Effect of preloading on soil setup effects of a soft clay through pile static and dynamic load tests

Kh. Behroozian[i] & K. Fakharian[ii]

i. PhD Student, Department of Civil & Environmental Engineering, Amirkabir University of Technology, Tehran, Iran
khalil.behroozian@yahoo.com

ii. Associate Professor, Department of Civil & Environmental Engineering, Amirkabir University of Technology, Tehran, Iran
kfakhari@aut.ac.ir , kfakhari@yahoo.com

ABSTRACT

Soil setup effect is a natural phenomenon where pile bearing capacity increases over time as a result of dissipation of excess pore water pressure as well as soil aging. Soil setup significantly contributes to the increase in shaft bearing capacity of prefabricated displacement piles installed in saturated clay. The past experiences indicate that shaft bearing capacity increases almost linearly with the logarithm of time elapsed after the pile initial drive, usually quantified using a dimensionless setup factor. The magnitude of setup factor is governed by pile geometry and type of surrounding soil. To quantify the setup rate potential, a minimum of two field measurements of the pile ultimate load are required. Whenever signal matching analyses are performed on pile dynamic load test results at end-of-drive and restrike data, the variations of setup effects along the pile shaft can be determined.

The main objective of this paper is to evaluate the effects of over-consolidation ratio (OCR) on setup factor using pile dynamic load tests (DLT) at end of initial drive and restrike as well as static load tests (SLT). For this purpose, an elaborate field testing program was conducted at Bidboland II Gas Refinery site. The program consisted of DLT and SLT on the prestressed spun concrete piles that have been driven at different locations of the site having variations of OCR as a result of preloading under different surcharge pressures. Results indicate that shaft resistance has considerably increased over time at the study site. The setup factor is affected by the over-consolidation ratio in such a way that with increasing OCR, the setup potential has reduced. The findings of the study are useful in engineering applications of piling in clayey deposits in which the time constraints of construction does not allow performance of dynamic load tests at different time steps as well as estimating the setup factor at sites having over-consolidated soil.

Keywords: Clayey deposits, OCR, soil setup, pile bearing capacity, dynamic load test, and static load test

1 INTRODUCTION

Precast and prestressed concrete piles as foundation of onshore, near shore and offshore structures have been widely used. The most significant and challenging aspect of pile design is proper estimation of pile ultimate resistance. Pile installation results in generation of excess pore water pressure (EPWP) and hence alternation of stress state in soil surrounding the pile shaft (e.g., Randolph and Wroth, 1981; Dijkstra *et al.*, 2011; Fakharian and Khanmohammadi, 2022). Pile bearing capacity is correlated with effective stress state and therefore, variations of PWP in soil surrounding the pile shaft is an important factor (O'Neill, 2001; Fellenius, 2008; Khanmohammadi and Fakharian, 2017). Therefore, eventual dissipation of EPWP results in increase of effective stress and hence pile resistance over time (Skov and Svinkin, 2000). This mechanism has been reported as soil setup in different types of cohesive and non-cohesive soils (Roy *et al.*, 1981; Hosseinzadeh Attar and Fakharian, 2013; Abu-Farsak *et al.*, 2015).

Other than contribution of dissipation of EPWP to soil setup, there is a second factor which is not dependent on PWP. This second mechanism is combination of several factors such as creep, thixotoropy, interlocking, ect., referred to as “aging” in literature. Therefore, the soil setup effects on piles are classified as: (1) generation of EPWP during pile penetration and subsequent dissipation over time, and (2) aging. Attempts have been made to quantify the contribution of each of the two components (Haddad *et al.,* 2012; Fakharian *et al,*. 2013; Fakharian and Khanmohammadi, 2022). The results agree that contribution of aging is generally less than about 15 percent and require longer periods of time to occur. In engineering practice, the first factor correlated with dissipation of EPWP is mostly considered in setup calculations. Pile static load tests (SLT) and dynamic load tests (DLT) are carried out to evaluate and quantify the soil setup effects on the pile bearing capacity (e.g. Skov and Denver, 1988; Fellenius *et al.*, 2004; Lee *et al.*, 2010). The load tests during construction are normally carried out at short time periods within hours to days after the pile installation as the pile execution cannot be delayed for longer times. During test pile study however, longer time periods in the order of weeks to several months could be planned, on the basis of which correlations are developed to enable predicting the soil setup effects on the pile bearing capacity for longer times, during service. Examples of correlations of setup prediction are proposed by Skov and Denver (1988), Long *et al.* (1999), Karlsrud *et al.* (2005) and Khan and Decapite (2011). The proposed correlations were developed on the basis of specific site data and hence calibrations are necessary for reliable predictions in other geological formations. Among the proposed correlations, Skov and Denver (1988) has been widely used in engineering practice either for total or skin frictional resistances. Many studies including full-scale tests, physical models and numerical simulations have revealed that mostly skin frictional resistance benefits from soil setup and effects on tip resistance are not significant. Skov and Denver correlation for skin frictional resistance is in the form of equation 1.

$$Q_s/Q_{s\text{-}t0} = 1 + A\,[\log (t/t_0)] \tag{1}$$

in which Q_s is skin friction at variable time t from end-of-drive (EOD), $Q_{s\text{-}to}$ is skin friction at the reference time t_0, and A is an empirical parameter related to soil layers type indicating the potential rate of soil setup. Reference time t_0 is considered the time when increase in pile resistance shifts from nonlinear to linear in log-scale time. The magnitudes of A and t_0 depend on a number of parameters including soil type (sand, silt, clay), permeability, pile dimensions, installation method (driven, vibrated, jacked-in).

The main objective of this manuscript is evaluating the effect of OCR and pile diameter on the basis of data available from a study site consisting of soft to medium stiff cohesive layers. The specific characteristic of the selected study site is execution of an initial preloading plan at circular locations of six liquid tanks under variable surcharge pressures for seven years. In fact preloading was initially adopted as a remedial measure of the tank foundations. However, the operation was stopped for several years while the locations were backfilled as high as 17 m and as low as only 2 m. After removal of the backfills, an extensive test pile study was performed comprised of 28 spun piles driven at different locations and DLT/SLT tests were carried out. Piles were having diameters of 450 and 600 mm and driven at center point and peripherals of the tanks preloaded under different surcharges and hence developing variable OCR magnitudes. Several piles were driven and tested out of the location of tanks to verify the *in situ* condition of the site. Geotechnical boreholes were drilled at center points of tanks and outside to evaluate the effects of pre-loading on OCR and other soil parameters, after removal of backfills.

The piles were DLT tested at different time intervals up to 60 days after EOD to evaluate the effect of OCR, pile diameter and overall site geotechnical conditions on setup parameters as wells load-movement response of the piles.

2 PROJECT DESCRIPTION

The study site is a gas refinery plant located in southwest Iran. Total of six cylindrical steel tanks including four double walls and two single walls were executed to contain gas and liquid fuels. The tank diameters are in the range of 38 to 50 m having heights of 20 to 27 m. The plan view of tank locations, dimensions and backfill geometry of each of the eight tanks are shown in Fig. 1 and the inset table in the figure.

Storage	Storage Tank Dimentions		Backfill Geometries		
	Diameter (m)	Height (m)	Bottom Diameter (m)	Top Diameter (m)	Height (m)
C3A	50	27	58.6	43.8	8.25
C3B	50	27	59.1	40.3	11.3
C4A	38	24	50.85	45.85	2.4
C4B	38	24	51.25	26.15	8.32
C5A	45.82	20	49.5	30.45	12.2
C5B	45.82	20	50.5	26.7	17.05

Fig. 1. Plan view of preloaded zones, dimensions of tanks and backfill geometries.

The incomplete heights as well as top and bottom diameters of truncated cone backfills are also listed in the table. Location of Tank C4A had the lowest backfill height equal to 2.4 m while the backfill diameters at the bottom and top, respectively, were 50.85 and 45.25 m. Backfill of Tank C5B on the other hand, was the highest equivalent to 17.05 m with bottom and top diameters of 50.5 and 26.7 m, respectively.

The truncated cone backfills remained at each tank position for a period of 5 years between 2011 through 2016 as execution had halted due to financial constraints. Although no drainage wells or fabric drains had been anticipated and installed, the time period of 5 years was sufficiently long for completing the consolidation process and dissipation of excess PWP. Diversity of backfill heights was a recipe for availability of variable OCR magnitudes at each tank location providing an attractive site for evaluating effect of pre-consolidation on soil setup potential at full scale. An aerial photo of the six tanks when backfills were present is shown in Fig. 1. The backfills were entirely removed in 2016 before starting the extensive test pile study at the location of each tank and outside on the natural (intrinsic) soil deposits.

3 GEOTECHNICAL CHARACTERISTICS

Geotechnical investigation was performed both before and after backfill operations for preloading purposes. Five boreholes were drilled at the center of each tank after removal of backfills, three of which were adjacent to the boreholes drilled before backfilling. SPT N-value profiles are therefore available for before backfilling and after removal. On the other hand, carefully supervised triaxial and oedometer tests were carried out on undisturbed samples from both natural and preloaded zones. The physical and mechanical parameters of different soil layers at both times are summarized in Table 1.

The top 1 to 3 m is a coarse-grained backfill (Layer A in the table). The soil deposits dominantly consist of very soft to soft marine clay with groundwater table between 0.9 to 2.3 m across the site. A soft silty clay in brown and sometimes green color extends to about 17 to 20 m deep (Layer B). This fine-grained cohesive layer is classified as CL and CL-ML in unified classification system and identified as soft to stiff under natural conditions and stiff to very stiff at preloaded zones. Underneath this cohesive layer is a frictional silty sand to sandy silt layer having variable thicknesses of 2 to 6 m of depths between 17 to 23 m (Layer C). This layer is identified as medium dense to dense under natural conditions and dense to very dense after preloading. Though identified as frictional, this layer has indicated reduction in void ratio under the surcharge pressure as it is dominantly composed of silt. Below 25 to 27 m is a very stiff to hard cohesive silty clay to clayey silt classified as CL and CL-ML (Layer D). The layers of interest in this study are B and C which are cohesive fined-grained.

It is worthwhile noting that unconfined cohesion (c_u) has almost doubled after preloading in both layers, i.e. increasing from “12 to 20 kPa” to “25 to 60 kPa” in layer B and “30 to 50 kPa” to “80 to 100 kPa” in layer D. Compression Index (C_c) does not show a remarkable variation as it refers to normally consolidation compression state which is not expected to change significantly with preloading. The magnitude of Over-Consolidation Ratio (OCR) has changed considerably in layer B (from “1 to 1.4” to “1.7 to 3.9”), and a small increase in the deeper layer D. The plots of OCR for different backfill heights shown in Fig. 3 reflect the correlation of the OCR increase with backfill height.

Table 1. Soil characteristics prior and after preloading.

Parameters	Unit	Natural Soil (Prior to Preloading)				After Preloading			
		Layer (A)	Layer (B)	Layer (C)	Layer (D)	Layer (A)	Layer (B)	Layer (C)	Layer (D)
Dry Density	kN/m^3	19-21	14-16	16-17	15-17	19-21	16-17	16-17	16.5-17.5
Moisture	%	5-7	16-24	19-25	19.5-25.5	5-7	16-21	17.5-26.5	17.5-26.5
Undrained Cohesion (c_u)	kPa	___	12-20	___	35-50	___	25-65	___	80-100
Drained Cohesion (c’)	kPa	5-8	8-12	3-5	10-15	5-8	10-15	3-5	15-20
Drained Friction Angle (Φ’)	Degree	28-32	17-21	24-28	21-23	28-32	21-24	24-28	22-26
Compression Index (C_C)	-		0.2-0.35		0.16-0.2	___	0.18-0.36	___	0.19-0.24
Swell Index (C_S)	-		0.02-0.03		0.015-0.02	___	0.01-0.04	___	0.02-0.03
Initial Void Ratio (e_0)	-		0.8-1.17		0.6-0.65	___	0.7-0.87	___	0.7-0.8
OCR	-		1.0-1.4		1	___	1.7-3.9	___	1.1-1.3

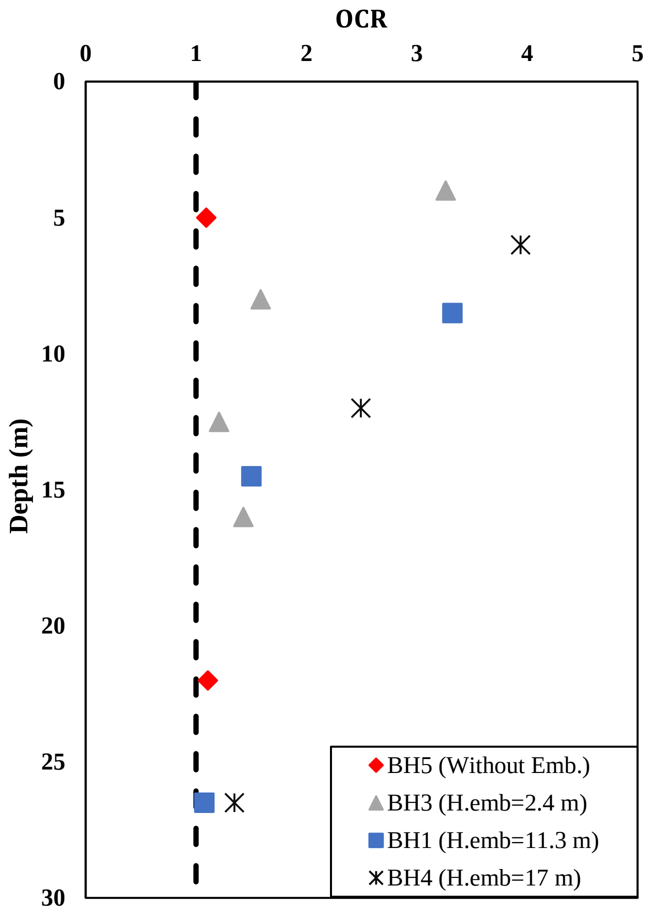

Fig. 2. Effect of surcharge height on OCR at different tank positions

4 TEST PILE STUDY OUTLINE

An extensive test pile study was planned and executed in 2016 on 28 spun piles onto which 159 dynamic load test (DLT) and 6 static load test (SLT) were carried out at different time intervals using pile driving analyzer (PDA). The piles are prstressed spun and closed-toe with two external diameters of 450 and 600 mm. Pile segments were supplied in 9 and 12 m lengths spliced by welding to reach the desired embedment depth in the range of 21 to 23 m. A single-acting diesel hammer (Delmag 46-32) having a ram of 46 kN with maximum deliverable energy of 166 kJ per blow was adopted for pile driving and DLT tests.

The DLT tests on all the 28 piles were carried out at the End-Of-Drive (EOD) and Beginning-Of-Restrike (BOR) from Feb. 24, 2016 through March 30, 2016. Signal matching analysis (using CAPWAP) was performed on 117 DLT results through which tip and skin friction resistances were distinguished. Moreover, 6 SLT tests including 3 axial compressive and 3 axial tensile were carried out according to ASTM-D-1143 and ASTM-D-3684, respectively. Compressive tests were carried out on test piles C3A-TP5, C4A-TP5 and C5A-TP5, respectively after 43, 60 and 48 days from EOD. The compressive testing system was topdown making use of 4 surrounding piles to provide reaction. Tensile tests were carried out on test piles C3A-TP3 C4A-TP3 and C5A-TP1, respectively after 61, 67 and 50 days from EOD. The tensile test piles had identical diameter and embedment depths compered to compressive test piles at each tank location in order to work out the tip and skin frictional resistances as a cross check with CAPWAP results. A 4,000 kN hydraulic jack and a calibrated load cell with the same capacity were employed to apply and measure the axial loads. Four dial gages having a resolution of 0.01 mm were mounted on reference beams to measure the pile head movement. Loading method of C3A-TP5 and C5A-TP5 was quick load test (QLT) with 20 load steps maintained 15 min. Test pile C4A-TP5 was loaded using maintained load test (MLT) method according to ASTM-D-1143.

5 RESULTS AND INTERPRETATIONS

5.1. Test results

Details of test results of four test piles are presented in Table 2. They include three piles onto which compressive SLT tests were carried out (C3A-TP5, C4A-TP5 and C5A-TP5) and Unit 537 at free zone with no preloading. The DLT results include skin friction, tip resistance, the blow number selected for interpretation, penetration per blow (set) and maximum delivered energy (EMX) for each time the test was carried out. Last row of each test pile corresponds to the SLT results. The load-movement graphs of compressive and tensile SLT tests are shown in Fig. 3. Ultimate resistances in Table 3 are derived employing Davisson Offset Limit (DOL). The tip resistances are obtained from subtraction of the skin friction resistance from tensile tests out of the total resistances. An assumption is made that skin frictional resistances are identical in both compressive and tensile tests. Clarke *et al.* (1993) showed that skin friction from compressive and tensile tests have negligibly small differences in over-consolidated stiff clays. Figure 3 shows clearly how the pile diameter and surcharge pressure of preloading have influenced the pile load-movement responses as well as the ultimate loads. It is worthwhile noting that all the six test piles are having almost the same embedment depths of 23.1 to 23.5 m. Test piles C5A-TP5 and C4A-TP5 are both 450 mm diameter, but the ultimate load of the pile at C5A, having experienced a surcharge of 12.2 m, was obtained 2,850 kN with respect to 2,000 N corresponding to pile at C4A having experienced simply a surcharge preloading of 2.4 m. Similar differences can be observed in tensile test results of Fig. 5 at the location of the same two tanks. The 600 mm diameter test piles at C3A tank resulted in ultimate loads greater than C5A tank even through its surcharge pressure was smaller, i.e. 8.3 versus 12.2 m, respectively.

Results of Table 3 show significant setup potential in skin frictional resistance over time, compared to EOD results. For example in C4A-TP5 test pile, the skin friction has increased 407% after 27.4 days from EOD in DLT test and 525% after 60 days in SLT test. The surcharge height at this tank is 2.4 m inducing little OCR. In C5A-TP5 test pile located at a zone with 12.2 m surcharge, however, the percentage increase in skin friction has reached 334% and 392% in 20-day DLT and 48-day SLT tests, respectively. Therefore, higher OCR of fine-grained soil could result in smaller setup potential. The effect of magnitude of surcharge pressure which is corresponding to higher OCR is evaluated in more details in subsequent sections.

5.2. Comparison of DLT and SLT

An attempt was made to compare the skin frictional resistances from DLT and tensile SLT. Figure 4a shows comparison results for the three tensile SLT piles at location of three different tanks. An inset table in the figure shows the test pile numbers and the number of days after EOD each test was carried out, indicating longer times of SLT tests (61, 67 and 50 days) compared to the corresponding DLT test on the same piles (13, 27 and 20 days), respectively. Lower frictional resistances are obtained from CAPWAP analyses on the basis of DLT tests compared to SLT tensile results. Attempt was made to harmonize the results through equalization of the time after EOD.

Table 2. The results of DLT and SLT tests on four piles.

		Shaft Resistance		Toe Resistance		Total Resistance				
Event	Time	kN	Percent Increase	kN	Percent Increase	kN	Percent Increase	Blows	Penetration (mm)	EMX (kN-m)
C3A-TP5										
EOID	Initial	380	----	1,381	----	1,760	----	----	----	38.5
BOR1	0.75 Day	889	134	1,247	-10	2,136	21	3	41	52.3
BOR2	4.7 Day	1,275	236	988	-28	2,263	29	5	22	38.5
BOR3	25 Day	1,663	338	1,188	-14	2,851	62	6	12	71.4
SALT	43 Day	2,000	427	1,428	3	3,428	95	----	----	----
C4A-TP5										
EOID	Initial	224	----	406	----	630	----	----	----	44
BOR1	3.1 Day	765	242	570	40	1,335	112	33	300	42.4
BOR2	4.15 Day	807	260	755	86	1,562	148	4	24	45.3
BOR3	5 Day	870	288	626	54	1,496	137	2	55	54.6
BOR4	11.3 Day	1,020	355	591	46	1,611	156	7	17	63.9
BOR5	27.4 Day	1,135	407	626	54	1,761	180	3	22	48.7
SALT	60 Day	1,400	525	600	48	2,000	217	----	----	----
C5A-TP5										
EOID	Initial	321	----	888	----	1,209	----	----	----	43.1
BOR1	1 Day	827	158	1,145	29	1,972	63	4	19	47.5
BOR2	8 Day	1,197	273	890	0	2,087	73	4	20	27.2
BOR3	20 Day	1,394	334	1,087	22	2,481	105	3	9	46.6
SALT	48 Day	1,580	392	1,270	43	2,850	136	----	----	----
Unit 535										
EOID	Initial	347	----	484	----	831	----	----	----	25.9
BOR1	3.9 Day	1,332	284	838	73	2,170	161	3	41	40.9
BOR2	14.9 Day	1,700	390	1,019	111	2,719	227	3	15	64.6

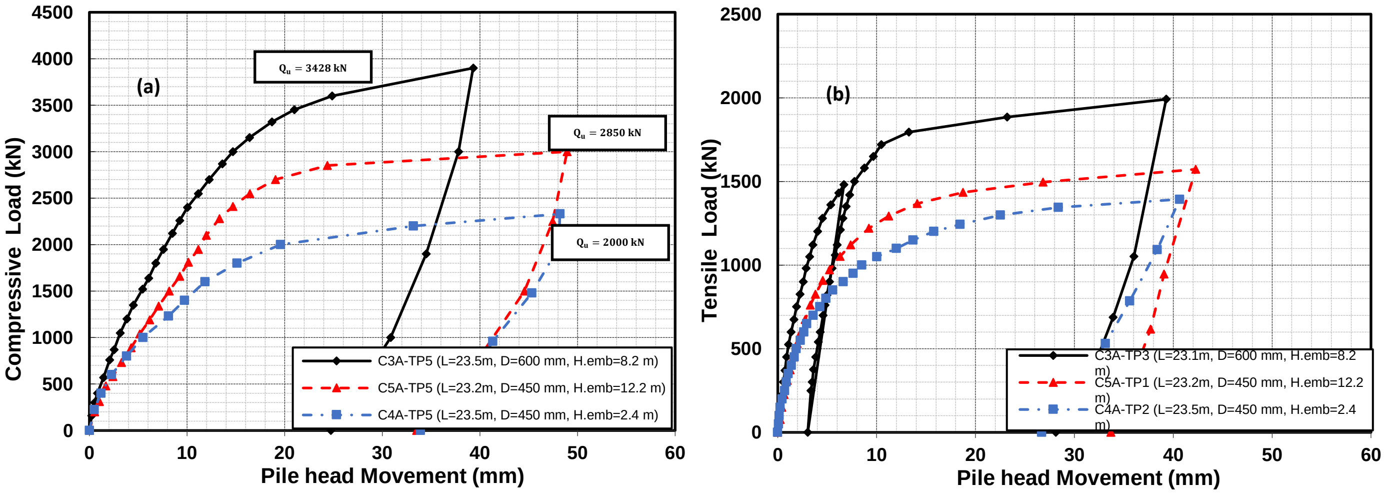

Fig. 3. Results of compressive (a) and tensile (b) static load tests at location of three different tanks having surcharge heights of 2.4, 8.2 and 12.2 m.

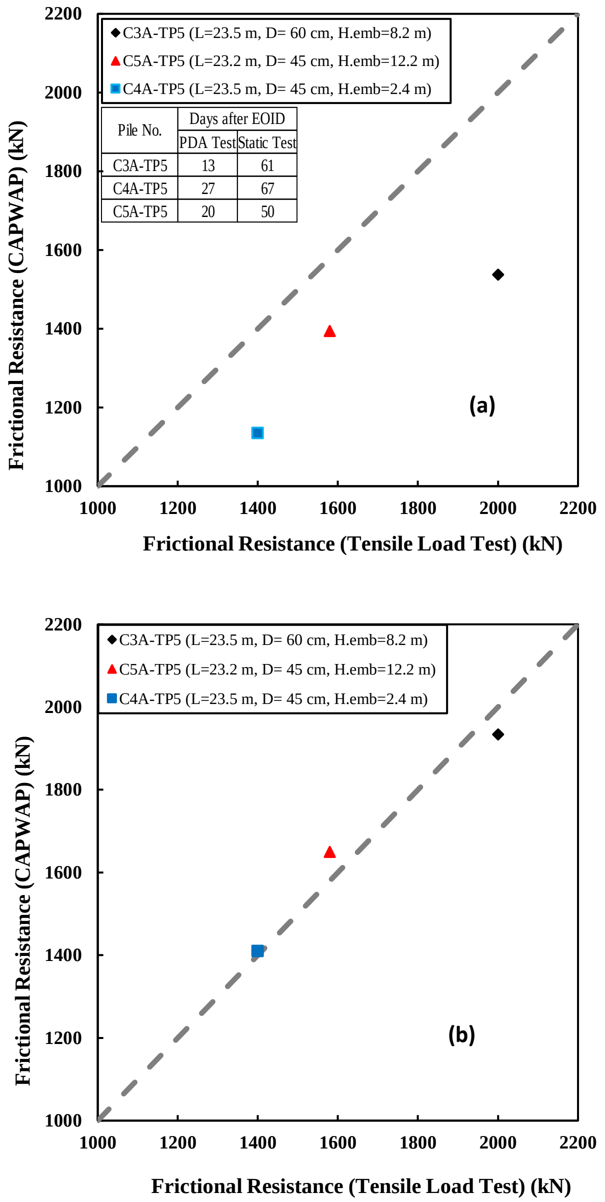

Fig. 4. Comparison of DLT and Tensile SLT for skin friction: (a) different time from EOD, (b) harmonized time from EOD.

Similar to the procedure by Fakharian *et al.* (2012), *A* and t_0 parameters of Eq. 1 (Skov and Denver) were determined at each tank location making use of the data from all the test piles. The reference time t_0 best estimate was 1 day which is reasonable in thick cohesive strata. The "*A*" magnitude at each tank location was also determined as presented in the next section. The DLT results of the three piles in Fig. 4a were corrected for the equivalent time of tensile SLT tests and plotted again in Fig. 4b. The agreement between DLT and SLT results has significantly improved after the compensation for the testing time differences.

5.3. Soil setup evaluation

According to various surcharge heights during the preloading operation, the site is divided into four zones: (I) free zone, i.e. no preloading and therefore zero surcharge, (II) tank C4A with 2.4 m surcharge, (III) tanks C3A and C4B with about 8 m surcharge, and (IV) tanks C5A, C5B and C3B with 11 to 17 m of surcharge. Table 2 presents the tip, shaft and total resistances on selective test piles of each zone at different time intervals. The percentage increase of each load with respect to EOD are mentioned in the table. It is noticed that skin frictional resistances have increased significantly over time, with the highest amount equivalent to 427% after 43 days from EOD. The tip resistance increases are much smaller and vary from 3 to 54% except at the free zone (I) that has shown to be 111%. The penetration per blow (set) magnitudes as well as EMX magnitudes in the table indicate that both tip and skin friction resistances have been fully mobilized.

Attempts are made to quantify the potential rate of setup for skin friction using Eq.1 of Skov and Denver (1988). The average "A" parameter at zones I, II, III and IV are respectively, 0.75, 0.65, 0.6 and 0.55. It can be conducted that the site is having a high setup potential, but the potential reduces with surcharge pressure of preloading. In other words, the setup potential has shown to be inversely proportional with OCR.

6 CONCLUSIONS

Dynamic and static load tests were performed on test piles at the Bidboland II Gas refinery site in Mahshahr region, Khuzestan province, Iran. By performing signal matching analysis on the DLT test results, toe and shaft resistances were calculated at EOD and BOR conditions. Also, signal matching analysis was validated using the results of tensile load tests. Based on the commonly used relation of Skov and Denver (1988), two parameters of "*A*" and reference time, "t_0" were calculated. The most important results of the present study can be summarized as follows:

- Taking into account the time elapse since the pile driving, the results of DLT tests and signal matching analysis in calculating both frictional and tip resistance have shown a good agreement with the results of SLT tests, in particular when the effect of soil setup is applied to DLT predictions.
- Tip resistance generally shows little increases over time which is in agreement with other studies in literature.
- A good correlation was observed between the embankment height and the magnitude of pre-consolidation after removal of the surcharge. The magnitude of pre-consolidation ratio considerably affects the setup potential on skin frictional resistance.

7 ACKNOWLEDGMENTS

The authors would like to thank the contributors to this study. Special thanks are due to EPC contractor

Tehran Jonob Inc. for financing the extensive test pile study. Pars GeoEnviro Inc. is gratefully acknowledged for providing the test data. Authors would like to acknowledge the sincere efforts of Dr. I.H. Attar for performing the CAPWAP analyses of the data from DLT tests.

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