Settlement Analysis of Axially Loaded Bored Piles: A Case History

M. Mert, M. T. Ozkan

***Abstract*—**Pile load tests should be applied to check the bearing capacity calculations and to determine the settlement of the pile corresponding to test load. Strain gauges can be installed into pile in order to determine the shaft resistance of the piles for every soil layer respectively. Detailed results can be obtained by means of strain gauges placed at certain levels into test piles. In the scope of this study, pile load test data obtained from two different projects are examined. Instrumented static pile load tests were applied on totally 7 test bored piles of different diameters (80 cm, 150 cm, and 200 cm) and different lengths (between 30-76 m) in two different project site. Settlement analysis of test piles is done by using some of load transfer methods and finite element method. Plaxis 3D which is a three-dimensional finite element program is also used for settlement analysis of the test piles. In this study, firstly bearing capacity of test piles are determined and compared with strain gauge data which is required for settlement analysis. Then, settlement values of the test piles are estimated by using load transfer methods developed in recent years and finite element method. The aim of this study is to show similarities and differences between the results obtained from settlement analysis methods and instrumented pile load tests.

***Keywords*—**Failure, finite element method, monitoring and instrumentation, pile, settlement.

I. Introduction

A specific number of load tests must be conducted on piles in most large projects. By conducting full scale tests on piles, the design of piled foundation can be done more economical and safer. Pile testing is also useful for determining settlement of single piles. Generally, only bearing capacity calculations are done for the design of piled foundation. However, pile settlement calculations also should be in considered to design piles safer and more economical. In every design of piled foundation, settlement analysis of piles must be evaluated [1].

Reference [7] firstly studied about load transfer method for analysis of pile settlement. If test piles are instrumented with strain gauges during the installation, the load-transfer mechanism of the piles can be modeled properly. Distribution of the shaft and base resistance during the loading process can be estimated by means of instrumentation of test piles [2]. References [3], [4], [13] and [14] studied with instrumented load tests for validation of their methods and they studied with layered soils in their investigations. Settlement analysis methods studied by [3], [8] and [9] are based on hyperbolic functions to describe individual shaft and base performance. Reference [12] presented a variational model for the settlement analysis of axially loaded piers, but this model is also valid for bored piles.

This paper reports a case history on the investigation of the pile load tests carried out for the Iskenderun Power Plant Project and Baku Hotel Project. The aim of this study is to compare the test results of the load-pile settlement curves to those evaluated by using four different load transfer methods and finite element method in the scope of Iskenderun Power Plant Project and Baku Hotel Project.

II. Geology And Soil Profile

A. Iskenderun Power Plant Project

Investigation area is located in the east coast of Iskenderun Bay where young alluvial deposit is found. The thickness of the alluvial deposits is maximum 41.5 m according to the boreholes applied in the site. There are older alluvial deposits under these young alluvial deposits as well.

The area of the power plant site is about 110000 m^2 and totally 99 boreholes are executed. The soil displays large variation in this huge area, however, generally the soil consists of the alluvial deposits which alternated with sandy-clayey-gravelly soil.

Standard penetration tests (SPT) and pressure meter tests (PMT) were conducted on different locations of the project site. Elasticity modulus of the different soil layers was evaluated from the graphs of pressure-volume change as by following usual practices.

Four test piles (TP1, TP2, TP3 and TP4) were constructed at the three different locations. The uppermost layer consists of an about 2.5 m thick fill layer and this layer was excavated from the site before installation of the test piles. In general, the natural soils consist of medium dense clayey gravelly sand, medium sandy clay, medium dense sandy gravel, and the lowermost layer consist of very stiff gravelly sandy clay. The three soil profiles and the soil parameters are listed in Table I.

B. Baku Hotel Project

The area of hotel foundation is approximately 19000 m^2 and it is not so huge as the investigation area of Iskenderun Power Plant Project. Therefore, only one idealized soil profile for three test piles was described (Table II). The soil profile of the site is quite homogenous, and the thickness of soil layers is not so variable. SPT were carried out in all boreholes at intervals of 1.5 m to 2 m. Pocket penetrometer tests were carried out in selected boreholes. Standard laboratory tests were also carried

M. Mert is MSc. In Geotechnical Engineering, Faculty of Engineering, Fatih Sultan Mehmet Vakif University, Istanbul, Turkey (corresponding author, phone: +90 530 5481122; fax: +90 212 3698164; e-mail: mmert@fsm.edu.tr).

M. T. Ozkan, is Associated Professor, Faculty of Civil Engineering, Istanbul Technical University, Istanbul, Turkey (e-mail: ozkantu@itu.edu.tr).

out to the samples taken from the project site. Elasticity modulus of soil layers were evaluated by considering both SPT values and the laboratory tests such as unconsolidated undrained (UU) triaxial tests and oedometer tests. As it can be seen in Table II, uppermost layer consists of an about 4 m thick fill layer and layer of clayey sand underneath it. Under the depth of 12 m, the soil consists of about 10 m thick stiff clay layer and 20 m thick interbedded sand and clay layer, respectively. The lowermost layer consists of plastic hard clay.

TABLE I
IDEALIZED SOIL PROFILE AND PARAMETERS (ISKENDERUN)

	Thickness of layer (m)	Cohesion (c_u) (kN/m^2)	Angle of Shear Strength (ϕ) (°)	Modulus of Elasticity (E) (kN/m^2)	Poisson's ratio (υ)
			TP1		
Medium Dense Sand	6.00	0	32	25000	0.3
Firm Gravelly Sandy Clay	21.50	60	0	20000	0.35
Very Stiff Clay	-	150	0	80000	0.3
			TP2		
Medium Gravelly Clay	24.50	40	0	15000	0.4
Medium Dense Sandy Gravel	3.00	0	34	35000	0.35
Very Stiff Clay	-	150	0	80000	0.3
			TP3-4		
Medium Gravelly Clay	3.00	50	0	15000	0.4
Medium Dense Sand	3.00	0	32	20000	0.3
Very Stiff Sandy Clay	10.5	100	0	30000	0.3
Medium Dense Sandy Gravel	4.50	0	34	35000	0.35
Medium Sandy Clay	9.00	50	0	20000	0.4
Very Stiff Clay	-	150	0	80000	0.3

TABLE II
IDEALIZED SOIL PROFILE AND PARAMETERS (BAKU)

	Thickness of layer (m)	Cohesion (c_u) (kN/m^2)	Angle of Shear Strength (ϕ) (°)	Modulus of Elasticity (E) (kN/m^2)	Poisson's ratio (υ)
Fill	4.00	0	32	10000	0.35
Clayey Sand	8.00	60	0	60000	0.37
Stiff Clay	10.00	170	0	75000	0.3
Interbedded Sand/Clay	20.00	130	0	75000	0.35
Plastic Hard Clay	-	210	0	90000	0.35

III. TESTING PROGRAM

A. Iskenderun Power Plant Project

In the scope of İskenderun Power Plant Project, test piles with Φ800 mm diameter were installed to depth of 30, 35, 40 and 45 m long at three different locations. This test covers four test piles and 16 reaction piles. Axial compressive Design Verification Load (DVL) of test piles was evaluated as 3000 kN by using pile bearing capacity equations. Instrumented load test systems were set up to 12000 kN as a maximum test load. Embedment type Vibrating Wire-Gauges (VWSG) are positioned on vertical reinforcement for measuring the strains at the levels. There are eight level strain gauges at 30 m long pile (TP1), 9 level strain gauges for 35 m long pile (TP2) and 11 level strain gauges for 40 m long pile (TP3) with vertical spacing of 4 m. TP4 pile was not instrumented with strain gauges. TP4 pile was installed at a distance of 30 meters from the TP3 pile. Therefore, the shaft friction and base resistance values obtained by strain gauges for TP-3 pile were assumed as similar with the shaft friction and base resistance values for TP-4 pile. The features of test piles are summarized in Table III.

The loading and unloading were carried out according to ASTM D1143-07 "Standard Test Methods for Deep Foundations under Static Axial Compressive Load" item 8.1.3 "Procedure B: Maintained Test". The loading was carried out with equal levels. The first load level and the increment of each level is 25% DVL (750 kN) of test piles. The unloading level is twice the increment of each loading level. Load tests were carried out with three cycles.

TABLE III
FEATURES OF TEST PILES (ISKENDERUN)

Test Pile No.	Diameter (mm)	Pile Length (m)	Design Verification Load (kN)	Maximum Test Load (kN)
TP-1	Φ800	30	3000	11250
TP-2	Φ800	35	3000	8250
TP-3	Φ800	40	3000	9000
TP-4	Φ800	45	3000	10500

B. Baku Hotel Project

In the scope of Baku Hotel Project, project piles have Φ1500-2000 mm diameter and 56-76 m length. Design verification load of the piles were calculated as 13000 kN and 14000 kN depends on the diameter and length of the bored piles. Pile load tests were carried out on three project piles which were chosen as test piles from different location of site. The project piles located around the chosen test piles were used as reaction piles. Therefore, test load applied to test piles was not much greater than the design load of test piles. The features of test piles are summarized in Table IV.

Pile load tests were performed in accordance with ASTM D1143-07. Load tests were carried out with two cycles.

TABLE IV
FEATURES OF TEST PILES (BAKU)

Test Pile No.	Diameter (mm)	Pile Length (m)	Design Verification Load (kN)	Maximum Test Load (kN)
BTP-1	Φ2000	59	13000	19000
BTP-2	Φ2000	67	14000	22000
BTP-3	Φ2000	76	14000	22000

Embedment type Vibrating Wire-Gauges (VWSG) were used for measuring the strains in all test piles. They were positioned on vertical reinforcement with interval of 7 m in average.

IV. Test Results

A. Iskenderun Power Plant Project

The load- settlement curves of test piles can be seen in Fig. 1. Maximum load of 11250 kN was applied to the TP-1 as it can be seen at Fig. 2 (a). When TP-1 was at the maximum load, 64.06 mm was measured as a maximum settlement. Maximum load of 9000 kN was applied to TP-3 which did not fail at maximum load. When TP-3 was at the maximum load, 14.52 mm was measured as a maximum head movement. TP-2 and TP-4 piles failed when the applied load reached at 8250 kN and 10500 kN, respectively.

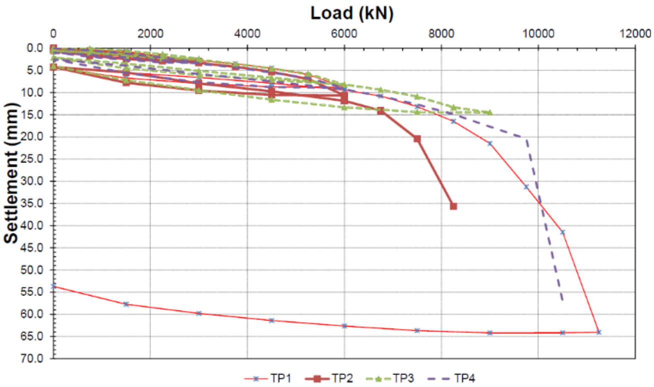
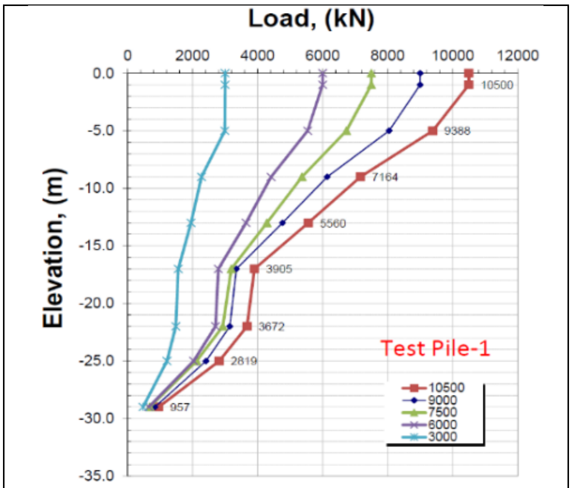


Fig. 1 Load-settlement curves obtained from pile load tests (Iskenderun)

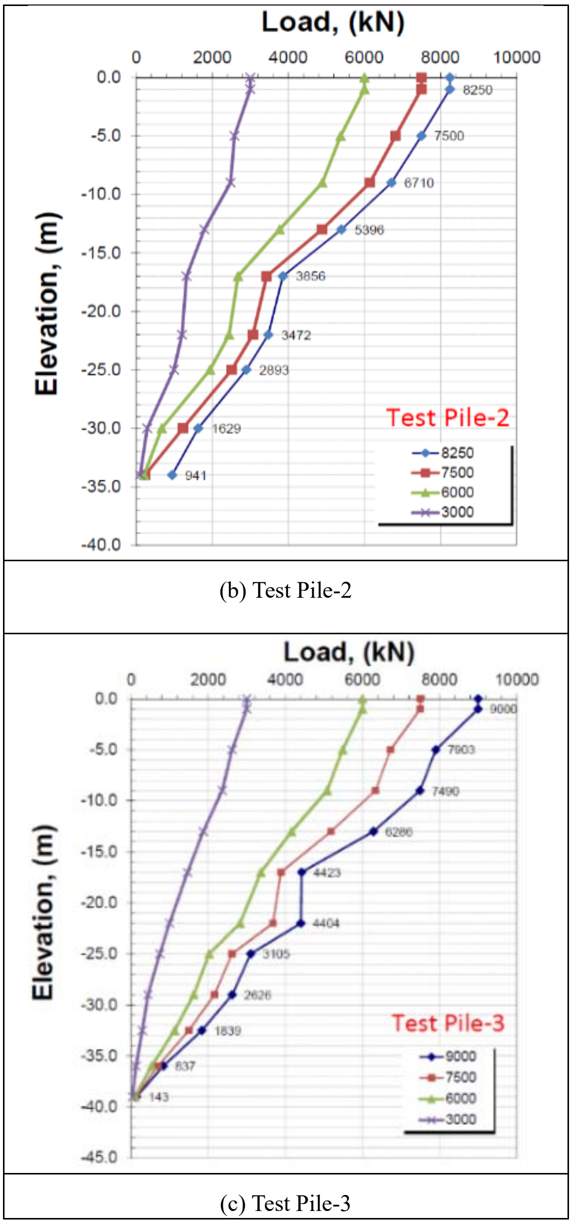
In order to convert the micro-strains obtained from the strain gauges (SG) to stress values (σ, kN/m²), the modulus of the pile at the SG level should be determined. The pile modulus at any gauge location would be affected by local conditions of concrete and reinforcement and is very difficult to determine. Therefore, the strain recorded by uppermost gauge, SG 1, was matched to applied load. As a result of that, one microstrain calibrates of a load of average 20 kN for all instrumented test piles. Estimated load transfer plots along the three instrumented test piles during various loading steps by interpretation of strain gauge readings are presented in Fig. 2. It can be inferred from Fig. 2 that there is a concordance between soil layers and ultimate unit shaft friction values obtained from strain gauges TP-1 and TP-2 piles failed after last loading steps. It can be inferred from the results that the ultimate tip resistance of the pile was estimated as 950 kN which is about 10% of pile total resistance. On the other hand, the ratio of the load affected to pile tip and pile head is evaluated as 0.03-0.10 for lower load steps. The ratio of the load affected to pile tip and pile head which is an important parameter for method developed by [3] is assumed as 0.05 for all load steps and test piles.

Estimated unit shaft friction values along the test piles during maximum loading steps are presented in Fig. 3. Unit friction values are obtained by dividing the pile load differences to each stage by the surface area of the pile. In generally, estimated shaft friction values are suitable with soil layers. The unit shaft friction values of sandy and gravelly layers are estimated less than unit shaft friction values of stiff clayey layers. As it is mentioned before, TP-1 and TP-2 piles were failed after the last strain gauge readings so estimated unit shaft friction values at maximum load steps are assumed as ultimate shaft friction values of soil layers in the calculations. Although TP-3 pile did not fail at 9000 kN which is maximum load step, estimated unit shaft friction values at maximum load step can be assumed as ultimate unit shaft friction values of soil layers. Since there was no significant difference between the shaft friction values estimated from TP-3 pile and the other test piles. The unit shaft friction values of TP-3 pile can also be used for TP-4 pile because the location of TP-4 pile and TP-3 pile are same and TP-4 pile was not instrumented as it is mentioned before.

Ultimate resistance (P_{tu}) of the piles is determined by using the method developed by [5], [6] which is used for pile settlement analysis by [8]. Ultimate resistance (P_{tu}) of the piles obtained from the method developed by [5] and [6] are listed in Table V. These values were used for validating the method of [8]. According to values obtained from strain gauges, base resistance of the piles evaluated as 950 kN.



(a) Test Pile-1



(b) Test Pile-2

(c) Test Pile-3

Fig. 2 Calculated load values at different strain gauge levels under different load steps

TABLE V
ULTIMATE RESISTANCE VALUES (ISKENDERUN)

Pile No	Ultimate Resistance (kN) [5]	Ultimate Base Resistance (kN)	Ultimate Shaft Resistance (kN)
TP-1	12450	950	11500
TP-2	10450	950	9500
TP-3	12800	950	11850
TP-4	12300	950	11350

B. Baku Power Plant Project

Load- settlement curves of chosen test piles can be seen in Fig. 4. As it can be seen in load settlement curves, test piles chosen from project piles were not failed, and the maximum settlement values obtained from pile load tests were very low. Maximum settlement value was obtained as 11.02 mm for BTP-3. Although the test piles were instrumented, it was difficult to predict the ultimate and shaft resistance of piles due to low maximum test load. Nevertheless, distribution of shaft stress along the pile and the ratio of load affected to pile end and pile head can be evaluated by means of strain gauges.

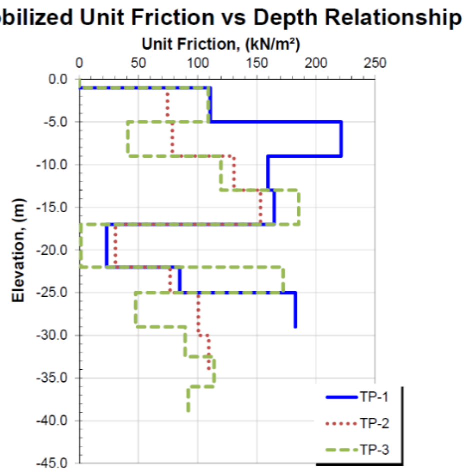


Fig. 3 Calculated unit shaft friction values between consequent strain gauge levels under maximum test load

Osterberg cell test was carried out to a test pile (BOTP) with Φ2000 mm diameter and 76 m length. This test was applied before the installation of project piles in order to calculate ultimate shaft, base and total resistance of the pile. The maximum load value applied from Osterberg cell was 26000 kN. Pile head behavior under load is provided by Cemset analysis which was described by [8]. The maximum top load was evaluated as approximately 50000 kN by using this method. Load-head movement curve predicted from Osterberg cell test considering elastic shortening of the pile is also given in Fig. 4 with other results of pile load tests conducted on project piles. Reason of difference in elastic region between the results obtained from Osterberg cell test and top-loading tests is about the elastic shortening calculations used in Cemset analysis.

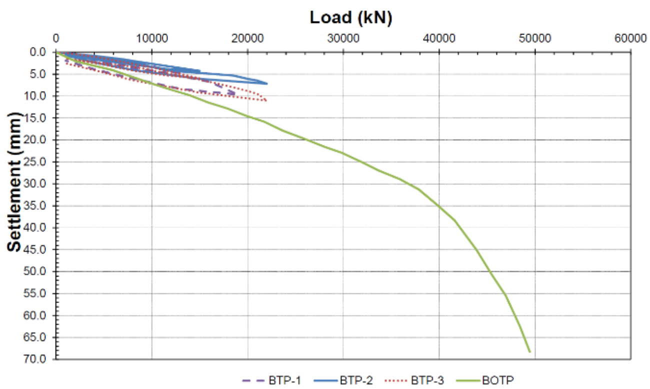


Fig. 4 Load-settlement curves obtained from pile load tests (Baku)

Tested project piles were instrumented as it was mentioned above. Calculations of shaft stress distribution were done in a similar way to those in Iskenderun Project. As a result of evaluations of strain gauge data, one microstrain calibrates of a load of average 25 kN for all instrumented test piles. Estimated unit shaft friction values along the test piles during maximum loading steps are presented in Fig. 5. Ultimate unit shaft resistance values along the test pile were obtained from Osterberg cell test and the results were also given in Fig. 4. Shaft friction distribution obtained from the Osterberg cell test represents the soil profile better than shaft friction distribution obtained from top loading tests conducted on project piles. The reason of that the test load applied to pile head for Osterberg cell test was evaluated much greater than the test load applied to project piles and failure mechanism of the test pile can be modeled properly in Osterberg cell test. Therefore, ultimate unit shaft and unit base resistance values obtained by Osterberg cell test were used for settlement analysis. Ultimate unit base resistance was evaluated as 2700 kPa.

Ultimate shaft, base and total resistance of project piles (BTP-1, BTP-2 and BTP-3) was evaluated by using the results of Osterberg cell test (see in Table VI). Ultimate base and unit shaft resistance and ultimate unit base resistance values evaluated from Osterberg cell test were assumed similar to those for project piles.

TABLE VI
ULTIMATE RESISTANCE VALUES (BAKU)

Pile No	Ultimate Resistance (kN) [5]	Ultimate Base Resistance (kN)	Ultimate Shaft Resistance (kN)
BTP-1	41100	8500	32600
BTP-2	46500	8500	38000
BTP-3	38200	4800	33400

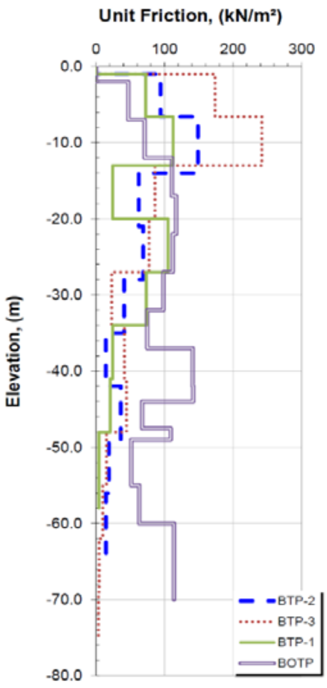


Fig. 5 Calculated unit shaft friction values between consequent strain gauge levels under maximum test load

V. ESTIMATION OF PILE SETTLEMENT

There are several load transfer methods developed by many researchers. In this study, some methods developed by [3], [4], [8] and [14] in recent years were investigated.

Application steps of the methods developed by [4] and [14]. In both methods, pile is divided into n segments and settlement of pile base is assumed as an initial value which is very small. Next, base resistance (τ_b *and* P_b) is calculated by using correlations between base settlement (s_b (mm)) and base resistance (τ_b *and* P_b). These correlations are evaluated differently in both methods [4] and [14] as written in (1) and (2) respectively.

$$P_b = A.(1 - e^{-Bs_b}) \quad [4] \tag{1}$$

$$\tau_b = \frac{s_b}{A + Bs_b} \quad [14] \tag{2}$$

A *and* B are the base parameters related to ultimate base resistance (τ_{bu}) and soil and pile parameters such as shear modulus of soil at pile base (G_b), Poisson's ratio of soil at pile base (ϑ_b) and radius of pile (r_0) described in [4] and [14] differently (see in Table VII). The shaft displacements are then calculated separately for each pile segment along the pile length by using correlations between the shaft resistance (τ_{sz}) at depth z and shaft displacements (s_s) as written in (3) and (4).

$$\tau_{sz} = a.(1 - e^{-bs_s}) \quad [4] \tag{3}$$

$$\tau_{sz} = \frac{(a + c + bs_s) \pm \sqrt{(a + c + bs_s)^2 - 4bcs_s}}{2bc} \quad [14] \tag{4}$$

a, b *and* c are the shaft parameters related to ultimate shaft resistance (τ_{su}) and soil and pile parameters such as shear modulus of soil at pile shaft (G_s), radius of pile (r_0) described in [4] and [14] differently (see in Table VII).

TABLE VII
PARAMETERS USED IN METHODS DEVELOPED BY [4] AND [14]

	[4]	**[14]**
Shaft Parameters	$a = \frac{\tau_{su}}{R}, b = \frac{G_s}{a.r_0 ln\left(\frac{r_m}{r_0}\right)}$	$a = c = \frac{r_0}{G_s} ln\left(\frac{r_m}{r_0}\right), b = \frac{R}{\tau_{su}}$
Base Parameters	$A = \frac{P_{bu}}{R},\ B = \frac{4r_0 G_b}{A(1-\vartheta_b)}$	$A = \frac{\pi r_0(1-\vartheta_b)}{4G_b},\ B = \frac{R}{\tau_{bu}}$

r_m can be calculate as $r_m = 2.5L \frac{\sum_{i=1}^{n_s} G_{si} h_i}{G_{sm} L}\left(1 - \frac{\sum_{i=1}^{n_s} \vartheta_{si} h_i}{L}\right)$ according to [10] for a pile embedded in multilayered soils. The hyperbolic curve fitting constant R can be adopted in the range of 0.80-0.95 [15].

Load transfer methods based on hyperbolic curves developed by [3] and [8] are also investigated in the scope of this study. References [3] and [8] suggest same hyperbolic functions to describe individual shaft and base performance (see in Table VIII).

The main difference between of these methods is the method to evaluate deformation parameters at the shaft (M_s) and at the base (M_b). These parameters were evaluated as constant number in the method of [3] by investigating 50 instrumented pile load tests ($M_s = 0.0038$ and $M_b = 0.01$). According to [8], deformation parameter at shaft depends on stiffness of soil layer around the pile. ($M_s = 0.0005$ for stiff soils to $M_s = 0.004$ for soft soils). Reference [11] also suggested that M_s values can be in the range of 0.001-0.004. The deformation parameter at base (M_b) is related to elasticity modulus of soil at pile base (E_b).

TABLE VIII
HYPERBOLIC FUNCTIONS USED IN METHODS DEVELOPED BY [3] AND [8]

	[3]	[8]
Unit Shaft Friction (τ_{sz})	$\tau_{sz} = \frac{\tau_{su} s_s}{M_s D + s_s}$	$\tau_{sz} = \frac{\tau_{su} \cdot s_s}{M_s D + s_s}$
Unit Base Resistance (τ_b)	$\tau_b = \frac{\tau_{bu} s_b}{M_b D + s_b}$	$\tau_b = \frac{\tau_{bu} \cdot s_b}{0.6 \cdot \pi \cdot \frac{B}{4E_b} \cdot \tau_{bu} + s_b}$

Settlement analysis of bored piles were carried out by using finite element method (FEM) in addition to load transfer methods in order to comparing them with each other. Plaxis 3D is a finite element program that is used worldwide for geotechnical engineering and design. Seven pile load tests are modeled in Plaxis 3D program. Layered soils and can be modeled with finite element method. Pile load tests were carried out with three cycles in Iskenderun Project and two cycles in Baku Project. This situation was modeled by using Plaxis 3D program.

VI. COMPARISONS BETWEEN CALCULATED AND MEASURED SETTLEMENTS OF TEST PILES

Pile settlement analysis for four test piles in İskenderun Project and three test piles in Baku Project was carried out by using five different methods explained in the 5th part. Pile settlement values corresponding to critical load steps are listed in Tables IX and X.

TABLE IX
PILE SETTLEMENT VALUES OBTAINED FROM DIFFERENT METHODS AND PILE LOAD TESTS (ISKENDERUN)

	ITP-1				ITP-2		
Applied Load (kN)	**3000**	**6000**	**9000**	**10500**	**3000**	**6000**	**8250**
[8] (mm)	4.92	11.72	25.85	48.39	5.69	14.99	36.97
FEM (Plaxis 3D) (mm)	5.05	12.21	25.30	32.95	5.18	11.21	16.44
[3] (mm)	3.52	10.56	36.51	75+	3.90	13.32	50+
[14] (mm)	6.03	16.50	44.52	75+	5.32	15.63	43.05
[4] (mm)	5.98	13.49	25.70	39.12	5.74	12.86	21.62
Measured (mm)	2.76	8.82	21.49	41.48	3.03	10.66	35.65
	ITP-4				**ITP-3**		
Applied Load (kN)	**3000**	**6000**	**9000**	**10500**	**3000**	**6000**	**9000**
[8] (mm)	5.45	11.99	22.65	36.02	4.94	10.84	19.99
FEM (Plaxis 3D) (mm)	4.60	10.13	17.69	21.00	4.64	10.13	17.69
[3] (mm)	2.88	7.88	18.84	31.69	3.21	12.24	23.68
[14] (mm)	3.97	9.72	20.46	33.21	4.37	11.35	28.46
[4] (mm)	5.42	11.49	18.79	23.32	5.63	12.28	21.57
Measured (mm)	3.34	9.01	17.71	56.99	2.39	8.22	14.52

Pile settlement-load plots obtained from different methods and pile load tests are presented in Fig. 6 for test piles located in Iskenderun. According to results, it can be said that in generally elastic region of load-settlement curves was modelled properly with all methods. The pile settlement values corresponding to the design load (3000 kN) and 200% of design load (6000 kN) evaluated from the hyperbolic curves obtained by [3] model and the method of [14] were the closest to the measured values and also to the values obtained by finite element method. It can be said that, there is a good match between the results obtained by method of [4] and test results considering in both elastic and failure region of test piles. There is an excellent match between the settlement values of ITP-4 obtained by using method of [4] and finite element method. The settlement values evaluated by method of [8] were also similar with measured values especially for ITP-1 and ITP-2 piles.

TABLE X
PILE SETTLEMENT VALUES OBTAINED FROM DIFFERENT METHODS AND PILE LOAD TESTS (BAKU)

	BTP-1		BTP-2		BTP-3	
Applied Load (kN)	**13000**	**19000**	**15000**	**22000**	**15000**	**22000**
[8] (mm)	6.84	12.60	8.75	17.34	11.56	19.52
FEM (Plaxis 3D) (mm)	6.43	10.15	6.95	10.35	10.41	18.11
[3] (mm)	5.13	10.68	5.05	10.53	4.81	11.17
[14] (mm)	5.84	9.75	6.40	10.74	8.22	14.18
[4] (mm)	6.97	10.63	7.89	12.07	10.96	17.07
Measured (mm)	4.66	9.66	4.13	7.17	5.96	11.02

Pile settlement-load plots obtained from different methods and pile load tests are presented in Fig. 7 for test piles located in Baku. It can be inferred from Table X and Fig. 7 that, the settlement values obtained by different methods are generally close to measured values for BTP-1 and BTP-2 piles. On the other hand, there is a difference in a range of 80% between the results obtained by settlement analysis methods (except the method of [3]) and measured values for BTP-3. This may be due to the fact that the elastic shortening calculations and assumptions of parameters may be incompatible due to the high pile length. Settlement values obtained by the method of [3] are very similar with the measured values. It can be inferred that the proposed curves obtained by using method of [3] match with curves obtained from pile load tests perfectly.

The failure region of test piles located in Baku was modeled by the method of [4] in accordance with the results obtained from Osterberg cell test.

As a result of this study, all methods investigated in this paper were provided in all test piles with normal and large diameters. The main point for analyzing the pile settlement perfectly is to evaluate ultimate base and shaft resistance of soil correctly.

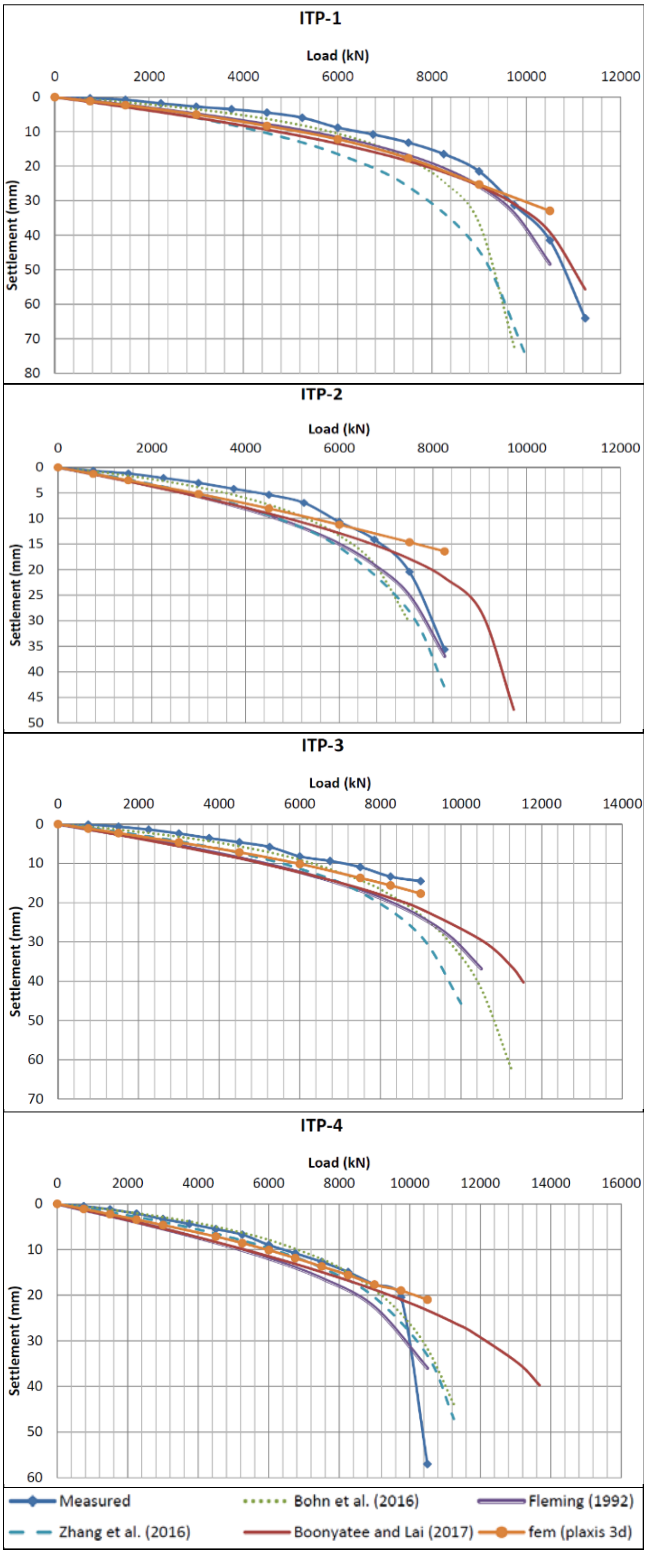


Fig. 6 Comparison of calculated and measured load-settlement plots (Iskenderun)

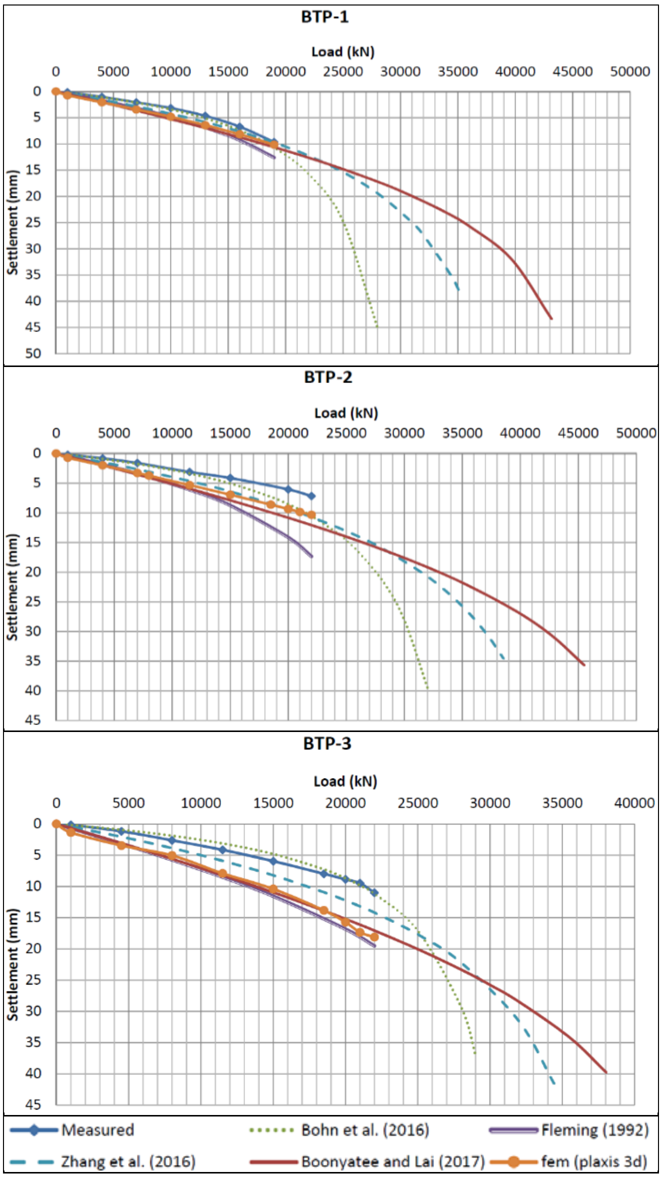


Fig. 7 Comparison of calculated and measured load-settlement plots (Baku)

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M. Mert was born in Eskisehir, Turkey in December, 1987. He received his Bachelor’s degree in Civil Engineering in 2009 and Master’s degree in Geotechnical Engineering in 2012 from Istanbul Technical University, Turkey. He is a PhD. Student in Istanbul Technical University now.

He worked in Temeltas Company located in Istanbul, Turkey for six years (2009-2015) as a Design Engineer in geotechnical projects. He has worked in Fatih Sultan Mehmet Vakif University since 2015 as a Research Assistant.