Penetration resistance and the bearing capacity of small-diameter steel piles

La résistance de pénétration et la capacité supportable des fondations en acier de petit diamètre

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ABSTRACT

The objective of this study was to develop an improved relationship between the penetration resistance of small diameter steel pipe piles and their ultimate bearing capacity. Static loading tests were conducted to failure on four 50-mm diameter pipe piles that were driven with a 43 kg jack hammer to four different penetration resistances.The loads and the ultimate bearing capacities interpreted from the jack pressure gauge were 0 to 15 % higher than the capacities interpreted from load cell measurements. The tangent-line method gave loads that were more conservative than those obtained with Davisson method.In the Seattle area, the current permissible load without performing a load test is 18 kN for 50-mm pipe piles continuously driven to a penetration resistance of 25 mm per 1 min with a 43 kg jack hammer. This value was found to be very conservative. Based on our test results, the ultimate observed plunging load was about five to ten times greater than 18 kN. Other interpretation methods also gave significantly higher values.

RÉSUMÉ

L'objectif de cette étude a été le développement d'une relation améliorée entre la résistance de pénétration d'une fondation des petits tuyaux en acier et leur capacité ultime de supporter. Des tests statiques chargés ont été accomplis jusqu'à l'échec des quatre fondations d'un diamètre de 50 mm, après avoir été foncés dessus par un marteau perforateur de 43 kilos à quatre résistances de pénétration différente. Les chargements et les capacités ultime de supporter interprétaient par la jauge de pression du marteau ont été jusqu'à 15% plus élevés que les capacités provenant des mesures de chargement cellule. La méthode de la ligne tangentielle a produit des chargements plus conservateur que celui de la méthode Davisson. À l'endroit de Seattle, le chargement permis actuel sans accomplir des tests de chargement est de 18 kN pour des fondations d'un diamètre de 50 mm, continuellement foncé dessus jusqu'à une résistance de pénétration de 25 mm pour chaque minute avec le marteau perforateur de 43 kilos. Cette valeur est très conservatrice. Selon nos résultats, l'ultime chargement était cinq à dix fois plus grand que 18 kN. Autres méthodes ont produit des valeurs significativement plus élevées.

1 INTRODUCTION

Small diameter pipe or pin piles 50 to 150 mm diameter are a common support system for small structures in the Seattle area. The piles are driven with internal couplings open or closed ended with pneumatic jack hammers for the 50 mm piles or modified pavement breakers for the larger diameter piles. The hammers typically operate at 1200 blows per min for the smaller hammers. Data on typical piles and driving equipment are given by Haggard et al. (2001). The present driving and support system was developed by the geotechnical consulting firm Shannon & Wilson Inc. about 1980.

The objective of the present study was to: (1) improve the relation between the penetration resistance of pipe piles and their ultimate bearing capacity; (2) examine the current commonly used allowable axial load capacity of 18 kN. in the Seattle area; and (3) determine if the current penetration criterion could be relaxed in order to reduce the damage to the piles and driving equipment.

2 PREVIOUS STUDIES

Small diameter non-grouted steel piles have been used for many years in Scandinavia, and they are still in use today (Fredriksson et al., 1989). They are designed as end-bearing piles with external galvanized couplings and toe shoes. After installation they are commonly filled with concrete. Several Reports from the Swedish Pile Commission have dealt with slender steel tube piles (Fredriksson et al. 1989, Camitz 1994, Bengtsson et al 2000). Today a system with polyethylene coated-steel piles is most common, due to corrosion and construction codes. According to Carville and Pack (1994), 50-mm diameter pipe piles were developed and patented in the United States in 1971 under the trade name Mini Pile. They presented five case histories in which 50-mm diameter pipe piles were driven with a 40 kg jack hammer. In two cases a correlation between the SPT N-value and the rate of penetration of the pipe pile was developed; in another the relationship between the capacity of a pile and the penetration rate was developed.

Druebert and Yamane (1980) of Shannon & Wilson Inc. used 50-mm steel pipe piles driven with 35-45 kg pneumatic jack hammers "for underpinning or supporting relatively light structures where access is restricted and a competent bearing layer is located at relatively shallow depth." A competent soil layer was defined as a soil with a SPT N-value >50 blows/300 mm. The suggested design load of 18 kN was based on three load tests, but only one was carried to a plunging failure and that one occurred during a reloading cycle. Consequently, no information was available on the real factor of safety.

Haggard et al. (2001) conducted a comprehensive study of small diameter pipe piles. First, data from 55 load tests at 27 sites were collected, and they demonstrated the overall reliability of small-diameter pipe piles in the Seattle area. Then load tests on 50, 75, 100, and 150 mm diameter steel pipe piles driven with jack hammers or modified pavement breakers at two different sites were analyzed. Although subsurface soil conditions were known from exploratory borings, lengths of the piles varied somewhat as did driving resistances, although almost all piles plunged when loaded. Haggard et al. (2001) concluded that a reasonable estimate of the ultimate axial capacity of pipe piles could be obtained with the tangent-line method. Accuracy of capacity prediction by pile driving formulas was not especially good. Pile capacities calculated using the β-method tended to over predict the test load capacities, probably due to the use of conservative soil property values and conservative estimates of failure loads in the test piles.

3 TEST PILE PROGRAM

In the present study, static loading tests were conducted to failure on a set of four 50-mm diameter pipe piles, all driven with a 43 kg jack hammer to four different penetration resistances. For most of the measurements of the applied load at the pile head, a calibrated load cell was used in addition to the pressure gauge on the hydraulic jack.

3.1 *Site Description*

A suitable test site at a pile contractor's yard in Renton, Washington was chosen so that the currently standard penetration resistance criteria of 25 mm penetration in 1 min would be obtainable at a depth of about 6 m. An exploratory boring was performed about 1.2 m away from the test piles, with SPT and soil samples taken as shown in Table1.
The soil profile consisted of about 0.6 m of loose sandy fill overlying 1.6 m of organic silt and peat. Below that was a 0.7 m stratum of clayey silt, 3.7 m of coarse to fine sand, then silty sand for another 2 m down to the end of boring at 8.8 m depth. The upper portion of the sand was moderately dense and below about 5.5 m it became very dense.

Table 1. HSA drilling with SPT sampling, hammer weight 63.5 kg with 0.76 m drop. Boring terminated at 8.8 m below existing grade. Groundwater encountered at 1.1 m during drilling.

Sample No	Depth in meter	Blows	N values	Core recovery %	Moisture %	USCS code
1	0 - 0.46	2/3/4	7	100	-	SM
2	0.76 -1.21	3/1/1		100	100	Pt/OL
3	1.51-1.97	(1/12)/0/1	1	81	100	Pt
4	2,.7 -2.73	-/-/-	0	100	47	MH
5	3.03 -3.48	7/8/7	15	40	-	SP
6	3.79 -4.24	13/9/11	20	100	13,6	SP
7	5.3 -5.75	27/23/27	50	100	-	SP
8	6.81-7.27	37/42/50	92	100	-	SM
9	8.33 -8.78	7/28/37	65	100	-	SM

3.2 *Test Pile Installation*

Four 50 mm (nominal diameter; actual OD = 60.3 mm) steel pipe piles (ASTM Grade 53) were driven with an American Pneumatic Tool Model 180 jack hammer (mass = 43 kg). The piles were driven in a row with a center to center distance of 0.35 m to four different penetration resistances as shown in Table 2. Each pile was driven to a specific predetermined penetration resistance that is different from the current commonly used penetration resistance criterion in the Seattle area of 25 mm penetration during 1 min of continuous driving. Figure 2 is a plan of the test site showing the location of the test piles and boring.

Table 2. Lengths and driving resistances for the Piles.

Pile No.	Depth	Penetration resistance
	(m)	(m/min)
1	7.0	0,06
2	5.0	0,1
3	5.1	0,2
4	4.7	0,3

3.3 *Load Test Setup*

Figures 1 and 2 also show the location of the reaction load. Large concrete blocks 1.8 X 0.6 X 0.6 m were used to provide a reaction load, estimated to be at least 180 kN, for the load test. See Vestberg (2002) for additional details about the load test setup.

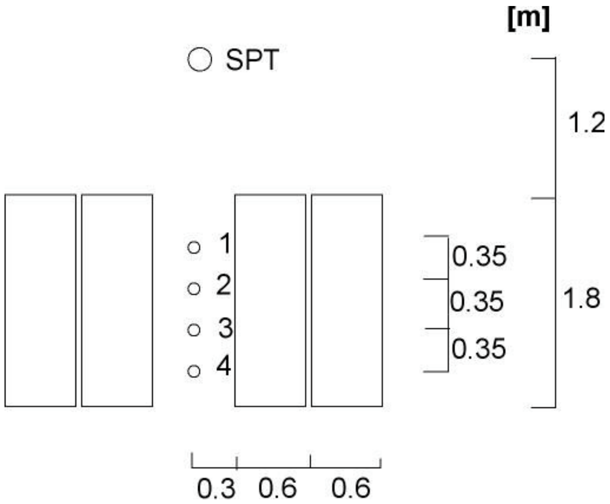

Figure 1. Plan of test site showing location of the four piles, the borehole, and the footprint of the reaction load.

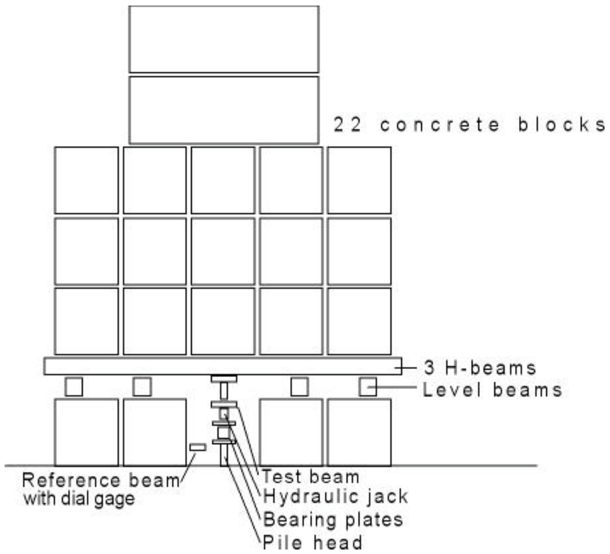

Figure 2. Side view of reaction load at the test site.

3.4 *Test Equipment and Instrumentation*

The test load was applied by a Power Team P 55 hydraulic jack with a 265 kN cylinder. The jack also had a calibrated pressure gauge, and each division represented a load of about 5.7 kN. The first readable mark on the gage corresponded to about 11.5 kN. The jack had supposedly been calibrated in 1998, but when a load cell was used, we found that the measured loads were not the same.
A nominal 110 kN capacity strain gage type load cell was chosen because of vertical space limitations between the pile heads and the reaction beam. Because of possible loading requirements in the field, it was calibrated up to 220 kN. It was read using a laptop computer, and after each field use, the cell was brought back to the laboratory and recalibrated to show that the calibration was still valid. Pile head movements were measured

with an extensometer dial gage accurate to 0.025 mm. The gage was attached to the reference beam, and the points of measurements were on the upper side of the base plate that had been attached to the pile head. A laser level was aimed at a scaled target to measure eventual uplifting of the reaction loads.

3.5 *Load Test Procedure*

To insure that all parts were in tight contact, the load tests were performed with an initial seating load of 2 to 4 kN as indicated by the load cell. This seating load was not readable on the jack pressure gauge. The times for jack loading and reloading were when bleeding in the system was observed on the jack pressure gauge. During testing the glass on the jack pressure gauge was tapped so that the gauge-pointer would follow pressure decreases due to bleeding and to pile head movement. Thus, it was only possible to measure larger pressure changes on the gauge, and examination of the load vs. time curves indicated that for higher applied loads, the load decreased during each load step. The load cell was much more sensitive to hydraulic jack bleeding and movement of the pile head than the jack pressure gauge. In all test sequences the deepest pile (Pile No. 1) was tested first. This was done to avoid possible disturbance to this pile when the shorter piles were tested. Thereafter Pile Nos. 2, 3, and 4 were loaded. Each load increment (about 6 kN) was held for 2.5 min, and measurements of the load were taken each second during loading. After the pile reached maximum load, it was unloaded and the permanent movement was read. The operation of the jack did not allow the same load steps for unloading as for loading.

3.6 *Test Results*

3.6.1 *Load Test Sequence 1*

Sequence 1 load tests were performed on October 19, 2001. Tests were made 2 to 5 h after pile driving. During this test Sequence it was noticed that the reaction load was jacked off its bearing platform. How much of the applied force was lifting the reaction and how much was applied at the pile head is not known. Thus, the results from that test Sequence 1 are considered unreliable and are, therefore, not reported in this paper.

3.6.2 Load Test Sequence 2

Additional concrete blocks were added for a total reaction load of more than 304 kN. The second sequences of tests on the same piles were performed on November 16, 2001, and a load cell was used in addition to the jack pressure gage for measurement of the applied load at the pile head. The load-movement curves for sequence 2 are presented in Figures 3 to 6 in which the filled symbols are the load cell measurements and the unfilled symbols are the hydraulic jack gauge readings.

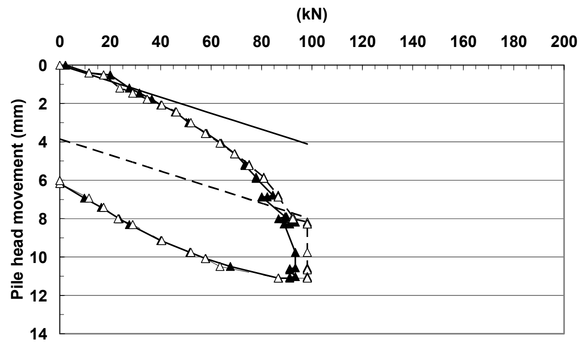

Figure 3. Load vs. movement at the pile head for Site C, loading sequence 2, Pile No. 1 measurements from load cell (filled) are shown and also measurements from the hydraulic jack . Also shown are the elastic shortening and the Davisson criterion for the pile.

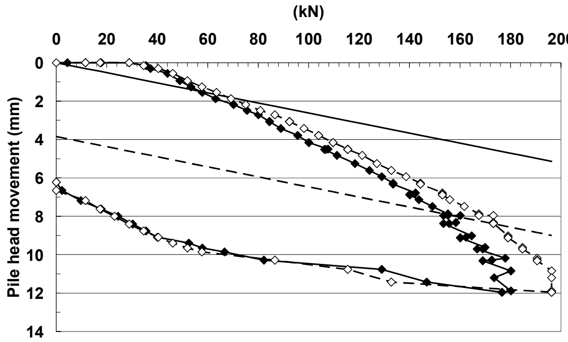

Figure 4. Load vs. movement at the pile head for Site C, loading sequence 2, Pile No. 2 measurements from load cell (filled) are shown and also measurements from the hydraulic jack . Also shown are the elastic shortening and the Davisson criterion for the pile.

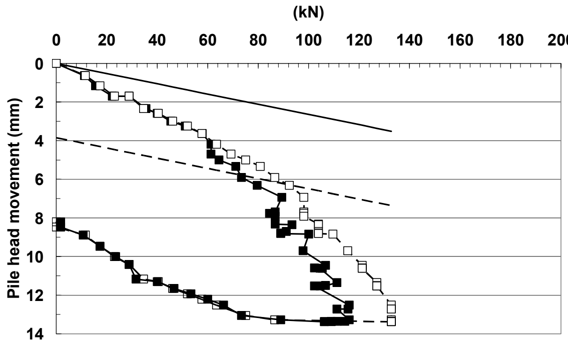

Figure 5. Load vs. movement at the pile head for Site C, loading sequence 2, Pile No. 3 measurements from load cell (filled) are shown and also measurements from the hydraulic jack . Also shown are the elastic shortening and the Davisson criterion for the pile.

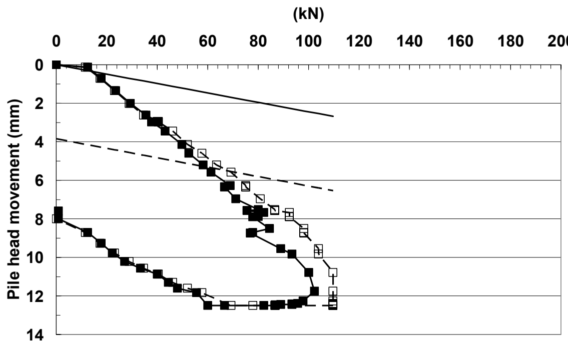

Figure 6. Load vs. movement at the pile head for Site C, loading sequence 2, Pile No. 4 measurements from load cell (filled) are shown and also measurements from the hydraulic jack . Also shown are the elastic shortening and the Davisson criterion for the pile.

3.6.3 Load Test Sequence 3

The third sequences of tests, also using a load cell, were performed November 24, 2001. The load-movement curves are presented in Vestberg (2002). The unexpected high bearing capacity for pile No. 2 was verified. It was also then not possible to further increase the load when capacity of the jack was reached. Thus, for pile No. 2 no plunging or creeping occurred at maximum load.

4 METHODS USED TO EVALUATE PIPE PILE LOAD TESTS

The ultimate bearing capacity of the four test piles were interpreted using Davisson (1972), Tangent-line, Butler and Hoy (1977) method and the plunging load. Haggard et al. (2001) concluded that the Tangent-line method yielded the most conservative ultimate bearing capacity values followed by Davisson. The offset in the Davisson criterion can be way too large for pipe piles, probably because it was derived for much larger piles. Haggard et al.(2001) found that at the deep soft clay site, the plunging failure occurred at a lower value than indicated by Davisson method. At another site with sand, the plunging load and Davisson ultimate bearing capacities were almost equal. Vestberg (2002) found that the plunging loads were much higher than indicated by the Davisson method. Fellenius (1991) noted that the load that induces pile-plunging is not an adequate definition of the failure load when plunging is more a function of man-pump system than of pile-soil system. However, even if the Davisson method is not suitable for determination of the ultimate bearing capacity of pipe piles at all sites, the method provides a single value that can be used for comparison with pile tests.

Both Haggard et al. (2001) and Vestberg (2002) found that static pile analysis (β-method) over predicted the test load capacity, probably due to use of conservative soil property values, and it is therefore not included in this presentation. Theoretically, the ultimate bearing capacity should be reduced because of the reaction load surcharge. The surcharge influence on ultimate bearing capacity is highly dependent on method of calculation and assumptions, Vestberg (2002). Therefore surcharge effects are also neglected in this presentation.

5 INTERPRETATION AND EVALUATION OF THE TEST RESULTS

In Figures 3 to 6 it is obvious that for higher applied loads the load cell it much more sensitive to hydraulic jack bleeding and pile head movement than the jack pressure gauge. Jack and load cell measurements agree up to about 70 kN. The loads and the ultimate bearing capacity interpreted from the jack pressure gauge loads were 0 to 15 % higher than the capacities interpreted from the load cell measurements. The seating load of 2 to 4 kN was not readable on the jack pressure gauge but was easily detected using the load cell. All four piles had an ultimate bearing capacity by all methods that far exceeded the currently used allowable load of 18 kN for 50 mm pipe piles in the Seattle area. In Table 3, the ultimate bearing capacity interpretations derived from load cell measurements Sequences 2 and 3 are presented. In Figures 3 and 4 the permanent displacements of the pile heads are approximately 6 mm at most and in Figures 5 and 6 the permanent displacements are about 8 mm.

Table 3: Ultimate bearing capacity interpretations at Site C derived from load cell.

Pile No. Load Sequence	**Penetration resistance**	**Tangent-line**	**Davisson**	**Plunging**
	m/min	(kN)	(kN)	(kN)
1, Sequence 2	0.06	71	85	93
2, Sequence 2	0.1	142	151	178
3, Sequence 2	0.2	80	71	89
4, Sequence 2	0.3	76	58	98
1, Sequence 3	0.06	76	80	91
2, Sequence 3	0.1	187	145	---

6 DISCUSSION

The Factor of Safety is more than four times larger than the allowable design load for a 50 mm pile in the Seattle area. Three of the piles in this test were driven with penetration rates far lower than the current criteria.

However, it should be stressed that our results are derived from a small number of full-scale load tests from one test site. A relaxation of the current penetration resistance criterion in order to reduce the damage on piles and driving equipment is possible. A consequence of reduced penetration resistance will reduce damage to driving equipment and piles, as well as yield potential economical benefits.

7 CONCLUSION AND RECOMMENDATION

The current allowable bearing capacity of 18 kN for a penetration resistance of 25 mm per 1 min of continuous driving with a 43 kg jack hammer is highly conservative for 50 mm pipe piles in the Seattle area. The permissible load can be increased significantly. From the load test data it appears that a penetration resistance criterion of 50 to 100 mm per min would still yield satisfactory safety. If 50 mm pipe piles are intended for high bearing capacities, they should be load tested to verify the actual capacity and factor of safety.

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