Journal of Zhejiang University-SCIENCE A (Applied Physics & Engineering)
ISSN 1673-565X (Print); ISSN 1862-1775 (Online)
www.zju.edu.cn/jzus; www.springerlink.com
E-mail: jzus@zju.edu.cn

Effect of soil set-up on the capacity of jacked concrete pipe piles in mixed soils*

Jun-wei LIU†, Zhong-miao ZHANG†‡, Feng YU, Cun-gang LIN

(*MOE Key Laboratory of Soft Soil and Geoenvironmental Engineering, Institute of Geotechnical Engineering, Zhejiang University, Hangzhou 310058, China*)

†E-mail: zjuljw@126.com; zjuzzm@163.com

Received Dec. 12, 2010; Revision accepted June 3, 2011; Crosschecked July 15, 2011

Abstract: The increase in capacity of displacement piles with time after installation is typically known as soil/pile set-up. A full-scale field test is carried out to observe the set-up effect for open-ended concrete pipe piles jacked into mixed soils. Both the total capacity and the average unit shaft resistance increase approximately linearly with logarithmic time. The average increase rate for unit shaft resistance is 44% per log cycle, while the average increase for total capacity is approximately 21%. A review on case histories for long-term set-up indicates an average set-up rate of approximately 40%. Based on this, the mechanism of pile set-up is discussed in detail and a three-phase model is suggested.

Key words: Concrete pipe pile, Jack piling, Set-up, Pile capacity
doi:10.1631/jzus.A1000503 **Document code:** A **CLC number:** TU473

1 Introduction

Soil/Pile set-up is defined as an increase in pile capacity over time after installation. This phenomenon was first reported by Wendel (1900), and has been recognized as occurring in most parts of the world. Set-up has been documented for virtually all displacement pile types including pipe pile and solid concrete pile. The consideration of set-up effect can efficiently improve the estimation of pile capacity, so that the length, diameter or the number of production piles may be economically reduced. The rate and magnitude of set-up is a function of a combination of a number of factors (Samson and Authier, 1986). Thus, the beneficial effect of capacity set-up is seldom used to any great extent in piling projects due to the high uncertainties of predicting the development of set-up.

To further investigate and demonstrate the long-term set-up, three full-size concrete pipe piles were jacked into mixed soils. Static load tests were undertaken at two different points in time for each pile, up to a maximum of 75 d after installation. The tendency of an increase in pile capacity with time was observed and compared with other case histories. Based on the mechanisms of pile set-up, a three-phase model is proposed. In addition, a thorough review of previous studies, including a compiled database of case histories and empirical formulas, is also undertaken in this paper. This study is an attempt to improve the understanding and application of the set-up effect on displacement piles.

2 Basic mechanism of set-up

During the installation of a pile, soil is displaced predominately radially along the shaft and vertically and radially beneath the toe. A remolded zone with a thickness of up to 100 to 150 mm is formed around the shaft (Flaate, 1972), accompanied by high excess

---

‡ Corresponding author

* Project supported by the National Natural Science Foundation of China (No. 51078330), and the Natural Science Foundation of Zhejiang Province (No. Y1090610), China

porewater pressure generated in this narrow band. After installation, as the excess porewater pressure dissipate, the effective stress of the affected soil increases, and set-up occurs as a result of increased shear strength and increased lateral stress against the pile (Komurka *et al.*, 2003). Accordingly, the set-up predominately takes place along the pile shaft (Samson and Authier, 1986; Bullock *et al.*, 2005b), and mainly on the lower part (Preim *et al.*, 1989; Fellenius *et al.*, 1992). Little or no set-up is observed to take place at the toe (Skov and Denver, 1988; Axelsson, 2000).

Based on Komurka *et al.* (2003), the soil/pile set-up mechanisms can be divided into three phases: (1) Phase 1: logarithmically nonlinear rate of excess porewater pressure dissipation; (2) Phase 2: logarithmically linear rate of excess porewater pressure dissipation; (3) Phase 3: independent of effective stress (ageing). The mechanism of set-up is shown in Fig. 1, where t is the time elapsed after initial driving, and t_0 is the time when Phase 2 begins; Q_{initial} is the initial capacity, and Q_{final} is the final capacity corresponding to time t.

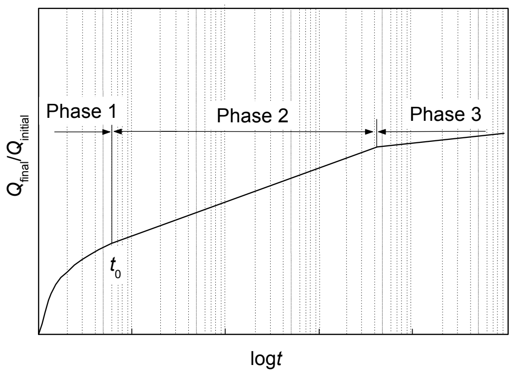

Fig. 1 Schematic idealized set-up phases (Komurka *et al.*, 2003)

During the first phase set-up is nonlinear with the logarithm of time. This phenomenon is possibly attributed to the non-constant rate of excess porewater dissipation, or other mechanisms that are complicated and have not been well understood. The duration of this nonlinear period, corresponding to the time parameter t_0 in predictive models (Skov and Denver, 1988), is a function of soil and pile properties. They recommended numerical values for t_0 as 1 d in clay whilst was 0.5 d in sand. Axelsson (1998) set t_0 equal to 1 d for pre-stressed concrete piles installed in non-cohesive soils. Long *et al.* (1999), however, recommended using t_0 equal to a time as short as 0.01 d. Moreover, a larger pile diameter will cause a larger t_0 (Camp III and Parmar, 1999).

In the second phase, set-up rate corresponds to the rate of excess porewater pressure dissipation. During the logarithmically constant rate of dissipation, the affected soil experiences an increase in effective vertical and horizontal stress, consolidates and gains shear strength according to the conventional consolidation theory (Komurka *et al.*, 2003). The length of this phase also depends on the soil and pile properties. In granular materials, complete porewater pressure equalization is expected within a few hours after installation. In cohesive soils, however, dissipation may continue for several weeks, several months, or even years (Skov and Denver, 1988). The time to dissipate excess porewater pressure is proportional to the square of the horizontal pile dimension (Soderberg, 1961), and is closely related to the diameter ratio (Randolph, 2003).

During the third phase, set-up rate is independent of effective stress, which is known as soil ageing. This effect is attributable to thixotropy, secondary compression, particle interference or clay dispersion (Long *et al.*, 1999; Komurka *et al.*, 2003). This effect also increases the soil's shear stiffness and dilatant behavior (Axelsson, 1998; 2000).

For cohesive soils, the majority of set-up is related to the dissipation of excess porewater pressure (i.e., Phase 1 and Phase 2). In addition, the reconsolidated soil in the remolded zone will gain higher shear strength than undisturbed soil (Randolph *et al.*, 1979). This effect transfers the failure under axial load to the surface of remolded zone and accordingly increases the effective perimeter of pile shaft, and thus improves the pile capacity. For granular soils, however, set-up is predominately associated with Phase 3 (i.e., ageing effect) due to relatively rapid dissipation of excess porewater pressure. Moreover, stress relaxation is also recognized as another main cause of long-term set-up for granular soils (Chow *et al.*, 1998; Axelsson, 1998; 2002; Bullock *et al.*, 2005a; 2005b). Based on the observations of Axelsson (1998), it can be understood that stress relaxation effect acts throughout Phase 2 and Phase 3.

3 Previous study

Pile set-up effect was first well-documented by Tavenas and Audy (1972) in non-cohesive soils. Subsequently, a number of other case histories of set-up have also been reported by Moe *et al.* (1981), Samson and Authier (1986), Hunt and Baker (1988), Seidel *et al.* (1988), Skov and Denver (1988), Wong (1988), Preim *et al.* (1989), Fellenius *et al.* (1989; 2000), Astedt *et al.* (1992), Eriksson (1992), York *et al.* (1995), Thomann and Hryciw (1992), Chow *et al.* (1998), Axelsson (1993; 1994; 1998; 2000; 2002), Long *et al.* (1999), Komurka *et al.* (2003), Tan *et al.* (2004), Bullock *et al.* (2005a; 2005b) and Zhang *et al.* (2009). Some of these cases are compiled and shown in Fig. 2. Here the reference capacity Q_0 was measured between 0.5 and 4 d after installation, and the maximum ageing time was up to 3310 d. It shows that the long-term set-up is generally in the region of 15% to 65% per log cycle, and the average value is approximately 40%. The most rapid set-up presented in the database is up to 106% per log cycle, observed by Tan *et al.* (2004) on 610 mm closed-ended pipe pile driven in medium dense sand.

A number of empirical equations have also been proposed to quantify the magnitude of set-up, most of which are summarised in Table 1. Amongst those proposed formulas, the logarithmic relationship proposed by Skov and Denver (1988) has been commonly adopted for the prediction of the set-up.

4 Field tests

4.1 Site condition

The field test was carried out at a construction site in Hangzhou, China. In-situ cone penetration tests (CPTs) were conducted and soil samples were also obtained from drilled holes to perform laboratory tests. Fig. 3 shows the soil profiles and CPT traces at

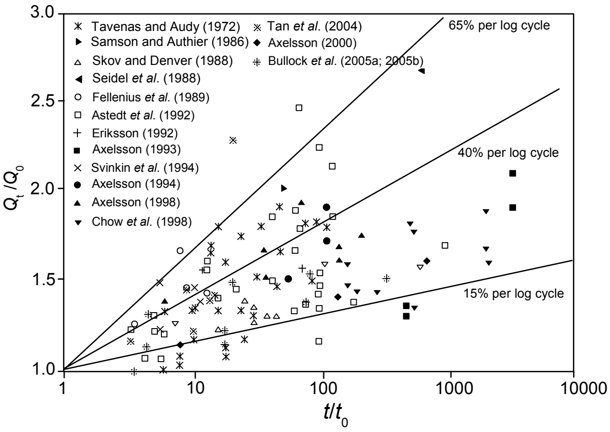

Fig. 2 Case histories of long-term pile set-up

Table 1 Empirical formulas for predicting pile capacities with time

Equation	Soil type	Reference
$Q_{14}=(0.375S_t+1)Q_{EOD}$, (1) where S_t is the sensitivity of soil, Q_{14} is the pile capacity at 14 d, and Q_{EOD} is the capacity at the end of driving (EOD)	Fine grained soil	Zhu (1988)
$Q_t=Q_{EOD}+0.236[1+\log t(Q_{max}-Q_{EOD})]$, (2) where Q_t is the capacity at time t, and Q_{max} is the maximum capacity	Soft soil	Huang (1988)
$\begin{cases} Q_t=1.4Q_{EOD}t^{0.1}, & \text{upper bound,} \\ Q_t=1.025Q_{EOD}t^{0.1}, & \text{lower bound} \end{cases}$ (3)	Sand	Svinkin *et al.* (1994)
$\begin{cases} Q_t=1.1Q_{EOD}t^{0.13}, & \text{upper bound,} \\ Q_t=1.1Q_{EOD}t^{0.05}, & \text{lower bound} \end{cases}$ (4)	Sands, clays, mixed soils	Long *et al.* (1999)
$Q_t=Q_0[A\log(t/t_0)+1]$, (5) where Q_0 is the capacity at time t_0, A and t_0 are two empirical parameters	Sand and clay	Skov and Denver (1988)
$q_t=q_0(0.3\log t+2.8)$, (6) where q_t is the unit shaft resistance at time t, and q_0 is the unit shaft resistance at EOD	Soft clay	Zhang (2009)
$Q_t=Q_u\left(0.2+\dfrac{t/t_{50}}{1+t/t_{50}}\right)$, (7) where Q_u is the ultimate capacity with 100% set-up, and t_{50} is the time corresponding to 50% set-up	Clay	Bogard and Matlock (1990)

the test site. From the ground surface down to a depth of 50 m, the profile consists of a stratigraphic sequence of fill (0–3.2 m), clayey silt (CS, 3.2–16.8 m), silty sand (SS, 16.8–30.1 m), silty clay (SC, 30.1–37.2 m), clayey silt (CS, 37.2–39.8 m) and silty clay (SC, 39.8–50.0 m). The major properties of soil layers are shown in Table 2. The ground water table was at 3.4 m below the ground surface.

4.2 Test setup

The test piles were all open-ended prestressed concrete pipe piles (PHC piles) with an outer diameter of 600 mm and wall thickness of 100 mm. The piles were jacked using a jacking rig with a capability of 9000 kN and were re-jacked 0.5 d after installation. Three test piles, P1, P2 and P3, were jacked to the depths of 32, 34 and 41 m, respectively. All the three test piles were performed twice using a maintained static load test carried out at a different time after installation.

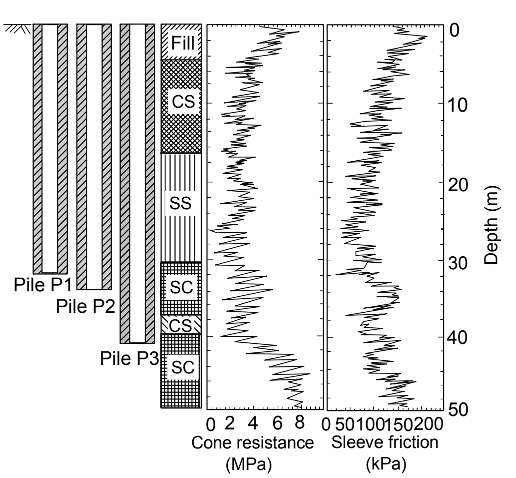

Fig. 3 Soils profiles and cone penetration test results

4.3 Test results

The load-movement curves from the static loading tests on the piles P1, P2 and P3 are depicted in Fig. 4. Evident difference can be found in the two curves for the first and second tests, especially for piles P2 and P3. The ultimate bearing capacities at different time confirmed following the Chinese Technical Code (JGJ106-2003) are summarized in Fig. 5 and Table 3. The pile resistances observed at the end of initial jacking and re-jacking procedure were taken as the pile capacities at that time. These are also plotted in Fig. 5. An evident increase in the bearing capacity takes place over time, although pile P1 shows a tendency to level off after approximately 20 d. This may be explained by the pile P1 being significantly disrupted during the static loading, thus decreasing the set-up process. During the eighth load increment for pile P1 at the first test, 500 kN weights

Table 2 Soil properties

Soil	Depth (m)	Natural water content (%)	Specific gravity	Liquid index (%)	Plastic index (%)	Cohesion (kPa)	Friction angle (°)	Compression modulus (MPa)
Clayey silt	3.2–16.8	29.8	2.71	1.145	9.2	10.0	29.5	3.6
Silty sand	16.8–30.1	24.4	2.69	1.359	8.1	5.9	39.1	11.5
Silty clay	30.1–37.2	26.9	2.72	0.340	13.6	43.7	19.6	9.0
Clayey silt	37.2–39.8	29.1	2.71	1.069	8.8	10.9	29.8	6.1
Silty clay	39.8–50.0	26.2	2.72	0.350	13.5	45.9	20.6	10.5

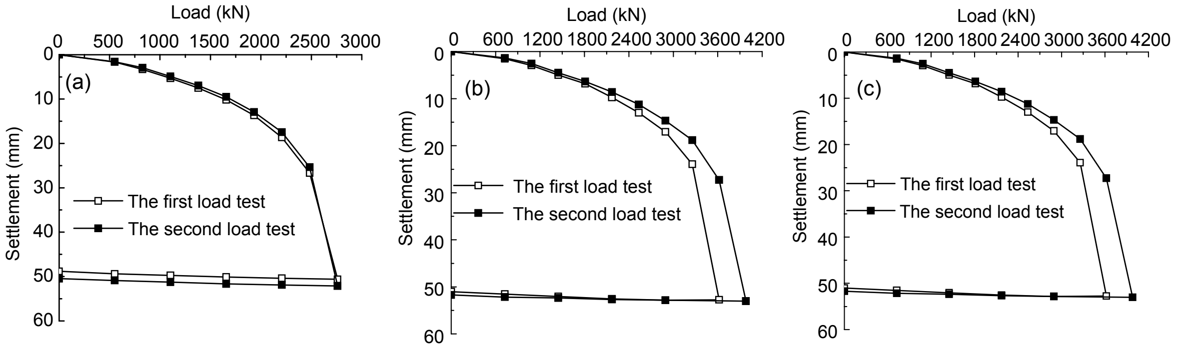

Fig. 4 Load-movement curves derived from the static loading tests
(a) P1; (b) P2; (c) P3

were added on a reaction system for assuring sufficient load reaction. Although the load-movement curve is modified and the influence is minimized in this test, the set-up process of the pile may have been changed to some extent. The different increase rate for the three curves could reasonably be due to the different lengths for the test piles. This is because set-up is mainly attributed to the increase in shaft resistance. Hence, the ratio of the shaft resistance to the base resistance plays an important role in set-up development.

Based on Skov and Denver (1988)'s relationship (Eq. (5) in Table 1), the normalized total pile capacity versus the logarithm of time is plotted in Fig. 6. Here the re-jacking resistance is used as the reference capacity (Q_0), (i.e., t_0=0.5). Although the scatter was relatively large, the three curves all display a distinct increase over time which is approximately linearly with the logarithm of normalized time. Average *A* factor according to Skov and Denver (1988)'s relationship is 0.19, 0.27 and 0.15 for P1, P2 and P3, respectively. The recorded increase in total resistance in this study is relatively small in comparison with other observations shown in Fig. 2. The pile installation method is possibly responsible for this as jacking installation generally creates soil disturbance much less than percussive methods.

The increase in pile capacity, as discussed above, is predominately derived from the increase in shaft resistance. Hence, it is of interest to separate the pile capacity into the shaft resistance (Q_s) and the base resistance (Q_b). For open-ended pipe pile the base resistance is composed of the annular resistance (Q_{ann}) and plug resistance (Q_p), which can be estimated from CPT cone tip resistance (q_c) and the incremental fill ratio (IFR) of soil plug. The ratio of unit annular resistance, q_{ann}, to q_c has been found to be independent of the penetration depth and the IFR (e.g., Lehane and Gavin, 2001; Doherty *et al.*, 2010). Hence, a value of $0.8q_c$ is adopted here for unit annular resistance (q_{ann}) based on the field observations (Doherty *et al.*, 2010), and the unit plug resistance (q_{plug}) can be evaluated through the relationship:

$$q_{plug} = (0.8 - 0.7\text{IFR})q_c \geq 0.1q_c. \qquad (8)$$

The observed developments of soil level inside the pile reveal that all the three piles achieved the

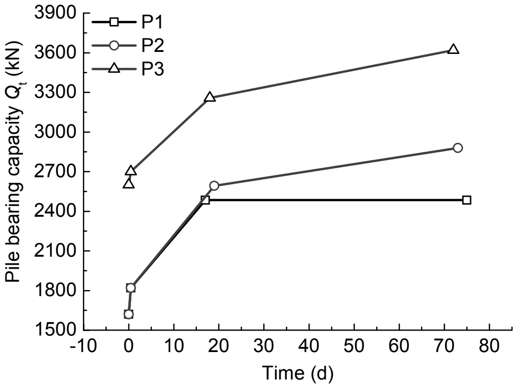

Fig. 5 Increase in total pile capacity with time

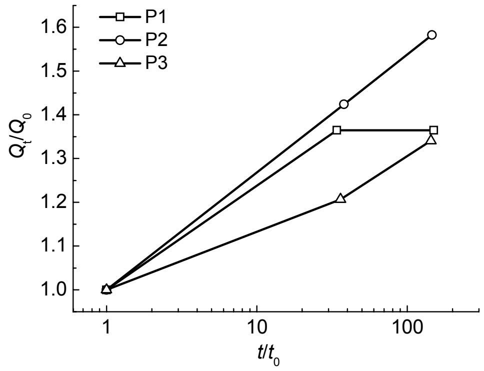

Fig. 6 Relative increase in pile total capacity with log-time

Table 3 Results of the jacking, re-jacking and load tests

Test	P1				P2				P3			
	Rest time (h)	Total resistance (kN)	Shaft resistance (kN)	Base resistance (kN)	Rest time (h)	Total resistance (kN)	Shaft resistance (kN)	Base resistance (kN)	Rest time (h)	Total resistance (kN)	Shaft resistance (kN)	Base resistance (kN)
End of jacking	0	1620	603	1017	0	1620	602	1018	0	2600	1357	1243
Re-jacking	0.5	1820	803	1017	0.5	1820	802	1018	0.5	2700	1457	1243
First load test	408	2484	1467	1017	456	2592	1576	1018	432	3258	2015	1243
Second load test	1800	2484	1467	1017	1752	2592	1862	1018	1728	3620	2377	1243

fully plugged mode during the re-jacking procedure. Furthermore, in most cases piles are found in a plugged mode under static loading (Randolph *et al.*, 1991), thus it is reasonable to assume that the three piles remain in the plugged condition during static loading tests, although no measurement was undertaken in the tests. Then the value of q_{plug} can be taken to be equal to $0.8q_c$ corresponding to the fully plugged (IFR=0) according to Eq. (8). This is close to the value 0.83 obtained from the field measurements by Axelsson (2000), indicating that the above analysis has provided reasonable predictions.

Assuming the base resistance remained constant after installation, the derived unit shaft resistance is plotted against the time in Fig. 7. All of the data points exhibit similar increase trend and appear to be an approximately logarithmic distribution. The normalized pile shaft resistance versus the logarithm of the normalized time is plotted in Fig. 8, where q_{st} is the average unit shaft resistance at time t, and q_{s0} is the initial average unit shaft resistance. The average long-term set-up on the shaft resistance is 44% per log cycle, which is approximately 108% larger than the value for total resistance. All the above data have also been presented in Table 3, including the total, shaft and base resistance as well as the rest time for each test.

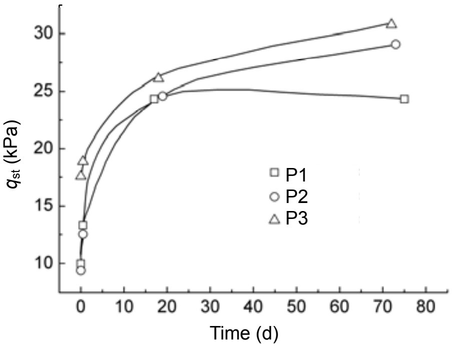

Fig. 7 Increase in average unit shaft resistance with time

5 Conclusions

A comprehensive study on the pile set-up effect has been reported and the following conclusions could be drawn.

1. The set-up on the total pile capacity is in the range of 15% to 27% per log cycle, which is approximately linear with the logarithm of normalized time. The set-up of pile resistance is predominately from the increase of shaft resistance. The increase in the average unit shaft resistance is also linear with respect to logarithmic time, but its increase rate is approximately 108% larger than that for the total resistance.

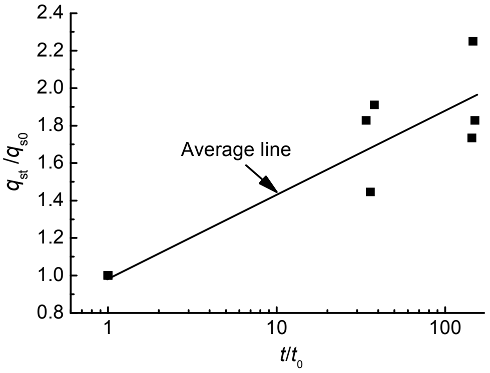

Fig. 8 Relative increase in average unit shaft resistance with log-time

2. An idealized mechanism explaining the pile capacity set-up mechanisms is presented. The complete set-up process is divided into three phases which are likely to overlap. For cohesive soils the majority of set-up is related to the dissipation of excess pore pressure. For granular soils, however, set-up is predominately associated with ageing effect.

3. A database of case histories for long-term set-up was compiled, and the set-up rate per log cycle is normally in the region of 15% to 65% with an average of approximately 40%. In addition, empirical equations for estimating the set-up effect have been summarised. The logarithmic relationship proposed by Skov and Denver (1988) has been commonly adopted.

Acknowledgements

The authors would like to acknowledge the technical staff in the Key Laboratory of Soft Soil and Geoenvironmental Engineering at Zhejiang University, China.

References

Astedt, B., Weiner, L., Holm, G., 1992. Increase in Bearing Capacity with Time for Friction Piles in Silt and Sand. Proceedings of Nordic Geotechnical Meeting, p.411-416.

Axelsson, G., 1993. Pile Test Report for the Bridge over Pite

River at Bole. Skanska Teknik AB, Danderyd, Sweden.

Axelsson, G., 1994. Pile Test Report for the Varby Bridge. Skanska Teknik AB, Danderyd, Sweden.

Axelsson, G., 1998. Long-Term Set-up of Driven Piles in Sand Evaluated from Dynamic Tests on Penetration Rods. Proceedings of the 1st International Conference on Site Characterization, Atlanta, **2**:895-900.

Axelsson, G., 2000. Long-Term Set-up of Driven Piles in Non-Cohesive Soils. PhD Thesis, Royal Institute of Technology, Stockholm.

Axelsson, G., 2002. A conceptual model of pile set-up for driven piles in non-cohesive soil. *Geotechnical Special Publication*, **116**(1):64-79. [doi:10.1061/40601(256)6]

Bogard, J.D., Matlock, H., 1990. Application of Model Pile Tests to Axial Pile Design. 22nd Annual Offshore Technology Conference, Houston, **3**:271-278.

Bullock, P.J., Schmertmann, J.H., McVay, M.C., Townsend, F.C., 2005a. Side shear setup. I: Test piles driven in Florida. *Journal of Geotechnical and Geoenvironmental Engineering, ASCE*, **131**(3):292-300. [doi:10.1061/(ASCE) 1090-0241(2005)131:3(292)]

Bullock, P.J., Schmertmann, J.H., McVay, M.C., Townsend, F.C., 2005b. Side shear setup. II: Results from Florida test piles. *Journal of Geotechnical and Geoenvironmental Engineering, ASCE*, **131**(3):301-310. [doi:10.1061/(ASCE)1090-0241(2005)131:3(301)]

Camp III, W.M., Parmar, H.S., 1999. Characterization of pile capacity with time in the cooper marl: study of application of a past approach to predict long-term pile capacity. *Journal of the Transportation Research Board*, **1663**(1): 16-24. [doi:10.3141/1663-03]

Chow, F.C., Jardine, R.J., Naroy, J.F., Brucy, F., 1998. Effects of time on capacity of pipe piles in dense marine sand. *Journal of Geotechnical & Geoenvironmental Engineering, ASCE*, **124**(3):254-264. [doi:10.1061/(ASCE) 1090-0241(1998)124:3(254)]

Doherty, P., Gavin, K., Gallagher, D. 2010. Field Investigation of the Undrained Base Resistance of Pipe Piles in Clay. Proceeding of the ICE Geotechnical Engineering, **163**(1): 13-22. [doi:10.1680/geng.2010.163.1.13]

Eriksson, H., 1992. Behaviour of Driven Piles Evaluated from Stress Wave Measurements Performed during Dynamic Probing. PhD Thesis, Royal Institute of Technology, Stockholm.

Fellenius, B.H., Riker, R.E., O'Brian, A.J., Tracy, G.R., 1989. Dynamic and static testing in soils exhibiting set-up. *Journal of Geotechnical & Geoenvironmental Engineering, ASCE*, **115**(7):984-1001. [doi:10.1061/(ASCE) 0733-9410(1989)115:7(984)]

Fellenius, B.H., Edde, R.D., Beriault, L.L., 1992. Dynamic and Static Testing for Pile Capacity in a Fine-Grained Soil. Proceedings of the 4th International Conference Application Stress-Wave Theory to Piles, Hague, the Netherlands, p.401-408.

Flaate, K., 1972. Effects of pile driving in clays. *Canadian Geotechnical Journal*, **9**(1):81-88.

Huang, S., 1988. Application of Dynamic Measurement on Long H-Pile Driven into Soft Ground in Shanghai. Proceedings of the 3rd International Conference on the Application of Stress-Wave Theory to Piles, Ottawa, Ontario, Canada, p.635-643.

Hunt, S.W., Baker, C.N., 1988. Use of Stress-Wave Measurements to Evaluate Piles in High Set-up Conditions. Proceedings of the 3rd International Conference on the Application of Stress-Wave Theory to Piles, Ottawa, p.689-705.

JGJ106-2003. China Academy of Building Research. Chinese Technical Code for Testing of Building Foundation. China Architecture and Building Press, Beijing, China, (in Chinese).

Komurka, V.E., Wagner, A.B., Edil, T.B., 2003. Estimating Soil/Pile Set-up. Wisconsin Department of Transportation, USA WHRP Report No. 03-05.

Lehane, B.M., Gavin, K.G., 2001. Base resistance of jacked pipe piles in sand. *Journal of Geotechnical and Geoenvironmental Engineering, ASCE*, **127**(6):473-480. [doi:10.1061/(ASCE) 1090-0241(2001)127:6(473)]

Long, J.H., Bozkurt, D., Kerrigan, J.A., Wysockey, M.H., 1999. Value of methods for predicting axial pile capacity. *Journal of the Transportation Research Board*, **1663**(1): 57-63. [doi:10.3141/1663-08]

Moe, D., Arvesen, H., Holm, O.S., 1981. Friction bearing pile piles at Calabar Port. Proceedings of the 10th International Conference Soil Mechanics and Foundation Engineering, Stockholm, **2**:781-786.

Preim, M.J., March, R., Hussein, M., 1989. Bearing Capacity of Piles in Soils with Time Dependent Characteristics. Proceedings of the 3rd International Conference on Piling and Deep Foundations, Brookfield, p.363-370.

Randolph, M.F., 2003. Science and empiricism in pile foundation design. *Geotechnique*, **53**(10):847-875. [doi:10.1680/geot.2003.53.10.847]

Randolph, M.F., Carter, J.P., Wroth, C.P., 1979. Driven piles in clay—the effects of installation and subsequent consolidation. *Geotechnique*, **29**(4):361-393. [doi:10.1680/geot.1979.29.4.361]

Randolph, M.F., Leong, E.C., Houlsby, G.T., 1991. One-dimensional analysis of soil plugs in pipe piles. *Geotechnique*, **41**(4):587-598. [doi:10.1680/geot.1991.41.4.587]

Samson, L., Authier, J., 1986. Change in pile capacity with time: Case histories. *Canadian Geotechnical Journal*, **23**(2):174-180. [doi:10.1139/t86-027]

Seidel, J.P., Haustorfer, I.J., Plesiotis, S., 1988. Comparison of Dynamic and Static Testing for Piles Founded into Limestone. Proceedings of the 3rd International Conference on the Application of Stress-Wave Theory to Piles, Ottawa, p.717-723.

Skov, R., Denver, H., 1988. Time-Dependence of Bearing Capacity of Piles. Proceedings of the 3rd International Conference on the Application of Stress-Wave Theory to Piles, Vancouver, BC, p.879-888.

Soderberg, L.O., 1961. Consolidation theory applied to foundation pile time effects. *Geotechnique*, **11**(3):217-225.

[doi:10.1680/geot.1962.12.3.217]

Svinkin, M.R., Morgano, C.M., Morvant, M., 1994. Pile Capacity as a Function of Time in Clayey and Sandy Soils. Proceedings of the 5th International Conference and Exhibition on Piling and Deep Foundations, Belgium, p.1.11.1-1.11.8.

Tan, S.L., Cuthbertson, J., Kimmerling, R.E., 2004. Prediction of pile set-up in non-cohesive soils. *Geotechnical Special Publication*, **125**:50-65. [doi:10.1061/40743 (142)3]

Tavenas, F., Audy, R., 1972. Limitations of the driving formulas for predicting bearing capacities of piles in sand. *Canadian Geotechnical Journal*, **9**(1):47-62. [doi:10.1139/t72-004]

Thomann, T.G., Hryciw, R.D., 1992. Stiffness and strength changes in cohesionless soils due to disturbance. *Canadian Geotechnical Journal*, **29**(5):853-861. [doi:10.1139/t92-092]

Wendel, E., 1900. On the Test Loading of Piles and Its Application to Foundation Problems in Gothenburg. Tekniska Samf Goteberg handl, **7**:3-62.

Wong, P., 1988. The Use of Pile Driving Analyzer at a Hospital Complex in Malasya. Proceedings of the 3rd International Conference on the Application of Stress-Wave Theory to Piles, Ottawa, p.762-770.

York, D.L., Brusey, G.B., Clemente, F.M., Law, S.K., 1995. Set-up and relaxation in glacial sand. *International Journal of Rock Mechanics and Mining Sciences and Geomechanics Abstracts*, **32**(6):265A-265A(1).

Zhang, M.Y., Liu, J.W., Yu, X.X., 2009. Field test study of time effect on ultimate bearing capacity of jacked pipe pile in soft clay. *Rock and Soil Mechanics*, **30**(10):3005-3008 (in Chinese).

Zhu, G.Y., 1988. Wave Equation Applications for Piles in Soft Ground. Proceedings of the 3rd International Conference on the Application of Stress-Wave Theory to Piles, Ottawa, Ontario, Canada, p.831-836.