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EMPIRICAL RELATIONSHIPS OF CPTu RESULTS AND UNDRAINED SHEAR STRENGTH

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ABSTRACT

Cone penetration testing (CPTu) and pore pressure measurement have long been used to estimate the undrained shear strength (S_u) of clay, although the evaluation of strength from CPTu results in fine-grained soil is mostly empirical. In fact, for various reasons, in-situ methods such as CPTu and laboratory methods may give different results for S_u for a specific soil. The present study correlated the results of 22 CPTu and 50 uniaxial compression tests conducted on quaternary alluvial clay from southern Iran. Empirical relationships with acceptable coefficients of correlation between S_u and CPT parameters were obtained and values of the empirical cone factors N_k, N_{kt}, N_{ke}, and $N_{\Delta u}$ are proposed for these clays. A comparison of these values with values proposed in previous studies suggests that the cone factors depend on the type of the reference test used. Although the plasticity and consistency index also affect cone factors, it was not possible to determine an acceptable empirical relationships for these parameters.

Key words: CPTu, undrained shear strength, empirical cone factors.

1. INTRODUCTION

The undrained shear strength (S_u) of soil is a widely used design parameter in engineering; however, the value obtained depends on the testing apparatus and procedure used (Myftaraga and Koreta 2013) as well as the direction of loading, boundary conditions, stress level, sample disturbance, testing method (failure mode), strain rate, stress path and other factors (Bond 2011; Mayne *et al*. 2009). Uniaxial, triaxial and direct shear tests on undisturbed samples are routine laboratory tests for the determination of S_u, whereas the field vane shear test is an in-situ test method favored by many.

The increasing use of the CPTu in ground investigations because of its increased reliability, high speed, cost effectiveness, continuous soil profile make it a valuable tool in characterizing subsurface conditions and in assessing soil properties. This has resulted in the need for methods to determine the value of S_u from the test results. Theoretical correlations based upon bearing capacity theory (Terzaghi 1943; de Beer 1977), cavity expansion (Skempton 1951; Vesic 1975), analytical and numerical methods (Ladanyi 1967) and strain path methods (Teh 1987) have been proposed, but because they require the use of many assumptions, they offer no advantage over empirical methods. Empirical methods allow determination of S_u using total cone resistance (Eqs. (1) and (2)), effective cone resistance (Eq. (3)) and pore water pressure (Eq. (4)) (Lunne *et al*. 1997) as follows:

$$S_u = (q_c - \sigma_v) / N_k \tag{1}$$

$$S_u = (q_t - \sigma_v) / N_{kt} \tag{2}$$

$$S_u = (q_E / N_{ke}) = (q_t - u_2) / N_{ke} \tag{3}$$

$$S_u = (\Delta u / N_{\Delta u}) = (u_2 - u_0) / N_{\Delta u} \tag{4}$$

where q_c is cone resistance, q_t is corrected cone resistance, q_E is effective cone resistance, u_2 is pore pressure measured immediately behind the cone tip, u_0 is hydrostatic pore water pressure, σ_v is total vertical overburden stress and N_k, N_{kt}, N_{ke}, and $N_{\Delta u}$ are empirical cone factors that depend on the geological conditions and type of reference test used. Table 1 summarizes the values proposed by different researchers for these empirical cone factors. In addition, Karlsrud *et al*. (1997) has proposed a method for determining S_u by averaging the results from different methods, as shown in last row in Table 1.

Quaternary geological interpretations prompted by the development of construction on recent alluvium are important (Hawkins 1994). Yim (1993) used geophysical methods, field sampling and field and laboratory testing to study offshore quaternary sediment in Hong Kong. Zastrozhnov *et al*. (2017) developed regional charts for quaternary deposit in European Russia. Fakher *et al*. (2007) proposed classification for Tehran alluvium based on a combination of geological and geotechnical data. He *et al*. (2017) investigated alluvium sediment in Longshan, China and found sediment ranging from boulders to clay in this area. El May *et al*. (2015) studied geotechnical characterization of quaternary alluvial deposits in Tunis. Ku *et al*. (2010) studied the reliability of CPT I_c as an index for mechanical behavior classification of in quaternary alluvium deposits (I_c is consistency index). Zein (2017) proposed a relationship between undrained shear strength and CPT for fine grained soil from three Sudanese states with different OCR, over consolidation ratio, values.

Hajimohammadi *et al*. (2010) presented a relationship between shear wave velocity and cone tip resistance in the silty clay soil of southern Iran. Cheshomi and Ezzedi (2016) proposed a new soil classification based on CPTu test results for some parts of the quaternary alluvium in southern Iran. Cheshomi *et al*. (2015) compared the results of 43 dissipation tests and 35 one-dimensional laboratory consolidation tests in quaternary alluvium in southern Iran with C_V values obtained from CPTu and one-dimensional laboratory consolidation testing (C_V is coefficient of consolidation).

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Table 1 Values proposed by researchers for empirical cone factors

Reference	Reference test	N_k	N_{kt}	N_{ke}	$N_{\Delta u}$	Soil type	Location
Kjekstad *et al*. (1978)	Triaxial compression	17	–	–	–	Non-fissured over-consolidated clays	North Sea
Lunne and Kleven (1981)	Field vane shear	11 ~ 19	–	–	–	Normally consolidated marine clays	North Sea
Senneset *et al*. (1982)		–	–	6 ~ 12	–	Clays (3% < *PI* < 50%)	
Lunne *et al.* (1985)	Triaxial compression	–	–	1 ~ 13	4 ~ 10	Clays (N_{ke} varies with B_q)	North Sea
Aas *et al.* (1986)	Triaxial compression, triaxial extension and direct shear	–	8 ~ 16	–	–	Clays (3% < *PI* < 50%) (N_{kt} increases with I_p)	Norway
La Rochelle *et al*. (1988)	Vane shear	–	11 ~ 18	–	7 ~ 9	Sensitive clay No correlation found between N_{kt} and *PI*	Canada
Rad and Lunne (1988)	Triaxial compression	–	8 ~ 29	–	–	Clays N_{kt} varies with OCR	Nigeria
Powell and Quarterman (1988)	Triaxial compression	–	10 ~ 20	–	–		
Karlsrud (1996)	Triaxial compression	–	6 ~ 15	2 ~ 10	6 ~ 8	Soft to medium stiff clay (N_{kt} decreases with B_q)	Norway
Jörß (1998)		15 ~ 20	–	–	–	Marine clay	Northern Germany
Chen (2001)		5 ~ 12	–	–	–	Klang clay	Indonesia
Gebreselassie (2003)		7.6 ~ 28.4	–	–	–	Sludge, marin young clay, lacustrine soft soil, quaternary clay and clay stone, tertiary clay	Germany
Hong *et al*. (2010)	Triaxial compression	–	7 ~ 20	3 ~ 18	4 ~ 9	Busan clay, 25% < *PI* < 40%	Korea
Almeida *et al*. (2010)	Vane shear	–	4 ~ 16	–	–	Very soft clay High plasticity, 42% < *PI* < 400%	Brazil
Average value Karlsrud *et al*. (1997)		15.2	13.5	8.1	7.1		

Note: *PI*: plasticity index; OCR: over consolidation ratio; B_q: piezocone pore pressure parameter

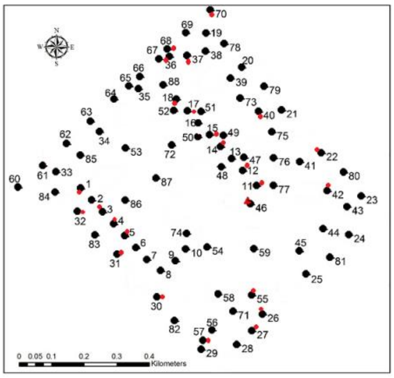
In the present study, S_u was determined by comparing the results of 50 uniaxial tests and the cone penetration test extracted from the corresponding depths in southern Iran. Values for the empirical cone factors (N_k, N_{kt}, N_{ke}, $N_{\Delta u}$) are proposed for the fine-grained soil tested. The effects of soil plasticity and stiffness on cone factors were also investigated. The values obtained in the present study were compared with the values proposed by different researchers.

2. STUDY AREA

Sampling and testing was performed in southern Iran, as shown in Fig. 1(a). Table 2 shows the number of boreholes and tests performed in the area and Fig. 1(b) shows the location of the boreholes and CPTu tests. Geotechnical investigation in this area was conducted with the aim of industrial construction. Disturbed and undisturbed samples were taken (usually at intervals of 2 to 3 m). The geological models shown in Fig. 2 were derived based on the results of classification tests.

This area has a surface layer (fill material, sand and gravel) with a thickness of about 2 m. Below this, the subsurface material consists mainly of fine-grained layers (lean clay, little fat clay and clayey silt) with low to high plasticity; thus, the soil variability is low. The groundwater table observed in boreholes was at a depth of 3 ~ 5 m below the surface. Below a depth of about 17.5 m, silt and sand layers of variable thickness (0.2 to 1.3 m) were observed in the fine sediment. In these areas, many seasonal and permanent rivers have carried large amounts of sediment eroded from the highland and have deposited the fine material on the plain. This process is the main factor in evaluation of the alluvium from the quaternary period.

(a) Location of the study area in southern Iran



(b) Borehole and CPTu tests location

Fig. 1 Sample and test area in southern Iran

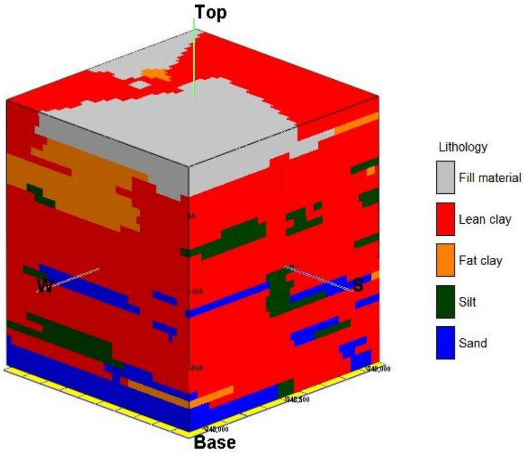


Fig. 2 Geological model for the area studied (in this area after the top soil is a layer of lean clay to depth of about 17.5 meter)

Table 2 Geographical location of study area and number of boreholes and tests performed in this area

Number of boreholes	Depth of boreholes (m)	Number of uniaxial tests	Number of CPTu tests	Geographical coordinates (in center area)	Location
50	15 ~ 30	50	22	30°74'38" N 48°42'41" E	Khouzestan plain

3. TEST RESULTS

3.1 Identification Tests

Identification testing was performed on all samples and comprised particle size analysis, Atterberg limits and soil classification tests according to ASTM D422-63:2007, ASTM D4318:2010 and ASTM D2487:2010. Table 3 shows the test results, including the designations using the unified classification system (CL and CH), determination of the liquid limit (*LL*) and plasticity (*PI*) and consistency indices (*CI*). Figure 3 shows the changes in the liquid limit, plasticity index and consistency index by depth for all samples. The liquid limit ranged from 25 to 55, the plasticity index from 8 to 28 and the consistency index from 0.18 to 1.5. As shown, the values of *LL*, *PI*, and *CI* varied greatly at any specific depth and did not show a recognizable trend by depth.

3.2 Uniaxial Compression Testing

This test was performed in accordance with ASTM D2166:2006 on undisturbed samples 38 mm in diameter and 76 mm high at loading rates of 1.5% to 2% strain (1.5 mm/min). This test method covers the determination of the unconfined compressive strength of cohesive soil, using strain-controlled application of the axial load. This test method provides an approximate value of the strength of cohesive soils in terms of total stresses. The bore holes were drilled using the rotary method and at 2 ~ 3 m intervals, thin-walled tube sampling (undisturbed) was carried out. The results of all the UCS tests are summarized in Table 3. Figure 4 shows that S_u varied from 17.85 to 104.38 kPa.

Table 3 Physical properties of soil, uniaxial test results and CPTu parameters

BH No.	Depth (m)	USCS	Fine content (%)	*LL* (%)	*PI* (%)	*CI*	q_u (kPa)	S_u (kPa)	CPTu data (kPa)		
									q_c	f_s	u
BH-1	7 ~ 7.5	CL	98	41	19	0.53	57.68	28.84	986	20	147
BH-3	11 ~ 11.4	CL	99	38	16	0.19	89.27	44.64	1204	27	234
BH-3	7 ~ 7.5	CL	99	37	15	0.53	144.21	72.1	1799	0	174
BH-4	14 ~ 14.4	CL	100	34	15	0.47	124.98	62.49	2108	80	326
BH-4	4.6 ~ 4.9	CL	99	30	11	0.36	75.54	37.77	926	0	108
BH-5	10 ~ 10.45	CL	100	40	21	0.38	109.87	54.94	1276	19	273
BH-11	9 ~ 9.45	CL	98	28	8	1.25	82.4	41.2	1366	7	219
BH-12	11.5 ~ 12	CL	97	40	19	0.89	151.07	75.54	2018	52	347
BH-12	7 ~ 7.5	CL	94	34	15	1	140.09	70.04	1826	102	335
BH-14	11 ~ 11.45	CL	100	49	23	1.09	178.54	89.27	2222	67	419
BH-15	7 ~ 7.4	CL	99	33	14	0.93	112.62	56.31	1323	31	202
BH-16	14 ~ 14.4	CL	99	30	11	1	188.16	94.08	2360	76	488
BH-16	7 ~ 7.5	CL	98	33	17	0.47	74.16	37.08	1040	32	192
BH-16	9 ~ 9.5	CL	95	33	12	0.75	115.37	57.68	1605	36	261
BH-17	12 ~ 12.5	CL	93	25	10	0.2	130.47	65.24	1469	50	273
BH-17	8.5 ~ 9	CL	78	29	12	1	50.82	25.41	956	24	177
BH-18	10.5 ~ 11	CL	83	42	19	1.05	130.47	65.24	1863	22	304
BH-19	13.5 ~ 14	CL	98	33	13	1.08	64.55	32.27	1168	52	268
BH-22	13.4 ~ 13.7	CL	84	45	19	1.21	203.26	101.63	2165	35	419
BH-26	8.5 ~ 9	CL	100	39	17	1	148.33	74.16	1813	107	358
BH-27	12 ~ 12.5	CL	97	35	16	1.06	185.41	92.7	1995	59	433
BH-29	13.5 ~ 14	CL	100	39	17	0.76	208.76	104.38	2170	75	472
BH-29	2.5 ~ 3	CL	100	44	22	1	104.38	52.19	1423	26	111
BH-29	7.5 ~ 8	CL	99	40	17	0.88	74.16	37.08	1245	34	158
BH-30	13 ~ 13.5	CL	99	32	13	0.69	123.2	61.6	1946	35	299
BH-31	13.5 ~ 14	CL	99	36	16	0.75	57.68	28.84	802	34	291
BH-32	12.5 ~ 13	CL	100	38	18	0.44	49.44	24.72	956	44	212
BH-32	8 ~ 8.5	CL	100	32	14	0.86	83.78	41.89	1395	40	130
BH-33	17 ~ 17.5	CL	99	43	21	1.14	171.68	85.84	2193	121	453
BH-35	11.5 ~ 12	CL	100	31	9	0.67	35.71	17.85	791	35	158
BH-36	7 ~ 7.5	CH	99	53	28	0.96	137.34	68.67	1300	24	289
BH-36	11 ~ 11.5	CL	100	36	15	1	107.13	53.56	1453	34	307
BH-37	16.5 ~ 17	CL	99	40	18	0.83	104.38	52.19	1436	59	353
BH-37	11 ~ 11.5	CL	99	33	14	0.64	100.26	50.13	1445	33	260
BH-40	17.5 ~ 18	CL	98	34	13	0.46	98	49	1546	63	334
BH-42	3 ~ 3.5	CL	99	28	10	1.2	111.25	55.62	1327	65	152
BH-44	7.5 ~ 8	CL	98	45	20	1.15	123.61	61.8	1838	21	320
BH-44	3 ~ 3.5	CH	99	53	24	1.13	135.97	67.98	1795	0	196
BH-46	11.5 ~ 12	CL	95	28	10	0.8	104.38	52.19	1316	18	263
BH-48	12 ~ 12.5	CL	100	49	25	0.92	182.66	91.33	1915	42	390
BH-48	5.5 ~ 6	CL	99	40	17	1.18	103.01	51.5	1062	34	208
BH-49	15 ~ 15.5	CL	98	34	15	0.73	178.54	89.27	2240	70	442
BH-49	8 ~ 8.5	CL	99	38	17	0.94	38.46	19.23	591	12	132
BH-50	7.5 ~ 8	CL	99	38	21	0.76	54.94	27.47	800	20	154
BH-52	7.5 ~ 8	CL	100	29	10	0.7	87.9	43.95	1131	32	204
BH-53	12.5 ~ 13	CL	99	28	8	0.63	86.52	43.26	1407	24	231
BH-54	12 ~ 12.5	CL	99	34	14	1	37.08	18.54	699	16	174
BH-55	7 ~ 7.5	CL	100	35	13	1.15	145.58	72.79	1506	41	270
BH-57	14 ~ 14.5	CL	98	42	18	0.67	178.54	89.27	1964	104	474
BH-70	7 ~ 7.5	CL	98	35	13	1.46	126	63	1278	46	277
Max.			100	53	28	1.46	208.76	104.38	2360	121	488
Min.			78	25	8	0.19	35.71	17.85	591	0.00	108
Ave.			98	37	16	0.84	114.15	57.07	1489	42	273

As with the classification data, the results span a range at any specific depth and showed a slight increase by depth. Figure 5 shows the distribution consistency of soil in the study areas. The stiffness of the clayey soil ranged from very soft to hard.

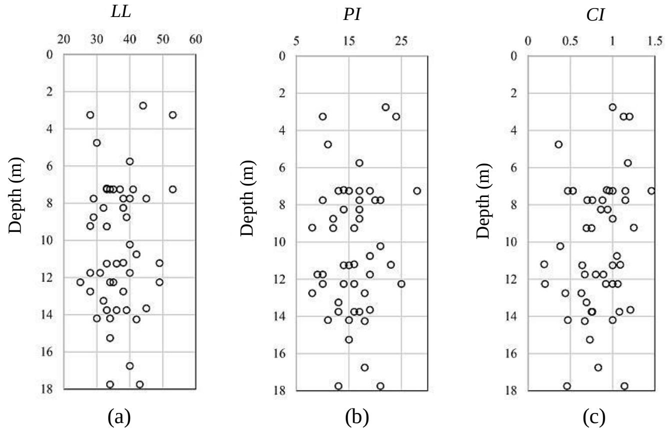


Fig. 3 Values of (a) liquid limit (*LL*); (b) plasticity index (*PI*); and (c) consistency index (*CI)* of samples

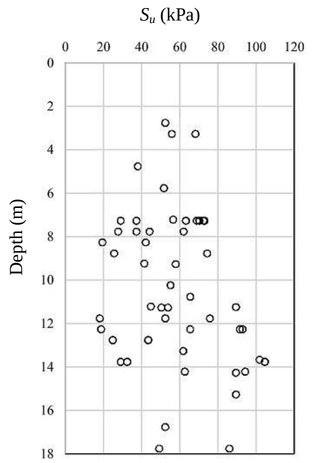


Fig. 4 Values of undrained shear strength (S_u) obtained from the uniaxial tests

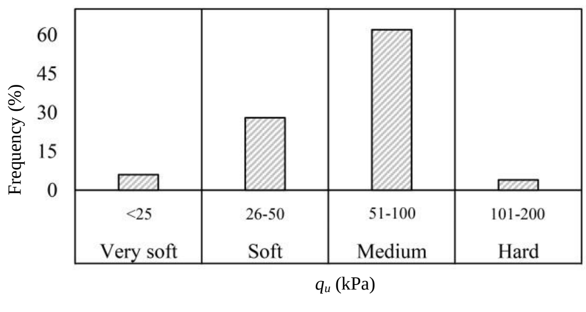


Fig. 5 Distribution consistency of soil in the study area. The stiffness of the clayey soils ranged from very soft to hard

4. CONE PENETRATION TESTING

CPTu was carried out in accordance with ASTM D5778: 2012. In this test, a penetrometer tip with a conical point having a 60° apex angle and cone base area of 10 cm^2 advanced at a constant rate of 20 mm/sec through the soil. The force on the cone required to penetrate the soil is measured as cone resistance (q_c). Sleeve resistance (f_s) represents the sleeve strength against penetration and is calculated by dividing the measured axial force by the sleeve surface area. The pore water pressure induced during advancement of the cone is measured using a pressure transducer (u_2). Total cone resistance q_t is given as:

$$q_t = q_c + u_2(1-a) \tag{5}$$

According to the definition offered by Lunne *et al*. (1997), cone area ratio *a*, is approximately equal to the ratio of the cross-sectional area of the load cell or shaft divided by the projected area of the cone. The CPTu equipment used was manufactured by van den Berg, A.P.

Figure 6 shows examples of CPTu profiles of the successive CPTu testing conducted from ground level to the desired depth and the relevant q_c, f_s, and u_2 values are shown in Table 3. Based on the profile, the top soil was followed by a layer of lean clay to a depth of 17.5 m. A silty sand layer about 1 m in thickness (at depths of 11 and 17.5 m) was then observed. Similar profiles for other CPTu tests have been produced and the following steps for comparison of S_u and CPTu parameters have been performed:

- The borehole nearest to the CPTu is selected according to studies conducted by Ku *et al*. (2010). Because soil variability in this area soil is low, the distance between the selected borehole and the CPTu is less than 3 m.
- A uniaxial test carried out on undisturbed sample is obtained from side-by-side boreholes.
- The average q_c, q_t, f_s, and u_2 values were extracted from the CPTu profile at the same depth at which the uniaxial test is done.

Table 3 lists the depths at which uniaxial testing was performed and the average values for q_c, q_t, f_s, and u_2 for the same depth are presented as well. Figure 7 shows the average values for q_c, q_t, f_s, and u_2 versus depth.

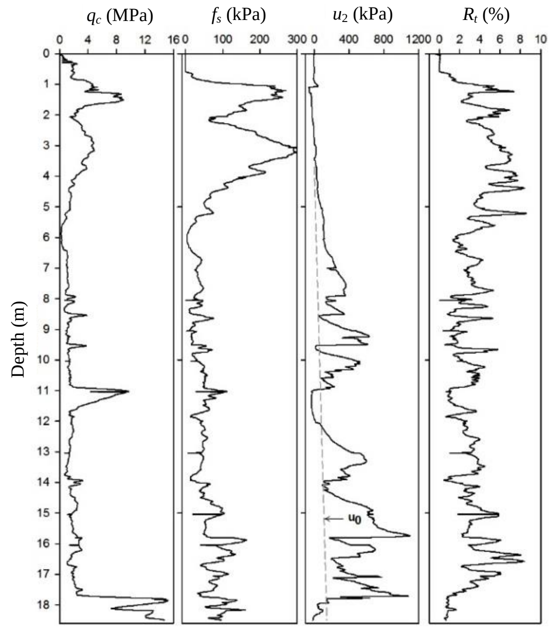


Fig. 6 Examples of CPTu profiles (q_c, f_s, u_2, and R_f). After the top soil is a layer of lean clay to depth of 17.5 m with silty sand layer with about 1 m thickness (at a depth of 11 and 17.5 meter)

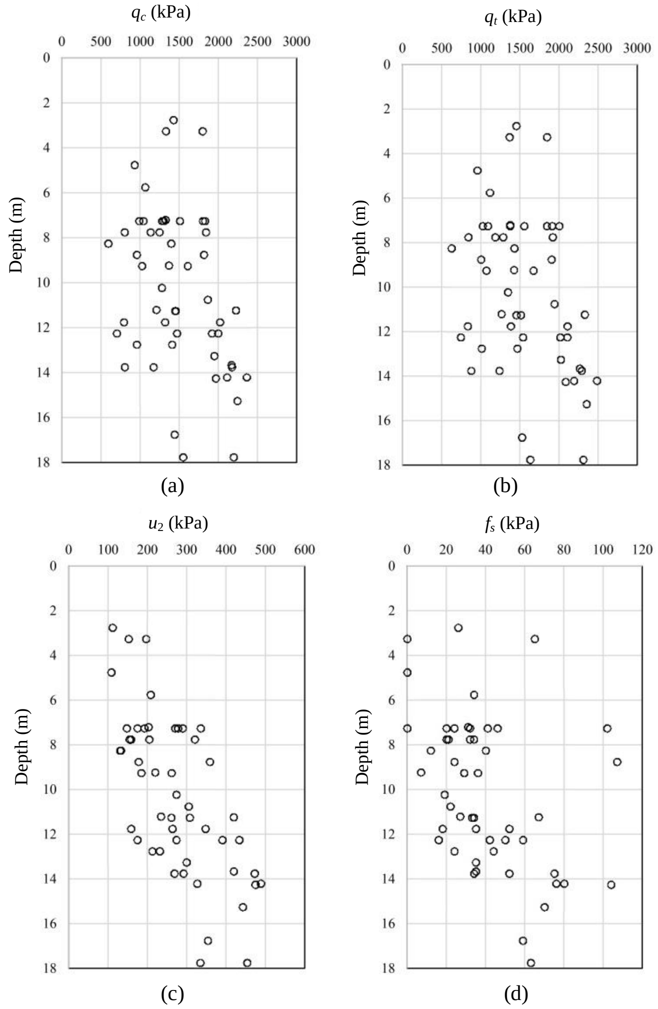


Fig. 7 **Values of (a) q_c, (b) q_t, (c) u_2 and (d) f_s with respective depth**

5. DETERMINATION OF EMPIRICAL CONE FACTORS

Empirical cone factors N_k, N_{kt}, N_{ke}, and $N_{\Delta u}$ were determined from the data listed in Table 3 using the S_u values from uniaxial testing as a reference.

5.1 Empirical Cone Factor N_k

Figure 8 shows the correlations for N_k using S_u from uniaxial testing versus $q_c - \sigma_v$. As shown, the upper and lower boundaries for N_k are 29 and 18, respectively. The proposed best value for the soil in the study areas was 22. Equation (6) describes the empirical relationship between these two variables as:

$$S_u = (q_c - \sigma_v)/22 \quad R^2 = 0.83 \tag{6}$$

5.2 Empirical Cone Factor N_{kt}

Figure 9 shows the results of determination of N_{kt} using S_u from uniaxial testing versus $q_t - \sigma_v$. As shown, N_{kt} varied from 18 to 31 for the study areas, with the best value equaling 23. Equation (7) describes the empirical relationship between these two variables as:

$$S_u = (q_t - \sigma_v)/23 \quad R^2 = 0.84 \tag{7}$$

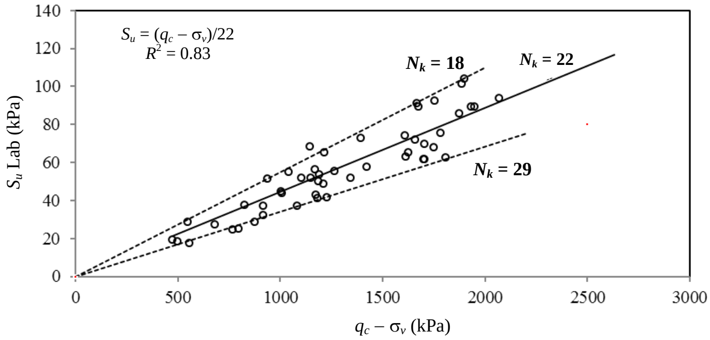


Fig. 8 **S_u from uniaxial tests vs. ($q_c - \sigma_v$) for determination of N_k. Based on the upper and lower boundaries max. and min. N_k are 29 and 18. The best value is 22**

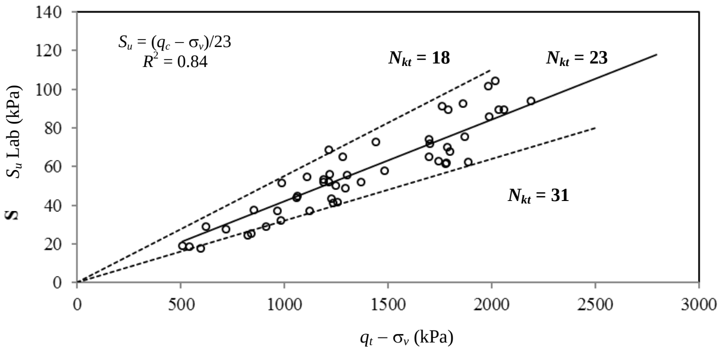


Fig. 9 **S_u from uniaxial testing vs. ($q_t - \sigma_v$) for determination of N_{kt}. Based on the upper and lower boundaries max. and min. N_{kt} are 31 and 18. The best value for N_{kt} is 23**

5.3 Empirical Cone Factor N_{ke}

Figure 10 plots the shear strength obtained in the laboratory versus the effective cone resistance obtained from CPTu to determine the N_{ke}. The best value for the factor was 22, which is equivalent to the value for N_k ($N_k = N_{ke} = 22$). The range of variation of this factor based on the upper and lower boundaries as shown in Fig. 10 is between 17 and 31, which is slightly wider than the range for N_k. Equation (8) describes the empirical relationship between these two variables as:

$$S_u = (q_E / N_{ke}) = (q_t - u_2)/22 \quad R^2 = 0.79 \tag{8}$$

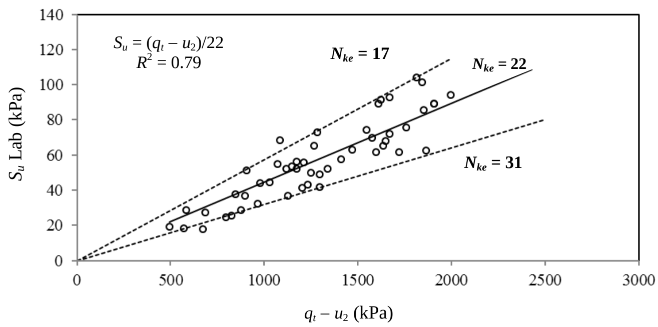


Fig. 10 **S_u from uniaxial testing vs. ($q_t - u_2$) for determination of N_{ke}. Based on the upper and lower boundaries max. & min. N_{ke} are 31 and 17. The best value for the factor is 22**

5.4 Empirical Cone Factor $N_{\Delta u}$

Figure 11 plots the shear strength obtained from uniaxial testing versus excess pore water pressure to determine $N_{\Delta u}$. The value for $N_{\Delta u}$ varies from 2.8 to 5.9 and the best value was 3.8. As seen, the amount and the range of variation of this factor is significantly smaller than for the other three factors. Equation (9) describes the empirical relationship between these two variables as:

$$S_u = (\Delta u / N_{\Delta u}) = (u_2 - u_0)/3.8 \quad R^2 = 0.83 \tag{9}$$

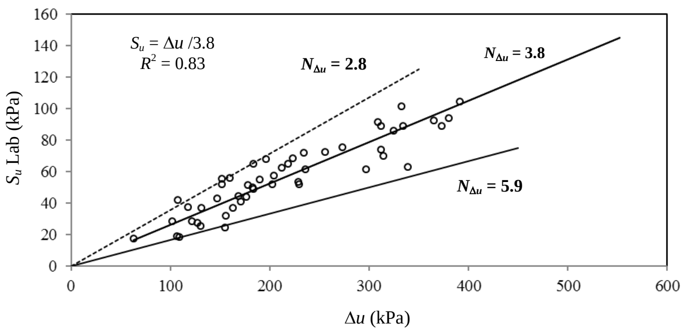


Fig. 11 S_u from uniaxial tests vs. Δu for determination of $N_{\Delta u}$. This factor varies from 2.8 to 5.9, and the best value is 3.8

Table 4 shows the best value and the range of the cone factors obtained for the study area in Figs. 8 to 11. In these figures, the upper and lower bounds are plotted to place the majority of the data between two lines, and these lines cross the origin of the coordinate.

Table 4 Proposed empirical cone factors using uniaxial test as reference

	N_k	N_{kt}	N_{ke}	$N_{\Delta u}$
Minimum	18	18	17	2.8
Best value	22	23	22	3.8
Maximum	29	31	31	5.9

SPSS software was employed to evaluate the normality of the variable and meaningfulness of the empirical relationships. Table 5 shows that the skewness and kurtosis of the variable fall between –2 and 2. The variable can be said to be normally distributed.

Table 5 Skewness and Kurtosis values for variablse

	Skewness		Kurtosis	
	Statistic	Std. Error *	Statistic	Std. Error
S_u	0.206	0.337	–0.691	0.662
q_c	0.069	0.337	–0.927	0.662
u_2	0.409	0.337	–0.715	0.662
q_t	0.066	0.337	–0.922	0.662
σ_v	–0.075	0.337	–0.936	0.662
Δu	0.492	0.337	–0.796	0.662

* The Std. Error or standard error of a statistic is the standard deviation of its sampling distribution or an estimate of that standard deviation.

Table 6 shows the values obtained from the t-test for the empirical relations (Eqs. (6) to (9)). Given that the meaningfulness is less than the amount of error (this test considered an error of 5%), the correlation coefficient was meaningful for the relationships between these parameters.

Table 6 Results of t-test to determine meaningfulness of proposed relationships

Model	Unstandardized coefficients		Standardized coefficients	t	Sig. *
	B *	Std. Error	Beta *		
$q_c - \sigma_v$	0.048	0.003	0.916	15.846	.000
$q_t - \sigma_v$	0.048	0.003	0.922	16.450	.000
$q_t - u_2$	0.051	0.004	0.896	13.949	.000
Δu	0.245	0.016	0.915	15.750	.000

* Beta coefficients is the estimates resulting from a regression analysis that have been standardized. B coefficients is the regression carried out on original (unstandardized. Sig. is level of significance. The Sig of .000 means the results are highly significant.

6. RELATIONSHIP BETWEEN CONE FACTORS AND PHYSICAL PROPERTIES

Figures 12 and 13 show the plasticity and consistency index

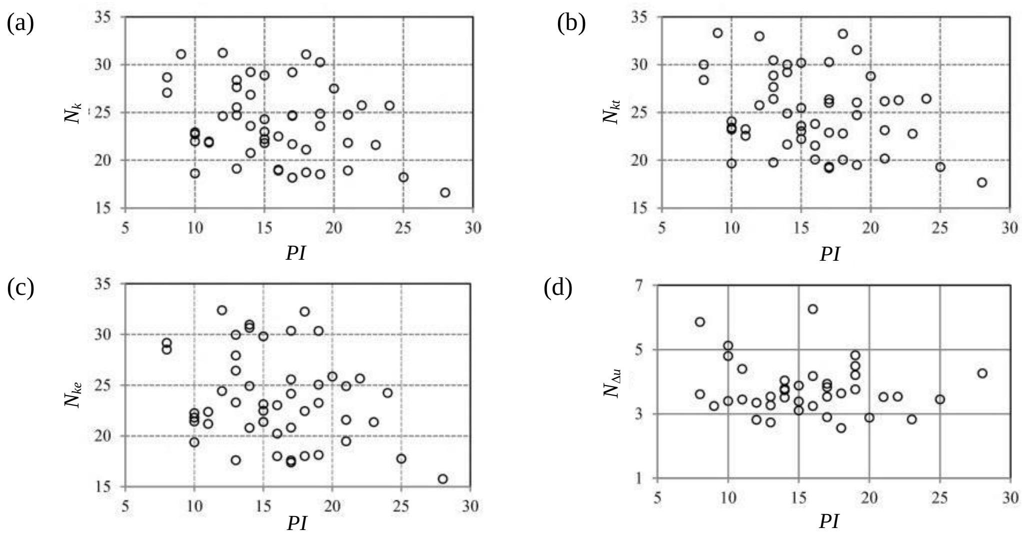


Fig. 12 Plasticity index vs. cone factor for the study area: (a) $N_k - PI$; (b) $N_{kt} - PI$; (c) $N_{ke} - PI$; (d) $N_{\Delta u} - PI$

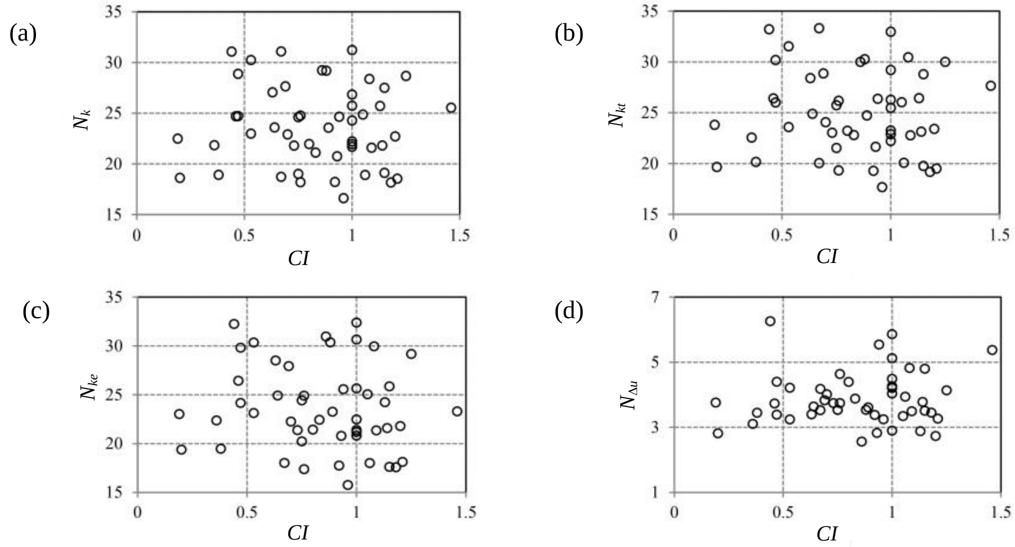


Fig. 13 Consistency index vs. cone factor for study areas: (a) $N_k - CI$; (b) $N_{kt} - CI$; (c) $N_{ke} - CI$; (d) $N_{\Delta u} - CI$.

values for different cone factors. There is no empirical relationship with acceptable correlation between plasticity index and the cone factors. It is not possible to offer an empirical relationship with an acceptable correlation between the plasticity index, consistency index and the cone factors. Aas et al. (1986) reported that the empirical cone factor has a direct relationship with the plasticity index, but subsequent studies have not confirmed this relationship (Remai 2013).

7. COMPARISON OF RESULTS WITH PREVIOUS RESEARCH

Figures 14(a) ~ 14(d) compare the values determined for the cone factors in the present study with those provided by previous research (Table 1). The values obtained for N_k, N_{kt}, and N_{ke} from the present study are slightly higher than those determined in previous studies and the value obtained for $N_{\Delta u}$ is slightly less than those determined in previous studies.

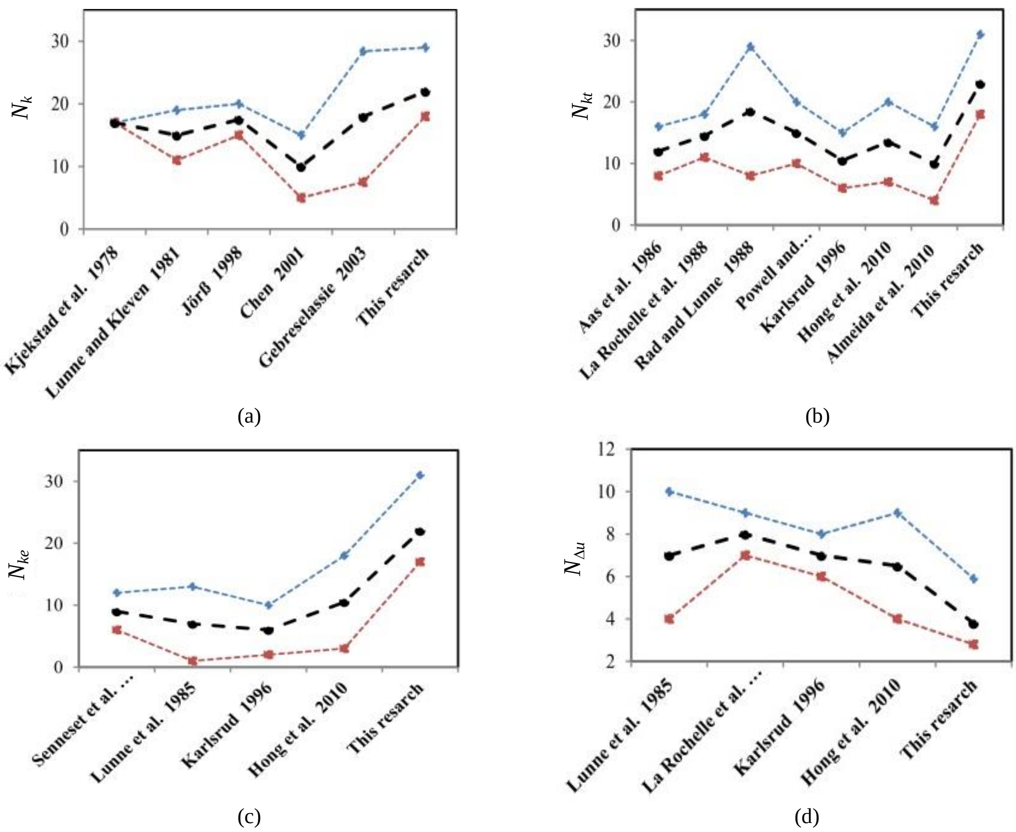


Fig. 14 Cone factors from present study vs. those from previous studies

The reference tests chosen in the earlier studies were the triaxial and field vane shear tests. In the present study, the uniaxial test was used as the reference test. It can be concluded that a major reason for the difference in value is the type of reference test. Other possible reasons are soil disturbance in the process of sampling and preparation of laboratory testing and the material properties of the study area. There are overall similarities between the values obtained in the present study and those form previous studies. There is also a significant and acceptable correlation between the variables.

8. CONCLUSIONS

A comparison of the results of 50 uniaxial and cone penetration tests on quaternary fine-grained alluvium (*CL* and *CH*) soil with q_u values of 35.71 to 208.76 kPa and *PI* values of 8 to 28 in southern Iran indicates that there is an acceptable and significant correlation between the undrained shear strength of the soil and the cone parameters (q_c, q_t, f_s, and u_2). The cone factors proposed for the study area were $N_k = N_{ke} = 22$, $N_{kt} = 23$, and $N_{\Delta u} = 3.8$. The results for these factors were compared with the results for cone factors presented by previous studies and showed only a slight difference, which could be the result of the use of different reference tests or from soil and local site specifications of the areas under study. The reference tests used in the previous studies were the triaxial and field vane shear tests. The present study used the uniaxial test as the reference test, so the test condition and sample disturbance in these tests differed. A comparison of cone factors with physical soil properties, such as the plasticity and consistency indices, did not reveal a reliable correlation.

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