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Evaluating CPT and CPTu based pile bearing capacity estimation methods using Urmiyeh Lake Causeway piling records

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Abstract Urmiyeh Lake is the largest super salt water situated in the north-west of Iran. A causeway embankment has been constructed in the narrowest part of the lake from both sides about 13.5 km, in order to connect two provincial capital cities of Tabriz and Urmiyeh of eastern and western Azerbaijan provinces to Europe through Turkey, while a 1280 m opening in between linked up by a bridge. Based on soil classification methods, utilizing CPTu data and soil sampling, the lake sediments consist of 150 m of soft and very sensitive clay. In order to evaluate the bearing capacity of driven piles of the bridge, eight long steel piles with diameters of 813 and 66 m and lengths of 75 m have been instrumented and monitored based on static and dynamic load testing program. Piezocone (CPTu) results are also available from adjacent pile locations. Results of pile capacity calculation based on direct CPT and CPTu methods demonstrate that reasonable accuracy can be achieved in reference to dynamic testing. Therefore, combination of CPTu data with dynamic testing results can be considered by engineers for predicting bearing capacity of piles in offshore and bridge structures, where the static pile load testing is difficult, time consuming and expensive in marine environment.

1. Introduction

In early 1980, construction of a highway, the Tabriz–Urmiyeh, began for the purpose of connecting west part of Iran to Turkey. The new road would shorten the distance between Tabriz and Urmiyeh cities in north-eastern of Iran by about 130 km, resulting in improved access and efficient transit between Iran and Europe through Turkey. The highway would cross the Urmiyeh Lake, an inland lake surrounded by mountains. The lake area is approximately 5500 km^2, the length is about 140 km and the width ranges from 15 through 50 km. The average depth of the lake is about 7 m and the maximum depth is about 12 m.

Due to the circulation of salt-water in the lake and the necessity to continue shipping between Sharafkhaneh and Rahmanlu Ports, a waterway should be kept open. Consequently, a bridge must be built to provide access for marine traffic. A bridge with the length of 1260 m, comprising of 19 spans is considered. The main span is in the form of an overhead tied arch structure and the side spans are in form of flat deck system. The bridge abutments land on the adjacent embankment in a manner ensuring adequate continuity for road and railway traffic in the abutment areas. In construction of the bridge, more than 400 piles having a total length of 32 km have been driven. Figure 1 shows the location of the lake and the causeway route, as well as the bridge longitudinal view inserted on the map of Iran.

The highway embankment was constructed in the narrowest part of the lake by rock fill quarried from the nearby mountains. When the rock fill placed on the lake bed, it started to subside. The rate of subsidence was monitored closely during the filling process, and the filling was carried out until the subsidence of the lake bed halted. The width of the embankment crest is about 30 m and the average height is about 20 m [1].

Geological and geotechnical investigations were performed to determine the thickness and the physical and mechanical properties of the soil layers. Investigations were carried out by

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Nomenclature

a	Net area ratio
A_{si}	Pile shaft area interfacing with layer i
A_t	Cross-section area of the pile toe
f_s	Cone sleeve friction
FS	Factor of safety
F_t	Normalized friction ratio
i	Number of studied pile
K_s	Earth pressure coefficient
n	Number of soil layers along the pile shaft
N_q	Bearing capacity coefficient
N_s	Empirical coefficient of sensitivity
PDA	Pile Driving Analyzer
P_{50}	Cumulative probability of 50%
P_{90}	Cumulative probability of 90%
q_c	Cone point resistance
q_E	Effective cone resistance
q_t	Corrected cone resistance
Q_{all}	Allowable load capacity of the pile
Q_m	Measured pile capacity
Q_p	Predicted pile capacity
Q_t	Normalized cone resistance
Q_{ult}	Bearing capacity or ultimate geotechnical capacity
r_s	Average unit shaft friction of the soil layer i
r_t	Unit end bearing capacity of pile
R_f	Friction ratio
R^2	Coefficient of determination
SRSS	Square Root of Sum of Squares
S_t	Sensitivity
S_u	Undrained shear strength
u_0	Hydrostatic pore pressure
u_2	Pore pressure measured behind cone point
μ'	Geometric mean
σ	Standard deviation
σ_v	Total overburden stress
σ'_v	Effective overburden stress.

drilling exploratory boreholes, including CPTu soundings and laboratory tests.

Due to the thick layers of sensitive and super soft clay sediments, the bridge foundations had to be placed on deep foundations, and steel pipe piles were selected. Design of the piles compiled different methods such as static analysis, pile loading test, dynamic testing and correlations to in situ tests like cone penetration test (CPTu). The CPTu was the major geotechnical tool and source of useful subsoil data in this project. This paper presents the results of the soil study, full-scale pile tests, and methods of analysis used in the evaluation of the test data [2]. The presentation includes an assessment of four current direct CPT methods and one CPTu method to calculate the bearing capacity of the piles.

2. Geological and geotechnical aspects of the site

The sedimentary rocks underlying the recent lake deposits are limestone and shale of the Permian and Cretaceous periods, respectively. The igneous rocks are typically granite, volcanic breccias and trachyandesite-dacit. The region has been subjected to complex faulting, folding and fracturing with numerous rock outcrops along the shoreline and lake islands.

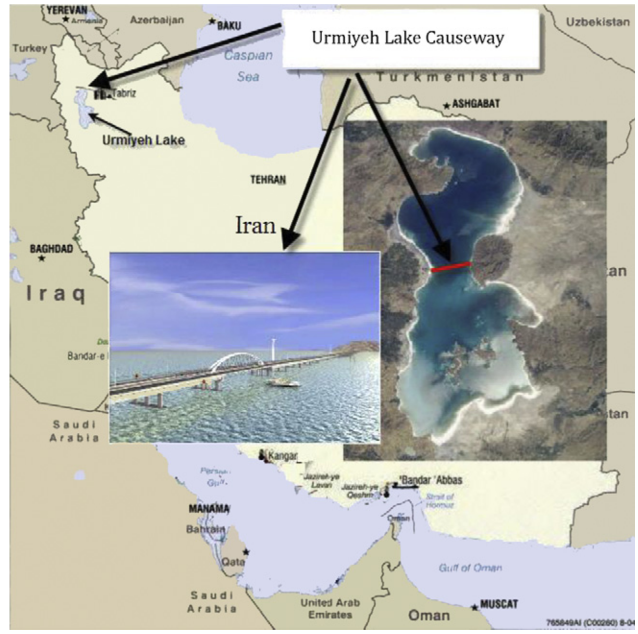

Figure 1: Iran's map with location of studied site and longitudinal view of the bridge.

Particular attention was given to the study of more recent geological sediments of the Pliocene–Pleistocene periods. In order to determine the geological properties of soil, three boreholes in the lake bed sediments were advanced to a depth of 150 m. Samples were collected, using thin-walled tube sampler from various depths. The soft clay in the upper 10 m was too soft to be sampled. The sample quality in the lower beds was generally good, although some disturbance was noted [2].

The variety and type of primary sedimentary structures found in the laminated beds during this investigation indicate that they have resulted from a traction and fall-out process from turbulent suspension. The complexity of the sediments can be rationalized by the use of the following depositional mode: This model which is very common in the lake describes the effect of Brownian forces between different particles. In this process, flocculated clay particles stick on silt and fine sand grain particles, which cause them to settle rapidly. It should be mentioned that concentration of ions and cations in super salt water accelerates the process. Under this condition, the top layers are largely underconsolidated. The upper soft clay layers are highly flocculated, compressible and under ongoing consolidation. They have collapsible structure under dynamic loading condition. Figure 2 is a photo of thin-walled tube samples from the site, showing fissures (stained) caused by seismic action and subsequently filled by fine sand. The lower part of sediments contains organic matter, i.e. the remnant of species such as Algae and Artemia salina of brine crustacean that can tolerate fluctuation in the salinity of water. Oxidation produced dark color sludge.

The range and sequence of structures can be summarized as five basic units, believed to have local stratigraphic significance [4]. A typical log of the boreholes is shown in Figure 3.

CPTu soundings were performed in 8 locations, along the bridge route, down to 100 m below the lake-bed. Two CPTu data profiles (Boreholes Nos. 5 and 7) are shown in Figures 4 and 5, respectively, and include q_c, f_s, u_2 and R_f.

Figure 2: Stained fissures filled by fine sand, by Jakobsen Co [3].

Depth (m) from-to	Description of Strata	Log symbol	Moisture content (%)	Dry density (gr/cm^3)	Peak shearing strength (kg/cm^3)
0-22	CL-ML		43	1.26	0.25
22-24	ML		44	1.24	0.24
24-33.5	CL-ML		37-41	1.27-1.35	0.25-0.38
33.5-35	ML		37	1.36	0.40
35-38	CL-ML		36	1.37	0.62
38-45	MH		25-37	1.35-1.5	0.39-0.85
45-75	CL-ML		27-42	1.26-1.50	0.17-1.03
75-82	CL		26-36	1.29-1.59	0.64-1.61
82-92	ML		27-32	14.42-1.56	0.5-0.54
92-101	CL-ML		32	1.39	***
101-105	CL		39	1.33	1.53
105-107.5	CH		29	1.36	***
107.5-127	CL-ML		22-32	1.22-1.7	0.9-1.11
127-128.5	SP-SM		***	***	****
128.5-133	CL-ML		***	***	****

Figure 3: Typical soil profile from boring exploration by Mandro Co [5].

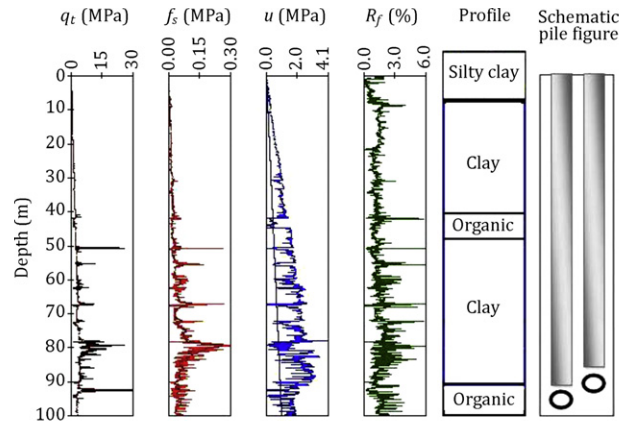

Figure 4: CPTu logs and soil profile for borehole No. 5 by Mandro Co [5].

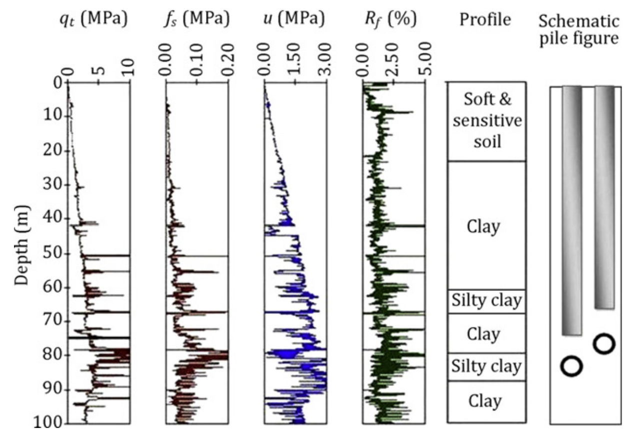

Figure 5: CPTu logs and soil profile for borehole No. 7 by Mandro Co [5].

A primary application of CPTu is stratigraphic profiling, which can be done with greater accuracy than that achieved from conventional boring and sampling. Robertson et al. [6] presented a chart, based on the piezocone, where the pore-pressure corrected cone resistance, q_t, is plotted versus friction ratio, R_f. Later, Robertson [7] proposed a refinement of the profiling chart. The normalization was proposed to compensate for the cone resistance dependency on the overburden stress, and therefore when analyzing deep CPTu soundings (i.e. deeper than about 30 m), a profiling chart developed for more shallow soundings does not apply well to the deeper sites. Robertson [7] proposed charts plot a normalized cone resistance, Q_t, against a normalized friction ratio, F_t, which are defined by Eqs. (1)–(3) as follows:

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

$$Q_t = (q_t - \sigma_v)/(\sigma_v'), \tag{2}$$

$$F_t = (f_s)/(q_t - \sigma_v), \tag{3}$$

where:

q_t	Cone resistance corrected for pore water pressure on shoulder;
u_2	Pore pressure measured at cone shoulder;
a	Net area ratio;
Q_t	Normalized cone resistance;
σ_v	Total overburden stress;
σ_v'	Effective overburden stress;
$(q_t - \sigma_v)$	Net cone resistance;
F_t	Normalized friction ratio;
f_s	Sleeve friction;
u_0	Hydrostatic pore pressure

Eslami and Fellenius [8,9] investigated several CPT and CPTu approaches for soil behavior classification. They proposed a new approach to classify the soil, based on CPTu data, plotting values of 'effective' cone resistance, q_E, defined by Eq. (4), versus sleeve friction, f_s.

$$q_E = (q_t - u_2). \tag{4}$$

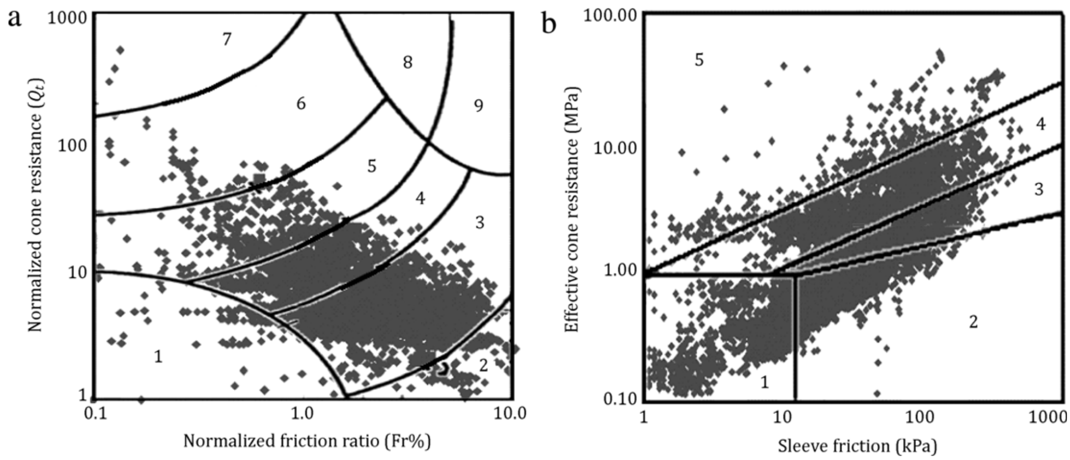

Figure 6: Soil Profiling by (a) Robertson [7] and (b) Eslami–Fellenius [9].

The soil profiling charts are shown in Figure 6(a) and (b) [7,9]. In order to classify the soil, two discussed methods were applied. According to Robertson's chart [7], normalized cone resistance, Q_t, versus normalized friction ratio, F_t, and according to Eslami and Fellenius's chart [8], the values of effective cone resistance, q_E, versus sleeve friction, f_s, are plotted, respectively. The obtained CPTu data are plotted, on mentioned charts of classification, as shown in Figure 6(a) and (b).

As illustrated in Figure 6(a), orientation of data shows that the layers consist of clays (clay to silty clay), silt mixtures (silty clay to clayey silt), sand mixtures (sandy silt to silty sand), sand (silty sand to clean sand) and sensitive clay. Considering Eslami and Fellenius's chart, Figure 6(b), orientation of data shows that most of the layers consist of sensitive and collapsible clay, silt and some other type of soil like silty clay and clayey silt, sandy silt and silty sand [10]. For these unusual sediments, special design aspect and considerations are required for CPTu data processing used in soil classification and geotechnical design.

3. Axial bearing capacity of piles for offshore structures

Several methods have been developed to calculate the axial capacity of piles. All include simplifying assumptions and/or empirical approaches regarding soil stratigraphy, soil–pile structure interaction, and distribution of soil resistance along a pile [8]. The axial capacity of a pile, Q_u, is comprised of toe-bearing resistance, Q_t, and shaft resistance, Q_s. The general rule is given in Eq. (5).

$$Q_{\mathrm{ult}} = Q_t + Q_s = r_t A_t + \sum_{i=1}^{n} r_s A_{si}, \tag{5}$$

where

r_t	Unit toe resistance;
A_t	Cross-section area of the pile toe;
r_s	Average unit shaft resistance of soil layer i;
A_{si}	Pile shaft circumferential area interfacing with layer i;
n	Number of soil layers along the pile shaft

In global factor of safety approach, the allowable load (Q_{all}) of the pile is usually calculated by dividing the pile capacity (Q_{ult}) by an appropriate Factor of Safety (FS).

Static analysis methods recognize that shaft and toe respond differently to an applied load. For shaft resistance, considerable uncertainty and debate exist over the appropriate choice of the horizontal stress coefficient, K_s. For estimating toe resistance in non-cohesive soils, bearing capacity theory is usually applied. However, the theory involves a rather approximate $\varphi - N_q$ relationship coupled with in-situ value of soil friction angle, φ. In cohesive soils, pile capacity is often estimated by direct correlation with undrained shear strength, s_u. However, s_u is not a unique parameter and depends significantly on the type of the test, the strain rate, and the orientation of failure plane.

In recent years, the application of in-situ testing techniques has increased in geotechnical designs. This is due to rapid development of instruments, improved understanding of soil behavior and subsequent realization of the limitations of conventional laboratory testing [8].

In contrast to the SPT, the cone penetration test, CPTu, is simple, fast and economical, and supplies continuous records with depth. A variety of sensors is incorporated with the penetrometer. Because of similarities between the cone penetrometer and a pile, the penetrometer can be considered as a model pile. Several methods have been proposed to predict the pile capacity from CPT and CPTU data. These methods can be classified into two approaches, as follows:

1. Direct approach: The unit toe resistance, r_t, is evaluated from the cone tip resistance, q_c, and the shaft resistance, r_s, is evaluated from either the sleeve friction, f_s, or q_c profiles.
2. Indirect approach: The CPT data, q_c and f_s, are first used to evaluate the soil strength parameters, such as the undrained shear strength, s_u, and the angle of internal friction, φ. These parameters are then used to evaluate the values of r_t and r_s, using formulas derived from semi-empirical or theoretical relations [4]. A completion of five current CPT and CPTu direct methods is shown in Table 1.

To obtain reliable capacity estimation, it is necessary to calibrate the results of CPT methods with full-scale tests, such as static pile loading tests and analysis of pile driving records. A number of investigators and researchers, e.g. [15–20] have compared the bearing capacity calculations using different CPT and CPTu methods with measured pile capacity in loading tests. Test on 30 piles in China [21] showed that the bearing capacity calculation, using Eslami and Fellenius [8] and Takesue et al. [22] methods, best fit the measured values among twelve current methods. Full-scale pile loading tests can reduce the uncertainty involved in pile analysis and design. However, such tests are expensive and time-consuming, and the costs are often not justifiable.

Table 1: Summary of current CPT and CPTu methods.

Current CPT and CPTu methods	Unit toe resistance r_t	Unit shaft resistance r_s	Note
Schmertmann [11]	$r_t = C_t.q_{ca}$	$r_s = C_s.q_c$ $r_s = K.f_s$	$C_s = 0.8$–1.8% clay and sand $K = 0.8$–2 sand, $K = 0.2$–1.25 clay
Beringen and De Ruiter [12]	$r_t = N_c.S_u$	$r_s = C_s.q_c$, $r_s = K.f_s$ $r_s = \alpha.S_u$	$K = 1$, $C = 0.3\%$ sand $\alpha = 1$ for NC, $\alpha = 0.5$ for OC clay
Bustamante and Gianeselli [13]	$r_t = C_t.q_{ca}$	$r_s = C_s.q_c$	$C_t = 0.4$–0.55 $C_s = 0.3\%$
Tumay and Fakhroo [14]	$r_t = C_t.q_{ca}$	$r_s = k.f_c$	$C_s = 0.8$–1.8% clay and sand $k = 0.5 + 9.5e^{(-0.009f_s)}$
Eslami and Fellenius [8]	$r_t = C_t.q_{Eg}$	$r_s = C_s.q_{Eg}$	$C_t = 1$ $C_s = (0.3$–$8)\%$

Table 2: Pile case records summary.

Case no	Pile name	Pile shape	Pile size			Measured	Type of test
			Length h (m)	Diameter r (mm)	Thickness s (mm)	R_u (kN)	
1	UCA4	Circular	66	813	38.1	5400	Dynamic test pile driving analyzer
2	UCA5	Circular	66	813	38.1	4700	
3	UCA7	Circular	66	813	38.1	5500	
4	UCB3	Circular	75	813	38.1	7300	
5	UCB4	Circular	75	813	38.1	5500	
6	UCB5	Circular	75	813	38.1	7000	
7	UCB7	Circular	75	813	38.1	6130	
8	UCB8	Circular	75	813	38.1	8000	
9	UCA4-C	Circular	30	356	12	1100	Pile load test
10	UCA5-T	Circular	70	305	16	3100	

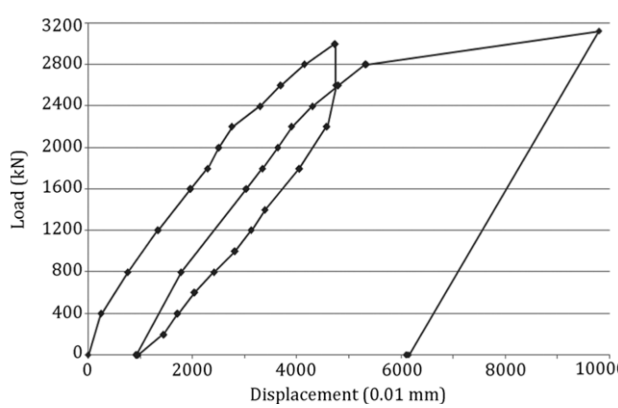

Figure 7: Load–displacement diagram for tension test by Sadra Co [27].

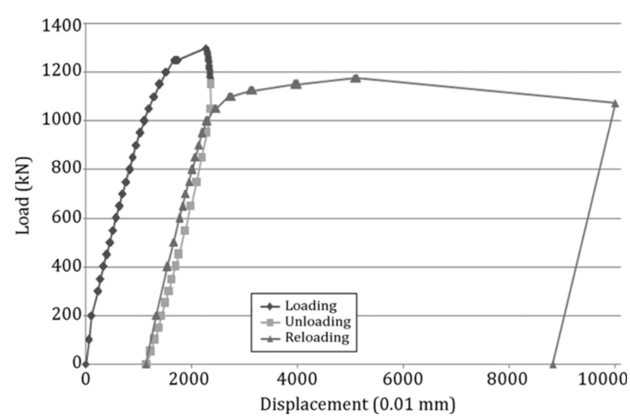

Figure 8: Load–displacement diagram for static compressive test by Sadra Co [27].

With the developments of modern device made during the last 30 years, it is now practical to measure the pile capacity by dynamic methods. These measurements have made possible the improved pile analysis and design. One of these methods is called Pile Driving Analyzer, PDA, which has become common for verification of capacity of both driven and bored piles [23]. Comparison of bearing capacity results for three piles determined by dynamic tests and CPT soundings indicated that cone method agreed closely with dynamic test results [24,11,25,26].

4. Pile testing records and bearing capacity estimation

The diameter of piles for dynamic tests is 813 m and embedment lengths are 66 and 75 m, with wall thickness of 38.1 mm. The measured pile capacity ranges from 3200 to 8000 kN. Table 2 summarizes the main case records including pile embedment length and measured bearing capacity, either by dynamic or static tests. During the preliminary design stage of the Urmiyeh Causeway project, in order to measure the pile capacity, two static pile load tests were performed in site, one was compressive test and the other was tension (pullout) test. The compressive 35 m test pile, with diameter of 365 m and wall thickness of 12 mm, is adjacent to Case No. 4 so called UCA4-C. The tension 70 m test pile, with diameter of 305 m and wall thickness 16 mm, is adjacent to Case No. 5, called UCA5-T. The 70 m tension test was then preformed for design stage. Load–displacement diagrams for compressive and tension tests are shown in Figures 7 and 8 respectively. As shown in Figure 7 the tension capacity for 70 m pile (almost shaft capacity) is 3100 kN and based on compression test in Figure 8 the total capacity for 30 m pile is 1200 kN [27,5].

Because of the difficulties in performance of static pile loading tests in the lake, the full scale pile load tests are limited to the mentioned two cases, and instead, dynamic testing was chosen. In order to measure the bearing capacity of driven piles, a database of case histories from the results of eight full-scale

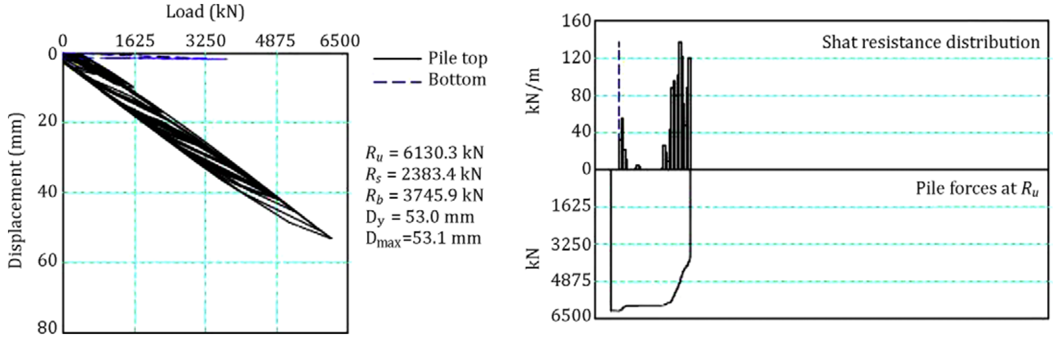

Figure 9: Capacity of Case No. 7 using CAPWAP, based on PDA data, by NGI [3].

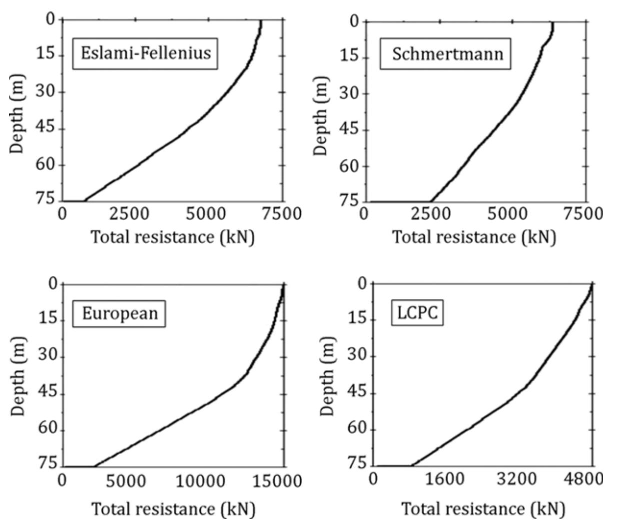

Figure 10: Pile total capacity distribution for case No.7 form three different CPT methods and one CPTu method (pile with length of 75 m, diameter of 813 m and wall thickness of 38.1 mm).

pile dynamic tests is compiled, with information on results of CPTu soundings which performed on adjacent piles [3,28]. The typical CAPWAP results for case No. 7, including total pile capacity and combination of toe and shaft resistances, are shown in Figure 9.

The pile capacity, Q_p, was calculated, using four CPT methods and one CPTu direct method [8,11–14]. Among these methods, the Eslami and Fellenius method [8] is CPTu and the others are CPT. Load distributions were calculated for Case No. 7 in which the toe and shaft capacities are separated for the predictive methods, as shown in Figure 10.

5. Discussions and comments

Considering soil profile, based on CPTu data, the subsoil consists of very soft and sensitive clays [27,5]. In such cases more accurate design is necessary. Usually traditional static methods do not have adequate accuracy for safe and economical pile design. In marine environment, especially in very soft deposits, CPTu sounding supplies continuous records with depth, which is very valuable for pile design. In such cases dynamic testings have been carried out during the investigation which are more reliable, fast and economical.

Recent investigations by Hosseini and Eslami [2] on 13 driven piles in Iran's marine environments showed that the pile capacity calculations based on DeRuiter and Beringen [12] method overestimates and based on Bustamante and Gianeselli [13] method underestimates pile capacity, in comparison to Pile Driving Analyzer testing data, PDA. This is because of using mechanical CPT data with low accuracy, instead of more accrue electrical cone data. Also, these methods were originally developed to fit specific regions and geology, and therefore they need to be calibrated for use in other regions, as presented in Figure 11(a) and (b). European method [12] is based on experience gained from offshore construction in the North Sea, and French method is based gained from offshore construction in the North Sea, and French method is based on experimental work of the French Highway Department [13]. Both methods were developed in sites which had very different geotechnical conditions in comparison with the case under study. Based on the best fit line validation method as presented in Figure 11(a) and (b), the European method [12] overestimates the pile capacity about 50%, and the French method [5] underestimates the pile capacity about 40%. Therefore, it is suggested that one uses the modification factors for making the results more logical and acceptable.

Four current CPT methods which are discussed in this paper apply total stress values. The total stress approaches govern short term behavior of piles capacity, whereas effective stress governs the long-term behavior of pile capacity, that is only considered in the proposed method [8]. The pore pressure effect is negligible in soils such as sands, where the excess pore pressures are small and dissipate quickly. However, in clays and silts, similar to the studied causeway site, the excess pore pressure can be significant. For instance, as shown in Figure 5, from depth 30 m to 70 m, the average value of q_t is about 3.5 MPa and the pore pressure is about 1.5 MPa. Consequently, the role of pore pressure value in pile capacity is not negligible. A method for determining pile capacity particularly in silts and clays necessitates the CPTu sounding. In sand, the pore pressures are assumed to be essentially unchanged during the cone penetration or pile driving. Therefore, CPT data from older types of CPT equipment, not recording the pore pressure, are still useful for design of piles in sand soils.

In order to determine the pile capacity after dissipation of excess pore pressure, Eslami and Fellenius method [8] was employed. The evaluation based on short and long term behavior is shown in Figure 12(a) and (b). In Figure 12(a), the short term pile capacity is obtained, regarding no dissipation pore pressure. According to the best fit line method, by considering dissipation, excess pore pressure, Figure 12(b) indicates that after dissipation, the pile capacity increases about 40%. Unfortunately, after restriking, no PDA test was performed in the causeway site. But, the report indicates that by restriking after 150 days, the energy to drive the piles increased in all

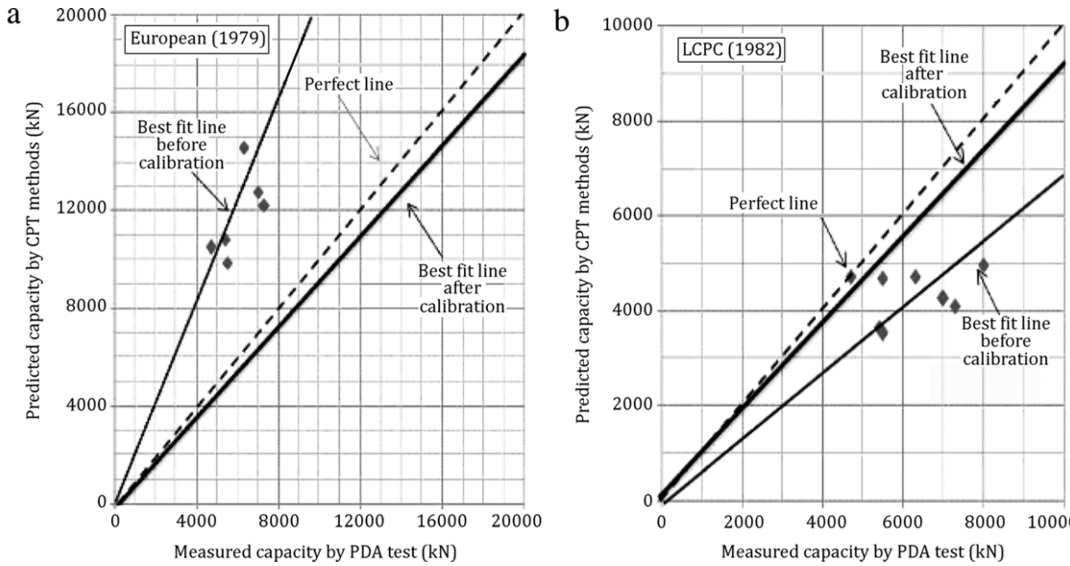

Figure 11: Comparison of predicted pile capacity with measured pile capacity before and after calibration for European (a) and LCPC (b) methods.

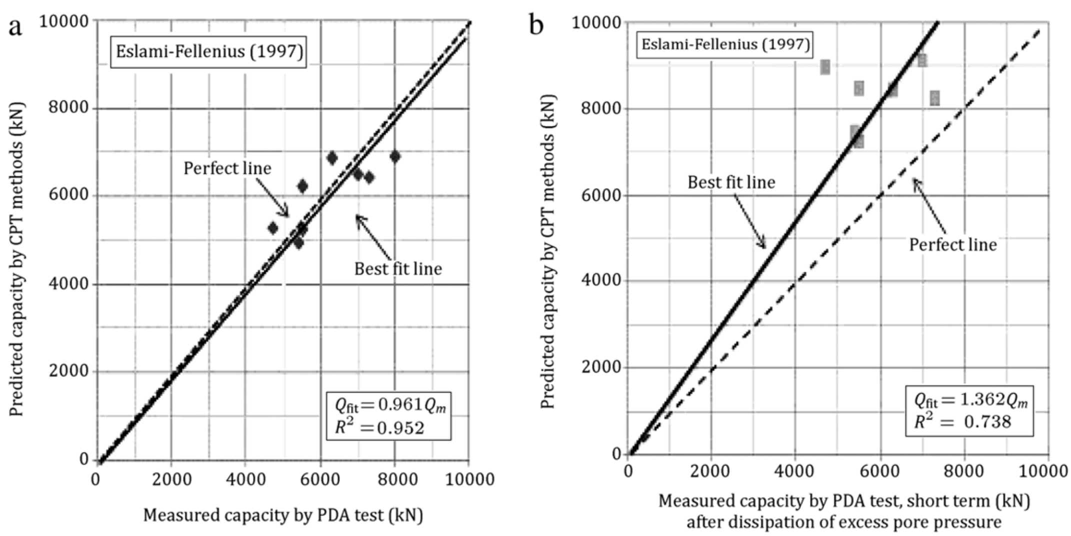

Figure 12: Eslami–Fellenius method. (a) Short term pile capacity; and (b) long term pile capacity (after dissipation of pore pressure).

cases [3]. Obviously it can be concluded that the pile capacity has been increased.

In order to evaluate the performance of different CPT and CPTu estimation methods, static pile compression and tension load tests records are employed. The results of estimated pile capacity and measured values are shown in Figure 13(a) and (b), respectively. As illustrated in Figure 13, the European [12], Eslami–Fellenius [8] and Tumay–Fakhroo [14] predictions are in close agreement with pile load tests results. On the other hand, the LCPC [13] and Schmertmann [11] methods have not shown an acceptable consistency in compression with pile load test results.

6. Validation of the CPT and CPTu methods

PDA test results have been considered as measured pile capacity, Q_m, in order to evaluate the applicability of current CPT and CPTu methods. To validate the methods, an evaluation scheme, using five different criteria, were considered in order to evaluate the accuracy and precision of the methods in predicting the axial capacity of piles by the following criteria employed:

1. The equations of the best-fit line of estimated Q_p versus measured capacity, Q_m, with the corresponding coefficient of determination, R^2.
2. The geometric mean, μ', and standard deviation, σ, for Q_p/Q_m ratio.
3. The 50% and 90% cumulative probabilities, P_{50} and P_{90} of Q_p/Q ratio.
4. The 20% accuracy level obtained from the histogram and log-normal distribution of Q_p/Q_m ratio.
5. The Square Root of Sum of Squares between Q_p and Q_m (SRSS method).

The result of quantified validations is presented in Table 3.

According to the first criterion, the estimated pile capacity, Q_p, is plotted against the measured capacity, Q_m, as shown in Figure 14. For each CPT method, regression analysis was conducted to obtain the line of the best fit for Q_p/Q_m. The relationship between Q_{fit}/Q_m and the corresponding coefficient

Table 3: Quantified evaluation of performance for different CPT and CPTu prediction methods by means of statistical and probability approaches.

Pile capacity method	Best fit calculations		Geometric calculation of Q_p/Q_m		Cumulative probability of Q_p/Q_m		Accuracy ±20%		SRSS method
	Q_p/Q_m	R^2	μ'	σ	P_{50}	P_{90}	Log normal	Histogram	SRSS value
Eslami and Fellenius [8]	0.96	0.952	0.98	0.11	0.93	1.13	0.93	1	1913
Tumay and Fakhroo [14]	1.08	0.895	1.12	0.15	1.13	1.30	0.65	0.75	3081
Schmertmann [11]	0.81	0.862	0.83	0.17	0.79	1.17	0.58	0.62	4460
Modified LCPC [13]	0.92	0.854	0.95	0.19	0.91	1.34	0.67	0.80	3257
Modified European [12]	0.94	0.914	0.094	0.15	1.01	1.18	0.80	0.97	2700

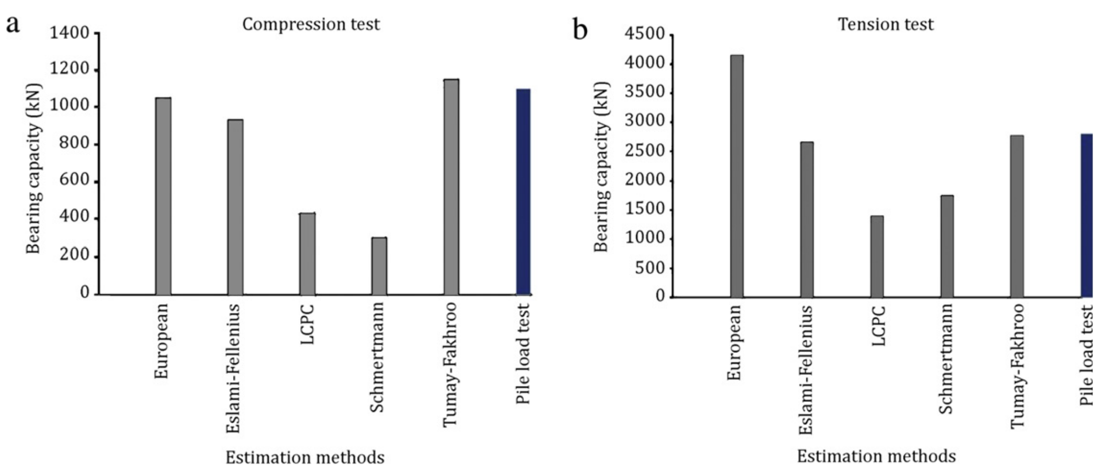

Figure 13: CPT/CPTu based methods bearing capacity in compression with pile load test result. (a) Compressive test; and (b) Tension test.

of determination, R^2, were determined for each CPT method. Figure 14 shows that the method with $Q_{\text{fit}} = 0.961Q_m$ and $R^2 = 0.245$ has better fit equation among the methods. This method, beside modified Eslami and Fellenius [8], modified Bustanmante and Gianeselli [13] and Schmertman [11] methods tend to underestimate the measured pile capacity, while Tumay and Fakhroo [14] method tends to overestimate the measured pile capacity (Table 3).

The geometric mean, μ', and standard deviation, σ, of the Q_p/Q_m ratio values for each method were determined and used as the second evaluation criterion. According to this criterion, Tumay and Fakhroo [14] method has $\mu > 1$, which means that this method on average is over-predicting the measured pile capacity. On the other hand, Eslami and Fellenius [8], Schmertman [11], the modified European DeRuiter and Beringen [12] and Bustamante and Gianeselli [13] all have $\mu < 1$ which means that these methods on average are under-predicting the measured pile capacity.

The third evaluation criterion is based on the 50% and 90% cumulative probabilities, P_{50} and P_{90} of Q_p/Q_m. Cumulative probabilities versus ratio Q_p/Q_m for the investigated methods are presented in Figure 15. P_{50} and P_{90} values were determined and presented in Table 3. The pile capacity prediction method with a P_{50} value closer to one, and with a lower $P_{50} - P_{90}$ range is considered to be the best method. Based on this criterion, the modified European DeRuiter and Beringen method [12] with $P_{50} = 1.01$ and $P_{90} = 1.18$ shows the best efficiency.

The fourth criterion used to evaluate the methods is based on the histogram and the log-normal distribution of Q_p/Q_m. First, the ratio Q_p/Q_m and then the mean and standard deviation were determined and used to identify the log-normal distribution of the density function for each method. The histogram and log-normal probability distribution were used to calculate the probability of predicting the bearing capacity within 20% accuracy. The probability corresponding to 20% accuracy is the likelihood that the estimated pile capacity will be within $0.8Q_m < Q_p < 1.2Q_m$. The probability of predicting the ultimate load capacity within 20% accuracy was estimated and presented in Table 3. Figure 16 depicts the comparison of log-normal distributions for different methods considered in this study. The area underneath each curve in Figure 16 is equal to one. Based on the 20% accuracy level, the Eslami and Fellenius [8] method with values 92.60% and 100% in histogram and log-normal showed the highest probability. At a specified accuracy level, the probability of predicting pile bearing capacity is determined by calculating the total area underneath the curve within the accuracy limits. The higher the probability, the better the performance of the method. The prediction accuracy obtained from the log normal distribution for the different methods is plotted in Figure 17.

The fifth criterion used to evaluate the methods is based on the square root of sum of squares (SRSS) method. The square root of measured pile capacity, Q_m, with predicted pile capacity, Q_p, is given by Eq. (6):

$$\text{SRSS} = \sqrt{\sum_{i=1}^{8}(Q_{mi} - Q_{pi})^2}. \tag{6}$$

According to this criterion, the square root for each CPT methods has been evaluated. The lower the square root, the better the performance. The results show that the Briaud and Tucker method [15] with value of 1913 has the lowest square root, where the modified methods [12–14] and Schmertman method [11] have the value of 2700, 3081, 3257 and 4460, respectively. The result of different CPT methods is presented in Table 3.

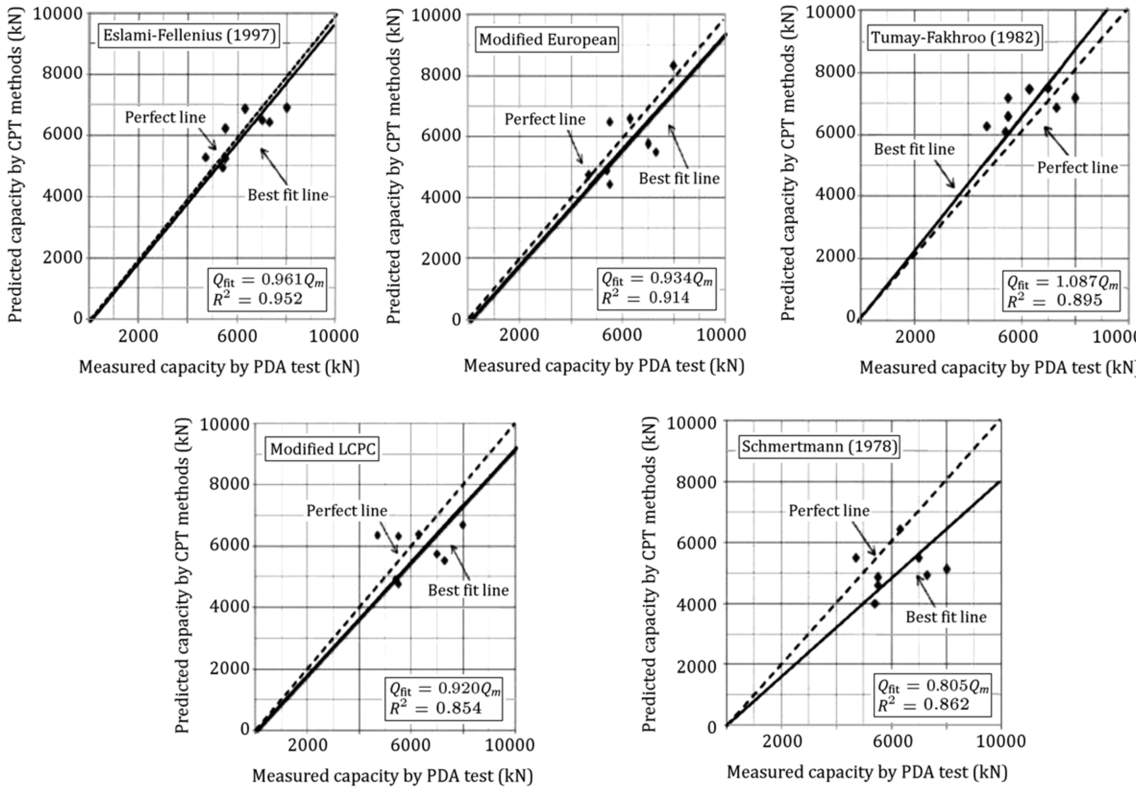

Figure 14: Estimated (Q_p) versus measured (Q_m) pile bearing capacity for different CPT and CPTu methods.

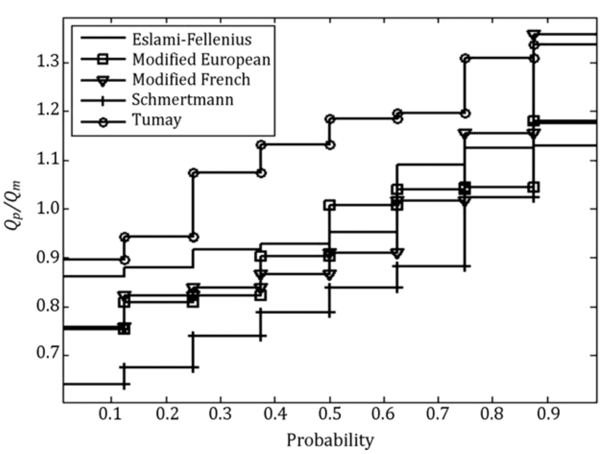

Figure 15: Cumulative probability plot of Q_p/Q_m for different methods.

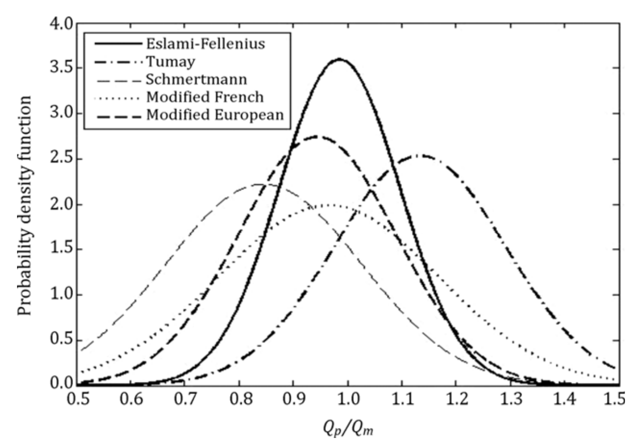

Figure 16: Log-normal distribution of Q_p/Q_m for different methods.

As validation showed, when using any of the four current direct CPT methods, some difficulties arise in calculating the pile capacity. The CPT methods have been developed before the advent of piezocone and therefore neglect the pore pressure acting on the cone shoulder. The subsequent error in the cone stress value is smaller in sand and larger in clay. The CPT methods apply total stress values, whereas effective stress governs the long-term behavior of piles. The current CPT methods are locally developed, that is they are based on limited types of piles and soils, and may not be relevant outside the local area. Moreover, the methods involve a judgment in selecting the coefficient to be applied to the average cone resistance to determine the unit toe resistance, size of rupture zone and filtering process of cone penetrometer data.

7. Conclusions

Four direct CPT methods and one CPTu method have been assessed to estimate the capacity of 10 circular steel driven piles with average embedment of 70 m for Urmiyeh Causeway project in Iran. During the different design stages, it has been found out that CPTu and dynamic tests are two suitable tools for rapid and optimum design and verification of piles. The following points were concluded through these studies:

- The soil profile obtained from CPTu data, based on Eslami and Fellenius [8] and Robertson [7] methods, showed that the most type of sediments in the lake bed consist of sensitive clay and soft clays. In order to cover the mentioned

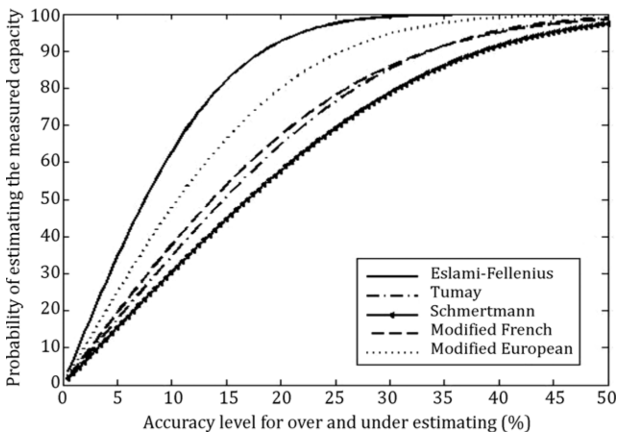

Figure 17: Accuracy level for CPT and CPTu predictive methods.

soils, special considerations are required for pile capacity determination in such an unusual sediments.

- Investigations show that the proportion of predicted pile bearing capacity by different CPT and CPTu methods to measured pile capacity, by pile driving analyzer (PDA) test is in close agreement, in the range of $0.92 \leq Q_p/Q_m \leq 1.08$, which shows acceptable accuracy of CPT and CPTu methods in comparison with traditional static analysis methods. Therefore, combination of CPTu data with dynamic test results can be considered by practicing engineers, for predicting the axial bearing capacity of piles in offshore structures practice.
- Based on the results, Eslami and Fellenius [8], modified European DeRuiter and Beringen [12], Bustanmante and Gianeselli [13] and Schmertman [11] methods tend to underestimate the measured pile capacity. Tumay and Fakhroo [14] method tends to overestimate the measured pile capacity. Among the predicted method's Eslami and Fellenius [8] method shows more consistency with the measured capacities, which is because of consideration of excess pore pressure, sufficient averaging and a logical modeling in CPTu data applying, in comparison with traditional CPT methods.
- The current CPT methods involve errors and scatters for pile capacity estimation. These difficulties are due to disregard of developed excess pore pressure, effective stresses, influence rupture zone, soil classification behavior and logic filtering of CPT data. The performance of the CPT methods may vary according to the procedure used to determine the pile bearing capacity from the load test. The results are also influenced by the characteristics of the soil at site. It is suggested that one uses modification factors 0.45–0.50 and 1.30–1.35 when European DeRuiter and Beringen (1979) and Bustanmante and Gianeselli [13] methods are used, respectively.

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