IOP Conf. Series: Materials Science and Engineering **394** (2018) 032097 doi:10.1088/1757-899X/394/3/032097

Determination of Bearing Capacity of Single Pile based on Seismic Piezocone Penetration Tests

Quan Cao[1, a], Shuhua Gong[1, b], Peigang Li[1, c], Heng Wan[1, d], Daolai Cheng[2, e]

[1]School of Railway Transportation, Shanghai Institute of Technology, Shanghai, 201418, China
[2]School of mechanical engineering, Shanghai Institute of Technology, Shanghai, 201418, China

[a]caoquanrang@126.com, [b]ggg05141226@163.com, [c]lipeigang@sit.edu.cn, [d]hengwan@sit.edu.cn, [e]daolaicheng@163.com

Abstract. Based on the similar characteristics of installation about the penetrometer of seismic piezocone penetration tests(SCPTu) and the single pile, a theoretical relationship between ultimate bearing capacity of single pile, time-effect of shaft bearing capacity of single pile, excess pore water pressure around the pile during pile driven and the data measured from SCPTu is developed according to the cavity expanded theory, the Terzaghi one-dimensional consolidation theory and effective stress theory. The result of field test in KunShan and the calculated result which used the theoretical relationship mentioned above are compared. The results indicate that the analytical solutions agree well with the in-situ tests, which show that the applications of seismic piezocone penetration tests have wide range in the design of pile foundation.

1. Introduction

Nowadays, there are many semi-empirical design methods linking the shaft capacity developed by jacked piles to the cone penetration test [1-8], and most of them are empirical regression formulas based on measured data [9-10]. However, applying any empirical formulation has a significant drawback, which is the extension of such methods to design situations which are outside the scope of databased used to drive the approach.

Because of their simplicity, empirical approaches remain popular in practice. However, the empirical parameters linking shaft resistance to the undrained strength of the soil are affected by multiple factors and the treatment of important factors of pile behavior. So analytical approaches as Cavity Expansive Theory offer considerable promise, even if this theoretical expression is not perfect [11-12].

The seismic piezocone penetration tests (SCPTu) provide the profiles of four independent readings with depth (tip resistance, shaft resistance shoulder power water pressure, and shear wave velocity), and it also possible to record the dissipation of the excess pore water pressure at the corresponding position after the penetration of the probe is stopped, which are employed to obtain pile capacity.

As discussed above, the theory of small cavity expansion, one-dimensional consolidation theory and effective stress principle are used to deduced the theoretical relation between the bearing capacity characteristics of jacked pile and the dates proposed by the SCPTu in the same theoretical framework.

2. Analytical solution of column cavity expansion theory in soft soil

Based on cavity expansion theory and one-dimensional consolidation theory and effective stress principle, some theoretical correlation equations are deduced.

(1) Excess pore water pressure caused by column cavity expansion

$$\Delta u=\frac{2c_u}{r_p-r_i}\{\left[(1-\beta)r_p-r_i\right]\ln\frac{r_p}{r}+\frac{3}{2}\beta\left(r_p-r\right)-\frac{1}{2}\left(r_p-r_i\right)\}+c_u+ \\ \left(1.733A_f-0.578\right)\left(1-\frac{r_p-r}{r_p-r_i}\beta\right)c_u \tag{1}$$

(2) Analytical formula for calculating the ultimate bearing capacity of the pile based on SCPTu

$$Q_s=\sum_{i=1}^{n}\frac{\dfrac{2c_{ui}}{\dfrac{r_{pc2i}}{b}-1}\left\{\left[(1-\beta)\dfrac{r_{pc2i}}{b}-1\right]\ln(\dfrac{r_{pc2i}}{b})+\beta(\dfrac{r_{pc2i}}{b}-1)\right\}+c_{ui}+p_{0i}}{\dfrac{2c_{ui}}{\dfrac{r_{pc1i}}{a}-1}\left\{\left[(1-\beta)\dfrac{r_{pc1i}}{a}-1\right]\ln(\dfrac{r_{pc1i}}{a})+\beta(\dfrac{r_{pc1i}}{a}-1)\right\}+c_{ui}+p_{0i}}\frac{tg\delta_{2i}}{tg\delta_{1i}}2\pi bl_i f_{si} \tag{2}$$

(3) Theoretical solution for calculating time-effect of bearing capacity of single pile

$$Q_{s(t)}= \\ \sum_{i=1}^{n}\frac{\dfrac{2c_{ui}}{\dfrac{r_{pc2i}}{b}-1}\left\{\left[(1-\beta)\dfrac{r_{pc2i}}{b}-1\right]\ln(\dfrac{r_{pc2i}}{b})+\beta(\dfrac{r_{pc2i}}{b}-1)\right\}+c_{ui}+p_{0i}}{\dfrac{2c_{ui}}{\dfrac{r_{pc1i}}{a}-1}\left\{\left[(1-\beta)\dfrac{r_{pc1i}}{a}-1\right]\ln(\dfrac{r_{pc1i}}{a})+\beta(\dfrac{r_{pc1i}}{a}-1)\right\}+c_{ui}+p_{0i}}\frac{tg\delta_{2i}}{tg\delta_{1i}}2\pi bl_i f_{si} \\ -4\pi b\sum_{i=1}^{n}tg\delta_{2i}\left\{\frac{l_i c_{ui}J_0\left(\mu_n^{(0)}\rho_0\right)}{\left(\mu_n^{(0)}\right)^2 J_1^2\left(\mu_n^{(0)}\right)}e^{\frac{-0.04c_{hi}\left(\mu_n^{(0)}\right)^2 t}{\left(r_{pi}\right)^2}}\sum_{n=1}^{\infty}\left[0.2A_iJ_1\left(0.2\mu_n^{(0)}\right)++1.61D\left(\mu_n^{(0)}\right)^2\right]\right. \\ +\frac{l_i c_{ui}J_0\left(\mu_n^{(0)}\rho_0\right)}{\left(\mu_n^{(0)}\right)^2 J_1^2\left(\mu_n^{(0)}\right)}e^{\frac{-0.04c_{hi}\left(\mu_n^{(0)}\right)^2 t}{\left(r_{pi}\right)^2}}\sum_{n=1}^{\infty}\left[B_i\left(J_0\left(0.2\mu_n^{(0)}\right)-0.32\mu_n^{(0)}J_1\left(0.2\mu_n^{(0)}\right)-1\right)\right] \\ +\frac{l_i c_{ui}J_0\left(\mu_n^{(0)}\rho_0\right)}{\left(\mu_n^{(0)}\right)^2 J_1^2\left(\mu_n^{(0)}\right)}e^{\frac{-0.04c_{hi}\left(\mu_n^{(0)}\right)^2 t}{\left(r_{pi}\right)^2}}\sum_{n=1}^{\infty}\left[0.33C_i\left(\mu_n^{(0)}\right)^5{}_1F_2\left(\frac{3}{2};1,\frac{5}{2};-0.01\left(\mu_n^{(0)}\right)^2\right)\right] \\ +\frac{l_i c_{ui}J_0\left(\mu_n^{(0)}\rho_0\right)}{\left(\mu_n^{(0)}\right)^2 J_1^2\left(\mu_n^{(0)}\right)}e^{\frac{-0.04c_{hi}\left(\mu_n^{(0)}\right)^2 t}{\left(r_{pi}\right)^2}}\sum_{n=1}^{\infty}\left[0.005D_i\left(\mu_n^{(0)}\right)^4{}_2F_3\left(1,1;2,2,2;-0.01\left(\mu_n^{(0)}\right)^2\right)\right] \\ \left.-\frac{l_i c_{ui}J_0\left(\mu_n^{(0)}\rho_0\right)}{\left(\mu_n^{(0)}\right)^2 J_1^2\left(\mu_n^{(0)}\right)}e^{\frac{-0.04c_{hi}\left(\mu_n^{(0)}\right)^2 t}{\left(r_{pi}\right)^2}}\sum_{n=1}^{\infty}\left[0.12D_i\left(\mu_n^{(0)}\right)^4{}_2F_3\left(1,1;2,2,2;-0.25\left(\mu_n^{(0)}\right)^2\right)\right]\right\} \tag{3}$$

In the above formula, r_p is the radius of the plastic zone, r_i is the radius of the current cylinder. $\frac{r_p}{r_i}=\frac{2I_r}{\sqrt{4I_r-1}}$, I_r is the stiffness index and it can be calculated from shear wave speed of SCPTu that $I_r=\frac{\gamma V_s^2}{C_u g}$, γ is the soil specific weigh, β is the softening coefficient. l_i is the i layer thickness of pile side, f_{si} is the i layer of soil cone shaft friction, c_{ui} is the pile side i layer of soil non - row shear strength, p_{0i} is the initial soil stress of fourth layers of pile side. r_{pc1i} and r_{pc2i} are the radius of plastic zone caused by penetration of cone probe and pile in pile side soil. δ_{1i} and δ_{2i} are friction parameters of pile side first layer soil and conical probe wall and pile body contact surface respectively a is the radius of the cone probe, and b is the radius of the pile.c_{hi} is the horizontal consolidation coefficient of the i soil layer. It can be obtained from the pore pressure dissipation test of the piezocone. $J_0(\mu_n^{(0)})$ and $J_1(\mu_n^{(0)})$ are zero order and first order Bessel functions respectively. $\mu_n^{(0)}$ Represents the positive zero of $J_0(\rho)$. In addition, $A=\left(\frac{3.8r_i-0.4r_p+0.87A_f r_p-1.73A_f r_i}{r_p-r_i}\right)$, B= $B=\left(\frac{(4.43A_f-1.45)r_p}{r_p-r_i}\right)$, $C=\left(\frac{r_p-2r_i}{r_p-r_i}\right)$, $D=(0.069A_f-0.023)$

3. Example analysis and verification

3.1. General situation of engineering geology in test site

The test site is located in Huaqiao Town, Kunshan City. The terrain is relatively flat and belongs to the alluvial plain of the Yangtze River Delta. The average values of soil types and main physical and mechanical indexes are shown in Table 1.

Table 1. Physical and mechanical properties of soil layer in test site.

Layer Number	Layer Thickness	Specific Weigh	Pore Pressure Coefficient	Three Axis(UU) Cohesive Force	Three Axis(CD) Friction Angle	Horizontal Consolidation Coefficient	Compression Modulus	Shear Wave Velocity
	(m)	γ(kN/m³)	A_f	c_u(kPa)	φ'(°)	c_h(10^{-3}cm²/s)	E_{s1-2}(MPa)	v_s(m/s)
①	0.5	17.8						
②	0.9	18.5	1.0	19.0	13.2	7.17	4.69	115.0
③	3.0	18.0	0.9	16.4	10.1	2.19-3.62	3.63	103.4
④	3.8	18.4	0.9	22.6	15.4	6.48-8.97	3.78	137.8
⑤	3.1	17.8	0.8	8.9	12.4	4.13-6.51	2.73	143.3
⑥	12.3	18.0	1.0	15.2	19.5	3.56-1255	5.11	145.8
⑦	6.0	18.1	0.9	13.0	12.6	5.24	3.56	145.0

By comprehensive comparison and selection, the loading test of S2 test piles at different intervals in the field, the latest hq-1 SCPTu results from the S2 test piles and their adjacent on-site monitoring values were selected to verify the rationality of the above theoretical solution.

S2 test pile is a friction pile, the model is PC-A-500-100-15, the concrete strength grade of the pile is C60, the outer diameter is d=500mm, the wall thickness is 100mm, the length of a single section is 15m, and the designed pile length is L=30m. The construction is divided into two sections (15m/section) and the static pile construction technology is adopted. A total of four intermittent load tests were performed on the S2 pile.

3.2. *Estimation of excess pore water pressure in pile surrounding soil during pile installation*

The stiffness indexes at depths of 5m (4th), 12m (5th) and 30m (7th) are calculated to be 55.8, 102.2 and 101.7 from the shear wave velocity measured by the hq-1 hole SCPTu. The relevant soil parameters in Table 1 are substituted into equation (1) respectively, and the calculated results are compared with the pore pressure monitoring data at the corresponding positions. It can be seen from Figure 1 that there are some differences between the prediction depth and the on-site monitoring value of the hole depths of 5 meters and 30 meters near the pile wall, but as the distance increases, the difference gradually becomes smaller and tends to be consistent. However, as the distance increases, the difference gradually becomes smaller and tends to be consistent. The prediction depth of the 12-meter hole depth is in line with the on-site monitoring value. The reason for the above phenomenon is that the soil layers at the depths of 5m and 30m are muddy silt clay sandwich silt and silt clay sandwich thin silt soil. The silt soil in the cohesive soil has changed the nature of the soil, which is slightly different from the soft clay in the theoretical solution of this study. Therefore, the difference between the theoretical value and the measured value appears. The 12m deep hole is muddy silt clay, which is completely consistent with the theoretical solution of the research object. Therefore, the predicted value of the treatment theory is in good agreement with the on-site monitoring value, indicating that the theoretical solution of excess pore water pressure caused by column cavity expansion in equation (1) has certain rationality.

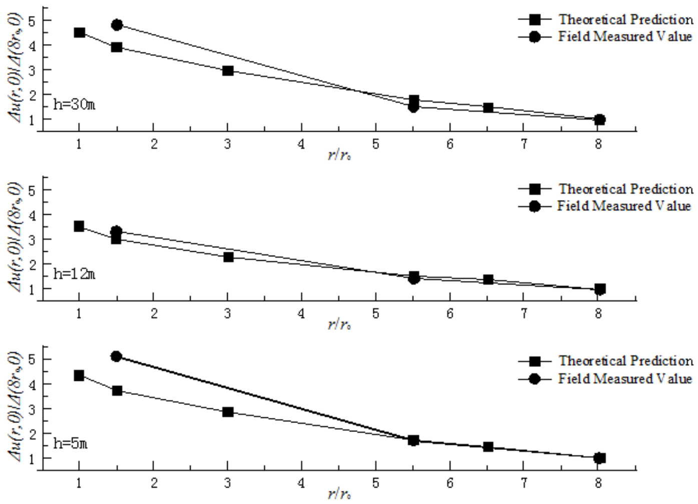

Fig. 1 Comparison between theoretical and measured values of maximum excess pore water pressure in pile surrounding soil at the end of pile installation

3.3. *Verify the ultimate bearing capacity of single pile*

Substituting the relevant soil parameters in Table 1 into equation (2), the ultimate bearing capacity of a single pile is calculated to be 2650 kN and the calculated value is 10% larger than the measured value. Two main reasons are contributed to the above phenomenon. One is that the equation (2) is based on the pore expansion theory in soft soils, many silt soil and sand interlayers are sandwiched in the soil layer at the site, which affects the strength index of the soil and leads to a large calculated value. The other is that the calculated value of 2838kN in equation (2) is the limit value for the assumption of complete consolidation of the soil around the pile, and 2420kN is the static load test value for the S2 pile after 146 days after the completion of the pile installation. At this time, the soil around the pile may not be completely consolidated. The time-effect of the bearing capacity of the pile is not yet completed, so this value is not the maximum value of the bearing capacity. In short, the ultimate bearing capacity of a single pile calculated from equation (2) is very close to the static load test value of a single pile, which shows that the theoretical relationship can well predict the ultimate bearing capacity of a single pile in soft soil and has certain engineering practical value.

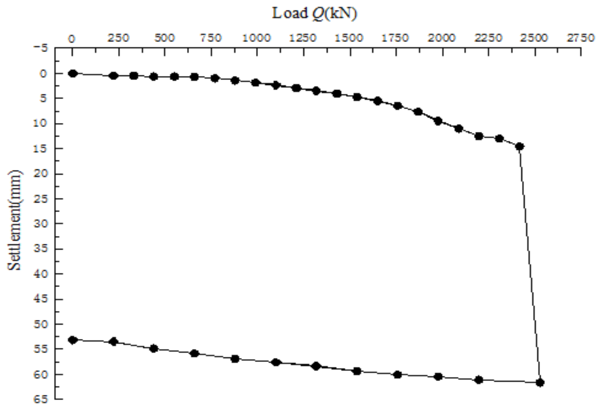

Fig. 2 Q~s curve of the static load test of the S2 test pile

3.4. *Verification of time effectiveness of single pile bearing capacity*

Substituting the hq-1 hole SCPTu results and the relevant soil parameters in Table 1 into the theoretical solution of the time-effect of the bearing capacity of the pile in equation (3). It can be seen from Figure 3 that both the theoretical curve and the measured curve of the lateral load bearing capacity of the pile show time-effect, and it increases quickly in the first 60 days after piled up, and then increases slowly. The consolidation was basically completed after three months and the lateral load capacity of the pile reached the maximum. The theoretical curve is in good agreement with the measured curve, but only in the early stage of dissipation, the theoretical value is smaller than the measured value. The main reason is that because the test site contains water-permeable silt and fine sand layers, the consolidation speed of the soil layer is accelerated. The theoretical formula uses soft clay as the research object, without considering the impact of the actual sand interlayer, so the calculated value is smaller than the measured value. With the consolidation, the two tend to be consistent, indicating that the theoretical formula has a certain degree of practicality.

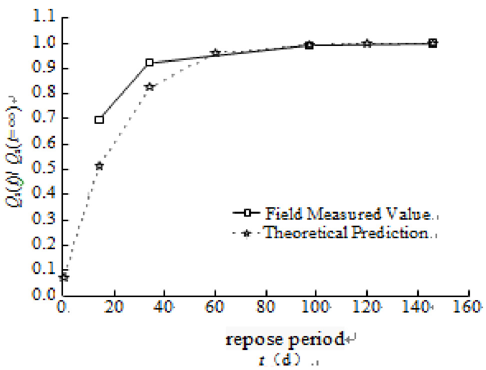

Fig. 3 Comparison between theoretical value and measured value of time-effect of jacked pile bearing capacity

4. Conclusions

From the above theoretical analysis and comparison with the results of field tests, we can draw the following conclusions.

(1) By using the function of SCPTu test soil shear wave velocity, the stiffness index of the soil layer can be determined, which lays the foundation for estimating the pore water pressure caused by the installation in soft soil.

(2) The formula for calculating the ultimate bearing capacity of a single pile in soft soil based on SCPTu not only makes full use of SCPTu test parameters such as tip resistance, shaft friction and pore water pressure, initial shear modulus, it also considers the effect of the pile and cone dimensions (a, b) and the effective friction angle $\emptyset'$, the undrained shear strength c_u, and the strain softening coefficient β of the soil as it traverses different soil layers. It is more reasonable and comprehensive than traditional in-situ test methods and empirical methods.

(3) Based on SCPTu, the formula for calculating the time-effect of bearing capacity of single pile in soft soil not only has a strict theoretical basis, but also fully utilizes the pore pressure dissipation test results of pizocone, which is a perfect combination of pile foundation design and in-situ test.

(4) Based on the above theoretical analysis and engineering example verification, it shows that SCPTu has important application value in pile foundation engineering.

Acknowledgments

This work was financially supported by Shanghai science and Technology Commission Local Colleges' Capacity building Project (1709050350), Shanghai Institute of Technology Collaborative Innovation Fund (XTCX2016-21) and The Key points of Graduation Project in Shanghai Institute of Technology. Construction Project (3911LW180037).

References

[1] Abu-Farsakh, M.Y., Titl, H.H. Assessment of direct cone penetration test methods for predicting the ultimate capacity of friction driven piles [J]. Journal of Geotechnical and Geoenvironmental Engineering, 2004, 130 (9): 935-944.

[2] Cudmani, R., Osinov, V.A. The cavity expansion problem for the interpretation of cone penetration and pressuremeter tests [J]. Canadian Geotechnical Journal, 2001, 38(6):622-638.

[3] White, D.J. Field measurements of CPT and pile base resistance in sand [R]. CUED/D-SOILS/TR327, 2003.

[4] Roy, M., Tremblay, M., Tavenas, F., Rochelle, P.L. Development of pore pressures in quasi-static penetration tests in sensitive clay [J]. Canadian Geotechnical Journal, 1982, 19(1):124-138.

[5] Brown, R.P. Predicting the ultimate axial resistance of single driven piles [Ph.D.Thesis]. Austin: University of Texas, 2001.

[6] Konard, J.M. Piezo-friction-cone penetrometer testing in soft clays[J]. Canadian Geotechnical Journal, 1987, 24 (4): 645-652.

[7] Watt, W.G., Kurfurst, P.J., Zeman, Z.P. Comparison of pile load-test-skin-friction values and laboratory strength tests [J]. Canadian Geotechnical Journal, 1969, 3:339-352.

[8] Chandler, R.J., Martins, J.P. An experimental study of skin friction around piles in clay [J]. Géotechnique, 1982, 32 (2): 119-132.

[9] O Eide,J N Huutchinson and A Landva． Short and long term test loading of a friction pile in clay [C]． Pro． 5th． int． conf． soil． mech． fdn． engng,1961: 309-312．

[10] Fellenius B H,Brusey W G and Pepe F． Soil set-up,variable concrete modulus and residual load for tapered instrumented piles in sand [C]． ASCE,Speciality Conference on Performance Confirmation of Constructed Geotechnical Facilities, Univ． Of Massachusetts, Amherst, 2000: 98-114．

[11] Wroth, C.P. The interpretation of in situ soil test[J]. Géotechnique, 1984, 34 (4): 449-489.

[12] Fernando Schnaid. Geo-characteristation and properities of natural soils by in situ tests [J]. Proceedings, 16th International Conference on Soil Mechanics and Geotechnical Engineering, Bali, Japan, 2001: 3-45.