, when the formulation for soil classification that considers excess pore water pressures (B_q or i) by Jefferies and Davies (1993), Been and Jefferies (1992), Li et al. (2007) and Bol (2013) were used with the method proposed in this paper, the undrained shear strengths estimated came out to be nearer to the s_u values measured in the laboratory.

The sensitivity analysis performed using all data points has shown that the undrained strengths estimated by the proposed method provide values that remain within acceptable bands of error. It is concluded that the capacity of the proposed equations to predict the undrained shear strength increases as their capacities to represent the soil physical properties increase.

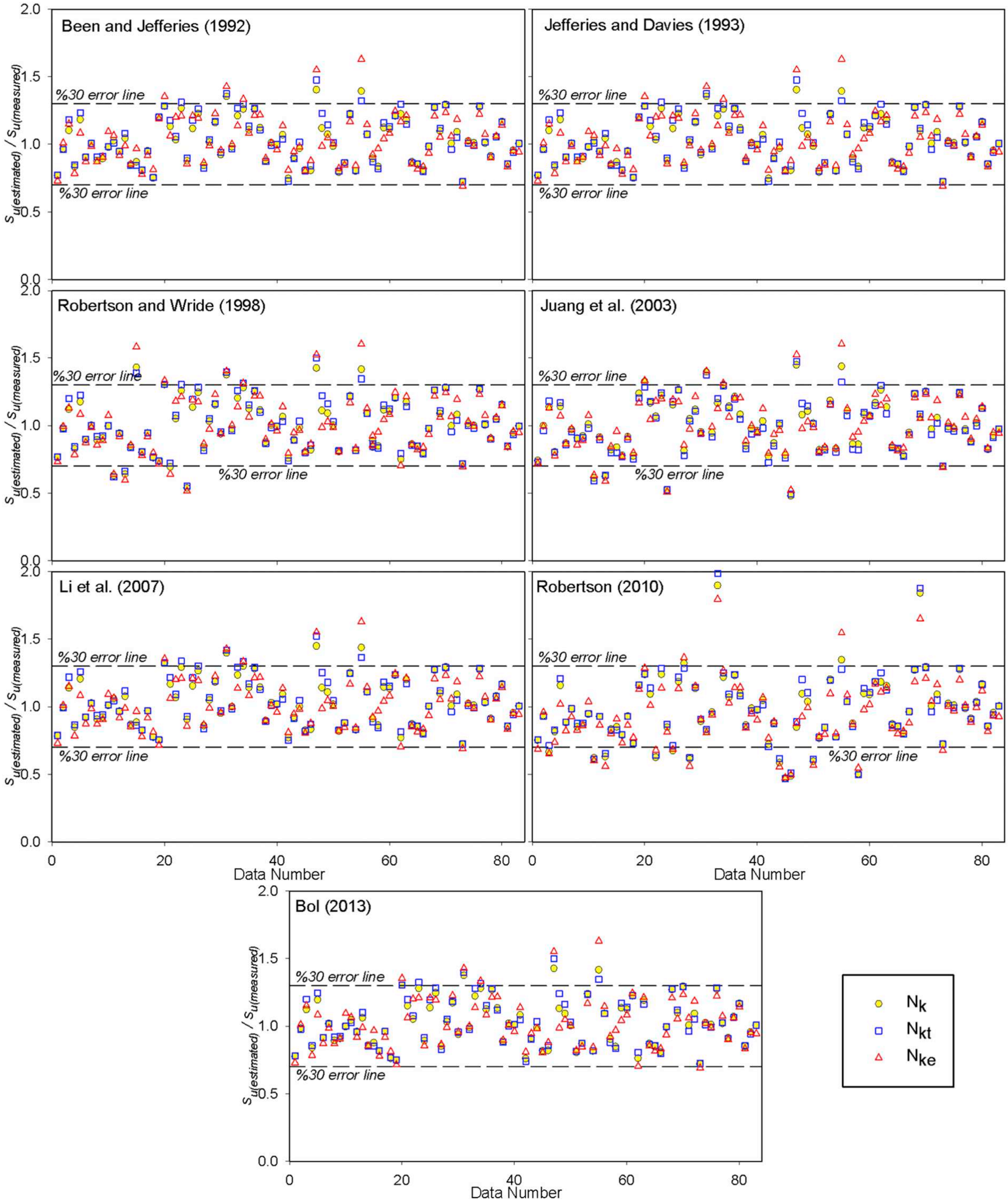

Fig. 12. Distribution of discrepancy ratios.

Table 11
Equation models and statistical evaluation of the results.

$I_{c\text{-}CPT}$ calculated from	Model ($I_{c\text{-}MLRA}$)	*AARE*	*MSE*	R^2
Been and Jefferies (1992)	$= 1.604 + (2.484\sigma_{vo}) + (0.00806w_L) + (0.00862w_P)$-$(0.000271w_n) + (0.00481\text{FC})$	8.6190	0.0813	0.493
Jefferies and Davies (1993)	$= 1.600 + (2.583\sigma_{vo}) + (0.00807w_L) + (0.00864w_P) + (0.0000558w_n) + (0.00482\text{FC})$	8.6686	0.0831	0.494
Robertson and Wride (1998)	$= 2.118 + (2.268\sigma_{vo}) + (0.00397w_L) + (0.00482w_P) + (0.00274w_n) + (0.00216\text{FC})$	5.3116	0.0356	0.469
Juang et al. (2003)	$= 2.035 + (0.966\sigma_{vo}) + (0.00469w_L) + (0.00716w_P) + (0.00259w_n) + (0.00402\text{FC})$	5.5506	0.0421	0.452
Li et al. (2007)	$= 1.795 + (2.931\sigma_{vo}) + (0.0110w_L) + (0.0113w_P)$-$(0.00225w_n) + (0.00612\text{FC})$	9.8096	0.1353	0.491
Robertson (2010)	$= 2.380$-$(0.766\ \sigma_{vo}) + (0.00393w_L) + (0.00780w_P) + (0.00241w_n) + (0.00334\text{FC})$	4.9969	0.0359	0.423
Bol (2013)	$= 2.214 + (2.811\sigma_{vo}) + (0.00896w_L) + (0.0118w_P)$-$(0.00216w_n) + (0.00492\text{FC})$	7.8753	0.1044	0.501

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