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Efficiency of methods for determining pile axial capacity-applied to 70 cases histories in Persian Gulf northern shore

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Abstract

Three common approaches to determine the axial pile capacity based on static analysis and in-situ tests are presented, compared and evaluated. The Unified Pile Design (UPD), American Petroleum Institute (API) and a SPT based methods were chosen to be validated. The API is a common method to estimate the axial bearing capacity of piles in marine environments, where as the others are currently used by geotechnical engineers. Seventy pile load test records performed in the northern bank of Persian Gulf with SPT profile have been compiled for methods evaluation. In all cases, pile capacities were measured using full scale static compression and/or pull out loading tests. As the loading tests in some cases were in the format of proof test without reaching the plunging or ultimate bearing capacity, for interpretation the results, offset limit load criteria was employed. Three statistical and probability based approaches in the form of a systematic ranking, called Rank Index, RI, were utilized to evaluate the performance of predictive methods. Wasted Capacity Index (WCI) concept was also applied to validate the efficiency of current methods. The evaluations revealed that among these three predictive methods, the UPD is more accurate and cost effective than the others.

Keywords: *Pile, Axial Bearing Capacity, Full scale load test, Predictive methods efficiency, Wasted capacity index (WCI).*

1. Introduction

The prediction of axial capacity of piles has been a challenge since the beginning of the geotechnical engineering profession. Several methods and approaches have been developed to overcome the uncertainties in the predictions. The methods include some simplifying assumptions and empirical approaches regarding soil stratigraphy, soil-pile structure interaction, and distribution of soil resistance along the pile. Therefore, they do not provide truly quantitative values directly useful in foundation design [1].

The evaluation made by different researchers like Tand & Funegard [2], Salgado & Lee [3], Eslami [4], Titi and Abu-Farsakh [5], Jamshidi et al. [6], Eslami et al. [7], Eslami et al. [8] and etc, have indicated that the performance of different methods show considerable differences. Based on this fact, a few methods have been developed considering local information rather than global results.

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Another main source of such differences can be due to simplifying assumptions and empirical approaches regarding soil profile and load transfer mechanism. Because of this reason, a predictive method has to be exhibited sufficiently accurate rather than it can be considered generally applicable.

So it would be mandatory to evaluate the accuracy of methods using site specific information and measurements. Another aspect which also should be considered to employ a method to estimate pile bearing capacity is cost effectiveness. For instance, however static loading is considered as the best reliable methods to estimate the bearing capacity but the high cost of this method makes it inconsiderable in small or even medium size projects. Therefore, choosing one of the methods for predicting bearing capacity with high relatively safety factor along with economical justification are considered as an important factor in geotechnical engineering practice.

In this study a data bank has been compiled including seventy case records, performed in the northern bank of Persian Gulf, Iran. The cases include of Standard Penetration Test (SPT) performed in the possible closest location of pile locations, surrounding soil properties and the results of static and/or dynamic pile load tests by means of PDA tests and CAPWAP analysis.

Three common methods to determine the vertical bearing capacity of piles were employed in this research. The performance of these three approaches was compared using the cases compiled in data bank.

The comparison was made based on the error analysis of employed approaches and cost efficiency of each method using the Wasted Capacity Index, WCI.

2. Data Bank

In homogeneous soil reinforced by stone-columns, if the A databank of 70 pile tests with different lengths and shapes (mostly pipes and a small number of H and square sections) was collected. All of these piles are performed in the northern bank of Persian Gulf, Iran. Fig. 1 shows the map of Iran and districts of the projects. The site locations are Rajaei Port, Bandar-e Abbas(Site A),Khalij-e Fars ship yard, Bandar-e Abbas(Site B), Tombak, Bushehr(Site C), and Mahshahr port, Khouzestan(Site D).

Fig. 1 The location of sites due to this research

2.1. Descriptions of sites

Site A: Shahid Rajaei Port

Shahid Rajaei port is located in Hormozgan Province and near Bandar-e Abbas city. In the development project of this port, to verify the design of piles under the rear crane beams, totally 8 pipe concrete piles were tested, with diameter of 800 to 1000 mm and embedment lengths of 17 to 27 m. The surrounding soil includes of sand, clay, silt and silty sand.

Site B: Khalij-e Fars Ship Yard

Two dry docks with dimension of 380×86 and 480×86 m, were built in Khalij-e Fars ship yard. In this project 47 driven steel pipe piles were statically and dynamically tested using PDA (Pile Driving Analyzer) device. The length of piles varies between 18 to 25m with two diameters of 1000 and 1200mm, (40 & 48 inches). Soil deposits consist of clay, silty sand and gravelly sand layers with density ranges from 18.5 to 22 KN/m3. Fig. 2 presents a typical SPT record and static load test result for a (A1-122a) pile in site B.

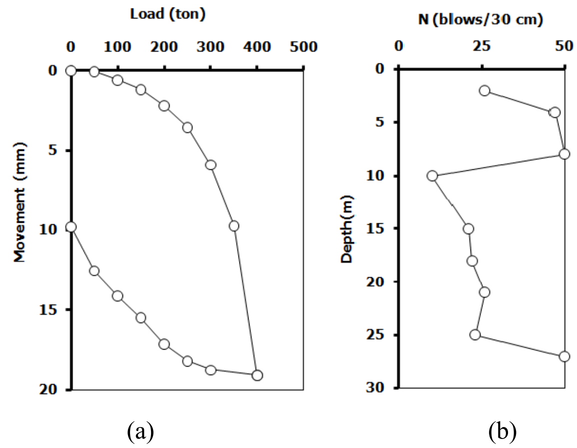

Fig. 2 A typical case records pile load test (a).SPT test result in Khalij-e Fars Ship Yard project (A1-122a pile) (b).

Site C: Tombak

Tombak is located in the north western bank of Persian Gulf and in Bushehr province. Steel pipe piles were utilized to construct different jetties in this port. Soil profiles include silty sand, clayey and silty gravel. Four pipe steel pile load tests were performed in this site, on piles with diameters varies between 400 and 1500 mm with 24 and 25 m of embedment lengths.

Site D:Imam-port:

Imam port is located in Khouzestan province in north western bank of Persian Gulf. To re-new and construct different jetties in this port, totally 11 load tests on different types of piles were executed. The site investigations showed a deep clay layer overlaid by silty sand layers with more than 40 meters of depth.

Table. 1 Summarizes the collected cases from these four sites employed in this investigation.

Table 1 Case records summary

No	Case	Reference	location	shape and material	diameter (mm)	Embedment Depth (m)	Ru (kN)	Soil profile
1	ICP1(D)	[9]	Imam's port	p,c	1000	36	4650	clay,silt,sand
2	125-C(D)	[9]	Imam's port	p,s	150	36	500	clay,silt,sand
3	112-B(D)	[9]	Imam's port	Hp,s	300	36	340	clay,silt,sand
4	62-B(D)	[9]	Imam's port	p,c	300	36	250	clay,silt,sand
5	ICP2(D)	[9]	Imam's port	p,c	1000	36	7770	clay,silt,sand
6	No.25-5(D)	[10]	Imam's port	Sq,c	400	24.5	680	CL,SM,SC,ML
7	B(D)	[10]	Imam's port	p,c	800	31	3000	CL,SM,SC,ML
8	No.19-4(D)	[10]	Imam's port	Sq,c	400	30	3000	CL,SM,SC,ML

No	Case	Reference	location	shape and material	diameter (mm)	Embedment Depth (m)	Ru (kN)	Soil profile
9	No.25-1(D)	[10]	Imam's port	Sq,c	400	24	1800	CL,SM,SC,ML
10	R1(B)	[11]	Rajaei's port	p,c	800	22	9790	SP,SM,CL,ML
11	R2(B)	[11]	Rajaei's port	p,c	1000	27	7730	SP,SM,CL,ML
12	R3(B)	[11]	Rajaei's port	p,c	800	22	8600	SP,SM,CL,ML
13	R4(B)	[11]	Rajaei's port	p,c	1000	27	6980	SP,SM,CL,ML
14	R5(B)	[11]	Rajaei's port	p,c	800	22	8590	SP,SM,CL,ML
15	TPE2(B)	[11]	Rajaei's port	p,c	1000	27	10000	SP,SM,CL,ML
16	TPE6(B)	[11]	Rajaei's port	p,c	1000	17	10000	SP,SM,CL,ML
17	TPE8(B)	[11]	Rajaei's port	p,c	1000	17	10000	SP,SM,CL,ML
18	B1(D)	[11]	Tombak	p,s	457	24	2230	CL,ML,SP,GP
19	B2(D)	[11]	Tombak	p,s	457	24	1200	CL,ML,SP,GP
20	K1(D)	[12]	Imam's port	Sq,c	350	30	1400	clay,silt,sand
21	K2(D)	[12]	Imam's port	Sq,c	500	30	202.5	clay,silt,sand
22	B-8(D)	[12]	Imam's port	Sq,c	350	28.8	1050	clay,silt,sand
23	T1(D)	[13]	Tombak	P,S	1016	25.07	8641.4	SM,GC,GM,CL
24	T2(D)	[13]	Tombak	P,S	1422.4	25.07	9040	SM,GC,GM,CL
25	A1-122a(D)	[14]	BandarAbbas	P,S	1200	23.5	3600	CM,SM,GS
26	A3-125a(D)	[14]	BandarAbbas	P,S	1000	22.5	6520	CM,SM,GS
27	D1(D)	[14]	BandarAbbas	P,S	1016	18	3500	CM,SM,GS
28	D1-28-N(D)	[14]	BandarAbbas	P,S	1000	25	4430	CM,SM,GS
29	E15-123(D)	[14]	BandarAbbas	P,S	1200	21.5	9600	CM,SM,GS
30	E19-108(D)	[14]	BandarAbbas	P,S	1200	18	4020	CM,SM,GS
31	F3-114(D)	[14]	BandarAbbas	P,S	1200	20	8000	CM,SM,GS
32	G11-122(D)	[14]	BandarAbbas	P,S	1200	25	5580	CM,SM,GS
33	G27-122(D)	[14]	BandarAbbas	P,S	1000	20.5	3210	CM,SM,GS
34	G7-107(D)	[14]	BandarAbbas	P,S	1200	19	5580	CM,SM,GS
35	H13-308(D)	[14]	BandarAbbas	P,S	1200	22.5	4000	CM,SM,GS
36	H7-121(D)	[14]	BandarAbbas	P,S	1200	22.5	3500	CM,SM,GS
37	H15-108(D)	[14]	BandarAbbas	P,S	1200	23.5	9230	CM,SM,GS
38	H19-117(D)	[14]	BandarAbbas	P,S	1200	24	9530	CM,SM,GS
39	H18-308(D)	[14]	BandarAbbas	P,S	1200	24	9600	CM,SM,GS
40	H21-324(D)	[14]	BandarAbbas	P,S	1000	20.5	2000	CM,SM,GS
41	i5-319(D)	[14]	BandarAbbas	P,S	1200	23	3510	CM,SM,GS
42	i25-321(D)	[14]	BandarAbbas	P,S	1200	21.5	3610	CM,SM,GS
43	i15-308(D)	[14]	BandarAbbas	P,S	1200	22	2810	CM,SM,GS
44	i21-309(D)	[14]	BandarAbbas	P,S	1200	17	2009	CM,SM,GS
45	i21-314(D)	[14]	BandarAbbas	P,S	1200	22	7180	CM,SM,GS
46	K1-310b(D)	[14]	BandarAbbas	P,S	1000	24	3600	CM,SM,GS
47	K3-312b(D)	[14]	BandarAbbas	P,S	1000	20.5	1800	CM,SM,GS

No	Case	Reference	location	shape and material	diameter (mm)	Embedment Depth (m)	Ru (kN)	Soil profile
48	K4-302b(D)	[14]	BandarAbbas	P,S	1000	19.5	4000	CM,SM,GS
49	K1-312b(D)	[14]	BandarAbbas	P,S	1000	24	3600	CM,SM,GS
50	L3-308(D)	[14]	BandarAbbas	P,S	1200	22.5	3000	CM,SM,GS
51	L13-310(D)	[14]	BandarAbbas	P,S	1200	20	9030	CM,SM,GS
52	L13-321(D)	[14]	BandarAbbas	P,S	1200	22	3000	CM,SM,GS
53	L19-314(D)	[14]	BandarAbbas	P,S	1200	23	10030	CM,SM,GS
54	M7-307(D)	[14]	BandarAbbas	P,S	1000	20.5	3500	CM,SM,GS
55	M21-314(D)	[14]	BandarAbbas	P,S	1200	25.5	10030	CM,SM,GS
56	N15-314(D)	[14]	BandarAbbas	P,S	1200	22.5	9025	CM,SM,GS
57	N19-321(D)	[14]	BandarAbbas	P,S	1200	22.5	4000	CM,SM,GS
58	N15-308(D)	[14]	BandarAbbas	P,S	1200	22.5	9025	CM,SM,GS
59	05-303-C(D)	[14]	BandarAbbas	P,S	1200	18	5860	CM,SM,GS
60	05-308(D)	[14]	BandarAbbas	P,S	1000	20.5	3200	CM,SM,GS
61	015-303(D)	[14]	BandarAbbas	P,S	1200	18	5860	CM,SM,GS
62	019-309(D)	[14]	BandarAbbas	P,S	1000	20.5	1600	CM,SM,GS
63	P9-321(D)	[14]	BandarAbbas	P,S	1000	19.5	3200	CM,SM,GS
64	P7-319(D)	[14]	BandarAbbas	P,S	1000	20	3200	CM,SM,GS
65	P5-306(D)	[14]	BandarAbbas	P,S	1000	20.5	3200	CM,SM,GS
66	P23-307(D)	[14]	BandarAbbas	P,S	1200	20	3200	CM,SM,GS
67	P3-314(D)	[14]	BandarAbbas	P,S	1200	22	6010	CM,SM,GS
68	P7-314(D)	[14]	BandarAbbas	P,S	1200	24	8380	CM,SM,GS
69	P13-314(D)	[14]	BandarAbbas	P,S	1200	22.5	8450	CM,SM,GS
70	P21-321(D)	[14]	BandarAbbas	P,S	1200	21.5	7734	CM,SM,GS

P,pipe; Sq, square; Hp, H section; D, driven pile; B, bored pile; C, concrete; S, steel

2.2. Failure criteria

The pile load test is considered the best method to evaluate the bearing capacity of piles and controlling design assumptions, however because of financial issues; this method is cost effective only in large size projects.

In pile load test, when a rapid movement occurs under sustained or slightly increase of the applied load, the pile plunges and ultimate bearing capacity is defined. However, reaching to this point of pile load-set behavior needs a considerable deformation which in the case of piles with large diameter would not be an easy task. Because of this reason, different interpretation methods to analyze the results of pile load tests has been proposed, such as Davison Offset Limit load [15], Decourt extrapolation [16] , and Maximum Curvature [17] are among these interpretation methods. CFEM [18] has suggested Davison Offset Limit load, Chin-Kondner and 80% criterion of Brinch Hansen for interpretation purpose of pile load tests. However according to Eurocode 7 [19], Geotechnical Design-General rules, when a pile shows a settlement equal to 10% of its nominal dimension, is considered to be failed.

For comparison purposes based on data bank, it is required that all the cases have the same failure condition. Therefore having a unique basis to compare the prediction of different methods, it is necessary that the failure load from the results of pile load tests to be determined based on a unique criterion.

In similar researches performed by Eslami & Fellenius [1], the plunging failure was chosen as the failure criterion; however for current study since the applied movement on the head of piles was limited, plunging failure was not achieved for some cases.

Because different methods may produce widely different values of failure load, the selection of the method for defining the failure condition should account for the characteristic shape of load-settlement curve and soil condition [3].

The plunging failure had not occurred for a majority of cases, but in all cases, the loading was continued beyond yield points to achieve the semi-plastic part of load-settlement curves. for this reason, Davisson offset limit load was chosen to estimate the failure load of piles from load-settlement curves.

UNITEST program (version 3.0), Fellenius & Goudreault [20] was employed to calculate the failure load of piles from load-settlement curve using Davison offset limit, Brinch-Hansen 80%, Chin-Kondner, DeBeer, Maximum curvature and Creep methods. Fig. 3 shows a typical result of this program for H18-313 pile with diameter of 1200 mm and embedment length of 27 m.

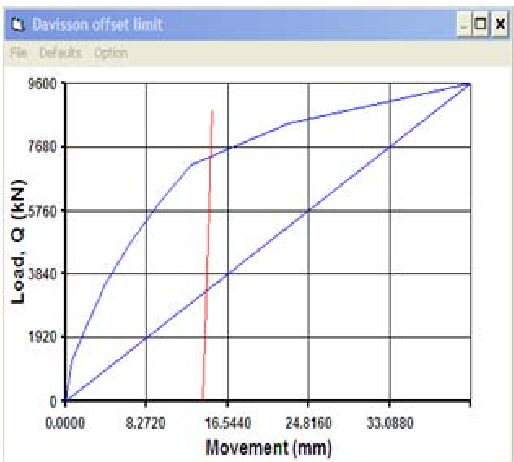

Fig. 3 Typical output of UNITEST software

3. Pile Capacity Estimation Approaches

The static analysis approaches using soil shear strength parameters which are modified form of the equation utilized to estimate the bearing capacity of shallow foundations always are the first step to estimate the bearing capacity of piles. This is because the site exploration and the results of geotechnical characteristics of sub-soils in the form of Mohr-Coulomb parameter are generally available in the preliminary stages of projects. The SPT records also could be generally found in the factual reports of site characterization, which might be employed to estimate the geotechnical engineering parameters. These parameters are used as an input for static approaches indirectly or the SPT records might be directly employed to estimate the bearing capacity of piles, which the latter shows higher consistency.

Among the common preliminary approaches coded in manual and guidelines, three methods were chosen for evaluation purposes in this study which two of them are static approaches and the third is based on SPT results.

API (American Petroleum Institute) method [21] and UPD (Unified Pile Design) recommended by CFEM [18], approaches are the static ones employed in this study. The API method, widely used in construction of marine, onshore and offshore projects.

The third approach is a method based on direct use of SPT records, proposed by Decourt [22], and also suggested by CFEM [18]. This method is one of the common methods to estimate the bearing capacity of piles from SPT records, not only for granular soils but also in clays and silts.

3.1. Methods description

In this section the above-mentioned methods will be described in more details.

I) API method: It is a well known approach to analyze and estimate the bearing capacity of piles mainly in marine environments. For cohesive soils, pile shaft and toe resistances are determined by:

$$r_s = \alpha C \quad (1)$$
$$r_t = 9C \quad (2)$$

Where α is a dimensionless factor calculated by two following relations:

$$\alpha = 0.5\,\Psi^{-.5} for \Psi \leq 1 \quad (3)$$
$$\alpha = 0.5\,\Psi^{-.25} for \Psi > 1 \quad (4)$$
$$\Psi = \frac{C}{p_0'} \quad (5)$$

and C is the undrained shear strength of soil in specific level and p_0' is the effective overburden stress. In this method as can be observed, the effect of both cohesion and overburden stress is considered in the bearing capacity of piles performed in cohesive soils which is similar to the well known α method [23] illustrated in Fig. 4.

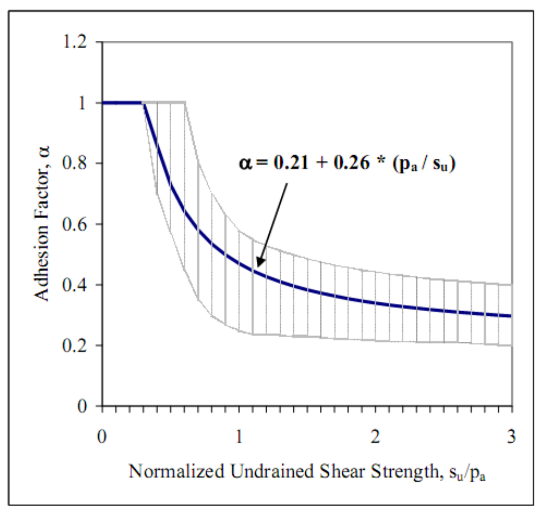

Fig. 4 Variation of α with c_u/p_a based on tested results by Kulhawy and Jackson (1989) [23]

These equations can be utilized with following regulations:

- The Pile toe must be penetrated 2 to 3 times of pile diameter in the cohesive bearing layer
- The Pile toe must be located approximately 3 times of the pile diameter upper the bottom of soft soil layer to prevent punching.

For Granular soils, the shaft and toe capacities can be calculated based on following equation

$$r_s = Kp_0^{'}tan\delta \tag{6}$$
$$r_t = N_q p_0^{'} \tag{7}$$

where K is the coefficient of lateral pressure soil equal 0.8 for both compression and tension loading conditions for open ended piles and, 1 in displacement piles. N_q is the embedment factor and ranges from 8 to 50, depending on soil types and its compaction state.

In this method, the concept of critical depth is considered in the form of an upper limit unit shaft and toe bearing capacity, increasing with the internal friction angle of subsoil or its compaction states. Researches performed by Poulos and Davis [24] confirmed the existence of such concept.

According to Fellenius and Altaee [25], based on their numerical analyses, this concept cannot be realized and related to interpretation of test data and should not be applied. Moreover, Fig. 5 illustrates the trend conceived by Randolph et al [31].

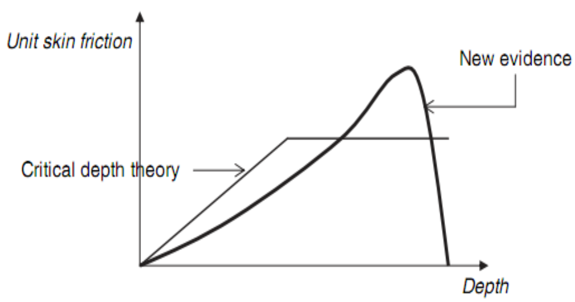

Fig. 5 Variation of skin friction (Randolph et al.,1994)[31]

II) UPD method: Based on this method which is recommended by CFEM (Canadian Foundation Engineering Manual) 2006, the shaft and toe resistance of piles are calculated using the following equations:

$$r_t = N_t\sigma_z^{'} \tag{8}$$
$$r_s = \beta\sigma_z^{'} \tag{9}$$

The β coefficient ranges from 0.25 to 0.7 for cast in place piles and 0.25 to 1.5 for driving piles in different boundary situations. The N_t coefficient varies from 3 to 150 for cast in place piles and 3 to 300 for driving piles in different soils.

III) The SPT based method: According to this method, the unit bearing capacity of shaft and toe might be estimated by following equations:

$$r_s = \alpha(2.8N_{60} + 10)\ (KPa) \tag{10}$$
$$r_t = K_b N_b \tag{11}$$

where α is equal to 1 for displacement piles in any soils and for cast in place piles in clayey deposits and 0.5-0.6 for cast in place piles in granular soils. N_{60} is the average of the quantities of SPT (normalized for 60% energy) along the length of piles.N_b is the average of the quantities of SPT in the adjacent of the toe and K_b is the based factor that ranges from 100 to 325 for displacement piles in different soils and 80 to 165 for cast in place piles in different soils.

4. Methods Evaluation

For comparison of the prediction of the pile's bearing capacity estimation approaches and evaluation of their accuracy and efficiency, the Rank Index, RI was utilized. This index is calculated as follows:

$$RI = R_1 + R_2 + R_3 + R_4 \tag{12}$$

Where R_1 is rank of the method based on highest value of coefficient of determination of Q_p/Q_m, R_2 is the methods rank based on statistical analysis using arithmetic mean and standard deviation, R_3 is methods rank based on cumulative probability analysis and finally R_4 is methods rank based on Log-Normal probability approach. The lower the RI, the more precise would be the method.

Fig. 6, illustrates the variation of the predicted capacities with measured capacities for different methods. According to this figure, the solid line in each diagram reveals perfect agreement between predicted and measured pile capacity passing the origin with a slope equal to unity.

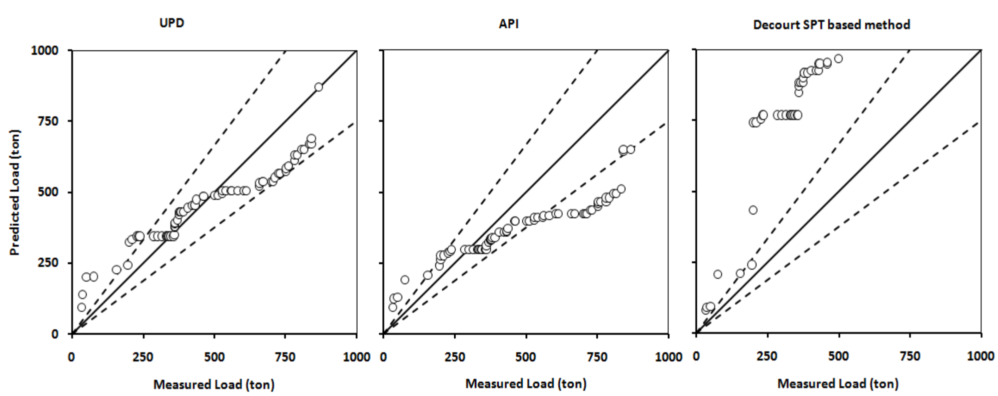

Fig. 6 predicted capacity versus measured capacity for different methods

Analyses of residual error, the difference between observed and predicted values, can be used to evaluate method performance by characterizing, i.e., systematic under or over-prediction [27,28,29,32].

In this approach, Coefficient of Determination (COD) or modeling efficiency is employed to check the compatibility of predictions and measured values.

$$COD = 1 - \frac{\sum_{i=1}^{n}(Q_{p_i} - Q_{m_i})^2}{\sum_{i=1}^{n}(Q_{m_i} - \overline{Q}_m)^2} \tag{13}$$

Where Q_{p_i} and Q_{m_i} are the predicted and measured values, and $\overline{Q}_m$ are the mean of the measured values, respectively, and *n* is the number of samples.

The COD provides a dimensionless statistic summary very similar to the coefficient of determination, R^2 from linear regression. It has been similarly interpreted as the proportional reduction in variation of observed values around the model expectation to variation around the observed mean value. Note $\overline{Q}_m$ represents the "worst case" regression line (slope = 0) indicating a lower bound of 0 for R^2, but Loehle [30] pointed out that no such lower bound exists for COD. In the case of 100% accuracy in method predictions the COD will be equal to one.

Based on this analysis presented in Table 2, UPD, API and Decourt SPT based method, have achieved ranks 1 through 3, exhibiting COD, 0.79, 0.43 and -3, respectively.

The arithmetic average (μ) and standard deviation (σ) of the Q_p/Q_m values were calculated and utilized as a second ranking criterion.

Table 2 Method's Ranking

Methods	COD	R_1	μ	σ	R_2	P_{50}	P_{70}	P_{90}	R_3	P(%)	R_4	RI
UPD	0.79	1	1.15	0.61	2	1.01	1.1	1.36	1	43	1	5
API	0.43	2	0.95	0.52	1	0.84	0.88	1.53	2	39	2	7
SPT	-3.05	3	2.08	0.53	3	2.07	2.33	2.70	3	2	3	12

The closer the arithmetic averages to one, the lower the methods prediction's error. Also, the closer the standard deviation to zero, the lower the scatter of the predictions.

Based on this analysis, the API method showing an arithmetic average of 0.95 and standard deviation of 0.52, is ranked in first place, however the Decourt SPT based method is ranked in third place.

The third approach employed to evaluate the accuracy of methods is cumulative probability measure. According to cumulative probability approach, the ratio of the predicted value (Q_p) to the measured value (Q_m) has been drawn versus cumulative probability [1,26]. For a series of numerals, Q_p/Q_m has been set ascending and indexed with 1 to n. Then for each of the relative amounts, the cumulative probability factor has been calculated as follows:

$$P\ (\%) = \frac{i}{n+1} * 100 \tag{14}$$

Where P is the cumulative probability factor, i is the index of considered case, and n is the number of total cases. To determine the convergence or deviation tendency of the output of prediction, the following criteria have been referred:

- The value of $\frac{Q_p}{Q_m}$ at the cumulative probability of 50% is a measurement of the tendency to overestimate or underestimate the pile capacity. The closer to a ratio of unity, the better the agreement. To estimate the average error, the following equation can be used:

$$E_{ave} = \left(\frac{Q_p}{Q_m}\right)_{\%50} - 1 \tag{15}$$

- The slope of the line through the data points is a measurement of the dispersion or standard deviation. The flatter the line, the better general agreement. Fig. 7 illustrates the cumulative probability analysis in this research.

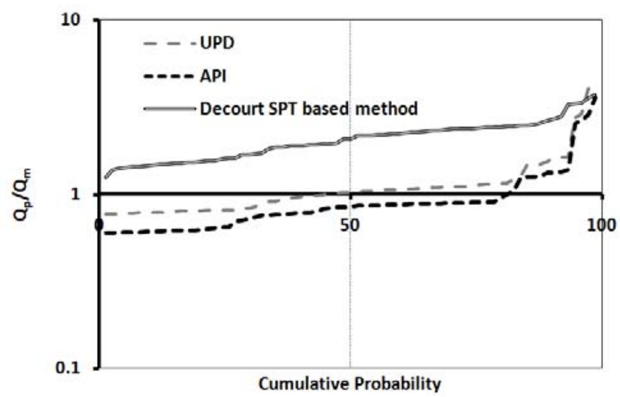

Fig. 7 Cumulative Probability for different methods

- The lowest average error is attributed to the predictions of UPD method with 1% of error. In the case of API and Decourt SPT based method these values are -16% and 107% respectively. It means that the prediction of API method is conservative in compare to the other methods.

- The difference between P50 and P90 also could be used as a scatter parameter. Based on this parameter also, UPD is ranked in first place. Generally based on cumulative probability approach, the UPD, API and Decourt SPT based method are ranked in first through third places.

That last approaches in this evaluation is Log-Normal Probability. The ratio of $\frac{Q_p}{Q_m}$ theoretically ranges from zero to an unlimited upper value, with an optimum value of one. Non-symmetric distribution of $\frac{Q_p}{Q_m}$ will be derived around the mean, therefore, equal weight of under-prediction and over-prediction cannot be reached [32]. A log normal distribution of $\frac{Q_p}{Q_m}$ to assess the performance of pile capacity prediction methods generally are employed [6].

In order to use Log Normal distribution, the mean ($\mu_{\ln}$) and standard deviation ($\sigma_{\ln}$) are evaluated for natural logarithm of $\frac{Q_p}{Q_m}$ as follows:

$$\mu_{\ln}\left(\frac{Q_p}{Q_m}\right)=\frac{1}{n}\sum_{i=1}^{n}\ln\left(\frac{Q_p}{Q_m}\right) \qquad (16)$$

$$\sigma_{\ln}\left(\frac{Q_p}{Q_m}\right)=\sqrt{\frac{1}{n-1}\sum_{i=1}^{n}\left(\ln\left(\frac{Q_p}{Q_m}\right)_i-\mu_{\ln}\right)^2} \qquad (17)$$

The ratio $\frac{Q_p}{Q_m}$ and the natural logarithm of the ratio ln ($\frac{Q_p}{Q_m}$) for each pile were calculated. Then, the mean ($\mu_{\ln}$) and standard deviation ($\sigma_{\ln}$), and the coefficient of variation (COV) of ln ($\frac{Q_p}{Q_m}$) for each method are determined.

The Log Normal distribution is defined as the distribution with the following density:

$$f(x)=\frac{1}{\sqrt{2\pi}*\sigma_{\ln}*x}Exp\left(-\frac{1}{2}\left(\frac{\ln(x)-\mu_{\ln}}{\sigma_{\ln}}\right)^2\right) \qquad (18)$$

Where x=($\frac{Q_p}{Q_m}$), $\mu_{\ln}$ is the mean of ln($\frac{Q_p}{Q_m}$) and $\sigma_{\ln}$ is the standard deviation of ln ($\frac{Q_p}{Q_m}$). The Log Normal distribution is used to evaluate the different methods based on their prediction accuracy and precision.

The length of the histogram peak shows the average error of prediction. The ideal value for this parameter would be unity, and The width of the peak also shows the level of scatter. The higher the width, the lower the scatter.

The Log-normal probability curve for the prediction of these three methods can be observed in Fig. 8. According to this graph, the API method shows lowest level of scatter in the predictions; however, the UPD method exhibits highest level of precision.

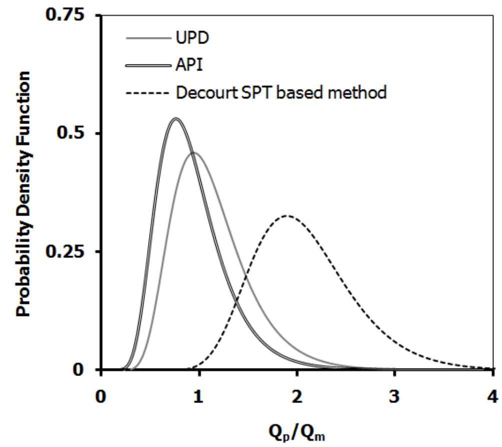

Fig. 8 Log-Normal Probability curve for different methods

Based on the Log Normal distribution analysis, the probability that predictions fall within ±20% accuracy level in these methods can be estimated as follows:

$$P(\%)=100*\int_{0.80}^{1.20}f(x)dx \qquad (19)$$

This parameter might be used as a criterion for ranking the methods.

Based on the analysis, the highest level of accuracy is allocated for the predictions of UPD methods with a probability level of 43%, followed by API methods with 39% and Decourt SPT based method with 2%.

As summarized in Table 3, the UPD method involve more accuracy than the API and Decourt SPT based methods.

Table 3 Wasted Capacity Index for different methods for 50, 70 and 90 percent

Methods	WCI 90%	WCI 70%	WCI 50%
UPD	1.97	1.37	1.10
API	2.05	1.42	1.19
SPT	2.24	1.53	1.28

4.1. Efficiency of the predictive methods

Common routine methods for estimation the bearing capacity of piles, like static approaches, SPT based methods and dynamic formula, involve large values of safety factors. On the other hand, more sophisticated methods like CPT based methods, static or dynamic pile load tests include of lower values of safety factors as they have lower levels of uncertainties capacity predictions.

According to PDCA, Pile Driving Contractors Association, the safety factor of piles in a project in the case of performing pile load tests on at least 1% of piles can be selected equal to 2.1. With increasing the pile load test numbers on the 5% of all piles, this value of safety factor, decreases to a value as low as 1.65.

The Wasted Capacity Index, WCI, proposed by Long et al. [33], is a measure of how inefficiently a method predicts capacity. A precise method will be very efficient and accordingly have a low WCI. On the other hand, a less precise method requires a more conservative design and thus a greater WCI. The value of this factor is calculated from the precision of the method and the reliability required for the pile foundation.

Wasted capacity simply referred to the extra capacity for which a foundation must be designed to account for uncertainties. It means the higher the level of uncertainties, the higher the wasted capacity, implies conservative design approach [33].

The mathematical expression of WCI is:

$$\mathrm{WCI} = \int_0^{(\frac{Q_p}{Q_m})_{\mathrm{required}}} P(x)\,\frac{(\frac{Q_p}{Q_m})_{\mathrm{required}}}{x}\,dx \qquad (20)$$

Where, $(\frac{Q_p}{Q_m})_{\mathrm{required}}$ is a desired level of uncertainty, and P(x) is the log normal distribution function. The x also is the ratio of$(\frac{Q_p}{Q_m})$.The distribution of probability for x, P(x), and$(\frac{Q_p}{Q_m})$ required, is controlled by bias, scatter and selected reliability however the WCI is independent to bias [33].

WCI is calculated for cumulative probability of 50, 70 and 90 percent for all methods and summarized in Table 3.

According to Table 3, the UPD method in all cumulative probabilities is ranked as first; that means has the lowest value of wasted capacity. In other hand, Decourt SPT based method shows highest values of WCI, indicating on the fact that its predictions are due to high level of uncertainties.

5. Conclusions

The accuracy of two static analyses methods i.e. UPD and API and one SPT based method for prediction of pile bearing capacity is investigated and evaluated. For achievement of this purpose, a data bank were collected consisting of 70 full scale pile load tests from four sites in the northern bank of Persian Gulf, Iran. The piles were generally driven with diameter ranges from 150 to 1200 mm and embedment depth of 15 to 36 m. Besides, for all cases, SPT records are available which performed close to pile locations. The Davisson Offset Limit Load method was used to determine the measured load carrying capacities from pile load tests (Q_m) in case of not reaching the plunging ultimate load.

The ultimate load capacity of each pile has been calculated using these predictive methods and the results were analyzed and compared to the measured values.

A systematic ranking approach called Rank Index, RI, is used to evaluate the accuracy and precision of three prediction methods, where R_1 is rank of the method based on highest value of coefficient of determination of Q_p/Q_m, R_2 is the methods rank based on statistical analysis using arithmetic mean and standard deviation, R_3 is methods rank based on cumulative probability analysis and finally R_4 is methods rank based on Log-Normal probability approach. The lower the RI, the more precise would be the method.

In addition, the Wasted Capacity Index, WCI, was employed to assess the efficiency of methods using cumulative probabilities. This index is considered as the general indicator of increased cost. Therefor wasted capacity for a special $\frac{Q_p}{Q_m}$ can be considered as the ratio of $(\frac{Q_p}{Q_m})_{required}$ divided by the value of $\frac{Q_p}{Q_m}$.

According to applied measures regarding the results of 70 pile case histories, the UPD among methods, exhibits highest level of accuracy and lowest values of WCI. Consequently, this method involve lowest values of uncertainties comparing to other methods. Following, the API method in both indexes is ranked as the second and finally, SPT based method, which shows the highest level of uncertainties and errors in pile capacity predictions.

References

[1] Eslami A, Fellenius B.H. Pile Capacity by direct CPT and CPTU methods applied to 102 case histories, Canadian Geotechnical Journal, 1997, No. 6, Vol. 34, pp. 886-904.

[2] Tand K.E, Funegard E.G. Pile capacity in stiff clays-CPT methods, 12th International Conference on Soil Mechanics and Foundation Engineering, Rio de Janeiro, 1982, Vol. 1349-1352.

[3] Salgado R, Lee J. Pile Design Based on Cone Penetration Test, Final Report, FHWA/IN/JTPR-99/8, Purdue University, 1999.

[4] Eslami A. Bearing capacity of piles from cone penetration test data. Ph.D. thesis, Department of Civil Engineering, University of Ottawa, Ottawa, 1996.

[5] Titi H.H, Abu-Farsakh M.Y. Assessment of direct cone penetration test methods for predicting the ultimate capacity of friction driven piles, Journal of Geotechnical and Geoenviromental Engineering, ASCE, 2004, pp. 935-944.

[6] Jamshidi R, Eslami A, Eslami A .Characterization of the correlation structure of residual CPT profiles in sand deposits, 2013, No.1, Vol.11, pp 29-37.

[7] Eslami A, Aflaki E, Hosseini B. Evaluating CPT and CPTu based pile bearing capacity estimation methods using urmiyeh lake causeway piling records, Scientia Iranica, 2011, Vol. 18, pp. 1009-1019.

[8] Eslami A, Veiskarami M, Eslami M. Study on optimized Piled-Raft Foundations Performance (PRF) with connected and non connected piles-three case histories, 2012, Vol. 10, pp 100-111.

[9] Pile load test in Imam-Port development Project, Final Report, Omran Sahel Co, 2006.

[10] Pile load test in Imam-Port development Project, Final Report, Omran Sahel Co, 2007.

[11] Pile load test in Shahid Rajaei Port development Project, Final Report, Omran Sahel Co, 2007.

[12] Compression loading tests on concrete piles in Bandar Imam Project-Iran-khak Co, 1999.

[13] Pile load test in Iran-LNG Project, Final Report, Omran

Sanat Co, 2007.
[14] Compression and Tension loading tests of steel piles in Bandar Abbas project Report-Azim Gostaresh and Rahvar Co, 2002.
[15] Davison M.T. High Capacity Piles, Proceedings, Lecture Series on innovation in Foundation Construction, American Society of Civil Engineers, ASCE, New York, 1972, pp. 81-112.
[16] Decourt L. Behavior of foundations under working load conditions, Proceedings of the 11th Pan-American Conference on Soil Mechanics and Geotechnical Engineering, FozDo Iguassu, Brazil, 1999, Vol. 4, pp. 453-488.
[17] Shen B, Niu D. A new method for determining the yield load of piles, Proceedings of the Fourth International Conference on Piling and Deep Foundations, Deep Foundation Institute, Stresa April 7-12, Balkema Publishers, 1991, Vol. 1, pp. 531-534.
[18] Canadian Foundation Engineering Manual, 4th Edition, Canadian Geotechnical Society, Bitech Publishers, Vancouver, 2006.
[19] Eurocode 7: Geotechnical Design-General Rules, Thomas Telford, London, 2004.
[20] Fellenius B.H, Goudreault P.A. UniTest Version 3.0 for Windows, User Manual, UniSoft Ltd, Ottawa, 1995, 36p.
[21] API, American Petroleum Institute, Recommended practice for planning designing and constructing fixed offshore platforms, 19th edition, Washington D.C, 1990.
[22] Decourt L. Prediction of load-settlement relationships for foundations on the basis of the SPT-T, Ciclo de Conferencias Internationale, Leonardo Zeevaert, UNAM, Mexico, 1995, pp. 85-104.
[23] Kulhawy F.H, Jackson C.S. Some observations on undrained side resistance of CIDH piles, Proceedings, Foundation Engineering: Current Principles and Practices, American Society of Civil Engineers, 1989, Vol. 2, pp. 1011-1025.
[24] Poulos H.G, Davis E.H. Pile Foundation Analysis and Design, John Wiley & Sons, New York, 1980.
[25] Fellenius B.H, Altaee A, The critical depth–How it came into being and why it does not exist, Proceedings of the Institution of Civil Engineers, Geotechnical Engineering Journal, London, 1995, No. 113-2, pp. 107-111, Discussion and Reply in No. 119-4, pp. 244-245.
[26] Long j.H, Shimel I.S. Drilled shafts a data-base approach, In ASCE Proceeding of the Foundation Engineering Congress: current Principles and Practices, Evantson.III, June 25-29, Edited by F.H. Kulhawy American Society of Civil Engineers, Geotechnical Special Publication 22, 1989, Vol. 2, pp. 1091-1108.
[27] Green I.R.A, Stephenson D. Criteria for comparison if single event models, Hydrological Sciences Journal, 1986, Vol. 31, pp. 395-411.
[28] Loague K, Green R.E. Statistical and graphical methods for evaluating solute transport models: Overview and application, Journal of Contaminant Hydrology, 1991, Vol. 7, pp. 51-73.
[29] Zarei Gh, Homaee M, Liaghat A. Modeling transient evaporation from descending shallow ground water table based on Brooks–Corey Retention function, Water Resource Management, 2008, Vol. 23, pp. 2867–2876.
[30] Loehle C. A hypothesis testing frame work for evaluating eco system model performance, Ecol, Modelling 97, 1997, pp. 153–165.
[31] Randolph M.F, Dolwin J, Beck R. Design of driven piles in sand, Geotechnique 44, 1994, No.3, pp. 427–448.
[32] Briaud J.L, Tucker L.M. Measured and predicted axial response of 98 piles, Journal of Geotechnical Engineering, 1988, No. 9, Vol. 114, pp. 984-1001.
[33] Long J.H, Bozkurt D, Kerrigan J.A, Wysockey M.H. Value of methods for predicting axial pile capacity, Transportation Research Record: Journal of the Transportation Research Board, 1999, Vol. 1663, pp 57-63.