Scale Effect Aspects for Correlation CPTu Data in Deep Foundation Analysis and Design

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Abstract: In recent years, the cone penetration test (CPT) is considered as one of the most important In-situ tests, because of its simple, fast, reliable, and economical nature. The scale effects between the pile toe resistance and CPT cone is considered by different researchers, while the effects of these factors are not taken into account in making a relation between CPT sleeve friction and pile shaft resistance. As the main purpose of this research, in order to study these effects on the shaft resistance, the shear strain produced under CPT sounding and pile load tests were studied. Then, there was an effort in relating the scale effect between CPT and pile to the shear strain levels. As a result of the difference in the rate of penetration, the effect of generating excess pore pressure that is important for fine grained soils is considered. The database consists of 42 case studies of pile load tests include the CPT profile is collected from 24 different sites. Next a method is presented in determining the shaft bearing capacity, with considering scale effects and this method is evaluated to four methods including: Clisby et al.,Tumay and Fakhroo, Price and Wardle and Takesue et al.The result of analysis showed that the proposed method estimated the shaft capacity with the highest accuracy in comparison to other methods.

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Keywords: Scale effect, Cone penetration test, Pile, Static load test, Shaft resistance.

1. Introduction

So many methods had been provided for determining deep foundation bearing capacity using cone penetration test as 50 years ago. This method is largely based on empirical relations and is little based on analytical relations. Various parameters such as soil type and pile construction method as well as its material and scale should be considered in these designs. Some of researchers would consider essentially existence of so many methods in this field because of researchers' knowledge evolution in geotechnical engineering as well as obtaining more accurate understanding out of soil behavior, foundation and various conditions prevailing on constructed projects in different countries, evolution of these methods and providing new methods.

Researches' results and investigating various case histories had shown that end and friction strength –recorded by penetrometer- would not be equaled to pile shaft and toe resistance. Because penetrated cone and pile had different scales considering geometric scales and penetration rate to soil. This would be in a way that penetrated cone has smaller scale in geometric scales and in larger scale in penetration rate and kind compared to pile. This would lead to differences in toe and shaft resistances' results between penetrometer and pile that would be expressed in form of scale effects. If these factors and their effects had been taken into account well, more appropriate relations would be made between cone penetration data and deep foundation bearing capacity values.

Researchers had considered scale effects in relation with toe resistance individually as well as in bearing capacity evaluation methods. But studies about these factors effect on friction resistance are scattered and no integral and unique method had been provided in this regard. This study had considered establishing relation between penetrometer friction strength and pile shaft resistance by considering scale effects.

2. Methods for determining shaft resistance using CPT data

Evaluating foundation bearing capacity using CPT data had been one of deep logging applications which would be administered in direct and indirect methods. In indirect method, parameters like friction angle (φ), untrained shear strength (Su) -obtained from CPT data- based on bearing capacity or cavity expansion theories should be used. These methods include errors due to ignoring horizontal stresses, soil compaction and strain softening. In engineering practices, direct methods including direct relations between CPT data and piles bearing capacities are more appropriate.

Methods determining pile shaft resistance using cone penetration data are divided into two groups:

- Methods in which cone resistance qc would be used to determine pile shaft resistance.
- Methods in which friction strength would be used to determine pile shaft resistance.

Because mechanism for creation and development of CPT friction strength compared to cone resistance would seem more like creating pile shaft resistance, fs could be considered an appropriate index for evaluating pile shaft resistance [1]. Methods for determining shaft resistance using friction strength include:

Penpile method [2]: This method provided by Clisby et al to handle Mississippi way in which CPT friction strength is used as follows:

$$\boldsymbol{r_s = f_{sa}/(1.5 + 14.47 f_{sa})} \qquad (1)$$

fsa is average CPT sleeve friction strength in MPa

Tummay and Fakhroo method [3]: This method is based on empirical study on clay in Louisiana. Unit shaft resistance would be determined by following equation:

$$\boldsymbol{R_s = k f_s} \qquad (2)$$

$$\boldsymbol{k = 0.5 + 9.5 e^{-0.09 f_s}} \qquad (3)$$

In this relation sleeve friction strenght fs are measured in KPa. Obtained results for K values range from 0.6 to 4.5 in order to fs to be in range 10 to 50 KPa. Upper bound unit shaft resistance rs would be equaled to 60 KPa.

Price and Wardle method [4]: In this method, CPT sleeve friction strength would be used as follows:

$$\boldsymbol{Rs = ks.fs \leq 120kPa} \qquad (4)$$

Ks for driven piles were 0.53, for jacking piles 0.62 and for bored piles 0.49 is recommended.

Takesue et al method: In this method, only pile shaft resistance had been estimated using sleeve friction strength and excess pore pressure according to Figure 1. Information used in this method includes driven and bored piles based in clay and sand [1].

$$\Delta u < 300kPa \quad => \quad f_p/f_s = \Delta u/1250 + 0.76 \qquad (5\text{-a})$$

$$300 < \Delta u < 1200kPa => f_p/f_s = \Delta u/200 - 0.5 \qquad (5\text{-b})$$

Accurately identifying various factors presented in existing differences between fs and rs would lead to achieving more appropriate results for evaluating shaft resistance by sleeve friction resistance values compared to cone point resistance values.

Scale effects had been considered as a method for foundations analysis and designing especially in deep foundations by researchers. Because cone penetration tests are somewhat similar to deep foundations considering geometry and appearance and penetration system to soil, there had been some efforts to establish an appropriate relationship between CPT data and pile analysis and design based on scale effects.

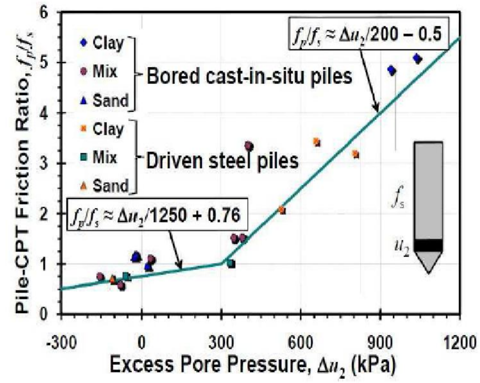

Fig 1. Direct method used in CPTu to evaluate shaft resistance

Differences between pile and CPT according to different directions had been shown in Figure 2. According to Fig 2-a, two primary differences including differences in pile and material differences as well as differences in their mechanism and penetration rate could be mentioned. CPT had a total displacement in soil and penetrates it with a standard rate of 20 mm/s, while pile had partial displacement and would be under loading test with so much lower rate. In CPT penetration, rate is distinct and in pile penetration, displacement is small and measurable. Another difference according to Fig 2-b is pile scales or in other hand, larger pile diameter compared to CPT ones.

Differences between pile material and penetrated cone would be considered in two matters of rigidness and difference in roughness of shaft. In this study, penetrated pile and cone had been assumed rigid for simplifying the problem. Many researchers such as Potyondy [5], Kishida and Uesugi [6] and Hammoud and Boumekik[7] performed studies in field of shaft roughness effect of materials such as steel and concrete on generated shear resistance between them and various soils. Moreover, DeJong et all [8] investigated CPT shaft roughness effect on friction strength results. Generally shaft roughness effect would be considered negligible according to large amount of surrounded soil around pile, even if this difference was effective between this penetrometer and pile, its impact on penetration rate and required force to penetrate pile and penetrometer in soil would be hidden. Studying scale effects influencing CPT and pile shaft resistance in their mechanism and rupture surface would be useful to investigate how these differences would affect their resistance. Rupture surface levels for a pile under axial compression loading had been shown in Fig 3.

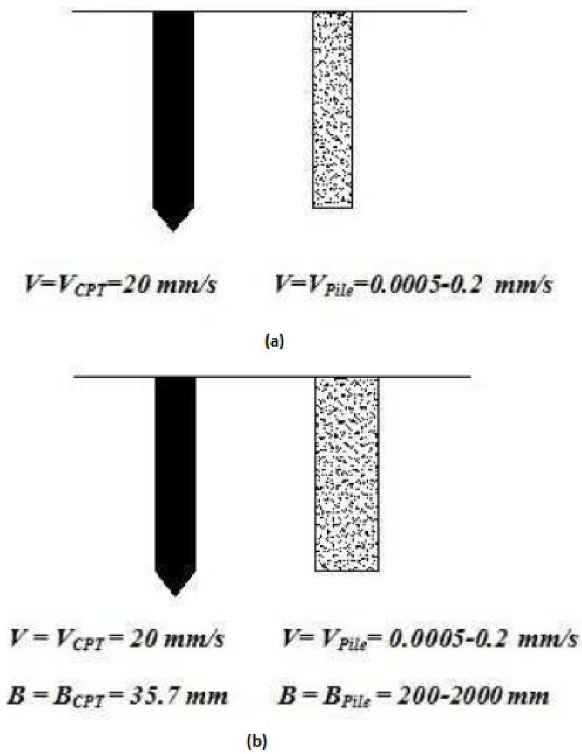

Fig 2. Schematic view of detailed differences between pile and CPT. a) differences in penetration rate and material. b) differences in scales.

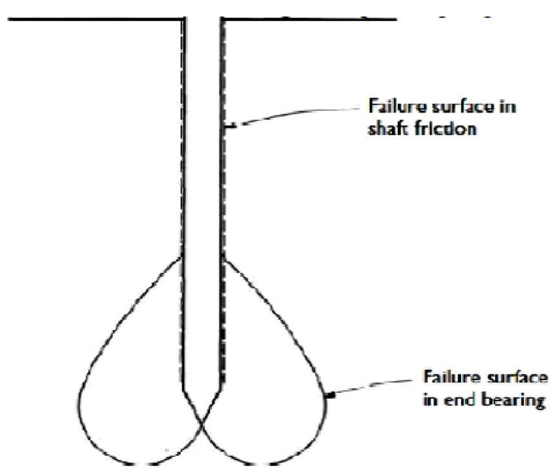

Fig 3.Rupture surface in pile toe and shaft under axial compression loading [19]

Generally two factors including scale effects, mechanism and penetration rate would be effective factors because of different and simple rupture surface in shaft.

Diameter: Diameter factor had not been used in any of methods for evaluating shaft resistance using cone penetration test results. Perhaps this issue would be because researchers didn't believe in significant effect of this factor. In this regard Meyerhof expressed that ultimate shaft resistance of driven and bored piles in rigid fractured clay and sand with specific density are practically independent of pile diameter.

Mechanism and penetration rate: This factor would be effective regarding two matters of strain-stress condition as well as generating pore pressure. Al-Mhaidib [11] put a model of pile in 30 mm diameter under various loading of 0.01, 0.05, 0.1, 0.5 and 1 mm/min and concluded that loading rate had significant effect on compressive and tensile capacity of pile model. Meanwhile increasing loading rate would increase bearing capacity, while loading would not affect displacement which is needed to reach rupture surface. Brown and Hyde [12] investigated penetration rate by performing loading test on concrete pile in place of setting a tool with 600 mm diameter and 12 m length in three modes including Rapid Load Test (RTL), Constant Rate of Penetration (CRP), Maintained Load Test (MLT) and concluded that increasing load rate lead to increase in axial force value and unit shaft resistance in pile length. Therefore it is clear that increasing penetration rate -without considering effect of generated excess pore pressure- would lead to increase in shaft resistance. Furthermore, considering strain values, CPT would generate more strains by full penetrating soil comparing to pile under load test and small displacements. Shaft resistance would be related to horizontal stress applied to pile shaft and friction angle between soil and pile according to equation (6) in order to investigate effect of generated pore pressure. Moreover relation between total horizontal stress and effective horizontal stress would be as equation (7):

$$\tau_s = \sigma'_h \tan\delta \qquad (6)$$

$$\sigma'_h = \sigma_h - u \qquad (7)$$

τ_s is the unit shaft friction in each point of pile length, σ_h is effective horizontal stress and δ is friction angle between soil and pile. Increasing pore pressure would lead to decrease in effective stress. Therefore shaft resistance would decrease. Campanella and Robertson [13] performed an experiment on cone penetration effect on clay with different penetration rate of 0.25, 0.4, 2 and 20 mm/s and studied its effect in drained condition and concluded that cone penetration would be in un drained condition until reaching 1 mm/s rate and in drained condition in lower penetration rated. They also investigated penetration rate effect on values of CPT sleeve friction resistance and understood that penetration rate had significant effect on above values. Takesue et al [1] achieved similar results by performing cone penetration test on various soils such as sand, clay, silty sand, with different rates.

3. Case history records

Database including 42 cases of constructed piles with CPT profile and pile loading test results were collected from 24 various site. Pile materials were in two kinds of concrete and steel material and their cross-section were in square, circle and H forms. Embedment pile depths were between 6 to 79m and its diameter were between 219 and 1500 mm. Shaft resistances values were measured between 135 and 24700 KN. Rate values shown in table 1 are pile penetration rate in rupture under plunging failure load in final load step. Generally case histories are divided into two groups: First group is used for calibration of proposed method and second group is used for verification of proposed method. Case histories data are represented in Table 1.

4. Proposed method

A relation between cone point resistance and pile shaft resistance would be obtained by associating scale effects to shear strain surfaces. What is clear is that generated shear stresses in CPT test are more than generated shear stresses in pile load test. Because there would be no excess pore pressure in sand which would be resulted in smaller rs/rf ratio, so generated shear stresses in CPT test are obviously more than generated shear stresses in pile load test according to basic formula of $\tau = G\gamma$.

Burland [14] expressed foundation static loads smaller in strain ranges and stated generated strains in these problems in range of 0.1 to 0.01 for most soils. Mayne et all expressed generated shear strains in CPT test is nearly 0.3 and strain associated to bearing capacity measures lower than this value.

Moreover, Teh and Houlsby [16] evaluated generated shear strain in a distance equal to cone diameter size in a point near that, between 0.1 and 0.5 by analytical investigation of cone penetration in clay. Therefore, factors that would cause larger strains and subsequent larger strains in CPT test compared to pile load test should be identified. Then a phenomenon called reduced shear modulus with increased shear strain should be evaluated to investigate generated stresses in different strains. Reduced shear modulus with increased shear strain would be often shown in normalized form in corresponding shear modulus divided to G0 or maximum shear modulus Gmax. Relation between G/G0 and shear strain logarithm for cyclic load condition and primary static load are shown in Fig 5 [15].

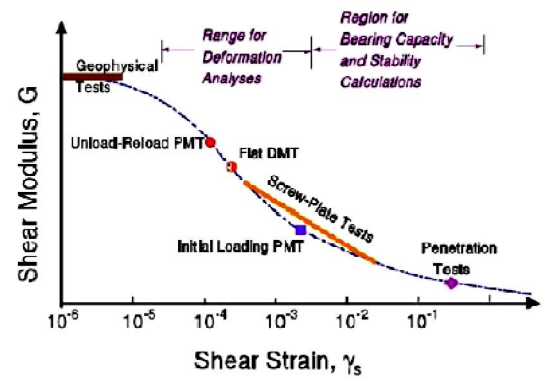

Fig 4. Shear modulus differences with strain values under monotonic load associated to bored tests [14]

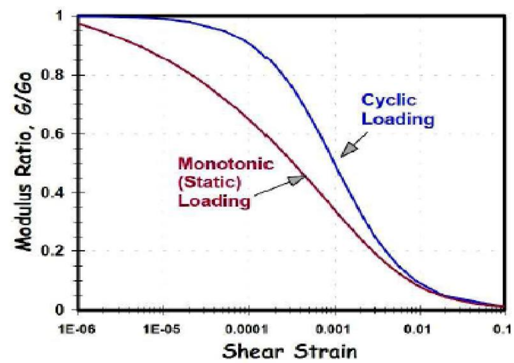

Fig 5. Reduced shear modulus with increased shear strain under static and dynamic load

Cyclic loading curve were the result of resonance column test sample, while static response had been shown in triaxial test by specific local and internal measurements [17]. So many researchers had evaluated how to reduce shear modulus with strain surface. Ishibashi and Zhang provided following empirical relations in this field:

$$\frac{G}{G_{max}} = \alpha(\sigma'_0)^{\beta} \tag{8}$$

$$\alpha = \frac{1}{2}+\frac{1}{2}\tanh\left[\ln\left(\frac{0.000102+n}{\gamma}\right)^{0.492}\right] \tag{9}$$

$$\beta = 0.272\left\{1-\tanh\left[\ln\left(\frac{0.000556}{\gamma}\right)^{0.4}\right]\right\}\exp(-0.0145PI^{1.3}) \tag{10}$$

$$n = \begin{cases} 0 & for \quad PI=0 \\ 3.37\times10^{-6}\,PI^{1.404} & for \quad 0<PI\le15 \\ 7\times10^{-7}\,PI^{1.976} & for \quad 15<PI\le70 \end{cases} \tag{11}$$

In above relation, G is shear modulus in shear strain γ, PI plasticity index and σ'_0 confining stress.

Table 1. Summary of case histories data

No.	Case	Reference	Pile Type	D(mm)	L(m)	V(mm/s)	R_s(kN)	r_s/f_s	Soil Profile
	GROUP 1								
1	BGHD1	Altaee et al.[20]	S_q,C,D	285	11	0.0033	640	0.63	Uniform Sand
2	FHWA	O'Neil [20]	P,S,D	273	9.1	0.0031	135	0.66	Sand
3	JPNOT1	Matsumoto et al.[20]	P,S,D	800	8.2	0.0024	3500	2.05	Stiffclay (softrock)
4	L&D35	Briaud et al. [20]	P,S,D	350	12	0.006	630	0.53	Sand
5	L&D38	Briaud et al. [20]	P,S,D	400	11	0.0064	945	0.77	Sand
6	NWUH	Finno [20]	P,S,D	450	15	0.0042	958	0.55	Sand, clay
7	OPELIKA	Mayne [14]	Rd,C,B	914	11	0.003	2200	0.51	Silt, Silty sand
8	PCCEP	Paik et al. [15]	P,S,D	356	6.9	0.003	425	0.89	Sand
9	TBTP1	Schneider et al. [16]	P,S,D	1500	67	0.042	16300	1.31	Clay, Clayes sand
10	TWNTP6	Yen et al. [20]	P,S,D	609	34	0.007	2810	0.82	Sand, clay , sand
11	UBC3	Campanella et al. [20]	P,S,D	324	17	0.033	315	1.21	Soft clay , sand
12	US95P	Fellenius et al. [17]	P,S,D	406	45	0.005	1685	1.47	Sand , clay
13	USPB1	Albiero et al. [20]	Rd,C,B	350	9.4	0.0049	405	0.61	Clayeysilt,siltysand
14	USPB2	Albiero et al. [20]	Rd,C,B	400	9.4	0.0046	415	0.55	Clayeysilt,siltysand
15	VILANOB	Felleniu & Infante [18]	S_q,C,D	457	12	0.1	1500	1.03	Silty sand , Sand
	GROUP 2								
16	A&N2	Haustorfer et al. [20]	S_q,C,D	450	14	0.004	2350	0.95	Sand
17	BGHD2	Altaee et al.[20]	S_q,C,D	285	15	0.0026	1120	0.68	Uniform sand
18	DUNKIRK	Chow et al.[19]	P,S,D	324	11	0.0003	535	0.4	Dense sand
19	FITTJA	Axelsson[9]	S_q,C,D	300	13	0.028	190	1.53	Clay,Siltysand,Sand
20	GIT1	Mayne[20]	Rd,C,B	760	17	0.1	3100	0.67	Silty sand
21	JPNOT2	Matsumoto et al.[20]	P,S,D	800	8.2	0.001	3190	1.87	Stiff clay(soft rock)
22	JPNOT3	Matsumoto et al.[20]	P,S,D	800	8.2	0.0014	3700	2.17	Stiff clay(soft rock)
23	L&D12	Briaud et al.[20]	HP,S,D	360	17	0.0018	1170	0.58	Sand
24	L&D21	Briaud et al.[20]	HP,S,D	360	17	0.12	2160	0.95	Sand
25	L&D32	Briaud et al.[20]	P,S,D	300	11	0.0063	560	0.61	Sand
26	L&D314	Briaud et al.[20]	HP,S,D	360	12	0.006	1170	0.98	Sand
27	L&D315	Briaud et al.[20]	HP,S,D	360	11	0.0025	817	0.73	Sand
28	L&D316	Briaud et al.[20]	HP,S,D	360	11	0.0022	870	0.77	Sand
29	N&SB144	Nottingham[20]	P,S,D	270	23	0.002	765	0.4	Sand
30	NWUP	Finno[20]	HP,S,D	450	15	0.0043	960	0.55	Sand, Clay
31	TBTP2	Schneider et al.[16]	P,S,D	1500	79	0.039	24700	1.02	Clay, Clayes sand
32	TWNTP4	Yen et al.[20]	P,S,D	609	34	0.007	2730	0.85	Sand, clay, sand
33	TWNTP5	Yen et al.[20]	P,S,D	609	34	0.007	2500	0.78	Sand, clay, sand
34	UBC5	Campanella et al.[20]	P,S,D	324	31	0.15	920	1.06	Soft clay, Sand
35	ISCC1	Fellenius et al.[21]	S_q,C,D	350	6	0.001	500	0.43	Siltysand,Clayes sand
36	ISCE9	Fellenius et al.[21]	Rd,C,B	600	6	0.001	700	0.35	Siltysand,Clayes sand
37	ISCT1	Fellenius et al.[21]	Rd,C,B	600	6	0.001	680	0.34	Siltysand,Clayes sand
38	MISAV	Olson & Shantz[22]	S_q,C,D	355	10	0.001	614	0.29	Sand
39	NEAST	Olson & Shantz[22]	P,S,D	410	13	0.001	1068	0.52	Sand, Clay
40	ORLANDO	Fellenius&Infante[18]	P,S,D	324	14	0.01	900	0.57	Sand
41	R351B1	Pando et al.[23]	Rd,C,D	592	18	0.01	1487	0.61	Sand, Silty sand
42	S&A2	Fellenius&Infante[18]	P,S,D	219	20.5	0.1	530	1.05	Siltyclay,Silty sand

There would be an estimation of how reducing shear modulus with increased shear strain for various soils according to above relation. Corresponding normalized shear stress ratio by maximum shear stress to shear strain would be plotted using Fig 6 [18].

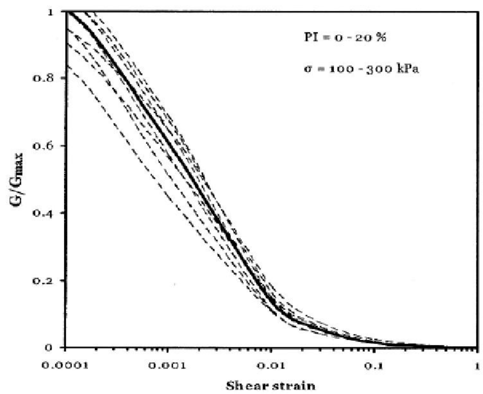

Fig 6.Reduced shear modulus with increased shear strain based on Ishibashi and Zhang relations [17]

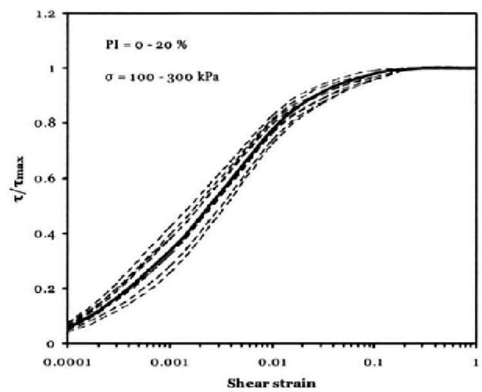

Fig 7. Increased shear stress with increased shear strain [17]

As it could be seen τ/τ_{max} value is related to shear strain differences. Therefore, identification of shear strain values obtained of CPT test and pile load test are imperative. According to fixing τ/τ_{max} ratio in strain value of 0.3 based on Fig 7, this strain would be considered as a basis for generated shear strain in CPT test [18].

As penetration rate increase, generated shear strain values and as a result, shaft resistance would increase. Furthermore, effect of pore pressure should be studied in sensitive soils to excess pore pressure. Positive excess pore pressure in CPT test would lead to reduced effective stresses due to high penetration rate and generated un drained condition therefore caused reduced effective stresses and as a result actual shaft resistance would be lower. While there would be adequate period for elimination of pore pressure in pile load test that would lead to pile shaft resistance ratio to CPT sleeve friction strength to be more than 1. in negative excess generated pore pressure in CPT test, increased effective stresses and more values for shaft resistance would be obtained. Due to this reason for considering differences in penetration rate, correction should be formed concerning two matters of differences in generated shear strains and differences in generated excess pore pressure in CPT test and eliminating it in pile load test in a long period.

Increasing diameter lead to increased confining pressure by surrounded soil around pile and as a result it caused increase in shaft resistance. Moreover, increased diameter would lead to increase in shear strain values and as a subsequent increase in shear stresses. To prove this matter it is just adequate to consider two piles with different diameters, one in cone size and the other one a standard pile with the same rates equal to common rate of pile penetration. Pile with a diameter in size of CPT cone had less potential for surrounded soil displacement around it compared to other pile and as a result there would be less shear strain and subsequently less shear stress.

Moreover, soil type had significant impact on generating shear strain. Thus, sensitive soils achieved more shear strain. Sensitivity in clays would be shown by St factor in form of Ns/Rf ratio in which Rf is friction ratio and Ns is a constant value. Although this ratio is defined just for clays and sands would be in friction ratio between 0.5 and 1 usually, therefore not much difference in this ratio lead to the fact that this factor would not be determinant in sands by itself. In fact sandy soils are low sensitive soils and sensitivity effect in shear strain values and generated shear stresses are negligible in them. In contrast, in clays friction ratio values would be in wide range near zero to more than 10. In fact, when clay friction ratio would be near to zero, it would be considered most sensitive soil and St values would be effective in computation.

Major factors in causing differences between shaft resistance results obtained of pile load test and CPT friction strength results would be expressed as follows:

- Mechanism and various penetration rate
- Differences in CPT and pile scales
- Soil type effect

Thus, for generating relation between generated shear strain in cone and pile penetration test under load test, following primary equation would be proposed:

$$\frac{\gamma_{pile}}{\gamma_{CPT}} = \left(\frac{V_{pile}}{V_{CPT}}\right)^{a} \left(\frac{D_{pile}}{D_{CPT}}\right)^{b} (S_t)^{c} \qquad (12)$$

Since Ns is a constant in provided relation for soil sensitivity, one value would be used in numerator instead of a specific number. Because first, its effect is hidden in amount of power and second, the aim here is providing an index for considering soil sensitivity. As a result pile shear strain ratio to penetrometer would be as follows:

$$\frac{\gamma_{pile}}{\gamma_{CPT}} = \left(\frac{V_{pile}}{V_{CPT}}\right)^{a} \left(\frac{D_{pile}}{D_{CPT}}\right)^{b} \left(\frac{1}{R_f}\right)^{c} \qquad (13)$$

Moreover, for considering generated excess pore pressure in cone penetration test, following relation would be proposed:

$$r_s = f_s(1 + \alpha\Delta u) \qquad (14)$$

Above relation would be similar to Takesue et al method. This method is shown in Fig 1 which is divided to two parts, in group one α value would be considered equal to 0.0008 and in group two would be considered equal to 0.005. Therefore α value would be in this range.

equation (13) had been calibrated by iteration method using case histories of group one including 15 cases for establishing relation between shear strains in pile shaft compared to penetrometer and a, b and c parameters had been proposed. Therefore calibration would be performed and the best fit compared to diagram $\tau/\tau_{max} - \gamma$ would be obtained according to rs/fs ratio for each case according to Table 1 using above relation. First cases associated with sand soils would be evaluated to pore pressure effect which would not deviate results. Penetration rate would be shown in $\tau/\tau_{max} - \gamma$ diagram to the best fit and would be considered for reduced shear stress with shear strain, then above pile shear strain values with specific ratio rs/rf and various diameters should be in a way to achieve best results. So a, b and c values should be selected 0.6, 0.45 and 0.5 respectively. As a result generated shear strain ratio in surrounded soil around pile to soil surrounding penetrometer would be obtained as follows:

$$\frac{\gamma_{pile}}{\gamma_{CPT}} = \left(\frac{V_{pile}}{V_{CPT}}\right)^{0.6} \left(\frac{D_{pile}}{D_{CPT}}\right)^{0.45} \left(\frac{1}{R_f}\right)^{0.5} \qquad (15)$$

Moreover, for method calibration in cases that effect of pore pressure would be considered, after investigating case histories of first group and applying combined effect of strain surface and pore pressure, appropriate value of α in equation (14) equal to 0.002 would have the best result. Therefore for considering excess pore pressure, following equation would be proposed:

$$r_s = f_s(1 + 0.002\Delta u) \qquad (16)$$

In summary, the step by step method for computing shaft resistance of a pile using friction strength values obtained of CPTu test and considering scale effects factors would be provided as follows:

- Applying excess pore pressure as:

$$f_{s-Modified} = f_s(1 + 0.002\Delta u) \qquad (17)$$

- Identifying generated shear strain ratio in pile compared to CPT diameter, pile penetration rate under plunging load and average friction ratio (Rf) and using equation (15)

- Identifying pile shear strain assuming generated shear strain in CPT test to 0.3.

- Identifying reduced ration caused by lower strain surface in pile penetration compared to penetrometer using $\tau/\tau_{max} - \gamma$ diagram in Fig 8.

- Applying k ratio on modified friction strenght for identifyng pile shaft resistance ($r_s = k.f_{s-Modified}$)

5. Evaluation and Discussion on results

Result of applying equations (15) and (16) on case histories of first group are shown in Fig 8. As shown here, above cases with approximation and appropriate form had been placed around reduced shear stress diagram with shear strain. Equation (16) is approximately similar to provided relation in Takesue et al method [1]. Fig 9 shows a comparison between how considering generated pore pressure effect in penetrometer penetration and proposed method by Takesue et al.

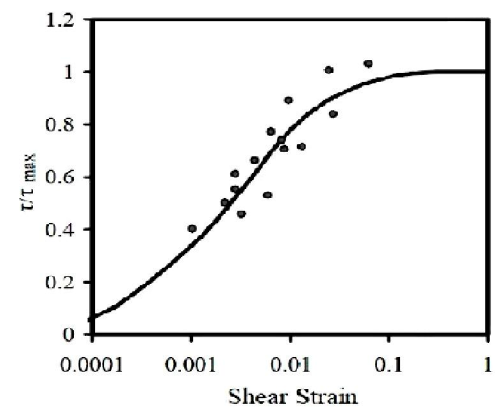

Fig 8. Reduced shear stress with shear strain after calibration for case histories

In Takesue et al method [1], if generated pore pressure in cone penetration test would be equal to hydrostatic condition (in sands), rs/rf ratio would be obtained equal to 0.76. But this ratio is equal to 1 in proposed method that would be more accurately justified. When excess pore pressure is zero, it has no effect on rs/rf ratio and it should be obtained to 1. Main reason for this difference between these two methods is that in Takesue et al method [1] only excess pore pressure effect had been considered, but it should be noted that scale effects had been considered in proposed method.

The ratio of estimated shaft resistance to measured shaft resistance, average value and standard deviation of this ratio for 15 cases of first group of case histories had been calculated in order to identify calibration accuracy which is provided in Table 2 and Fig 10. Average and standard deviation value of proposed method had been 0.97 and 0.13 respectively which would be representative of appropriate accuracy in calibration.

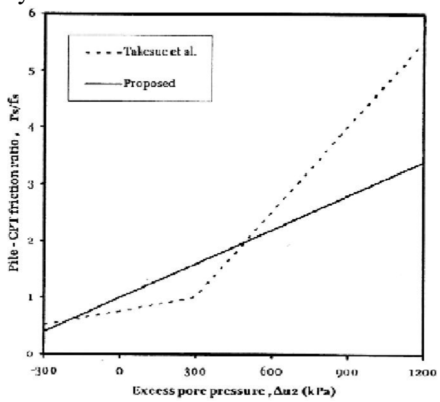

Fig 9. Pore water pressure effect modification

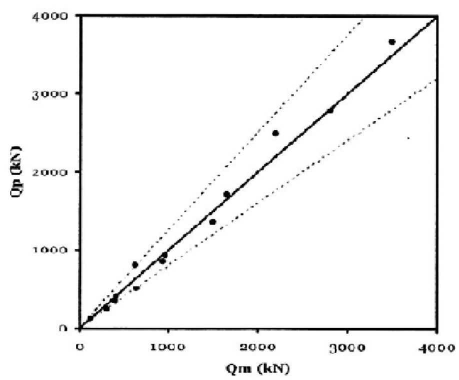

Fig 10. Estimated bearing capacity to measured one for case histories of first group

For analysis of methods to identify piles shaft bearing capacity using CPT friction data, shaft bearing capacity values had been estimated using four methods included Tummay and Fakhroo [3], Price and Wardle [4], Penpile by Clisby et al, Takesue et al as well as proposed method had been evaluated in accuracy and correction matters. For better comparison, anticipated shaft resistance to measured one for two methods had been shown in Fig 11. Diagonal line represent best estimation and dash lines represent error ranges to +-20%. Moreover, average and standard deviation values had been shown in each figure.

Price and Wardle method with a 0.76 average, especially Penpile method with average equel to 0.53, underestimated shaft bearing capacity and Takesue et al method by 1.37 average overestimated shaft bearing capacity. At first it seems that Tummay and Fakhro method were equal to 0.92 and proposed method by 0.98 averages had high evaluation of bearing capacity.

Table 2. Estimated shaft bearing capacity and measured one for case histories of group one after calibration of proposed equation

No	Case	Q_p	Q_m	Q_p/Q_m
1	BGHD1	540	640	0.79
2	FHWA	126	135	0.93
3	JPNOT1	3666	3500	1.05
4	L&D35	809	630	1.28
5	L&D38	854	945	0.9
6	NWUH	930	960	0.97
7	OPELIKA	2492	2200	1.13
8	PCCEP	362	408	0.89
9	TBTP1	16980	16300	1.04
10	TWNTP6	2789	2810	0.99
11	UBC3	246	315	0.78
12	US95P	1710	1650	1.04
13	USPB1	351	405	0.87
14	USPB2	409	415	0.99
15	VILANOB	1357	1500	0.9
				Mean = 0.97
				S.D. = 0.13

But average value was not appropriate index for methods evaluation. Although in Tummay and Fakhro method average estimated shaft resistance ratio to measured one is equal to 0.92, due to standard deviation equal to 0.52, dispersion is large ,and moreover, in spite of relative error to be equal to 8%, absolute error would be 39% and as a result there would not be high reliability. Standard deviation of Takesue et al method as well as Price and Wardle method would be 0.56 and 0.34 respectively that represent high dispersion in results. Relative and absolute error and standard deviation values had been provided in investigated methods in Table 3. In summery proposed method with relative error of -0.02 and absolute error of 0.17 and corresponding standard deviation of 0.22 and 0.13 is acceptable compared to other methods.

6. Conclusion

In recent decades so many methods had been provided for deep foundation bearing capacity using CPTu data by researchers. In these methods differences in scales, mechanisms and pile and CPT cone penetration rate as well as generated pore pressure effect had not been considered in identification of piles shaft resistance. In this research scale effects between CPT cone and pile with relation to these factors to strain surfaces had been investigated and a relationship had been proposed databank including 42 pile load test case histories and cone penetration test and had been compared and evaluated by methods of other researchers. The results are briefly as follows:

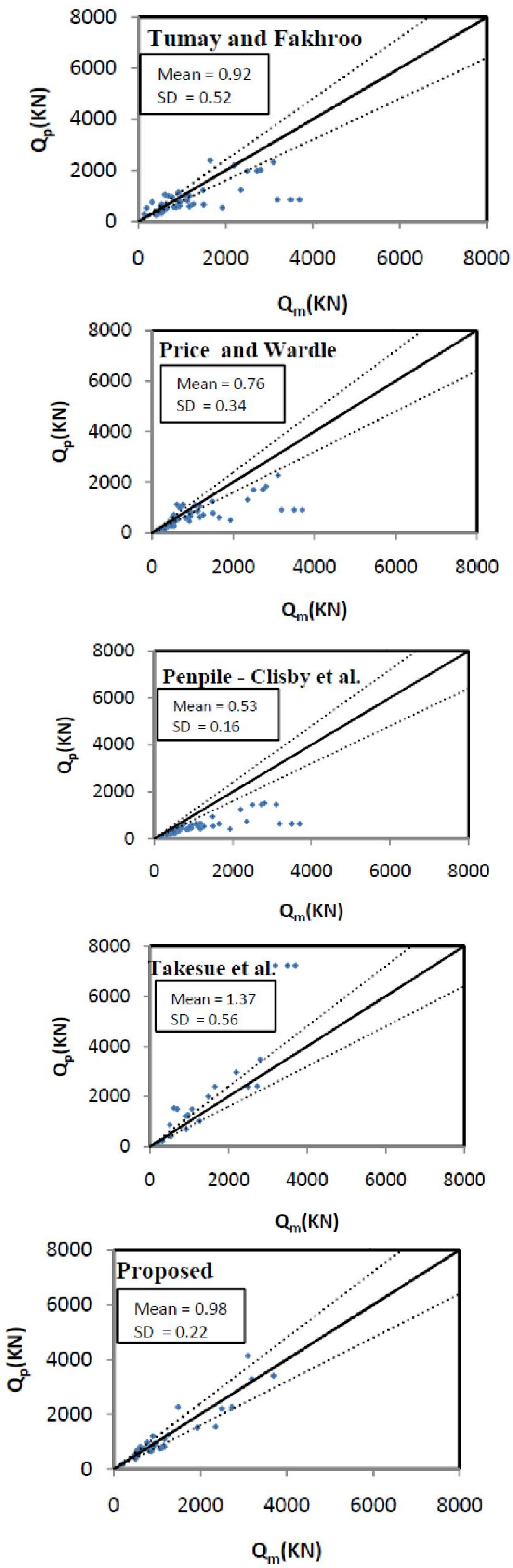

Fig 11. Anticipated shaft bearing capacity to measured one by various methods

Table 3. Absolute and relative errors and standard deviation values for shaft resistance identification methods

Method	Error Value		
Tumay and Fakhroo	Relative	Mean	0.08
		S.D.	0.52
	Absolute	Mean	0.39
		S.D.	0.35
Price and Wardle	Relative	Mean	0.24
		S.D.	0.34
	Absolute	Mean	0.36
		S.D.	0.21
Penpile	Relative	Mean	0.47
		S.D.	0.16
	Absolute	Mean	0.47
		S.D.	0.16
Takesue et al.	Relative	Mean	0.37
		S.D.	0.56
	Absolute	Mean	0.51
		S.D.	0.43
Proposed	Relative	Mean	0.02
		S.D.	0.22
	Absolute	Mean	0.17
		S.D.	0.13

1. In investigating penetrometer friction strength and pile shaft resistance, differences between them had been associated to generated shear strain values in penetrometer and pile penetration as well as measured pore pressure in cone penetration test. Three factors consist of diameter and penetration rate of pile had been considered in matters of scale effects as well as1/R_f factor as soil effect type in shear strain values. In a way that by increasing diameter and pile penetration rate in soil as well as increasing soil sensitivity , more generated shear strain values and as a result larger shear stresses would be obtained.

2. Generated pore pressure in cone penetration test due to high penetration rate and generating un drained condition especially in fine grained soils, because of reducing effective stresses, had direct impact in shaft resistance evaluation using CPT friction data. This impact had been considered by establishing a direct relation.

3. According to represented factors, a method for evaluation of shaft bearing capacity using cone penetration test data as well as considering scale effect would be proposed and calibrated by 15 case histories and would be provided in step by step method.

4. Proposed method with Penpile, Tummay and Fakhroo, Price and Wardle, Takesue methods using 42 case histories had been analyzed. Performed investigations by various statistical and probability methods by computation of average compared to

anticipated shaft resistance and absolute and relative errors and standard deviation for different methods represent more accuracy, less dispersion compared to other four methods.

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