Kolodiy, E., Vardanega, P., & Patel, D. C. (2015). Settlement prediction of bored piles in stiff clay at a site in the Moscow region. In M. G. Winter, D. M. Smith, P. J. L. Eldred, & D. G. Toll (Eds.), Geotechnical Engineering for Infrastructure and Development: Proceedings XVI European Conference on Soil Mechanics and Geotechnical Engineering. (Vol. 6, pp. 3619-3624). London: ICE Publishing.

Proceedings of the XVI ECSMGE
Geotechnical Engineering for Infrastructure and Development
ISBN 978-0-7277-6067-8

doi:10.1680/ecsmge.60678

Settlement prediction of bored piles in stiff clay at a site in the Moscow region

Prédiction des tassements des pieux forés in situ dans une argile raide dans la région de Moscou

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ABSTRACT Geotechnical Engineers should devote a considerable amount of time to understanding foundation settlements. When designing in soil deposits that are unfamiliar, pile load testing is essential for investigating likely foundation movements. Simple models to compute pile settlement are also useful for design engineers, especially in soil deposits that are not widely studied or reported. Pile load test data is presented along with relevant site investigation data for a site in the Jurassic clay in Moscow. This clay is of similar age to the Oxford clay found in England. The available site investigation data included Atterberg limit tests and CPT tests. Settlement analysis is carried out from the load testing of 20m long circular bored piles in this stiff over-consolidated clay. A simple MSD-style settlement model was used to backanalyse the results of the load testing conducted at this site. The backanalysis results are compared against the general methods in Russian codes of practice (SNiP/SP). The prediction of pile settlements in Russia generally assumes use of numerical modelling or emprical calculations given in Russian codes of practice, and this paper provides an alternative approach based on a semi-empirical method.

RÉSUMÉ Les ingénieurs géotechniciens doivent consacrer leur attention pour comprendre les tassements des fondations. Lors de la conception des fondations dans des sols qui ne sont pas familiers, des essais sur des pieux issolés – pieux d'essai - sont essentiels afin de characteriser leur comportement in situ. Des modèles simples pour l'éstimation des tassements des pieux peuvent également etre utiles en particulier pour le cas des sols qui ne sont pas largement étudiés ou connus. Des resultats sur des pieux d'essais et des investigations in situ dans un dépôt d'argile Jurassique à Moscou sont presentés dans ce papier. Cet argile est du même âge que l'argile d'Oxford trouvé en Angleterre. Les données disponibles ont inclus les limites d'Atterberg et des essais du CPT. L'analyse des tassements des pieux est effectué sur des pieux circulaires de 20 m de longueure, forés dans cette argile surconsolidé raide. Un modèle simple de tassement de type MSD a été utilisé pour confirmer les résultats des essais de charge effectués et aussi comparés à la procedure proposée par le code de conceptions des fondations russe (SNIP/SP). Les deux approches montrent un accord raisonnable avec les données sur les pieux d'essai. La prédiction des tassement des pieux en Russie est bassée généralement sur l'utilisation de modéls numériques ou des calculs empriques suivant les codes en vigueure, tandis que cet étude montre l'utilisation et les avantages d'une approche alternative basée sur une méthode semi-empirique.

1 INTRODUCTION

Estimating the settlement (performance) of piled foundations is useful for practicing geotechnical consultants. This paper uses the results of a recently published MSD-style method for pile settlement prediction (Vardanega et al. 2012b) that utilizes the mobilization strain framework (Vardanega & Bolton, 2011) to backanalyze pile load test data from a site in the Moscow region. The paper shows that the method produces results that are roughly comparable to the SNiP/SP method of pile settlement calculation.

2 SITE DESCRIPTION

The site is located in the Moscow region in the Russian Federation. The proposed development comprised several blocks of multi-storey mixed development buildings, which share a 12.5m deep (or 3 level) basement over the whole site. The foundation consists of 1200mm bored piles and a 1.0 to 2.0m thick raft.

3 GROUND CONDITION

The site topography is generally flat. The ground level varies between +157.5mMD and +158.5mMD. The site stratigraphy comprises made ground overlying stiff glacial clayey silt, medium dense fluvoglacial sand and stiff over-consolidated Jurassic clay. The clay is of similar age to the Oxford clay found in England. The Jurassic clay is underlain by Cretaceous limestone. All deposits are layered subhorizontally. The typical design stratigraphy is given in Table 1.

The basement extends into the fluvioglacial sands. It is founded on a raft bearing onto the fluvioglacial sands and piles extend through this layer and into the underlying Jurassic clays. The ground water was typically encountered at +150.0mMD, within the fluvioglacial sand.

Table 1. Site stratigraphy

Stratum	Elevation,top of stratum, (mMD)	Thickness (m)
Made Ground	+158.0	4.0
Glacial Clayey SILT	+154.0	5.5
Fluvioglacial SAND	+149.5	8.5
Jurassic CLAY	+141.0	14.0
Cretaceous Limestone	+127.0	Not proven

4 UNDRAINED SHEAR STRENGTH PROFILE

Examination of data of the variation of liquidity index (I_L) with depth for the site in question, see Figure 1, reveals little obvious trend of decrease with depth (decrease of I_L indicates an increase of c_u e.g. Wroth & Wood, 1978). Liquidity index data is necessary for the Russian design method for bored piles (SP20.13330.2011 and SP24.13330.2011). However, recent research has called into question the use of correlations for c_u from I_L where $I_L < 0.2$ as little published data is actually available to corroborate the empirical equations generally used (Vardanega & Haigh, 2014). Therefore, data from Cone Penetration Testing (CPT) was used to assess the c_u-profile for the site.

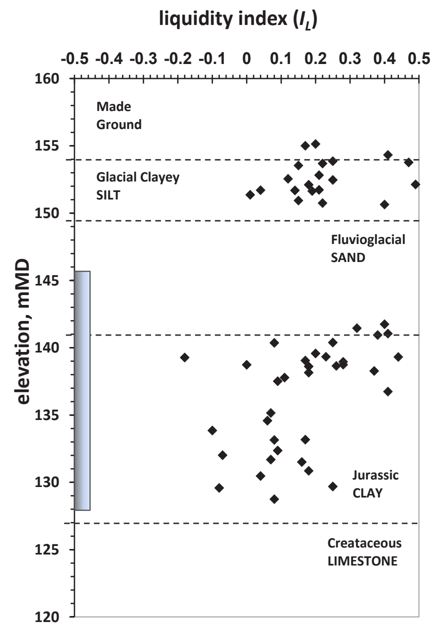


Figure 1. Variation of liquidity index with elevation

The CPT data collected on the site was used to assess the c_u-profile of the Jurassic clay. The undrained shear strength is assessed using equation 1 and the results are shown on Figure 2.

$$c_u = (q_c - \sigma_0) / N_k \quad (1)$$

Where, c_u = undrained shear strength (kPa); q_c = cone resistance, (kPa); σ_0 = total overburden pressure (kPa); N_k = empirical cone factor (taken as 16-18 for sites where little local experience is available, e.g. Robertson & Cabal, 2010 but taken as 20 for Moscow soil based on the local experience).

The CPT equipment was not able to penetrate the stiff Jurassic clay all the way to the pile toe level – data was only obtained for the first 8m of the pile. Evidence from Figures 1 and 2 suggests that there is minimal increase of c_u with depth down the pile shaft. Therefore, in this paper a constant c_u-profile (c_u= 155kPa) has been assumed.

5 PILE LOAD TESTS

5.1 Pile installation details

Two static load tests were performed on 1200mm, circular dry bored piles installed from existing ground level +158.0mMD to about +128.0mMD (Table 2).

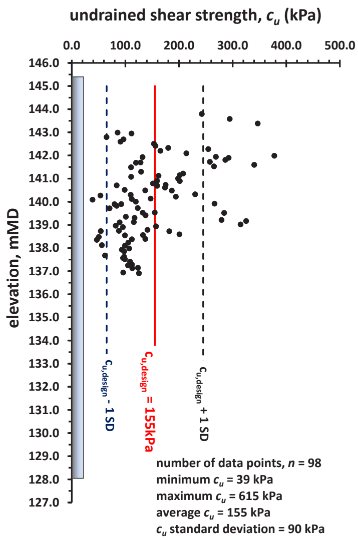


Figure 2. Variation of CPT undrained shear strength (c_u) with elevation

Table 2. Test Pile details

	Installation method	Size, mm	Top of concrete , mMD	Toe level, mMD
Pile 1	Temporary sleeved and dry bored	1200	+145.5	+127.8
Pile 2		1200	+145.5	+128.2

The pile installation sequence comprised the following:

- Installation of 1500mm dia. oversized temporary casing from ground level to +145.5mMD;
- Installation of a nominal 1200mm dia. casing into Jurassic clay to provide a seal;
- Boring 1200mm dia. pile below the casing to the required toe level (Table 2);
- Installation of reinforcement and concreting from toe level to +145.5mMD;
- Extracting 1200 mm casing.

After the test pile was constructed, a 1000mm diameter steel pile with the welded plate at the bottom was installed into the bore from ground level to +149.5mMD to be able to transfer the load from the ground level to the test pile at that level, Figure 3. The gap between the 1500mm shaft and the steel pile was backfilled with loose granular material to reduce the shaft friction, Figure 3.

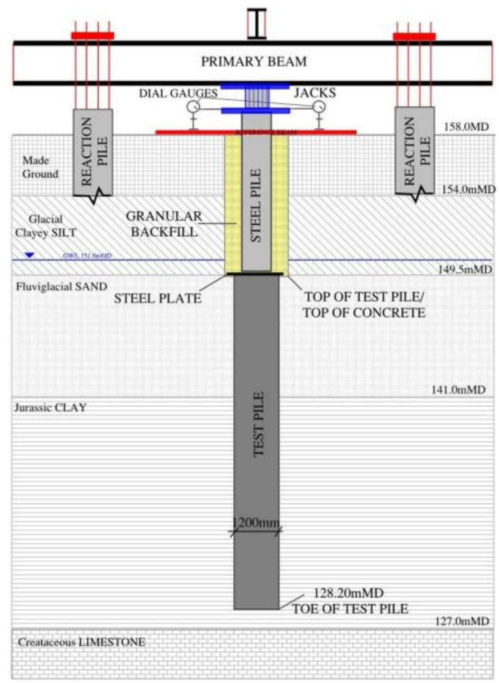


Figure 3. Pile test arrangement in section

5.2 Testing procedure and results

The test piles were loaded from the ground level by hydraulic jacks installed on the steel plate at the top of 1000mm steel pile. The load from the steel plate transferred into the steel pile and then via the plate at the bottom of the steel pile into the test pile. No instrumentation had been installed in the pile itself.

The reaction load was arranged through the four 1200mm anchor piles located side by side to the test pile, Figure 4.

Pile 1 was loaded in two cycles, and Pile 2 in one cycle. Each load increment was maintained for a maximum period of 2.5hrs or when the settlement rate observed to be no more than 0.1mm per hour. The last load step was held for six hours.

All test piles were loaded to their failure to the maximum load of 9.5MN (Pile 1) and 9.0MN (Pile 2), Figure 5. The end bearing component of this failure is governed by the clay not the limestone layer below.

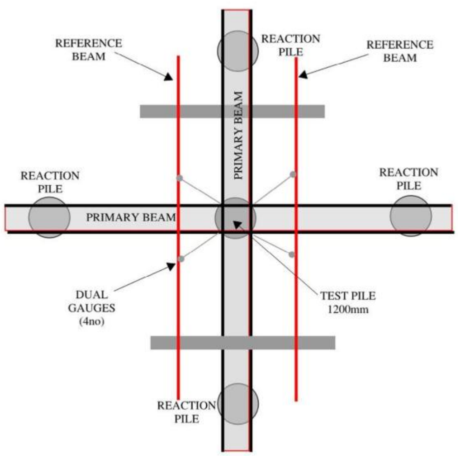


Figure 4. Pile test arrangement in plan

6 SNIP/SP BACKANALYSIS

The general SNiP method of calculation for bored pile capacity is reviewed in more detail in Vardanega et al. (2012a). The method relies on liquidity index to compute shaft and base resistances. However, the approach often leads to very conservative ultimate bearing capacity (F_d) and hence working load (N) in stiff clay. Also compared are the back-analyzed F_d and N directly from the pile test using SP (2011). A summary of the calculations for Piles 1 and 2 are shown.

6.1 Bearing capacity

The calculation of the bearing capacity of the pile has been carried out using SP24.13330.2011 (the latest revision of SNiP 2.02.03-85) along with the loading code SP20.13330.2011. The following partial factors have been applied to get the working load from calculated bearing capacity (Table 3): the partial factor on bearing capacity based on the calculation using liquidity index data equals 1.4 and the typical partial factor on variable and permanent structural loads equals 1.2. This gives a global factor of 1.68 when no pile testing has been carried out.

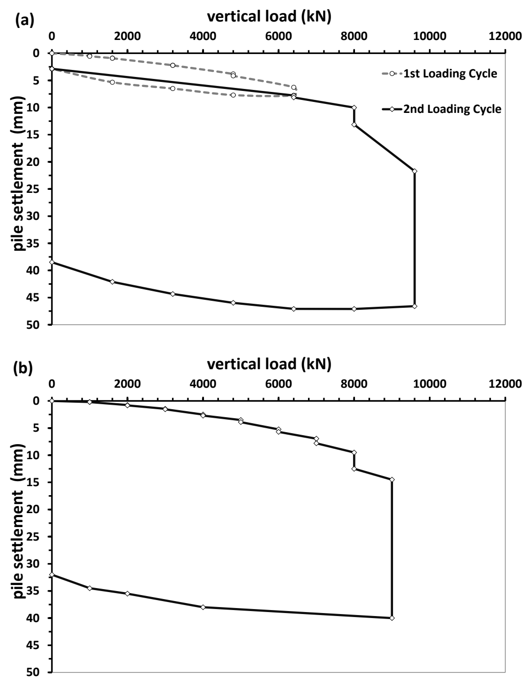


Figure 5. (a) Load settlement plot for test Pile 1; (b) Load settlement plot for test Pile 2

Table 3: Ultimate Bearing Capacity (F_d) and Working load (N) of test piles SP (2011) (I_L calculation), note that $N = F_d/1.68$

	F_d, (kN)	N, (kN)
Pile 1	4874	2901
Pile 2	4932	2935

The following partial factors have been applied to get the working load from the pile test: partial factor on ultimate load from pile test equals 1.2 and typical partial factor on variable and permanent structural loads equals 1.2. This gives a global factor of 1.44 when a pile test has been carried out: see Table 4.

Table 4: Ultimate Bearing Capacity (F_d) and Working load (N) of test piles (from pile test), note that $N = F_d/1.44$

	F_d, (kN)	N, (kN)
Pile 1	9500	6597
Pile 2	9000	6250

The calculation results show that the working load is underestimated by about 55% in comparison to the pile test results for the test piles.

6.2 Settlement of single pile

Settlement of the single pile which is cutting through the soil with shear modulus G_1 and founded on the ground with shear modulus G_2 can be calculated according to SP24.13330.2011, using a simple empirical equation if the following condition satisfied: $L/D > G_1L/G_2D > 1$.

For the piles being reported in this paper, both conditions are satisfied i.e. $L/D = 14.6$ and $G_1L/G_2D = 1.9$.

Therefore, equation 2 can be used to estimate pile settlement (s):

$$s = (\beta N)/(G_1 L) \tag{2}$$

Where; N = vertical load on the pile, kN; G_1 = average shear modulus along the pile; kPa; L = the pile length in m and β is determined using equation (3).

$$\beta = [\beta'/\lambda_1] + [(1 - [\beta'/\alpha'])/\chi] \tag{3}$$

Where, β' = coefficient of the completely rigid pile (EA=∞), equation 4; α' = coefficient for the case of the uniform soil along the pile, equation 5; χ = relative pile stiffness, equation 6 and λ_1 = settlement due to pile compression, equation 7.

$$\beta' = 0.17\ln(k_v G_1 L / G_2 D) \tag{4}$$

$$\alpha' = 0.17\ln(k_{v1} L / D) \tag{5}$$

$$\chi = (EA)/(G_1 L^2) \tag{6}$$

$$\lambda_1 = (2.12\chi^{0.75})/(1 + 2.12\chi^{0.75}) \tag{7}$$

Where; G_2 = shear modulus of soil below pile toe, kPa; D = pile diameter, m; E = elastic modulus of the pile, kPa; A = pile cross-sectional area, m^2; k_v and k_{v1} are calculated using equations 8 and 9.

$$k_{v,v1} = 2.82 - 3.78\mu + 2.18\mu^2 \tag{8}$$

$$\mu = 0.5(\mu_1 + \mu_2) \text{ for } k_v \text{ or } \mu = \mu_1 \text{ for } k_{v1} \tag{9}$$

Where μ_1 is the average Poisson's ratio along the pile and below the pile toe and μ_2 is the Poisson's ratio at the pile toe level.

The results of the single pile settlement calculation from SP 24.13330.2011 are presented in Section 8.

7 MSD BACKANALYSIS

Equation 10 was developed in Vardanega et al. (2012b) based MSD-principles and a Randolph-style calculation for soil displacement of a single pile. Development of this generalized form of the equation is presented in Vardanega (2015).

$$\frac{w_h}{D} = \eta\left(\frac{\gamma_{M=2}}{M^{(1/b)}}\right) + \left(\frac{\overline{c_u}}{M}\frac{2}{E_c}\right)\left(\frac{L}{D}\right)^2 \tag{10}$$

Where, w_h = pile head settlement; D = pile diameter; η and b are non-linearity factors; $\gamma_{M=2}$ is strain to $0.5c_u$; c_u-bar is the average shear strength along the active pile shaft; M is the mobilization factor; E_c is the concrete elastic modulus and L is the length of the constructed pile.

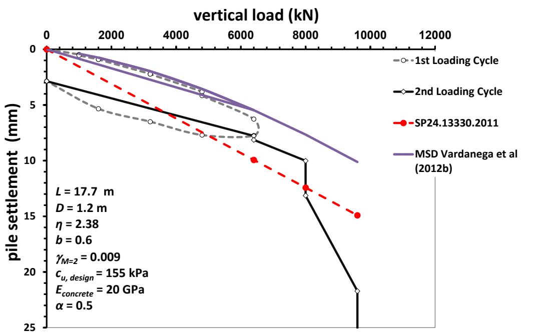
The database of Vardanega & Bolton (2011) showed that b is on average 0.6 for natural clays, with a standard deviation of 0.15. The same database showed that $\gamma_{M=2}$ is on average 0.0088 with a standard deviation of 0.0066. Table 5 shows the values of η derived for various b values. Based on the mean values of b and η, the settlement can be predicted for the test pile.

Table 5. Variation of η with b

b	η
0.45 (-1 SD)	1.91
0.60 (mean)	2.38
0.75 (+1 SD)	3.78

8 COMPARISON OF MODELS

Figure 6 shows the back-analysis of the pile loading curves using both equation 10 and the SP method outlined previously. The match to the loading curve is reasonable for both methods. The SP method appears to produce a more conservative estimate of settlement. Vardanega et al. (2012b) explain $F = \alpha M$ for piles in stiff deposits. In this paper α is taken as 0.5. This value is based on pile tests back-analysed by Patel (1992) in the London clay deposit (assuming use of U100 triaxial specimens).



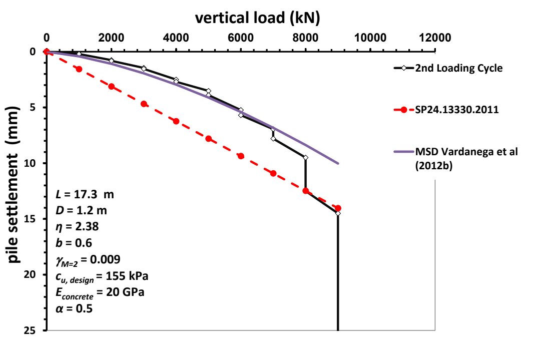


Figure 6. (a) Back-analysis of test Pile 1 (b) Back-analysis of test Pile 2 (parameters used in equation 10 shown)

9 SUMMARY

This paper has presented load-test data for two test piles from a site in the Moscow region. The bored piles are constructed in the stiff over-consolidated Jurassic clay, a clay of similar age to an Oxford clay found in England.

Two simple settlement predictive models were used and compared to the actual test pile behaviour. Both methods match the load-test data reasonably well up to the failure condition when the end bearing is mobilized. Further sensitivity studies using the method are given in Vardanega (2012) and Vardanega (2015).

The bearing capacity of the pile was also predicted using the SP method based on liquidity index and this shows that without pile testing, the working load on the pile would be significantly under-predicted compared with the test pile.

This is because in stiff clays when $I_L < 0.2$ for the shaft component and $I_L < 0.0$ for the base component, then the Russian SP method of predicting bearing capacity can lead to very conservative designs if no pile testing is carried out.

ACKNOWLEDGEMENT

Thanks are due to S. Alonga and Dr E. Ibraim for their help preparing the French abstract.

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