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Influence of soil setup on shaft resistance variations of driven piles: Case study

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Abstract

Pile foundations are frequently used in industrial projects in southwest lowlands of Iran. Although high setup of shaft resistance is usually reported in the area, no reliable formulation or guidelines are available for considering the increased capacity in design applications. Therefore, the pile design practices are usually not optimized. The main objective of this paper is presenting a site specific formulation for setup effects of a utility plant in southwest Iran in which a good database of prestressed concrete driven piles is available. Fajr-II Petrochemical site in PetZone of Mahshahr accommodating a utility plant is selected as the database of the current study. The setup factor (A) and the reference time (t_0) are evaluated through processing of a relatively large database of this well-supervised piling project. As the main portion of variations of driven piles capacity with time is related to shaft, only shaft resistance variations are considered in this research. The shaft capacity variations are derived from signal matching analysis on PDA tests. Reliability of PDA tests has been confirmed through comparing with the static load test results. Influence of driving the surrounding piles on setup factor is also investigated. The results show that the average setup factor (A) and the reference time (t_0) of 0.30 and 0.01 day, respectively, are proper values for estimating the long term capacity in this region. Evaluation of the results indicates that driving 8 piles around the test pile has increased the "A" factor average of 40% resultingin increase of the shaft capacity about 19% in one month and 22% in one year, in comparison with the tested piles with no surrounding piles driven.

Keywords: *Driven pile, PDA test, Static load test, Signal matching analysis, Soil setup, Surrounding piles.*

1. Introduction

Determination of real pile capacity is important in optimized design of pile foundations. Many studies are available in literature on the basis of static and dynamic load tests or in situ tests such as SPT and CPT (e.g. Eslami et al., 2013[1]; Fellenius, 2008 [2]; Feizee and Fakharian, 2008[3]; Fakharian and Vaezian, 2007[4]; Fellenius et. al, 1989 [5]). One of the important issues in driven piles is variation of bearing capacity with time after the initial drive. This important issue is well-understood in literature (e.g. Fellenius et. al, 1989[5]; Chow et. al, 1998[6]; Bullock et. al, 2005a[7]). It is pointed out that depending on the soil type, either "soil setup" or "soil relaxation" may occur with time. Soil setup results in eventual increase in the pile capacity, whereas in the soil relaxation condition, the bearing capacity decreases with time.

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Different reasons are stated for these phenomena and the types of soil in which either of the setup or relaxation may occur (Svinkin, 1996[8]; Seidel and kolinowski, 2000[9]; Rausche et al., 2004[10]; Bullock et al., 2005a[7]). In majority of reported cases however, the setup has occurred and relaxation has seldom been reported (Komurka et al., 2003[11]; Axelsson, 2000[12]). The bearing capacity variations are observed to be rapid with time initially, the rate of which substantially decreases with the time elapse.

The stated reasons for setup can be summarized as: (1) generation of excess pore water pressure during pile driving and subsequent dissipation with time, (2) aging. Most of the studies available in literature, however, have simply focused on pile capacity variation with time and it is stated that (for example by Svinkin and Skov, 2000[13]) the most portion of setup is related to dissipation of excess pore water pressure. A recent investigation shows that the aging effect has also a considerable contribution to set up process (Hadad et al., 2012[14]). As Yan and Yuen (2010) [15] mentioned, the second mechanism (aging) is a collective term used by Axelss on (2002) [16] to describe the particle rearrangement around the pile shaft. This rearrangement may accompany by collapsing of temporary arches formed around the shaft which increases the lateral stress on the pile and thus the soil's shear strength and pile's axial capacity.

The setup effects on bearing capacity of piling projects play a significant role in construction time and material savings. Different relations have been proposed to estimate setup effect (e.g. Skov& Denver, 1988[17], Svinkin&Skov, 2000[13], Rausche et al., 2004[10], and Axelsson & Hintze, 2000[18]). In most of the proposed relations, a constant coefficient has been established to correlate the total capacity or skin friction to the logarithm of time. This coefficient, however, has been emphasized to be substantially site dependent (e.g. Rausche et al., 1996 [19] and Bullock and Schmertmann, 2003[20]). Skov and Denver (1988)[17] relation has been applied in most studies in literature. The relation is as follows:

$$\frac{Q_t}{Q_0} = 1 + A\left[\log\frac{t}{t_0}\right] \quad (1)$$

Where, A is the setup factor (dimensionless), t_0 is the reference time, and t is the real time elapsed from end of initial drive. Q_t and Q_0 are the total (or shaft) pile capacity at times t and t_0, respectively.

In the past, the setup effects were attributed to both tip and shaft resistances and the total capacity would have been considered in the relation for Q and Q_0. More recent studies have attributed the setup to shaft capacity and state that the effects on tip are not significant (e.g. Attwoll et al, 1999[21], Bullock and Schmertmann, 2003 [20], Bullock et al, 2005a& b [11] & [22]). The observations and processing results of the data collected for this study confirm this and therefore, the skin friction variations are considered in this paper.

To obtain the setup factor, Q_0 and Q need to be measured at times t_0 and t, respectively. The method adopted in this study was to carry out dynamic load test (DLT) at times t_0 and t and then calculate the tip and shaft resistance using signal matching analysis.

Different studies have reported that the setup factor magnitudes are very scattered and have to be used cautiously from region to region. According to Bullock and Schmertmann (2003) [20] report, values of the setup factor A range from 0.2 to 0.6 (Skov and Denver, 1988 [17]), 0.25 to 0.75 (Chow et al., 1996 [23]), 0.2 to 0.8 (Axelsson, 2000 [12]), and -0.07 to 1.60 (Bullock, 1999 [24]). Konrad and Roy (1987) [25] found pile capacity in over-consolidated soft sensitive marine clay to reach 12 times the initial capacity over a period of 25 days. The maximum capacity was reached after excess pore-water pressure had fully dissipated. Bartoomey and Yushkov (1985) [26] represented skin friction increases of 80% for a four-pile group and 70% for a nine-pile group after 45 days.

Rausche et al. (1996)[19] concluded that for any geological region, the setup factor has to be adjusted performing DLT at the End-Of-Drive (EOD) and Beginning Of Restrike (BOR) and further fine-tuned with the support of SLT.

In this paper, the Fajr-II utility plant as a case study is considered in which the setup factor (A) and the reference time (t_0) are evaluated through processing of a relatively large database. A database of shaft capacity variationsis derived from signal matching analysis on DLT tests performed on “test piles” and “construction piles” at EOD and BOR conditions. Also, positive or negative influences of driving the surrounding piles on setup factor are investigated.

2. Construction site

Mahshahr in Khuzestan Province near Persian Gulf, located in southwest Iran, was selected for developing petrochemical industries in the past two decades. The general geology of this industrial zone is very peculiar, in that many narrow branches of the gulf (so-called fiord) are extended onto the land in tree-shape patterns. The main branch is started from the northwest corner of Persian Gulf, so called Musa Fiord and then divided into several branches such as Jafari Fiord, Zangi Fiord, etc. The fiords are subject to tidal effects and because of high rate of evaporation, layers of salt have been deposited reaching to several meters at some areas. Therefore, treatment of salt or salty layers of soft soil (such as replacement with granular materials) during earthwork is a major challenge.

The soil layers of the region are made of two different geological formations. The upper formation is Deltaic and Estuarine deposits formed of frequent clay, silt and sand layers. These layers are deposited in marine condition and have not experienced appreciable amount of pre-consolidation pressure. The lower formation is formed of dense to very dense sand and non-plastic silt along with stiff to hard clay with silt. These layers are formed of bedrock erosion and decomposition.

Fajr II is a 32-Hectar utility plant in PetZone of Mahshahr accommodating a power plant, pre-treatment and treatment water unit and air unit. Total of 38 boreholes were drilled across the site during the geotechnical study. Table 1 has summarized the geotechnical parameters for layers I to V. Figure 1 shows the overall soil profile and SPT values across the site along 5 distinguishable layers. The dominant soil layering across the construction site is a very soft to stiff clay, average of 15 m thick, overlain a medium dense to dense sand, 4 to 8 m thick (the pile tips are mostly embedded within this sandy layer), continued by a stiff to very stiff clay, 3 to 5 m thick, overlain a dense to very dense sand, 4 to 8 m thick, continued by a stiff to hard clay.

Different types of precast and prestressed driven concrete piles at 8700 points with a total length of 150,000 m were constructed within three years. About 7000 points include 450 mm outside diameter prestressed spun piles with wall thickness of 80 mm and closed-toe driven in water and air units. The spun piles have been driven with Kobe-35 and Kobe-45 single-acting diesel hammers, or equivalents, down to embedment depths ranging between 14 through 22 m.

As the construction was in progress, SLTs were carried out on 33 piles in compression, 2 in tension and 7 lateral loading. Total of 462 piles were monitored by 566 DLTs (PDA) for QC/QA purposes. Out of 462 piles PDA tested, 82 piles were monitored during initial driving only, 282 piles were tested at restrike (one time or more) and 98 piles were tested both during initial driving and at restrike (one time or more).

Table 1 Summary of the geotechnical parameters for layers I to V

Parameter		Unit	Layer I	Layer II	Layer III	Layer IV	Layer V
Soil Type		*USCS*	CL or ML	SM or ML	CL or ML	SM or ML	CL or ML
Depth (Min / Max /Average)		*m*	4.5 / 16 / 14	1.5 / 16 / 4	14 / 2 /3	18 / 1 / 4	*
Liquid Limit (LL)		%	30	-	32	-	37
Plasticity Index (PI)		%	10	-	12	-	18
Moisture Content (ω)		%	30	-	31	-	-
TotalUnit Weight (γ_t)		kN/m^3	20	20	20.7	21	21
Angle of Internal friction (Ø)		*degrees*	0	29-36**	0	41	0
Cohesion (c)		*kPa*	***	0	***	0	-
ElasticModulus (E)		*MPa*	20	80	40	125	90
Poisson's Ratio (ν)		-	0.5	0.3	0.5	0.3	0.5
Consolidation	$C_c/(1+e_0)$	-	0.124	-	0.127	-	-
	$C_s/(1+e_0)$	-	0.012	-	0.014	-	-
	OCR	-	1-3	-	1	-	-

*Down to the maximum drilled depth as 40 m, 18 m as a maximum and 2 m as a minimum.
**widely scattered (average 33^0). At some location and at deep level equal to 41^0.
*** c = 30 – 2.5D for 0<D<6 m & c = (D+8.4)/0.96 for D>6 m (D is depth in term of meter)

3. Pile load test program

A total of 4650 spun piles, 450 mm OD were driven to support 12 water tanks which are the focus of the current research. Pile DLTs and SLTs were carried out on 30 "test piles" and 221 "construction piles". The statistical overview of the "test piles" and "construction piles" is presented below:

3.1. Test piles

Total of 30 "test piles" were driven and DLTs carried out on them. All the 30 "test piles" were monitored by DLT during continuous initial driving and 21 piles were also tested at restrike. Three static compression load tests and 2 lateral tests were carried out on 5 test piles.

Figure 2 presents the water unit structures, location of the 30 "test piles" and the geotechnical boreholes. Blow count numbers and SPT variations for the nearest borehole are shown in Fig. 3 for TP#11 with 15.4 m embedment depth and BH39, as an example. Table 2 has summarized the range and average values of embedment depth, blow counts, hammer energy, pile maximum compressive stress, pile maximum tensile stress and beginning of restrike (BOR) PDA capacity of driven "test piles".

3.2. Construction piles

The capacity and quality controls were carried out with support of pile DLTs and SLTs during construction on 221 construction piles, including 251 DLT tests and 20 compressive SLTs. In fact about 5% of the construction piles were DLT tested and average of 2 piles were SLT tested at each tank (about 0.5%). With support of the testing program, the factor of safety was lowered to about 2 through 2.2 and sometimes as low as 1.8 (with additional tests), that resulted in considerable savings compared to previous projects in the region. More details on design and construction process can be found in Fakharian et al. (2012)[27].

The results of 12 water tanks with 4650 pile points are selected as the database in current research. Water unit tanks were constructed with different capacities ranging from 5000 to 54000 m^3, different diameters from 21 to 70 m, but all having the same height of 14.5 m. All the water tanks are supported by the prestressed spun piles. The details of each tank geometry and number of piles are presented in Table 3.

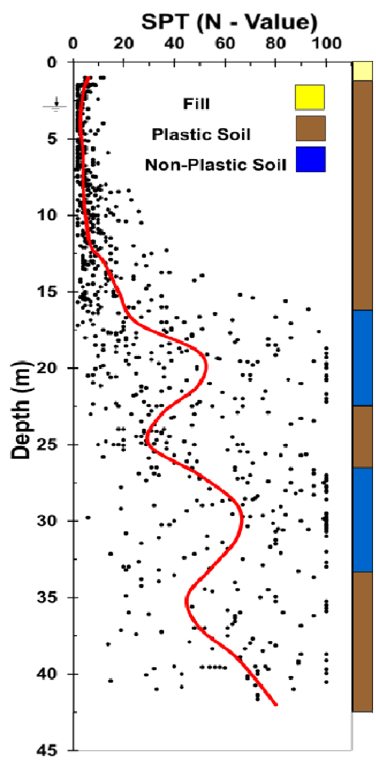


Fig. 1 SPT versus Depth for 38 BHs across the 32-Hectar site

Figure 4 presents the sample results of PDA and static tests on pile 522 of raw water B tank. PDA signal at EOD and BOR conditions are shown in Figs 4a and 4b, and load-settlement chart of static test is presented at Fig. 4c. Setup phenomenon

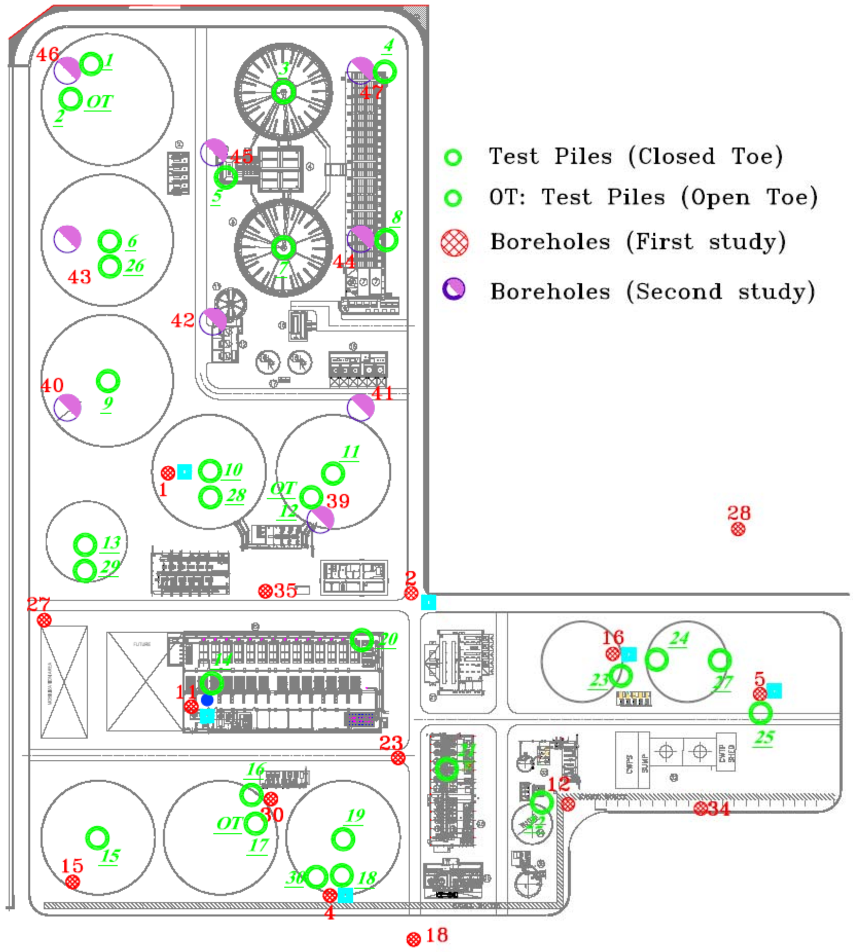


Fig. 2 Plan view of water unit structures, boreholes and test pile locations, Fajr II, Mahshahr, Iran

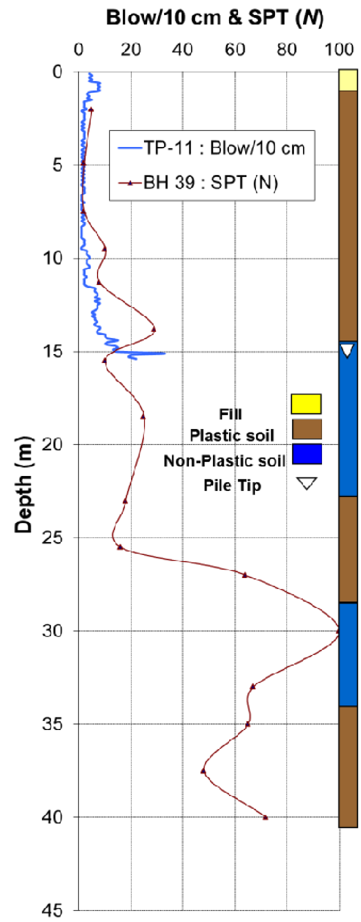


Fig. 3 Blow count, SPT N-value and soil profile for TP11

Table 2 Average of 30 "Test piles" PDA tests and CAPWAP analysis results

	Embedment depth (m)	Blow count*	Hammer energy (kJ)	Max compressive stress (MPa)	Max tensile stress (MPa)	Ultimate Load (kN)
Average	18	23	44	33	3	2598
Max	22.7	51	68.5	42.6	6.7	3258
Min	14.2	12	22.8	21.2	0	1942

*: average of blow count per 10 cm penetration for the last four 10 cm counts

Table 3 Tank geometries and No. of piles (Research Database)

Structure	No.	Diameter (m)	Height (m)	Piles C/C spacing (m)	Average Embedment Depth (m)	Each Tank # of Piles	Total # of Piles	Total # DLT (PDA)	Total # ofSLT
RAW	**3**	70	14.5	2.65	20.1,17.6,15.5*	577	**1731**	**81**	**8**
RO	**3**	60.4	14.5	2.65	16.5,17.2,16.4	421	**1263**	**74**	**7**
Clarified	**2**	60.4	14.5	2.65	17.6,17.1	421	**842**	**57**	**5**
DM	**2**	43	14.5	2.4	17.7,17.9	261	**522**	**30**	**2**
Fire	**1**	43	14.5	2.65	17.6	223	**223**	**29**	**1**
Healthy	**1**	21	14.5	2.65	16.2	69	**69**	**2**	**-**
Sum	**12**	**-**	**-**	**-**	**-**	**-**	**4650**	**273 (5%)**	**23 (0.5%)**

* For RAW-A, B and C, respectively.

in shaft capacity is the main reason of changing the shape of EOD signal to BOR one. Area between the force and velocity line from impact pick point and 2L/C after it, represents the shaft capacity (L is distance between pile toe and gages in meter and C is the wave speed in meter/mili second). Comparing this area between 2 signals, the shaft capacity approximately doubled from EOD to BOR conditions. Signal matching analysis on these signals show that the shaft capacity increased from 707 kN at EOD condition to 1391kN at BOR condition. Static test load-settlement chart interpretation is done by Davisson Offset Limit method as presented on Fig. 4c.

4. Results

The purpose here is to back-calculate the setup factor, A, from DLT results at different times from EOD. Having used

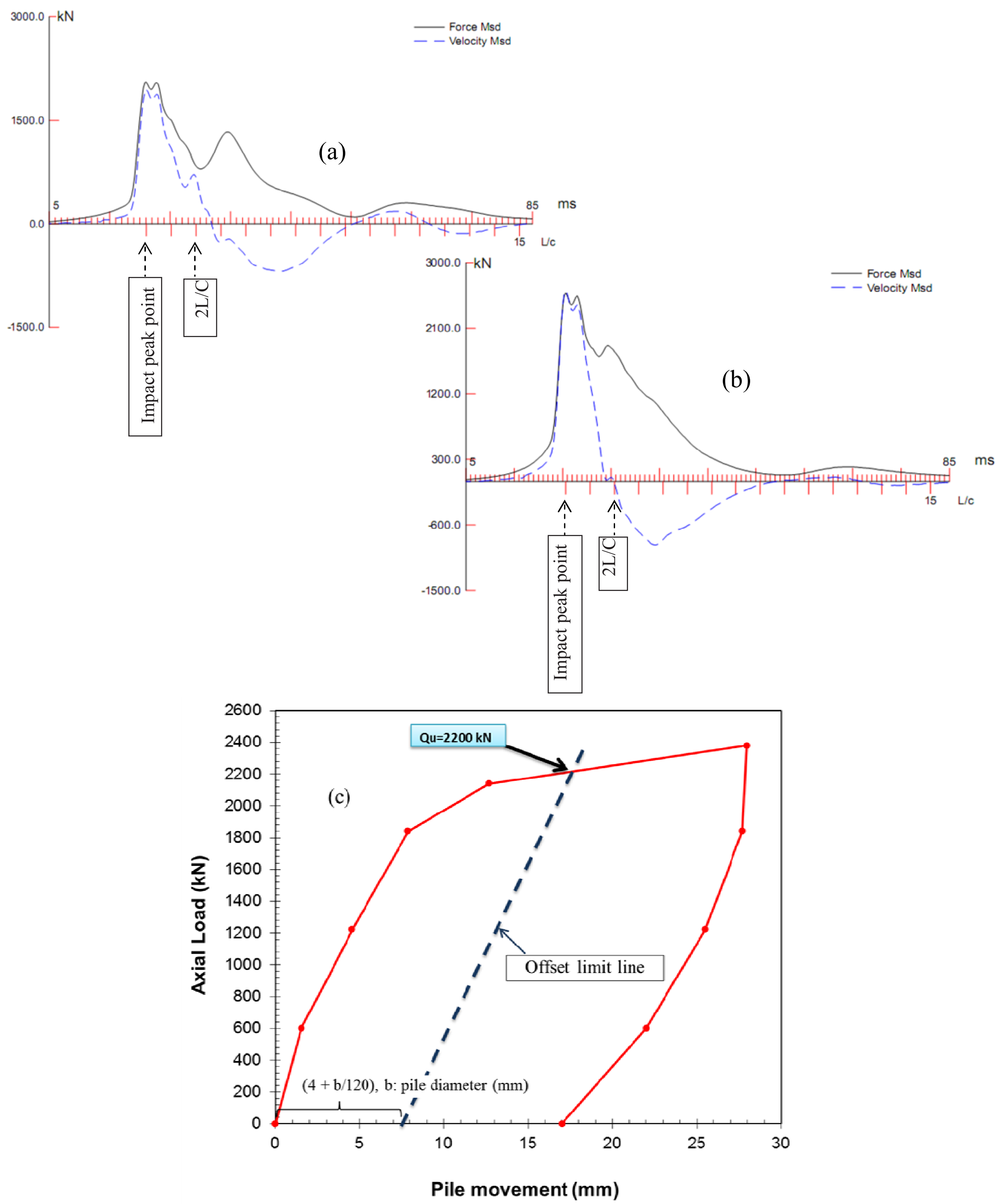


Fig. 4 Sample results of PDA and static tests on pile#522 of raw water B tank, (4a): PDA signal at EOD condition, (4b): PDA signal at BOR condition, and (4c): load-settlement chart of static test

signal matching analysis, the tip and shaft resistances are distinguished first. Then Q_t/Q_0 (shaft resistance) for corresponding t/t_0 is calculated for each pile, out of which A has been determined using Skov and Denvor relation. The range of A and its average for all piles of the water tank are then determined.

Different values are proposed for t_0 ranging 0.01 to 1 day in previous studies. From the existing test data in the study area, t_0 was obtained equivalent to 0.01 to best estimate the variations of shaft capacity with time. This could be attributed to the fact that capacity variations are very quick in the region as has been approved by the restrike tests conducted in less than 24 hours. The details of determination of A is shown for one of the representative tanks and then the results of all tanks are presented and discussed.

Raw water B tank (RAW-B) is among the largest tanks with 70 m in diameter. To transfer 140 kPa of uniform load to more competent layers, 577 piles were driven to embed the tips in the medium to dense sand of layer 2 with embedment depth ranging 16.5 to 19 m (with average of 16.5 m). All the piles are 450 mm outside diameter high-strength prestressed spun piles driven Center-to-Center distance of 2.65 m or S/B ratio of nearly 6. For quality control (QC) and capacity verification purposes, 23 piles were static or dynamic load tested at different times from EOD. Two of the piles were dynamic load tested after one year from initial drive. The break down of conducted tests is 5 EOD test, 19 BOR, and 3 SLT. The BOR tests have been conducted as early as ½ hour to as long as 574 days.

The variations of normalized shaft capacity (Q/Q_0) with respect to $\log(t/t_0)$ for RAW-B Tank are plotted in Fig. 5. The lower and upper limits of the plotted points are shown in Fig. 5a and the average is shown in Fig. 5b, indicating a range of setup factor values between 0.26 to 0.36 with an average of 0.32. This range is wide within the size of simply one structure. One reason could be the relatively high variability of geometry and properties of soil layers. But it was also found out that the number of surrounding piles driven when the pile has been restrike tested contribute to variations in capacity. This observation has been discussed in more detail in subsequent sections. Processing the data for the all tanks has indicated a range of setup factor values between 0.23 to 0.38 with an average of 0.30.

5. Practical implications

5.1. Static versus dynamic tests

To show the reliability of DLTs at the study area, comparisons between DLT and SLT for 15 piles onto which both tests have been carried out are presented in Fig. 6. It is noticed that in most cases the ultimate capacity obtained from SLT is greater than the DLT results. It is reminded that a 45 kN diesel hammer has been used for restrike tests, having sufficient energy to mobilize the ultimate capacity. The main reason of underestimating the capacity in DLT results is differences in testing time. Most SLTs were performed after a longer time compared to DLTs. At the time of PDA tests, soil setup effects have not been completed yet. As an example, for pile 502 in RAW-A Tank, pile 562 in RAW-B Tank and pile 33 in Clarified-B Tank that the time difference between the two tests has been short, the prediction points are very close to the 45º line. But in pile 387 at RAW-A Tank, onto which the DLT was performed after several hours, while the SLT was carried out 111 days after, the maximum deviation is observed.

To compensate for the difference in test times, the skin friction of PDA test results has been predicted using Skov and

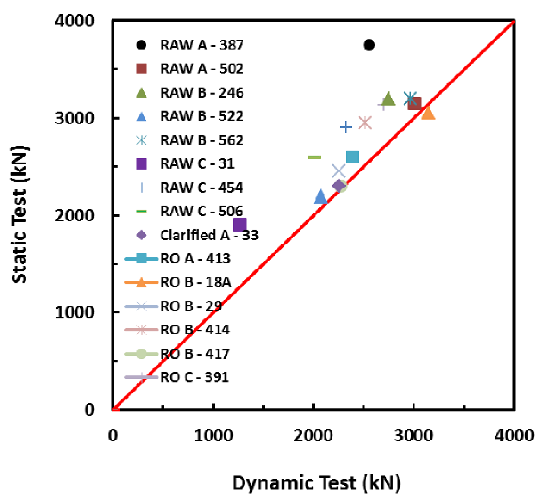
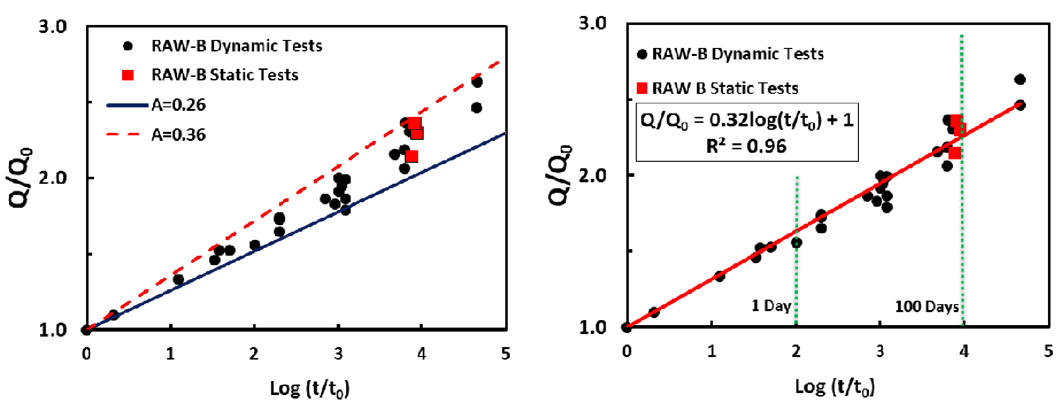


Fig. 6 Comparison of static (SLT) and dynamic (DLT) test results



(a) (b)

Fig. 5 Setup factor (A) for DLT tested piles of tank RAW-B; a) upper and lower bounds, b) average

Denver (1988)[17] relation specifying an "A" value of 0.3. In fact the increase in skin friction for the time difference between DLT and SLT has been calculated and added to PDA predictions. The corrected capacity is referred to as "modified dynamic test". The modified results are presented in Fig. 7. A much better correlation is observed between SLTs and DLTs, after the time difference modifications. For example, consider pile No. 387 in RAW-A Tank, as shown in Fig. 6.Total capacity of 2558 kN was obtained in 2.5 hours (0.104 day) PDA restrike test, whereas 3750 kN capacity has been achieved after 111 days SLT. Signal matching analysis on the results of the 2.5 hour (0.104 day) PDA restrike test show that the 2558 kN total capacity can be divided to 1328 kN and 1230 kN shaft and toe resistances, respectively. To compensate for the 110.9 days difference in test times, the skin friction of PDA test results in 111days has been predicted from skin friction of PDA test results in 0.104 day using Skov and Denver (1988) [17] relation specifying an "A" value of 0.3 and "t_0" value of 0.01 as follows:

$$\frac{Q_{s(t)}}{Q_{s(t0)}} = 1 + A\left[\log\frac{t}{t_0}\right] \to \frac{Q_{s(t1)}}{Q_{s(t2)}} = \frac{1 + A\left[\log\frac{t1}{t_0}\right]}{1 + A\left[\log\frac{t2}{t_0}\right]} \xrightarrow{t1=111days,\ t2=0.104day,} Q_{s(111)}$$

$$= Q_{s(0.104)} \times \frac{1 + A\left[\log\frac{t1}{t_0}\right]}{1 + A\left[\log\frac{t2}{t_0}\right]} \to Q_{s(111)} = 1238 \times \frac{1 + 0.3\left[\log\frac{111}{0.01}\right]}{1 + 0.3\left[\log\frac{0.104}{0.01}\right]}$$

$$= 2252\ kN \to Q_{(111)}$$

$$= 2252\ (predicted\ shaft\ resistance\ after\ 111\ days)$$

$$+ 1230\ (toe\ resistance) = 3482\ kN$$

5.2. Surrounding piles influence

Figure 4a indicates that the setup factor for different piles of RAW-B is between 0.26 through 0.36. The question is why there exists such a wide range within the foundation zone of simply one structure? It would have been expected to come up with a narrower range of setup factor considering that the test time effects had been compensated for. Further evaluation of results indicated that the number of piles driven around the tested pile before the test date has influenced the setup factor too. In other words, the more piles driven around the tested pile, the higher the set up factor. Therefore, pile driving at the study site contributes to increase in shaft capacities of the adjacent piles, even for the S/B ratio of nearly 6. The increase in shaft capacity could be attributed to increase in normal stress on the skin of the tested pile resulted from driving the surrounding piles.

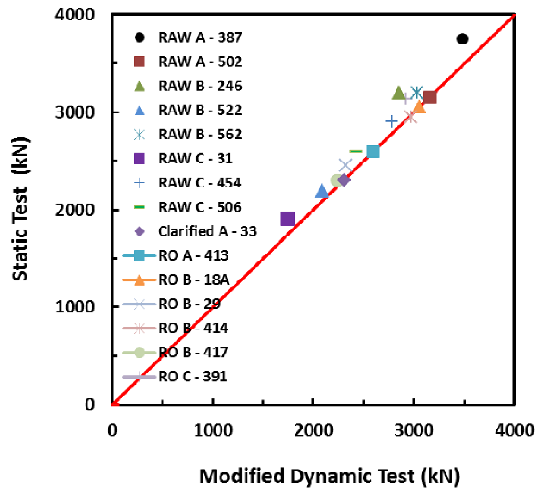


Fig. 7 Comparison when modification on soil setup effects are done on dynamic tests

Up to 8 piles could have been driven all around each pile at testing time. For example, piles 6 and 26 with no piles driven around them (after 12 and 9 days, respectively) resulted in a setup factor of 0.28. but the same piles at 457 and 453 days from initial drive, respectively, resulted in 0.35 for pile 6 with 6 piles (out of 8) driven around it, while pile 26 resulted in 0.31 with only 4 piles driven around it. Similar trend was observed for other tested piles. Table 4 presents several examples.

Evaluation of 221 piles tested in 12 tanks of Fajr II indicated that driving 8 piles around the test pile at testing time has increased the A factor average of 40%. Applying the 40% increase in A factor for 8 surrounding piles for RAW-B Tank (A increasing from 0.28 to 0.39) and Clarified-A Tank (A increasing from 0.23 to 0.32), the shaft capacity has increased 17 to 19% in one month and 20 to 22% in one year as presented in Table 4. The calculated 1 month and 1 year increases in capacity are calculated assuming that all the 8 piles are driven around each pile.

The available results in literature also indicate that the bearing capacity of piles within the group is different than single piles. Most studies have shown that pile group capacity in clay decreases while in sand increases (e.g.Meyerhof, 1960 [28], Kishida, 1967 [29], O'Neill, 1983 [30], Liu et al.,

Table 4 Setup factor (A) comparison for sample piles with different number of driven surrounding piles

Tank	Pile #	#of surrounding piles	Setup factor (A)	Average increase in capacity (1 month) (%)	Average increase in capacity (1 year) (%)
RAW-B	6	0	0.28		
RAW-B	26	0	0.28		
RAW-B	6	6	0.35	19	22
RAW-B	26	4	0.31		
RAW-B	526	7	0.36		
Clarified-A	4	0	0.23		
Clarified-A	362	6	0.31	17	20
Clarified-A	152	7	0.33		

1994[31], Zhang et al., 2001[32], and Ismael, 2001[33]). MaCabe and Lehane (2006)[34] showed that in a group of five 250 mm square piles in clayey silt with S/B of 2.8, the average group coefficient was 0.98 in full-scale compressive tests. Ismael (2001)[33] conducted aload test on a group of five 100 mm piles in medium dense weakly cemented silty sand (SPT about 15 to 20) with S/B equal to 2 and 3 and concluded group efficiencies of 1.22 and 1.93, respectively.

The experimental studies of Al-Mhaidib (2007) [35] on groups of 2,3, 4, 6 and 9 piles with S/B ratios of 3 or 9 embedded in clay showed that the group efficiency decreases with increase in number of piles, but increases as the pile spacing increases. For example, for a loading rate of 1 mm/min, the group efficiency of a 2×1 group increases respectively, from 0.91 to 0.97 for S/B ratios of 3 and 9, but the efficiencies decreases respectively, to 0.83 and 0.87 for a 2×3 group. Figure 8 shows a model investigation conducted by Meyerhof (1960)[28] in loose sand and soft clay on large and small piles driven in group. Group efficiency of about 1.2 in current study with long piles (L/B>35), mixed soil of clay and sand and S/B=5.9 is in agreement with Meyerhof (1960)[28] investigation. Figure 9 shows the results of group efficiency driven in sand conducted by Kishida and Meyerhof (1965)[36].

Vesic (1968)[37] conducted some experiments on instrumented pile group in sand and concluded that the group effect should be taken into account only for the shaft resistance (Fig. 10). Comodromos et al. (2003)[38] carried out numerical analyses to study the group efficiency for different S/B ratio at a site with soil layers comprised of gravel, sand, silt and clay. The numerical model was calibrated with full-scale SLTs. The results showed that increase in S/B for a group of 9 piles from 3 to 4.5 and 6 resulted in group efficiency increases respectively, from 0.94 to 1.02 and 1.17. This is while the results presented in the current study revealed that bearing capacity increased about 19% for the piles tested when surrounded by 8 piles (Table 4). This finding compares very well with 17% increase of Comodromos et al. (2003)[38] for a pile group with S/B of 6. This is while in both studies the soil layers were consisted of granular and fine-grained soils and pile spacings were comparable too, i.e. S/B of 5.9 (this study) versus 6 for Comodromos et al. (2003)[38].

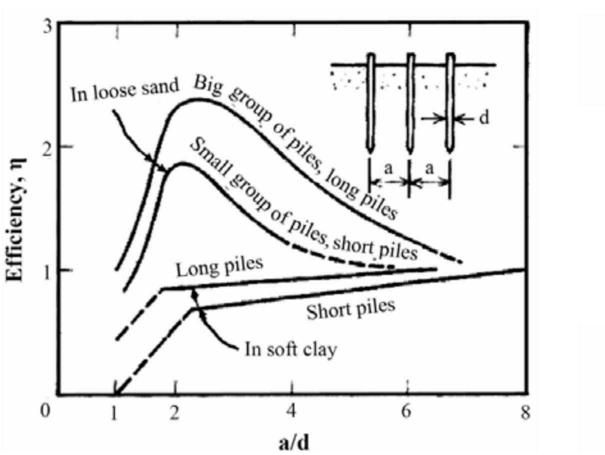


Fig. 8 Group efficiency for driven piles in soft clay and loose sand (After Meyerhof, 1960[25])

5.3. Effective stress analysis

Effective stress analysis is a practical method in pile design. Normally the effect of soil setup on the effective stress parameter for skin friction calculation, β, is not considered in pile design practices. But it is of practical importance to propose realistic parameters for design applications. Shaft capacity in β-method is determined through the following equation (CFEM, 2006)[39]:

$$q_s=\sigma_v' K_s \tan\delta=\beta\sigma_v'$$

where

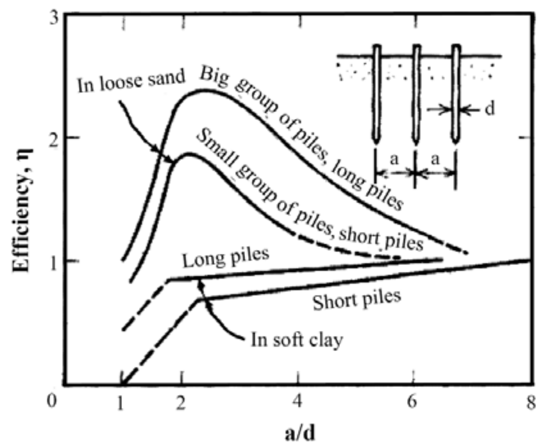


Fig. 9 Group efficiency for driven piles in sand (After Kishida and Meyerhof, 1965 [33])

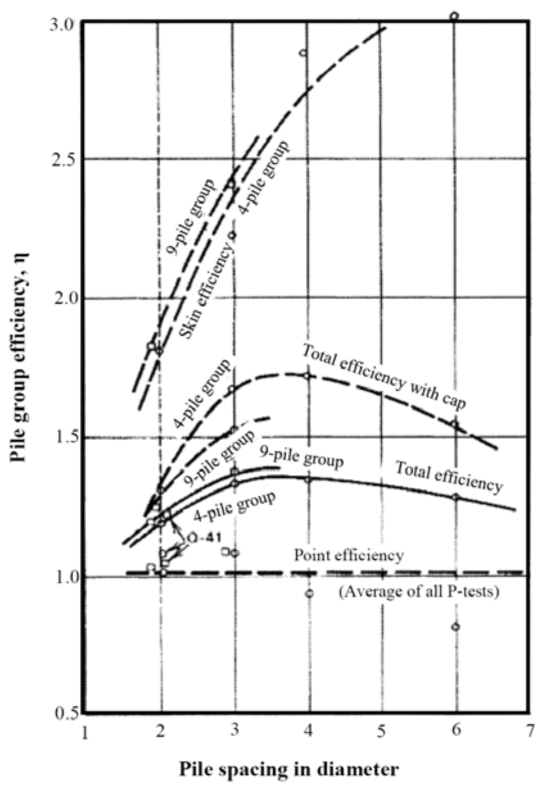


Fig. 10 Group efficiency for piles in sand (After Vesic, 1968 [34])

q_s = pile shaft capacity
β = a combined shaft resistance factor (Bjerrum-Burland factor)
K_s = coefficient of lateral earth pressure
σ_v'= vertical effective stress adjacent to the pile at depth z
δ = angle of friction between the pile and the soil

As the skin friction distribution and tip resistances are distinguished in this study using signal matching analysis, it is possible to calculate the β-factor using back-analysis for different soil layers and both at the EOD and restrike.

Soil layer I of the study site (Table 1), very soft to stiff clay, may be divided to two layers of soft clay, and medium stiff to stiff clay. If so, dominant soil layering along the piles shaft are classified in three layers, from top to bottom respectively, layer I-1 with 7 to 8 m thickness as soft clay, layer I-2, 6 to 7 m thick as medium stiff to stiff clay, and layer II, sand down to 20 m.

Figure 11 presents the β variations with depth for all the data points at EOD and BOR tests. The trend lines of EOD and BOR are plotted separately. Effect of setup on β parameter is quite noticeable, in particular in layers I-2 and 2. An exponential equation and corresponding R^2 values are shown for each trend line in the figure. It is noticed that the lowest increase in β ratio has occurred in soft clay layer I-1, while the maximum ratio has occurred in sandy layer 2, measured 28% and 98% respectively. The increase in layer I-2, medium stiff to stiff clay, is 91% which is also significant.To come up with design parameters for the study area, the range of β values at restrike for each layer is presented in Table 5 and compared to the proposed ranges by CFEM (2006)[39].

The data in Fig. 11 seem to be somewhat scattered. Two reasons may be stated: (1) very diversified geology and stratification of the region; (2) wide time period of restrike tests that have beenas early as 30 minutes and as long as 450 days.

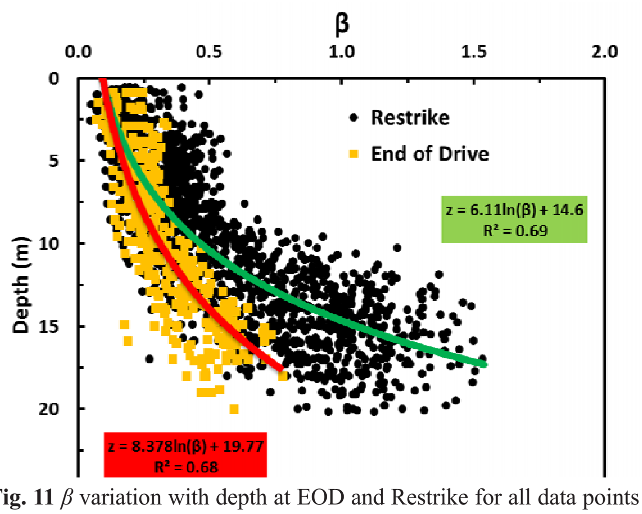


Fig. 11 β variation with depth at EOD and Restrike for all data points

Table 5 Proposed range of β for study area and comparison with CFEM (2006) [36]

Soil	Proposed β	CFEM β
Soft Clay	0.2 - 0.4	0.25-0.32
Medium Stiff to Stiff Clay	0.5 - 0.8	0.25-0.32
Medium Sand	0.8 - 1.2	0.6 - 1

6. Conclusions

Signal matching analysis on PDA tests performed on "test piles" and "construction piles" at EOD and BOR conditions of a case study, made a good database of shaft capacity variations. The setup factor (A) and the reference time (t_0) were evaluated through back-calculating of the pile shaft capacities. Reliability of DLTs was verified and the influence of driving the surrounding piles on setup factor was presented and compared to those reported in literature.

The following conclusions are made out of the presented results:

• The average setup factor (A) and the reference time (t_0) are estimated 0.30 and 0.01 day, respectively, form back-calculating of the case study database.

• Comparison between SLT and DLT results at first has shown that capacity obtained from SLT was greater than the dynamic test results, but with compensating the time differences of performing SLTs and DLTs, a much better correlation was obtained.

• Driving the 8 surrounding piles for RAW-B Tank, increased the shaft capacity equivalent to 19% in one month and 22% in one year, compared to tested piles with no surrounding piles driven.

• Evaluating the results of 221 piles tested in 12 tanks of Fajr II indicated that driving 8 piles around the test pile at testing time results in average 40% increase in the "A" factor.

• Interpretation of results on influence of surrounding piles is in agreement with the limited data reported in literature.

• The EOD and BOR dynamic test (PDA) results have shown 28% increase in β-value in the upper soft clay, 91% in the medium stiff to stiff mid layer and 98% medium dense to dense sand, within which the pile tip is embedded.

• The comparison between proposed ranges of β with CFEM (2006) [39] showed that β is 25% higher for medium sand of Mahshahr. The proposed range of β for clay in CFEM compiles with the upper soft clay in the study site, but the medium stiff to stiff layer has indicated higher values.

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