International Journal of Earth, Energy and Environmental Sciences
ISSN: 2517-942X
Vol:3, No:10, 2009

Evaluation of Tension Capacity of Pile (Case Study in Sandy Soil)

Shooshpasha[1] I., Kiakojoori[2] M., and Mirzagoltabar[1] R. A.

***Abstract*—**High building constructions are increasing in south beaches of the Caspian Sea because of tourist attractions and limitation of residential areas. According to saturated alluvial fields transfer of load from high structures to the soil by piles is inevitable. In spite of most of these piles are under compression forces, tension piles are used in special conditions. Few studies have been conducted because of the limited use of these piles. Tension capacity of open-ended pipe piles in full scale was tested in this study. The length of the bored piles was 420 up to 480 cm and all were in 120 cm diameter. The results of testing 7 piles were compared with the results of relations given by researches.

***Keywords*—**piles, tension capacity, sand, shaft friction

I. Introduction

THE most studies were conducted on the behaviors of piles, are mainly related to piles subjected to axial compression load, while in many cases due to uplift forces, piles will act in tension. Knowing behavior of piles under tension as well as parameters effecting on tension bearing capacity of piles is very significant.

The ultimate shearing resistance that can develop on the shaft of a pile (τ_f) in sand has been confirmed by [1] to be a function of the radial equalized stress after installation, (σ'_{rc}), added radial stress due to loading ($\Delta\sigma'_{rc}$), and the interface friction angle (δ_f):

$$\tau_f = (\sigma'_{rc} + \Delta\sigma'_{rd})\tan\delta_f = \sigma'_{rf}\tan\delta_f \tag{1}$$

Lehane & Jardine [2] show that in sands the value of $\Delta\sigma'_{rd}$ is relatively small for pile with diameters greater than 300mm and hence the radial effective stress at peak friction σ'_{rf} may be considered equivalent to σ'_{rc} for offshore piles. It has long been established that τ_f correlates well with the CPT end resistance q_c [3]. This observation used in conjunction with an observation made by the Imperial College (study on closed-ended instrumented pile) that σ'_{rc} varies with depth up to pile tip, h. Different equation has been established in the following form for σ'_{rc} [2, 4, 5 and 6]:

[1]Assist. Prof., Civil Eng. Faculty, Babol Noshirvani University of Technology, P.O. Box 47144, Babol, Iran, Telfax: 981113231707, Shooshpasha@nit.ac.ir
[2]Post-graduate student in geotechnic, Babol Noshirvani University of Technology, Mokiya_6211@yahoo.com

$$\sigma'_{rc} = \alpha q_c \left(\frac{\sigma'_{vo}}{p_a}\right)^b \left(\frac{h}{R}\right)^{-c} \tag{2}$$

Where σ'_{vo} is the vertical effective stress, p_a is a reference stress =100 kPa, R is the outer radius of the pile (with diameter D), α, b and c are constant values and qc is CPT end resistance. Equation (2) is used both by the Imperial College (IC) design method [4] and the Fugro design method [5].

II. Pile Tension Capacity

A. Alawneh Method

On the basis of 34 pull out pile load tests collected from the literature, Alawneh [7] proposed a method for estimating the ultimate tensile shaft resistance of pile in sand. According to, Alawneh method, the ultimate uplift shaft resistance is written as follows:

$$\tau_{m(z)} = \sigma'_{rf(z)} \tan\delta_f \tag{3-a}$$

$$\sigma'_{rf(z)} = k_{(z)}\sigma'_{v(z)} \tag{3-b}$$

$$k_{(z)} = k_{\min} + (k_{\max} - k_{\min})\exp\left[-0.03\left(\frac{L-Z}{D}\right)\right] \tag{3-c}$$

$$K_{\max} = 0.35\exp(0.03D_r)\left(\frac{D+0.45}{2D}\right)^{0.005D_R}\left(\frac{\sigma'_{vtip}}{P_a}\right)^{-0.84} \tag{3-d}$$

Where τ_m(z) is the ultimate uplift shaft resistance; σ'_{rf} (z) is the radial effective stress at failure; δ_f is the pile-sand friction angle at failure; $k_{(z)}$ is the uplift earth pressure coefficient; $\sigma'_{v(z)}$ is the effective vertical stress; z is the depth below ground surface, σ'_{vtip} is the effective vertical stress at location of pile tip; Pa is the atmospheric pressure (101.3 kPa), D_r is the sand relative density, D is the pile diameter, L is the pile length, k_{max} is the maximum earth pressure coefficient value at the pile tip obtained from Eq. (3-d) and k_{min} is the minimum earth pressure coefficient value.

The minimum earth pressure coefficient k_{min} can be linked to Rankien's active earth pressure coefficient k_a. However, Alawneh recommended to use k_{min}=0.23. The average minimum k value calculated from data by Alawneh is 24% less than the corresponding k_a value.

B. American Petroleum Institute (API) Method

Field experiments performed by Imperial College London reported by [1, 8, 9 and 4] on instrumented closed-ended displacement piles, show that the peak local shaft friction q_s can be related to the radial effective stress at failure σ'_{rf} by the simple Coulomb failure criterion;

$$q_s = \sigma'_{rf}.\tan\delta_f \tag{4}$$

Where the interface friction angle at failure δ_f is measured in appropriate laboratory interface tests [10]. After pile installation and prior to static loading, the radial effective stress would be equal to pore pressure σ'_{rc} acting at any point on the pile shaft which was found to be almost equal to q_c which is in directly proportional to the depth of the point, h. Based on measured σ'_{rc} from the medium dense and dense sand the following relationship is proposed [4]:

$$\sigma'_{rc} = 0.029 q_c \left(\frac{\sigma'_v}{p_a}\right)^{0.12} \left(\frac{h}{R}\right)^{-0.38} \tag{5}$$

Where P_a is atmospheric pressure (equal to 100 kPa), σ'_v is effective overburden pressure and R is the pile external radius. The Imperial College experiments also indicated that radial effective stress during static loading test, increases to a maximum value σ'_{rf} of about $1.4 \pm 0.2\,\sigma'_{rc}$.

C. ICP-05 Method for Shaft Capacity

The ICP-05 design method is based on results from load tests on jacked closed-ended instrumented piles [1 and 9] and was calibrated for open-ended piles primarily using tests on driven piles at Dunkirk [11]. Neglecting increases in radial stress due to interface dilation, equations (1) and (2) were combined to deduce the following expression for local ultimate shaft friction τ_f:

$$\frac{\tau_f}{q_c} = 0.029 \left(\frac{\sigma'_{vo}}{P_\alpha}\right)^{0.13} \frac{f}{f_c} \max\left[\frac{h}{R^*}, 8\right]^{-0.38} \tan(\delta_f) \tag{6}$$

Where f/f_c is capacity ratio (f/fc =1 for compression and 0.72 for tension loading on a pipe pile) and R^* is the equivalent pile radius (R_{eq}) defined as:

$$R_{eq} = R^* = \sqrt{R^2 - R_i^{\,2}} \tag{7}$$

Where R is the outside radius of the pile and R_i is the internal radius of the pipe pile (which is zero for a closed-ended pile).

D. Fugro-04 Method for Shaft Capacity

The Fugro-04 method for shaft friction is based primarily on the results of loading on instrumented large diameter open-ended piles cast in place in dense sands. Separate equations are provided for tension and compression:

$$\frac{\tau_f}{q_c} = 0.144 \left(\frac{\sigma'_{vo}}{p_a}\right)^{0.05} \left(\frac{h}{R^*}\right)^{-0.90} \tan\delta_f \tag{8-a}$$

$$\frac{\tau_f}{q_c} = 0.144 \left(\frac{\sigma'_{vo}}{p_a}\right)^{0.05} \left(\frac{h}{4R^*}\right) \tan\delta_f \tag{8-b}$$

$$\frac{\tau_f}{q_c} = 0.081 \left(\frac{\sigma'_{vo}}{p_a}\right)^{0.15} \max\left(\frac{h}{R^*}, 4\right)^{-0.85} \tan\delta_f \tag{8-c}$$

Where equation 8-a applies to piles in compression with $h/R^*>4$, equation 8-b applies to piles in compression with $h/R^*<4$, and Equation 8c applies to piles in tension. The constant volume interface friction angle δ_f is assumed to be 29^o, which is the interface friction angles used to calibrate the model.

E. The German Code of Practice – DIN 4014

This code does not distinguish between the shaft resistance in tension and compression. The relation is based on a large number of tests for both cased and uncased borings. The shaft friction can be obtained based on SPT as:

$$F_S = 2.86N \tag{9}$$

In which F_s is the shaft resistance in kN/m^2 and N is the SPT value.

F. British/American Methods

It is suggested to calculate the unit side friction from the following equation [12]:

$$\tau_S = \sigma'_r \cdot \tan\delta = k \cdot \sigma'_v \cdot \tan\delta \tag{10}$$

Where k = 0.90 for all sands and 0.6 for silt, σ'_v is the vertical effective stress and δ is the angle of friction in the interface between the pile and the soil which can be taken between φ_{peak} to φ_{cv}. No distinction is made between the values in tension and compression.

Based on 41 piles test, the following equation is proposed for the unit side friction [13]:

$$\tau_S = \beta \cdot \sigma'_Z \tag{11}$$

Where $\beta = 1.5 - 0.245 Z^{0.5}$, z is the depth below ground level and, σ'_Z is the vertical effective stress at depth z. It is assumed that $0.25 < \beta < 1.20$ and $\tau_s <$ *200 kPa. For SPT values lower than 15 it is* recommended to scale down the side resistance by the reduction factor N/15 [14]. Kraft and Lyons [15] also suggest using $\beta = Ftg(\phi' - 5°)$ which F = 0.5 and 0.7 for piles in tension and compression, respectively.

G. UWA-05 Methods for Shaft Capacity

The UWA database of static loads, as discussed by [16], was employed to assess the predictive performance of the proposed UWA-05 method.

The UWA-05 method simplifies to the following form for full scale offshore piles, as IFR=1 and ignoring dilation term $\Delta\sigma'_{rd}$ one has;

$$\tau_f = 0.03 \times q_c \times Ar^{0.3} \left[Max\left(\frac{h}{D},2\right)\right]^{-0.5} \tan\delta \qquad (12)$$

$$\mathrm{Ar} = 1 - \left(\frac{D_i^{\,2}}{D^2}\right) \qquad (13)$$

$$Q_s = 0.75 \times \pi \times D \times \int \tau_f .dz \qquad (14)$$

Where τ_f is local shaft friction, D is diameter, Ar is effective area ratio, q_c is cone tip resistance, h is pile length, D_i is inner diameter of pile and Q_s is ultimate tension capacity of pile.

H. Geological Conditions and Geotechnical Factors of Soil in the Studied Area

The area of study is located in city of Mahmoodabad, Iran (South Caspian Sea the central of Alborz mountain). The area has sediments from Paleozoic era and the quaternary which has turned into what it is now through years of weather changes and tides of the sea. Based on the evaluations made on samples from bored holes, one could say that sediments existing in the project area belong to the fourth geological era. The field study started boring two 25m boreholes in 400 square meter site. Layer changes were recorded while boring holes and samples were taken for testing from various depths. The Standard Penetration Test (SPT) was carried out according to AST-DIS86. Table 1 and 2 respectively summarize soil parameters of boreholes BH1 and BH2. the necessary laboratory tests were performed on samples obtained in filed in order to find out the physical and mechanical properties of soils.

TABLE I SOIL PARAMETERS OF BH1

Depth (m)	SPT	C	ϕ	e	G_s	C_c	C_u	Soil
1.0	22	0	27.7	0.75	2.69	0.94	2.43	sp
2.5	20	--	--	0.51	2.68	0.51	6.55	sp
4.0	47	--	--	0.41	2.67	0.49	2.93	sp
5.5	68	--	--	0.52	2.68	1.19	6.15	sp
7.0	47	0	25.1	0.59	2.73	1.13	2.21	sp
9.0	45	--	--	0.55	2.70	1.06	2.23	sp
11.0	51	--	--	0.56	2.68	1.14	1.99	sp
13.0	44	--	--	0.57	2.71	1.12	2.16	sp
15.0	57	0	29.2	0.60	2.71	1.14	2.13	sp
17.5	53	--	--	0.56	2.71	1.12	2.17	sp
22.5	53	--	--	0.48	2.65	1.00	2.15	Sp
25.0	61	0	32.1	0.51	2.67	0.98	2.68	sp

TABLE II SOIL PARAMETERS OF BH2

Depth (m)	SPT	C	ϕ	e	G_s	C_c	C_u	Soil
1.0	22	--	--	0.67	2.81	1.05	1.94	sp
2.5	21	0	25.1	0.68	2.80	1.08	2.01	sp
4.0	32	--	--	0.49	2.66	0.49	7.73	sp
5.5	37	-	29.8	0.43	2.66	0.51	7.19	sp
7.0	57	--	--	0.49	2.67	0.95	2.61	sp
9.0	40	--	--	0.59	2.67	1.11	2.02	sp
11.0	38	0	30.2	0.58	2.70	1.12	2.03	sp
13.0	27	--	--	0.64	2.78	1.12	1.99	sp
15.0	46	--	--	0.61	2.72	1.11	2.02	sp
17.5	60	-	31.8	0.50	2.65	1.03	2.16	sp
22.5	50	--	--	0.42	2.64	0.98	2.11	sp
25.0	61	0	32.1	0.53	2.63	0.97	2.08	sp

The following diagram of SPT changes with depth shows that the soil in the site is made of a layer of coarse soil in depth that is considered poor sand (SP) based on Unified soil classification.

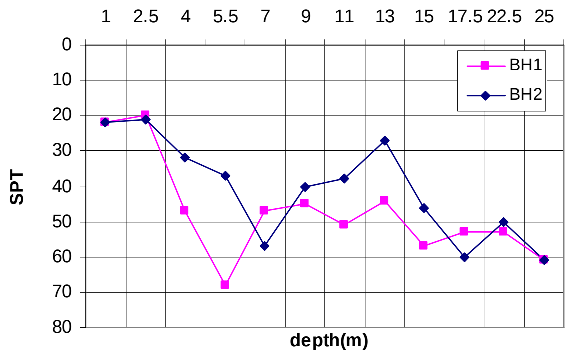

Fig. 1 Standard penetration with depth in BH1 and BH2.

The under ground water level is 4.5 m deep based on the evaluations from different holes. Besides SPT values the direct shear test in drain condition was done on samples prepared in various depths based on ASTM-D3080 for evaluating the mechanical properties of soil which the results are presented in table 1 and 2.

III. Tension Capacity of Piles

In the experimental part, a test apparatus was designed and fabricate for measuring the tension capacity of piles. In the following section the test apparatus and setup is explained.

A. Test Apparatus and Procedures

A metal frame was fabricated and used with a hydraulic jack with a reading gauge to measure pile tension capacity. This frame was put in the direction of tension force imposed by a 40 ton crane. The mechanism of this frame is to transfer force from the crane to the piles is like a chain, so that the first ring takes force from the crane by its top part and transfers it from the bottom part to the top part of the second ring. The second ring also transfers this force from its lower part into the pile with a cable to pull out the pile. Hence from the gage, the amount of applied force to pull out the pile is recorded. Figures 2 to 3 show the test setup and how tension force is transferred from crane to pile.

Fig. 2 Hydraulic Jack installation in the frame.

Fig. 3 Imposing tension force to pile

To install the pile, at first a 120 cm deep hole is bored then the pile is aliened. Digging inside the pile, casing is moved downward to the desired depths by its weight and additional weight. Measuring tension force needed to pull out casing which was done in two steps; the first step was after installation of casing i.e. before concrete casting. In this step the casing is pulled out about 10 cm to measure the force required to come over the side resistance resulted due to friction between the pile and soil.

The second step was taken after casting. The measured force included both outer shaft friction with soil and inner shaft friction with concrete. Figure 3 and 5 shows the process of measurements.

B. *Results and discussion*

Tests were carried out on 7 open-ended pipe piles with 420 cm to 480 cm long and a diameter of 120 cm.

Fig. 4 Reading imposed force.

Fig .5 Displaying the amount of force required to pull out the pile.

The following tables summarize the results of reading the tension capacity of piles neglecting pile weight.

TABLE III THE SUMMARY OF RESULTS FROM READING TENSION CAPACITY

Pile No.	Buried length of Pile L (cm)	L/D	Maximum pile displacement (cm)	Maximum pullout loading (ton)	Tension Load (ton)
P_1	480	4.00	12.0	10.25	8.00
P_2	420	3.50	8.0	8.50	6.25
P_3	450	3.75	10.0	9.75	7.50
P_4	420	3.50	8.0	8.50	7.25
P_5	480	4.00	10.0	10.50	8.25
P_6	450	3.75	10.0	9.50	7.25
P_7	465	3.875	12.0	10.0	7.75

Pile Diameter D = 120 (cm)   Weight of pile = 2.25 (ton)

The experimental results for each pile are shown in figure 7. As one can see, tension bearing capacity of piles increases with increasing L/D ratio.

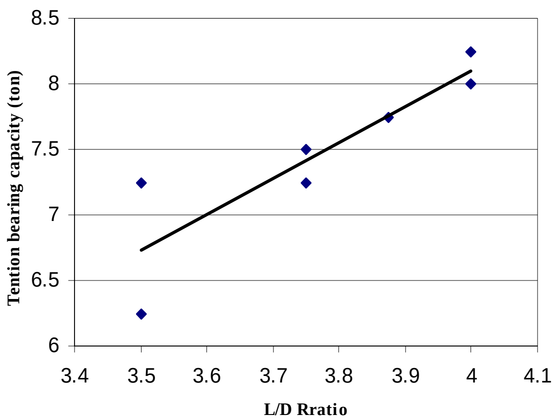

Fig. 7 Bearing capacity variation with L/D ratio

Based on aforementioned eight methods, the tension bearing capacity of piles was calculated using parameters obtained from the field. The results were presented in table IV in accompany with field measurement results for piles with different L/D ratio.

TABLE IV CALCULATION RESULTS FOR PILES WITH DIFFERENT L/D RATIO.

Pile length (cm)	420	450	465	480
L / D	3.50	3.75	3.875	4
Field measurement results	6.75	7.37	7.75	8.12
Imperial College	10.63	10.79	11.12	11.28
ICP-05	7.61	7.73	7.96	8.07
Fugro-04	6.72	6.81	6.99	7.07
DIN-4014	11.36	11.55	11.94	12.13
Fleming et. al.	4.73	4.80	4.94	5.01
Rees & O'Neill	7.85	8.00	8.28	8.43
UWA-05	7.31	7.42	7.62	7.72

In order to compare the results, variation of measured and calculated tension bearing capacity of piles with L/D ration for each method were shown in figures 8 to 14. These figures show that results obtained from the methods of ICP-05, FUGRO-04, Rees & O'Neill and UWA-05 has a good correlation with experimental results.

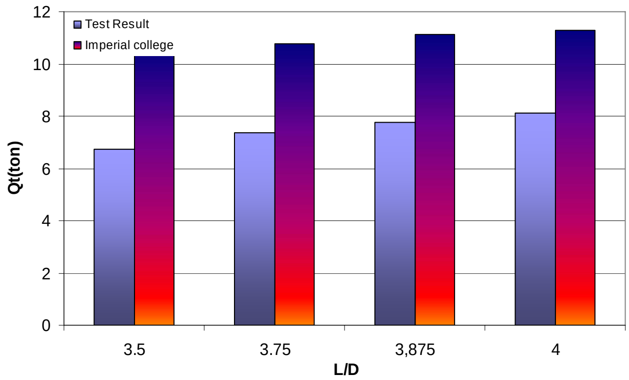

Fig. 8 Comparison of results obtained from Imperial College method with field measurements.

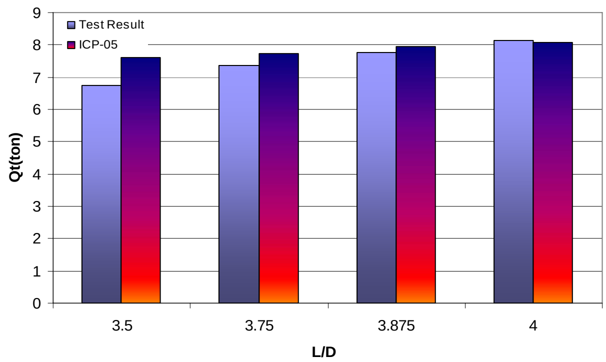

Fig. 9 Comparison of results obtained from ICP-05 method with field measurements.

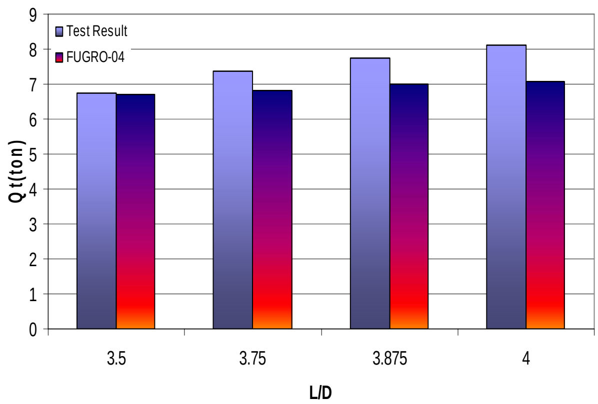

Fig. 10 Comparison of results obtained from FUGRO-04 method with field measurements.

International Journal of Earth, Energy and Environmental Sciences
ISSN: 2517-942X
Vol:3, No:10, 2009

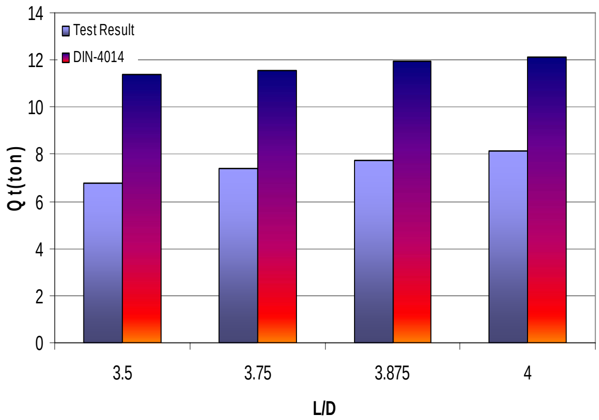

Fig. 11 Comparison of results obtained from DIN-4014 method with field measurements.

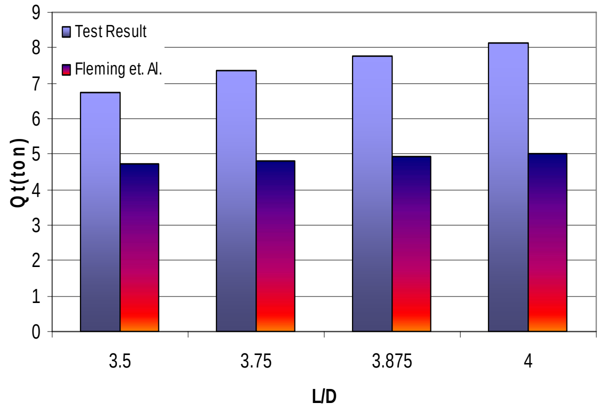

Fig. 12 Comparison of results obtained from Fleming et al. method with field measurements.

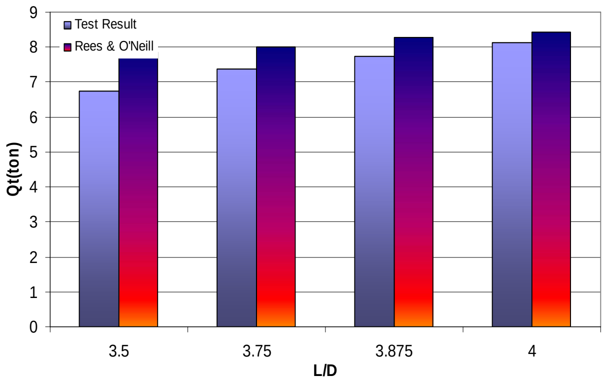

Fig. 13 Comparison of results obtained from Rees & O'Neill method with field measurements.

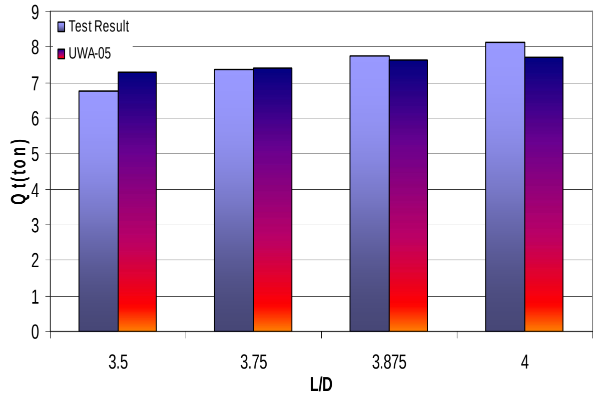

Fig.14 Comparison of results obtained from UWA-05 method with field measurements.

Calculated pile tension capacity based on different methods and field measurements for different L/D ratio are shown in figures 15 to 18.

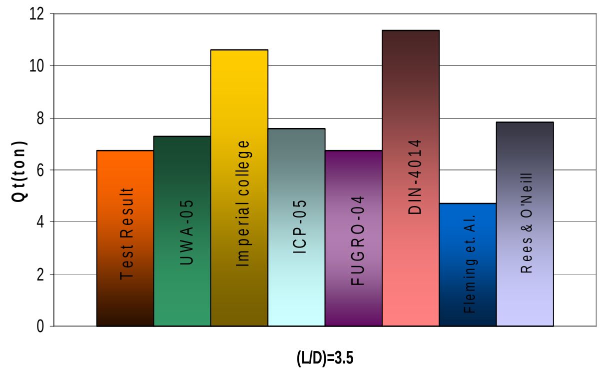

Fig. 15 Comparison of results obtained from different method with field measurements for L/D=3.5.

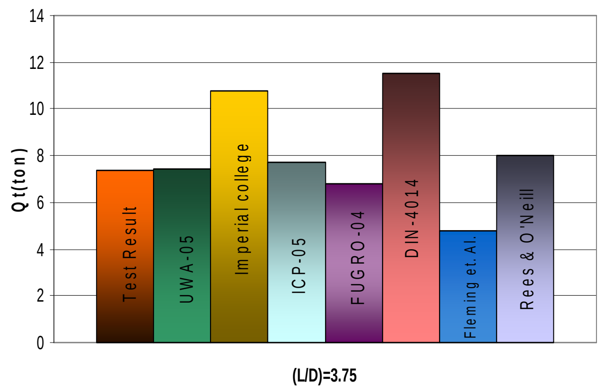

Fig. 16 Comparison of results obtained from different method with field measurements for L/D=3.75.

International Journal of Earth, Energy and Environmental Sciences
ISSN: 2517-942X
Vol:3, No:10, 2009

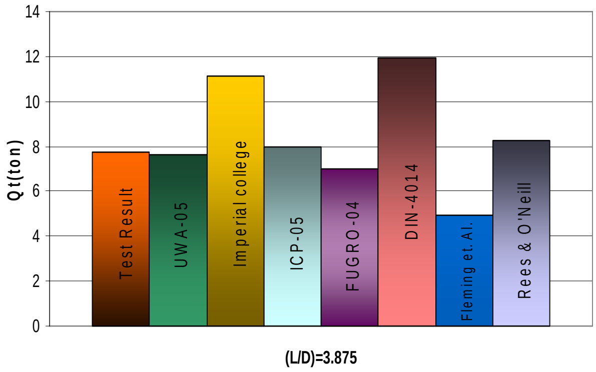

Fig. 17 Comparison of results obtained from different method with field measurements for L/D=3.875.

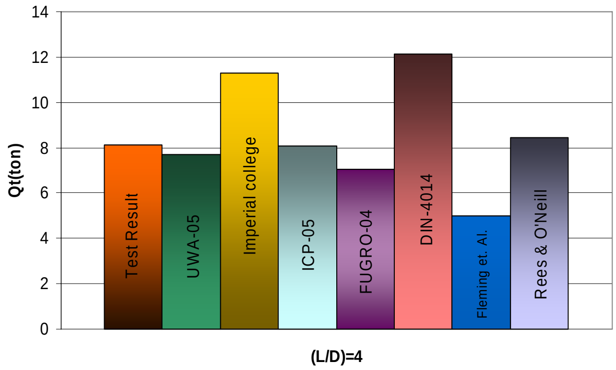

Fig. 18 Comparison of results obtained from different method with field measurements for L/D=4.0.

As one can see from figure 8, results obtained from Imperial College method are higher than measured one. This could be due to assumption is used by this method that the peak local shaft friction on the pile can be related to the radial effective stress at failure by mean of simple Coulomb failure criterion. Based on this criterion soil is homogenous with inner friction and cohesion as well as the friction capacity is constantly distributed along the failure plain. This theory suggests that failure wedge is a rigid body and the friction force is produced due to displacement of failure wedge between soil and shaft.

The formula provided by ICP-05 for estimating tension capacity is based on results from load tests on jacked closed-ended instrumented piles and was calibrated for open-ended piles too. As shown in figure 9 the results obtained from this method is too closed to the measured one.

As one can see from figure10, the method FUGRO-04 as ICP-05 method also gives acceptable tension capacity in compare to field measured one. A good correlation between calculated and measured could be due to that this method is based on field results and especial equation is presented for tension capacity of piles.

Results obtained from DIN 4014 method as presented in figure 11 is far away from actual results i.e. field measurements. This method has a low accurate to estimate shaft tension capacity because it doesn't consider difference between side friction in tension and compression for piles. Moreover, this code tries to estimate tension capacity only by SPT numbers. So choosing value N to estimate tension capacity needs a high accuracy while SPT is carried out in deep points. In order to use SPT results estimating tension capacity suggested by German code, number of impactions can be considered for layers, unless layers are very deep. For a deep layer it is better to divide the layer into several sub-layers in order to get the average N value for the method. The N value can be corrected based on information of soil layers. Correction factor of C_N related to depth equal to $\sigma_v = 99.76\ kPa$ and diminishes significant depth effect. So to calculate the shaft friction by N value with the relationship proposed by this method one needs a lot of patient. That is why there is difference between results obtained with this method with field measurements.

Figure 12 shows that Fleming et al. method is a conservative method in estimating tension capacity. In compare to other methods, since no distinction is made between the values in tension and compression and also estimates shaft friction by a fixed value of k=0.9 for all sandy soil without considering parameters such as relative density, moisture etc.

Rees and O,neill method estimated shaft friction by imposing a coefficient (which is a function of depth) to vertical effective stress. Since this method was obtained from 41 piles testing and considers a limitation of 0.25-1.2 for β as a coefficient for vertical effective stress, the results obtained from this method has no significant difference with field measurements (as shown in figure 13).

In UWA-05 method shaft friction appearing on piles is related to displacement of surrounding soil during installing piles. Displacement is determined by effective area parameter both for closed-ended piles and open-ended one. Results obtained from this method as shown in figure 14 is in good agreement with field measurement.

IV. Conclusion

In the present study, different theoretical and empirical methods were used to evaluate shaft friction capacity of piles. These methods show differences in tension capacity value of piles buried in sand. Field measurement results provided in full scale are in the range of results obtained from some methods.

Differences observed in results of these methods are made because of different parameters influential in shaft tension capacity in sandy soils and lack of enough suitable information from tests conducted in full scale especially on open-ended pipe pile and each method considered a few parameters to estimate shaft friction capacity.

However, assessment on pile tests show that two methods of ICP-05 and UWA-05 gives the shaft friction capacity closed to field measurements in full scale.

References

[1] Lehane BM. Jardine RJ. Bond AJ, Frank R. 1993. Mechanisms of shaft friction in sand from instrumented pile tests. J Geotechnical Eng., 119 (1):19-35.

[2] Lehane, B.M., and Jardine, R.J. 1994. Shaft capacity of driven piles in sand: a new design approach. In proceedings of a Conference on the Behavior of Offshore Structures, Boston, Mass., Vol. 1, pp. 23-36.

[3] Bustamante, M., and Gianeselli, L. 1982. Pile bearing capacity by means of static penetrometer CPT: In Proceedings of the 2nd European Symposium on Penetration Testing. Amsterdam, pp. 493-500.

[4] Jardine, R.J., Overy, R.F., and Chow, F.C. 1998. Axial capacity of offshore piles in dense North Sea sand. Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 124(2): 171-178.

[5] Fugro Engineers B.V. (Fugro) 2004. Axial pile design method for offshore driven piles in sand. Fugro Report No.P1003, Issue 3 to API, 5 August 2004: 122 pp.

[6] Lehane, B.M., White, D.J. 2005. Lateral stress changes and shaft friction for model displacement piles in sand. Canadian Geotechnical Journal 42 (4): 1039-1052 August 2005.

[7] Alawneh AS. 1999. Tension piles in sand: a method including degradation of shaft friction during driving. Transportation Research Record No. 1663. National Research Council. Washington, DC, Paper No.990092; p.41-9.

[8] Lehane, B.M. 1992. Experimental investigations of pile behavior using instrumented field piles. Ph.D. thesis, University of London (Imperial College), London, U.K.

[9] Chow, F. 1997. Investigations into the behavior of displacement piles for offshore structures. PhD thesis, University of London (Imperial College), London, U.K.

[10] Ramsey, N., Jardine, R.J., Lehane, B.M., and Ridly, A. 1998. A review of soil-steel interface testing with the ring shear apparatus. In proceedings of the 6th Conference on Offshore Site Investigation and Foundation Behavior. Society for underwater technology, London, U.K., pp. 237-258.

[11] Brucy, F., Meunier, J., & Nauroy, J.-F. 1991. Behavior of pile plug in sandy soils during and after driving. OTC 6514, proc., 23rd Annual OTC, Houston: 145-154.

[12] Fleming W.G.K., Weltman A.J., Randolph M.F., Elson W.K. 1992 Piling Engineering. Taylor & Francis Group, London and New York.

[13] O'Neill, M.W., Hassan, K.M. 1994. Drilled Shafts: Effects of construction on performance and design criteria. Proc., Int. Conf. Des. Constr. Deep Founds. Orlando, FHWA, 1,137-187.

[14] O'Neill, M.W. 1994. Drilled Shafts. Proc., International Conf. on Design and Construction of Deep Foundations, Fed. Highway Admin., Washington, D.C., Vol. 1, 185-206.

[15] Kraft, L.M., Lyons, C.G. 1974. State of the art- ultimate axial capacity of grouted piles. Proceedings of the 6th Annual OTC, Houston, Texas, pp.485-504.

[16] Lehanc, B.M., Schnider, J.A., and Xu, X. 2005. Evaluation of design methods for displacement piles in sand. UWA Report, GEO: 05341.1.