Challenge of driven piles in Southern Ontario

Laifa Cao
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ABSTRACT
Case studies of driven steel piles at three sites in Southern Ontario are presented in this paper. Pile resistances evaluated from static load tests, dynamic pile driving formulae, a pile driving analyzer (PDA), and static analyses are compared and analyzed. It is found that the resistance of driven piles can be reliably determined by PDA testing. A low resistance factor should be used when the dynamic pile driving formulae or static analyses is used to estimate the pile resistance at the ultimate limit state (ULS). The ULS, rather than the serviceability limit state (SLS), controls the design of driven steel pile.

RÉSUMÉ
Des études de cas de pieux en acier entraînés à trois sites dans le Sud de l'Ontario sont présentées dans le présent document. Les résistances des pieux évaluées à partir de tests de charge statique, de formules dynamiques d'entraînement des pieux, d'un analyseur d'entraînement des pieux (PDA) et d'analyses statiques sont comparées et analysées. Il est constaté que la résistance des pieux entraînés peut être déterminée de manière fiable par des tests PDA. Un facteur de faible résistance doit être utilisé lorsque les formules d'entraînement dynamique des pieux ou les analyses statiques sont utilisées pour estimer la résistance des pieux à l'état limite ultime (ULS). L'ULS, plutôt que l'état limite de facilité d'entretien (SLS), contrôle la conception des pieux en acier entraînés.

1 INTRODUCTION

The need for pile foundations results from soft or loose soil conditions at shallow depth that would result in unacceptable settlement or inadequate bearing resistance from shallow foundations. The pile size and length are designed based on the ultimate limit states (ULS, which considers the load resistance) and serviceability limit states (SLS, which considers the deformations or settlements), and the selection of the pile type is usually based on subsurface conditions as well as local experience and practice. When the vibration and noise are not concerned and the site is accessible, driven piles are usually selected due to several advantages such as the relatively low cost, variation in size, length and shape, minimum soil spoil, minimum supervision and quality assurance.

Prior to driving the production piles, a pile-driving criterion, which is defined as a specified pile-driving resistance that triggers a pause of driving operation, needs to be selected through pile load tests. The process to establish the pile-driving criterion are as follows: (1) evaluate soil and groundwater conditions and estimate static pile resistance; (2) select a preliminary driving criterion using the wave-equation analysis or dynamic pile driving formula; (3) drive test piles for static load testing using the preliminary driving criterion; (4) test the test piles; and (5) establish the final driving criterion based on the load test results. Lately a pile driving analyzer (PDA) test, rather than static load tests, is often performed to evaluate the pile resistance, driving stress and hammer performance, for the establishment of the pile-driving criteria.

This paper describes driven steel piles at three sites in Southern Ontario. The pile capacities determined from the static load tests, PDA, dynamic pile driving formulae, and static analyses are compared and discussed. The phenomenon of pile capacity increasing with time for pile driven into the saturated sand to silt deposits is discussed. Based on the results of pile load tests and PDA tests, resistance factors to estimate the ULS pile resistance using dynamic pile driving formulae or static analyses are recommended. Engineering judgement for the assessment of driven pile resistance based on the driving records are also discussed.

2 CASE STUDIES

Driven piles at three sites in Southern Ontario are discussed as following sections.

2.1 Ashbridge Bay Treatment Plant Upgrades

The site stratigraphy was made up of 9 m of very loose to loose organic sandy silt to sand overlying 3 m of soft clayey silt to silty clay which in turn rested on 5 m of very dense sand. The groundwater level was at 1 to 2 m below the existing ground surface (mbgs).

The 12 m thick very loose to loose and soft soils could not support the proposed digesters. Driven small diameter tubular pile was considered the best option of deep foundations due to relatively deep weak soils and low bearing capacity requirement. 200 mm dia. closed-end tubular piles with wall thickness of 5.5 mm were driven into the very dense sand to support the proposed digesters. The design axial compressive and tensile resistances were 250 kN and 35 kN in the SLS, respectively. A single acting diesel – direct driving Model B9 hammer at an Energy setting of 24.4 kJ at 40 blows per minute was used to drive three test piles (Pile Nos. 1 to 3) into the very dense sand.

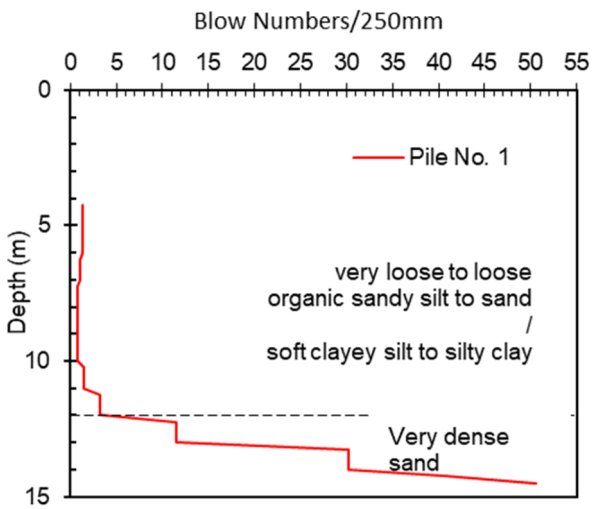

Figure 1. Driving records of a tubular pile in Toronto

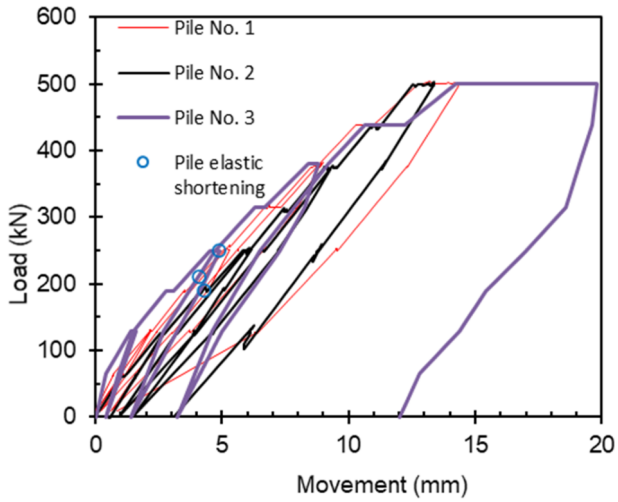

Figure 2. Results of static compressive load tests of Pile Nos. 1, 2 and 3 in Toronto

2.1.1 Pile Driving

Three piles were driven for static load testing. As the existing ground level was about 4 m higher than the pile cut-off level, 4 m long casing sleeves were installed prior to pile driving. The soil inside the casing sleeve was excavated prior to pile driving. The numbers of blows required for advancing Pile No. 1 during driving were recorded and are presented in Figure 1. The numbers of blows were found less than 4 per 250 mm penetration before the pile was driven 12 m into the ground. From 12 to 14.5 mbgs, the numbers of blows increased to 50 per 250 mm penetration and 2.5 m into the very dense sand as indicated in the boreholes based on SPT (standard penetration test) N-values. Therefore, the pile driving was temporarily halted and when continued, Hiley's graph was plotted. At the completion of plotting, the permanent set was 5 mm and the rebound 10 mm. The estimated pile ultimate capacity by Hiley's Formula was 1,364 kN. Pile No. 2 was driven 15.2 mbgs with the estimated pile ultimate capacity by Hiley's Formula of 1,481 kN and driving records were similar to those of Pile No. 1. Pile No. 3 was driven 13.1 mbgs with the estimated pile ultimate capacity by Hiley's Formula of 1,178 kN.

2.1.2 Static Load Tests

Compressive and tensile load tests on Pile Nos. 1 to 3 were carried out to establish the pile driving criteria.

The test setup follows the guidelines of ASTM standard (2020; 2022). Load was applied to the pile by a hydraulic jack acting against a test beam which was loaded by concrete blocks for compressive tests and was supported by timbers for tensile tests, respectively. A load cell was placed between the hydraulic jack and the test beam to measure the applied load.

The design compressive working load was 250 kN and the maximum test load was 500 kN. The design tensile working load was 35 kN and the maximum test load is 70 kN. The tests were performed in general accordance with ASTM standard (2020; 2022). Loads were applied in increments of 25% of the design working load to reach a total of 200% of the design working load. Cyclic loading was applied at 50%, 100% and 150% of the design working load. The test load was maintained for 12 hours and then the pile was unloaded in decrements of 25% of the maximum test load.

Figure 2 shows the results of compressive load test on the three piles. For Pile No. 1, the movement of pile head was equal to the pile elastic shortening when the load reached 210 kN. The movement of pile head at the maximum test load (200% of the design load) was 14.3 mm, which was less than the movement of pile head at the offset limit load which is defined as the sum of elastic shortening of pile plus 4 mm plus 8 times pile tip diameter divided by 1,000. Therefore, the ultimate capacity for Pile No. 1 was greater than 500 kN. During the first 1 hour while maintaining the maximum test load, the rate of settlement was 0.84 mm/hour but in the following 11 hours, the rate of settlement at the maximum test load was less than 0.25 mm/hour, the creep failure criterion as recommended in ASTM standard (2022). This indicated that Pile No. 1 did not fail during the creep test.

For Pile No. 2, the movement of pile head was equal to the pile elastic shortening when the load reached 190 kN. The movement of pile head at the maximum test load (200% of the design load) was 13.4 mm, which is smaller than that of Pile No. 1 and also less than the movement of pile head at the offset limit load. The ultimate capacity for Pile No. 2 is also greater than 500 kN. During the first 1 hour while maintaining the maximum test load, the rate of settlement was 0.55 mm/hour but in the following 11 hours, the rate of settlement at the maximum test load was less than 0.25 mm/hour. Pile No. 2 did not fail during the creep test.

For Pile No. 3, the movement of pile head was equal to the pile elastic shortening when the load reached 250 kN. The movement of pile head at the maximum test load (200% of the design load) was 19.8 mm, which is greater

than the movement of pile head at the offset limit load. During the 12 hours while maintaining the maximum test load, the rate of settlement was 0.46 mm/hour, which is greater than 0.25 mm/hour. Pile No. 3 was considered creep failure.

The factored pile resistances at the ULS from Hiley's formula using the penetration/rebound graph obtained from initial driving and static analysis were assessed using the resistance factor as recommended in the Canadian Foundation Engineering Manual (CGS 2006). Table 1 summarizes the results. The factored resistance at the ULS determined from the offset limit load is the same as that determined from the failure load when a creep test was conducted in the pile load test for Pile No. 3. Hiley's formula overestimated the factored resistance at the ULS, mainly because the hammer efficiency was not considered. When a resistance factor of 0.25 is applied, the factored resistance at the ULS determined from Hiley's formula is comparable to that determined from the failure load for Pile No. 3. Because plotting the penetration/rebound graph during pile driving is considered an unsafe activity, Hiley's formula based on the graph is rarely applied at present in driven pile projects. The factored resistance at the ULS, using the semi-empirical analysis based on borehole information, was similar to that determined from the failure load for the short pile.

Table 1. ULS resistances of Piles Nos. 1 and 2 in Toronto

Method	Resistance Factor	Factored Resistance (kN)		
		Pile No. 1	Pile No. 2	Pile No. 3
Offset limit load from pile tests	0.6	> 300	> 300	300(*)
Hiley's formula	0.4	546	592	471
Static analysis	0.4	382	425	303

(*) The ULS resistance was 300 kN determined from the failure load.

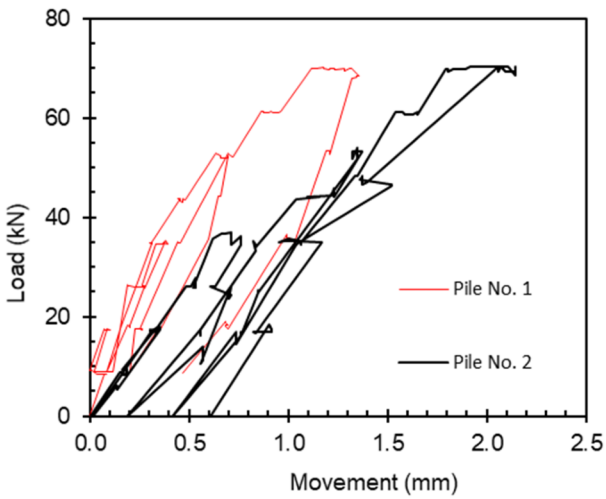

Figure 3. Results of static tensile load tests of Pile Nos. 1 and 2 in Toronto

Figure 3 shows the results of tensile load test on Pile Nos. 1 and 2. For Pile No. 1, the movement of pile head at the maximum test load (200% of the design load) was 1.36 mm, which is smaller than the elastic elongation of pile. Therefore, the ultimate tensile capacity for Pile No. 1 is greater than 70 kN. During the 12 hours while maintaining the maximum test load, the creep movement was 0.2 mm and the rate of movement was 0.017 mm/hour, much less than 0.25 mm/hour. This indicted that Pile No. 1 did not failure during the creep test.

For Pile No. 2, the movement of pile head at the maximum test load was 2.15 mm, which is smaller than the modified offset limit defined as pile elastic elongation plus 4 mm (Kulhawy and Hirany, 1989). Therefore, the ultimate tensile capacity for Pile No. 2 is greater than 70 kN. During the 12 hours while maintaining the maximum test load, the creep movement was 0.2 mm and the rate of movement was 0.017 mm/hour, which is much less than 0.25 mm/hour. This indicted that Pile No. 2 did not fail during the creep test.

The factored tensile resistance at the ULS, using the semi-empirical analysis and a resistance factor of 0.3 based on borehole information, ranged from 80 to 146 kN, which is higher than the maximum test load.

2.2 Schomberg Water Pollution Control Plant Upgrades

The site consists of 10 to 32 m firm to stiff silty clay/clay overlying very dense silt to sandy silt deposits. The groundwater measured in the silty clay/clay ranged from 2.0 to 5.5 mbgs, whereas the groundwater level in the lower silt to sandy silt was 0.3 mbgs, indicating an artesian condition in the silt to sandy silt deposits.

Driven steel H-pile was considered the best option of deep foundations to support buildings, a bridge and aeration tanks due to relatively deep weak soils and high bearing resistance requirement. The test piles were steel HP 310x110 fitted with flange plate tip protection (i.e., the driving shoe). A Delmag D30-32 diesel hammer, with maximum rated energy setting of 85 kJ, was used to drive two test piles (Pile Nos. 1 and 2) into the very dense silt to sandy silt.

2.2.1 Pile Driving and PDA Testing

The number of blows ranged from 1 to 3 per 305 mm penetration before Pile No. 1 was driven 22.5 m into the ground. From 22.5 to 29.7 mbgs, the number of blows increased from 13 to 27 per 305 mm penetration (Figure 4). Because, based on the borehole information, the pile had been driven more than 3 m into the very dense silt to sandy silt, pile driving was halted and the penetration/rebound graph was plotted. At completion of plotting, the permanent set was 10 mm and the rebound was 15 mm. The estimated ultimate pile resistance using Hiley's formula was 2,910 kN. This value was less than the estimated ultimate resistance of 3,420 kN using the values of bearing capacity factor and combined shaft resistance factor for driven piles, as recommended in CGS (2006). For this estimate, the pile resistance was calculated from the entire soil-pile cross-sectional area and the shaft

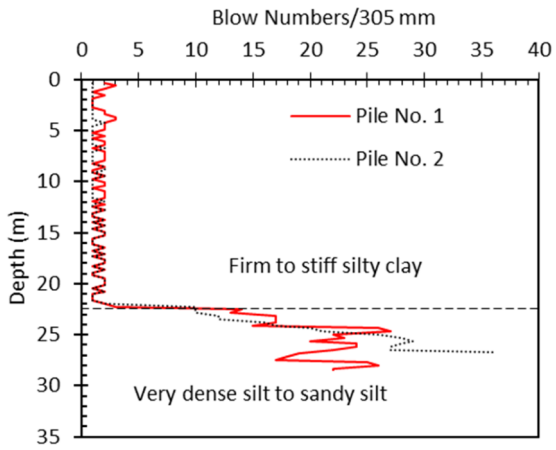

Figure 4. Driving records of Pile Nos. 1 and 2 in Schomberg

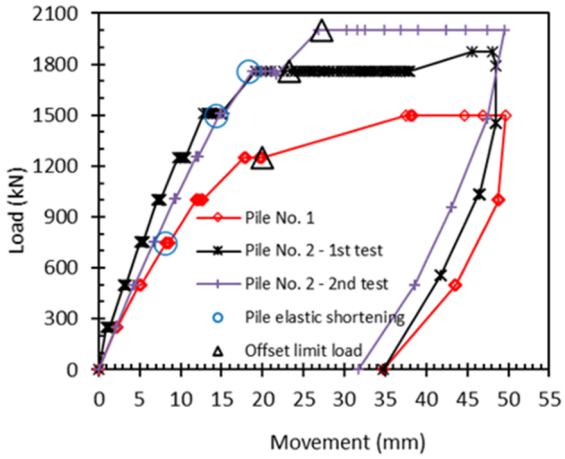

Figure 5. Results of static compressive load tests of Pile Nos. 1 and 2 in Schomberg

resistance was assumed as the plugging pile condition. The low resistance could be due to the excess porewater pressure around the test pile and below the pile tip during pile driving.

To allow the dissipation of excess porewater pressure, Pile No. 1 was restruck with the same 85 kJ energy 1 day and 4 days after initial driving. The recorded permanent set upon restriking ranged from 4.5 to 4.9 mm and the rebound from 17 to 20 mm. The estimated ultimate pile resistances were 3,920 and 3,420 kN, respectively, for restriking 1 day and 4 days after initial driving, therefore, not increasing with time. Four days after initial driving, the PDA test was performed. The PDA test estimated an ultimate pile resistance of 1,800 kN, which is significantly lower than the values from Hiley's formula, in which the rated energy, not the actual energy, was used. Since the actual energy transferred to piles normally ranges from 50% to 85% of the rated energy, it is not surprising that Hiley's formula provides a high pile resistance. Four days after initial driving, the static load test was conducted on Pile No. 1.

Similar to Pile No. 1, the number of blows was less than 3 per 305 mm penetration for Pile No. 2 until the pile reached a depth of 22.0 mbgs, and then the number of blows increased to 21 per 305 mm penetration from 22.0 to 24.5 mbgs. From 24.5 to 26.7 mbgs, the number of blows was 26 to 29 per 305 mm penetration, which was higher than that at the location of Pile No. 1 at the same depth (Figure 4). Comparing the driving records of Pile Nos. 1 and 2, it was found that the compactness of the very dense silt to sandy silt determined from SPT N-value were significantly different at the two locations. The penetration/rebound graph was plotted at a depth of 26.7 mbgs. At completion of plotting, the permanent set was 10 mm and the rebound 14 mm. The estimated ultimate pile resistance using Hiley's formula was 2,930 kN.

Pile No. 2 was also restruck with the same 85 kJ energy 1 day and 4 days after initial driving. The recorded permanent set upon restriking ranged from 3.2 to 2.2 mm and the rebound from 22 mm to 21 mm. The estimated ultimate pile resistance ranged from 3,510 to 4,090 kN, therefore, increasing with time. The PDA test was also performed on this pile 4 days after initial driving. The PDA estimated an ultimate pile resistance of 2,100 kN. Fourteen days after initial driving, the static load test was conducted on Pile No. 2.

2.2.2 Static Load Tests

The static load test setup generally followed the ASTM standard (ASTM, 2020). An axial compressive load was applied to the test pile by a hydraulic jack acting against a reaction beam.

Figure 5 shows the results of compressive load tests on Pile Nos. 1 and 2. For Pile No. 1, settlement of the pile head was equal to the theoretical elastic deformation of the pile when the load reached 750 kN. The settlement at the load of 1,250 kN was the same as that at the offset limit load. When the load reached 1,500 kN, pile failure occurred with continuous settlement. For Pile No. 2, settlement of the pile head was less than the theoretical elastic deformation of the pile when the load was less than 1,500 kN. Continuous settlement was observed when the load reached 1,760 kN. The maximum load reached was 1,872 kN.

Since the first test of Pile No. 2 did not reach the proposed maximum test load of 2,000 kN, this pile was driven a further 2 m to a depth of 28.7 mbgs. A very high blow account was recorded from the additional 2 m driving. The number of blows ranged from 59 to 80 per 305 mm penetration. Hiley's formula indicated the ultimate pile resistance of 3,780 kN. Three days after the additional 2 m driving, a second compressive static load test was carried out. Continuous settlement was observed when the load reached 2,000 kN. The offset limit load method also indicated the ultimate resistance of 2,000 kN (Figure 5).

The factored pile resistances at the ULS from the failure load, offset limit load, PDA tests, Hiley's formula using the penetration/rebound graph obtained from initial driving, and static analysis were then assessed using the resistance factor as recommended in CGS (2006). Table 2 summarizes the results. The factored resistance at the ULS determined from the offset limit load is conservative and less than that determined from the failure load. The factored resistance at the ULS determined from the PDA

test is equal to or slightly less than that determined from the failure load. This confirms that the PDA is a reliable method to estimate the driving pile resistance. Hiley's formula overestimated the factored resistance at the ULS, mainly because the hammer efficiency is not considered. When a resistance factor of 0.33 is applied, the factored resistance at the ULS determined from Hiley's formula was comparable to that determined from the failure load. Because plotting the penetration/rebound graph during pile driving is considered an unsafe activity, Hiley's formula based on the graph is rarely applied at present in driven pile projects. The factored resistance at the ULS, using the semi-empirical analysis based on borehole information, was 1 to 1.5 times of that determined from the failure load, indicating that the semi-empirical analysis can be only used as reference for the estimation of driving pile length and resistance. This is mainly because variation in the soil conditions could not be determined from limited borehole information and SPT N-values.

Table 2. Summary of factored resistance of Pile Nos. 1 and 2 at ULS in Schomberg

Method	Resistance factor	Pile No. 1	Pile No. 2 (1st test)	Pile No. 2 (2nd test)
Failure load from pile tests	0.6	900	1123	1200
Offset limit load from pile tests	0.6	750	1056	1200
PDA	0.5	900	1050	N.A.
Hiley's formula	0.4	1164	1172	1521
Static analysis	0.4	1368	1116	1292

The pile settlements at the factored pile resistance ranged from 8 to 11 mm (Figure 5). These settlements were less than the allowable settlement of 25 mm at the SLS. Thus, the ULS resistance controls the design of driven H-pile.

2.3 Railway Bridge in Vaught

The site consists of 1.2 to 1.5 m thick very soft to stiff silty clay fill underlain by 3.1 to 4.0 m thick upper very stiff to hard silty clay till overlying 13.8 to 14.9 m thick relatively weak (generally firm to stiff) silty clay till followed by hard clayey silt till and lower dense to very dense sandy soils based on SPT N-values. The groundwater measured in the lower sandy soils was 22.7 mbgs.

Steel HP 310x110 was proposed to support the bridge foundations. The designed axial resistance was 1200 kN in the ULS for H-pile driven 35 m below the grade (1.5 mbgs), corresponding to elevation (El.) 164.1 m.

A B-5505 diesel hammer, with a maximum rated energy setting of 146 kJ, was used to drive the H-piles into the lower dense to very dense sandy soils.

Figure 6 shows the driving records for 11 H-piles, for which the PDA testing were performed 11 to 22 days after the initial driving. The number of blows was generally less than 10 per 200 mm penetration for the first 10 m driving.

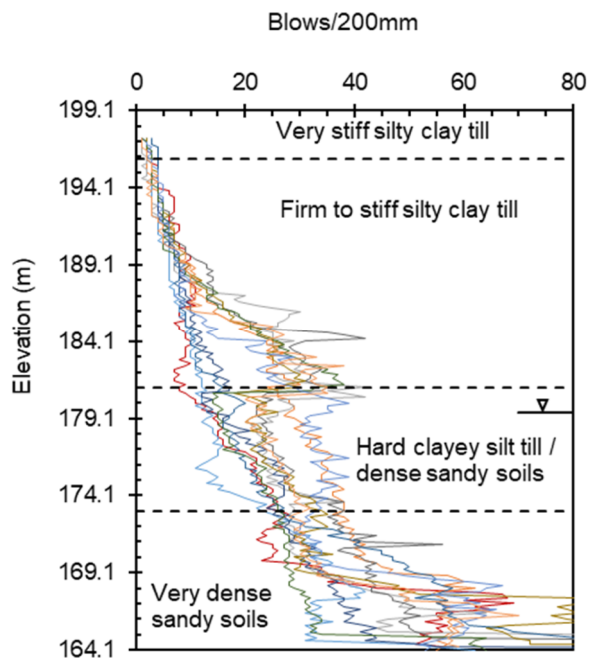

Figure 6. Driving records of 11 PDA tested piles in Vaught

From El. 189.1 to 173.1 m, the number of blows varied from 8 to 42 per 200 mm penetration, indicating significant variation in the soil consistency or compactness condition. Below El. 173.1 m, the number of blows varied from 22 to greater than 100 per 200 mm penetration, although the soil was considered in a very dense compactness condition based on SPT N-values. The number of blows was greater than 30 per 200 mm penetration for last 3 m driving and not less than 53 blows for the last 200 mm driving.

During the PDA testing, the transferred energy ranged from 29 to 53 kJ, much smaller that the rated energy; the equivalent penetration resistance ranged from 13 to 250 blows per 25 mm, much greater than that during initial driving. The PDA tests estimated ultimate pile resistances of 2,500 to 2,600 kN 11 days after initial driving and 3,000 to 3,300 kN 16 days after initial driving. The factored ULS resistance ranged from 1250 to 1300 kN, greater than the design requirement. Therefore, the pile resistance was considered as 1,250 kN for the pile driven about 35 m below grade with penetration resistances of generally not less than 30 blows per 200 mm penetration for last 3 m driving.

The PDA testing performed on three piles on the same days after initial driving only estimated the ultimate resistance of 1600 to 2200 kN. Pile ULS resistance of 1150 to 1250 kN was proposed for these piles by comparing their pile driving resistance with those confirmed by the PDA testing.

Three numbers of piles were found low penetration resistance when the piles were driven into the very dense sandy soil as shown in Figure 7, in which the number of blows ranged from 12 to 16 per 200 mm penetration. Pile ultimate resistance of 1600 kN was initially proposed for these piles by comparing their pile driving resistance with

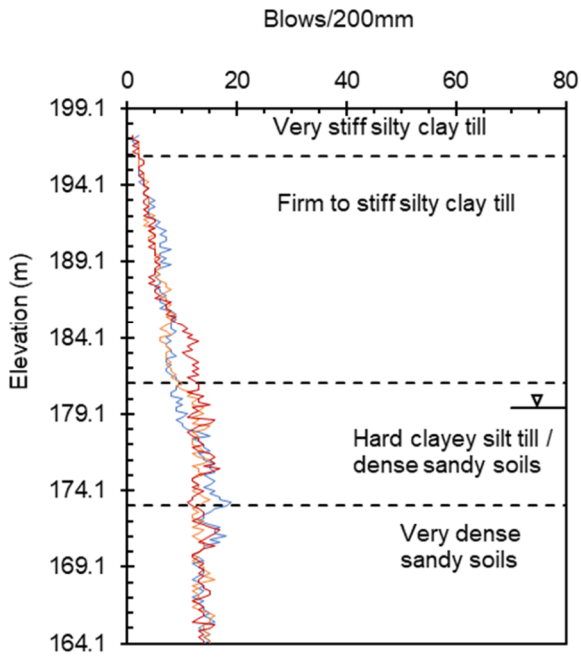

Figure 7. Driving records of 3 PDA tested piles in Vaught

those confirmed by the PDA testing. Since the recommended pile resistance could not meet the design requirement, PDA testing was performed on these piles 30 to 37 days after the initial driving. During the PDA testing, the equivalent penetration resistance ranged from 12 to 36 blows per 25 mm penetration, much greater than that during initial driving. The PDA tests estimated ultimate pile resistances of 2,500 to 3,200 kN.

Comparing Figures 6 and 7, it if founded that the penetration resistance as well as the ultimate pile resistance increases significantly with time for pile driving in the saturated sandy soils.

3 CONCLUSIONS

Based on the case study, the design method for driven steel based on semi-empirical analyses should only be used as a reference due to limited borehole information. A resistance factor of 0.25 to 0.4 could be considered when the semi-empirical analyses are used to estimate the ULS pile resistance. Hiley's formula is not recommended because the involved procedure is considered unsafe. A resistance factor of 0.25 to 0.33 could be considered when Hiley's formula is used to estimate the ULS pile resistance under the condition of safety measures in place. The resistance of H-piles can be reliably determined by the pile load test and PDA test. The PDA test has been widely used due to its relative low cost. The pile resistance increasing with time can be easily determined by PDA test. The ULS, rather than the SLS, controls the design of driven steel H-pile.

4. REFERENCES

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