Modeling and Observations of Pile Installation using Vibro Hammers in Fraser River Delta Soils

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ABSTRACT There has been significant recent interest in the use of vibratory drivers/vibro hammers to install piles in offshore and nearshore applications around the globe. Recent experience using vibratory drivers/vibro hammers is reviewed including lessons learned at the Riffgat Wind Farm project. Soil parameters derived for two local sites, based on the results of cone penetration test profiling and high strain dynamic testing conducted during and/or after completion of installation of piles by impact and/or vibratory methods, can be directly input into Wave Equation Analysis of Pile (WEAP) driving simulation software such as AllWave-PDP to model the vibratory installation process. The results of the analysis are shown to compare well with rudimentary observations during pile installation. Recommendations for better monitoring and analysis of vibro hammer projects are presented as are some of the advantages and challenges of using vibro hammers.

Introduction

There has been significant recent interest in the use of vibratory drivers to install piles in offshore and nearshore applications around the globe. Viking (2006) provides a thorough treatise on the vibratory installation method. More recently, Middendorp and Verbeek (2012), Verbeek et al. (2013) and Fischer et al. (2013) have demonstrated the successful use of vibratory drivers for the installation of piles in a variety of situations.

Previous work (Mosher 1987, Tucker and Briaud 1988) demonstrated that the average initial stiffness and often the ultimate vertical compression resistance of vibratory driven piles tend to be lower than that of impact driven piles. Fischer et al. (2013) essentially replicated this work and showed that the distribution of resistance along the pile shaft varied significantly between impact and vibratory driven piles. However, Fischer et al. also showed that, overall, the resistances were similar when tested at large displacements.

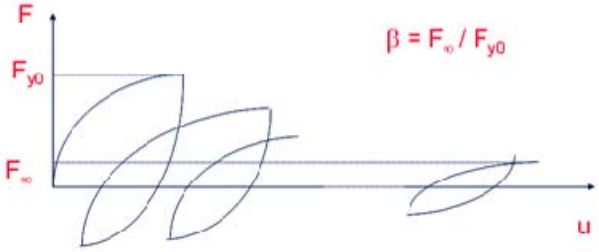
Many pile specifications have adopted the findings of Mosher (1987) and Tucker and Briaud (1988). For instance, many local infrastructure projects require that all vibratory driven piles be terminated by impact driving. Even the monopole foundations for the recently completed Riffgat Wind Farm project required that final driving be completed using an impact hammer (Fischer et al. 2013 and Verbeek et al. 2013). Given the interest in large marine facilities and offshore and nearshore wind turbine generators in British Columbia, the authors are of the opinion that this practice deserves review in light of recent experience and advances in measurement and modelling techniques.

This paper will provide a brief overview of a project in Europe and review recent experience at two Fraser River Delta sites near Vancouver, British Columbia.

Vibratory Driving Process

The vibratory driving technique was developed in the 1930s according to Rodger and Littlejohn (1980) and Holeyman (2000). The process involves a pile, vibrator and soil into which the pile will be driven. Typical piles installed using vibro hammers include steel H, sheet and pipe piles. Concrete piles may also be installed using vibratory drivers under the right conditions. Vibrators are typically hydraulic with counter-rotating eccentric masses located within the driving assembly or exciter block (Holeyman 2000). The exciter block is connected to the pile via a clamp to impart typically vertical vibratory movement in the pile to be driven. Through this process the soil adjacent to and below the pile toe is exposed to a high number of loading cycles and the soil strength thereby degraded as the pile is advanced into the soil as shown in Figure 1.

Fig. 1. Soil resistance reduction under cyclic loading (from Jonker 1987).



Vibratory drivers typically operate in the range of 20 to 40 Hz or about 1300 to 2500 rpm. As such, soil situated along a pile shaft can be subjected to a very high number of cycles in a matter of a few minutes. For comparison, White (2005) and Randolph et al. (2005) demonstrated the process of degradation of shaft resistance or friction fatigue with impact driven piles with increasing number of load cycles or impacts in silica and carbonate sands as shown in Fig. 2. By inspection, the shaft resistance ratio drops to about 18% to 36% of peak at 10000 cycles. For typical operating frequencies of vibro hammers, the 10000 cycle threshold is reached in about 4 to 8 minutes. This ratio is the same as the friction fatigue parameter β shown in Table 1 (Jonker 1987 and Fischer et al. 2013).

Riffgat Windfarm

The Riffgat Wind Farm project in the German part of the North Sea involved the installation of 30 wind turbines on steel monopiles (Middendorp and Verbeek 2012, Verbeek et al. 2013 and Fischer et al. 2013). These monopiles weigh between the 4700 and 7100 kN, are between 53 and 70 m long, with a diameter of 4.7 m at the pile head and between 5.7 and 6.5 m at the pile toe.

Initially the vibratory driver was meant to be used as a stabbing tool but given strict German environmental rules and to mitigate potential environmental issues due to noise and vibrations, the pile installation contractor suggested the use of a vibro hammer for full pile penetration. Using this hammer type also allowed the easy repositioning of a pile when the initial installation angle is exceeds tolerance, as experienced during project execution.

Based on the results of driveability studies by Allnamics Pile Testing Experts BV, a modular hammer consisting of four APE 600 units was built specifically for the project (the Super Quad Kong) as shown in Fig. 3. After driving the first monopiles it became evident that the capacity of this modular hammer was more than adequate for the piles in sandy soils. However, other locations were underlain by strong clay layers where pile driving with a vibratory hammer is typically considered more difficult given the higher shaft resistance ratio compared to sand as well as the higher damping values. Because the driveability predictions showed potential refusal before the target depth was reached, a small scale vibratory driving test was conducted at the clay locations using a pile instrumented with strain and acceleration transducers near the pile head to perform Vibratory Driving Analysis (VDA). The VDA results were then used to fine-tune the driveability studies for the monopoles yet to be installed. The results confirmed that the modular hammer was suitable to install all piles to a stable position.

Fig. 2. Degradation of shaft resistance vs. number of cycles (from White 2005).

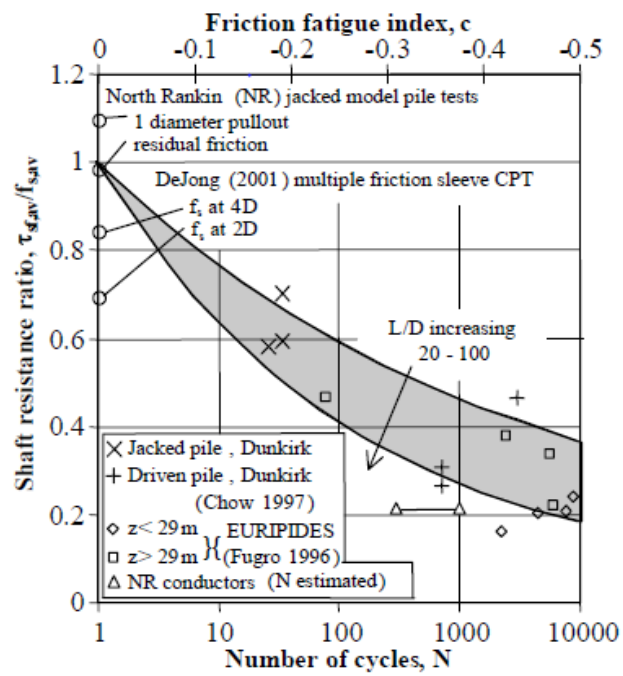


Table 1. Typical Soil Fatigue β Values (from Middendorp and Verbeek 2012).

Type of Soil	Fatigue Factor β
Round Coarse Sand	0.10
Soft Loam/Marl, Soft Loess, Stiff Silt	0.12
Round Medium Sand, Round Gravel	0.15
Fine Angular Gravel, Angular Loam, Angular Loess	0.18
Round Fine Sand	0.20
Angular Sand, Coarse Gravel	0.25
Angular/Dry Fine Sand	0.35
Marl, Stiff/Very Stiff Clay	0.40

Due to concerns regarding soil strength degradation by vibratory driving, the Owner required pile installation to be completed using an impact hammer so that a blow count could be obtained as

an indication of the pile's vertical compression resistance. As such, the last 10 m embedment of the piles was completed using an IHC S-1800 hydraulic hammer with a ram weight of 735 kN. Allnamic's driveability studies demonstrated this to be a rather conservative approach which was confirmed during the installation. After switching from the vibro hammer to the impact hammer (a process that took several hours, and required the installation of Noise Mitigation System around the pile), the recorded blow count at the start of impact driving was approximately 100 blows/250 mm, dropping to approximately 25 blows/250 mm once the pile began to move. This relatively high blow at beginning of driving using the impact hammer clearly demonstrated a strong and immediate recovery of the soil strength after vibratory driving.

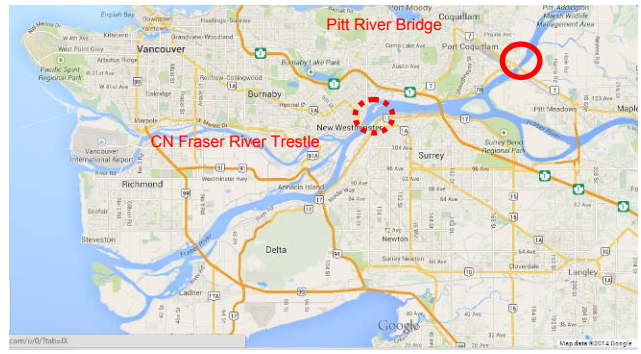
Fig. 3. Super Quad Kong (from Verbeek et al. 2013).

Vancouver Experience

Location and Geology

The locations of the two sites near Vancouver are shown on Fig. 4 with the solid red circle corresponding to the Pitt River Bridge site and the dotted red circle to the CN Fraser River Trestle site. As noted in Tara (2012), significant portions of the Lower Mainland of British Columbia including the Richmond, Pitt Meadows and lowland areas of Delta, Surrey and Port Coquitlam are underlain by Fraser River Delta deposits. Pleistocene (glacially over ridden) deposits often occur at depths of 100 m or more. When the last glacier ice retreated, sea level was much higher than today and marine and glaciomarine depositional conditions generally prevailed. During this period much of deep silt and clay deposits were formed. Excluding the discontinuous fills, the upper deposits of silt, sand and/or silty sand with clay seams are believed to have been deposited more recently in a shallow water deltaic environment.

Fig. 4. Site locations (from Google Maps).



Soil Conditions

Below the discontinuous fill, the general sequence of soils comprises an interbedded unit of silt, clay and/or sand of variable thickness over a thick deposit of silty clay extending to considerable depth (Tara 2012). These materials overlie very dense Pleistocene deposits of glacial till or drift and inter glacial sediments. Fig. 5 and 6 show the soil profile determined from CPT at the Pitt River Bridge and CN Fraser River Trestle sites, respectively. The soil types in the profile are inferred from the I_c value (Robertson 2009) derived from the CPT data.

Table 2. Soil Behaviour Type and I_c from CPT (from Robertson 2009).

Zone	Soil Behaviour Type	I_c
1	Sensitive, fine grained	N/A
2	Organic soils – peats	> 3.6
3	Clays – silty clay to clay	2.95 – 3.6
4	Silt mixtures – clayey silt to silty clay	2.60 – 2.95
5	Sand mixtures – silty sand to sandy silt	2.05 – 2.6
6	Sands – clean sand to silty sand	1.31 – 2.05
7	Gravelly sand to dense sand	< 1.31
8	Very stiff sand to clayey sand*	N/A
9	Very stiff, fine grained*	N/A

*Heavily overconsolidated or cemented

As is obvious from the pore pressure readings, ground water was encountered at shallow depth and the level was influenced by the water level in the adjacent Pitt and Fraser Rivers. Further, sub-artesian conditions are present below about 43 m depth at the Pitt River site.

Pitt River Bridge 2005 Test Pile Program

In the fall of 2005, a test pile program was conducted by BC Ministry of Transportation and Infrastructure (Golder 2006 and Tara 2012). Two indicator piles, a 610 mm diameter closed toe and 1067 mm diameter

Fig. 5. Soil Profile at Pitt River Bridge.

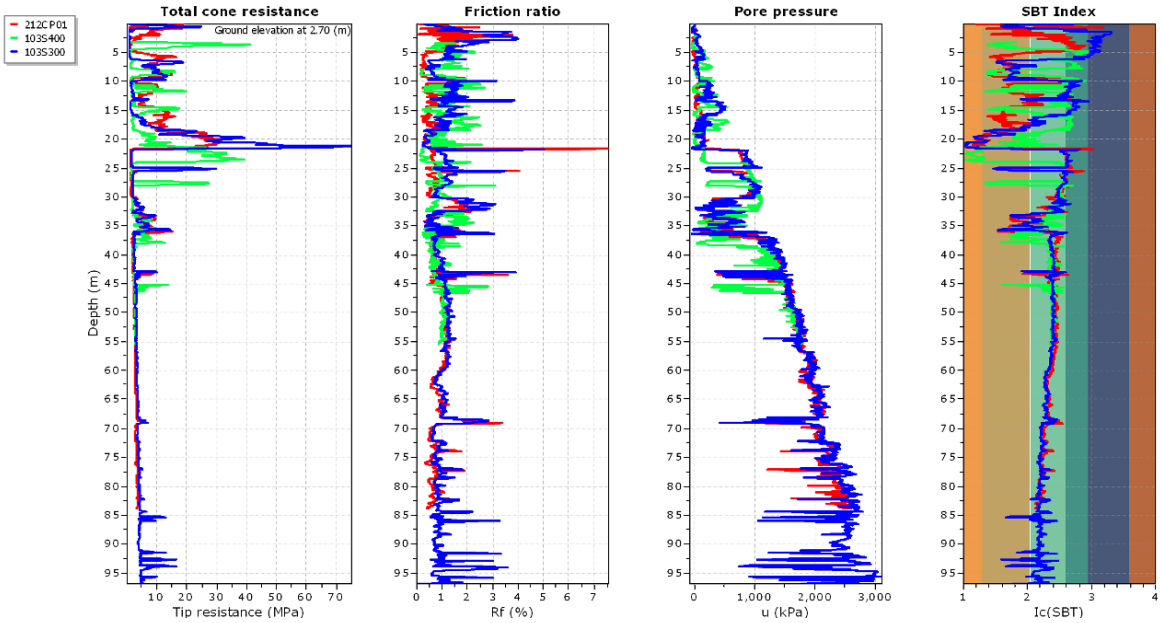
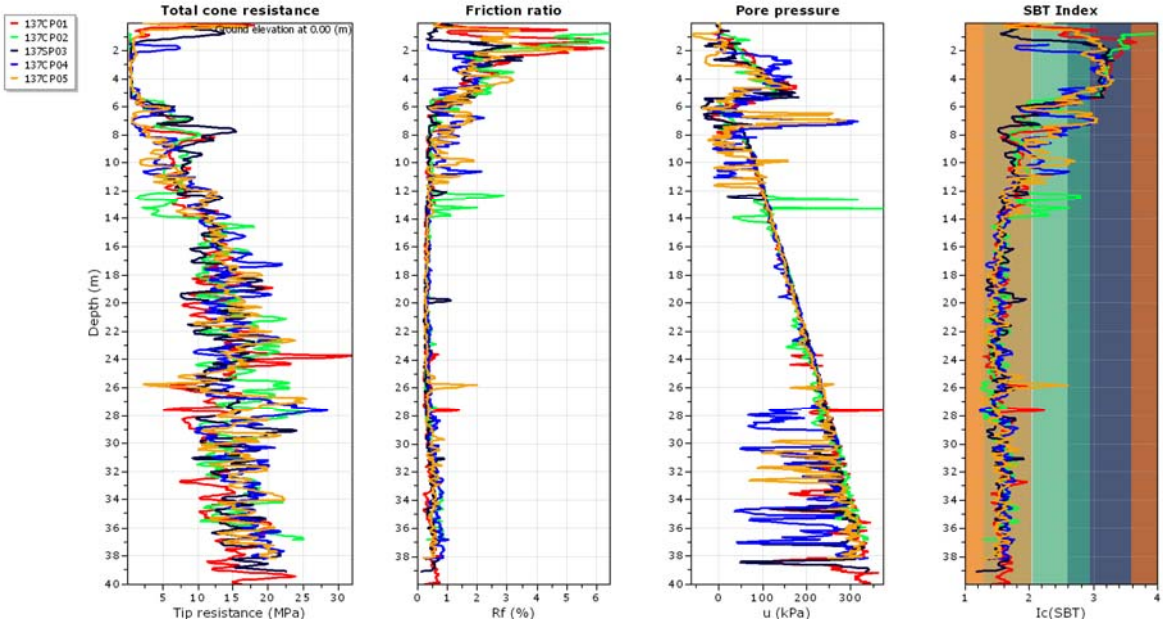


Fig. 6. Soil profile at CN Fraser River Trestle.



described herein. The 1067 mm diameter pile was reported to have a wall thickness of 19 mm and an internal shoe that effectively doubled pile wall thickness at the toe.

The 1067 mm pile was installed to 23 m penetration using an APE 300 vibratory driver and then driven to nominally 100 m penetration using diesel hammers. Installation records included in Golder (2006) indicate that approximately 37 minutes were required to advance the pile from 6 to 23 m. The penetration resistance in seconds/305 mm is shown on Fig. 7. It is assumed that pile advancement was not restricted by the crane winch.

Pitt River Bridge Production Piling

Installation of production piles for the new bridge began in the summer of 2007. Work started at pier E1 at the east or left bank of Pitt River where a static load test was planned. The piles were 1830 mm diameter by 24.4 mm thick wall open ended steel pipe. A driving shoe was included that effectively doubled the pile wall thickness at the toe. Although it was planned to install the piles using an APE D180 diesel hammer, the Manitowoc 4100 crane could not safely lift the hammer and pile at the same time. Consequently the diesel hammer could not be safely used until the piles had sufficient embedment to support the weight of the hammer and therefore pile

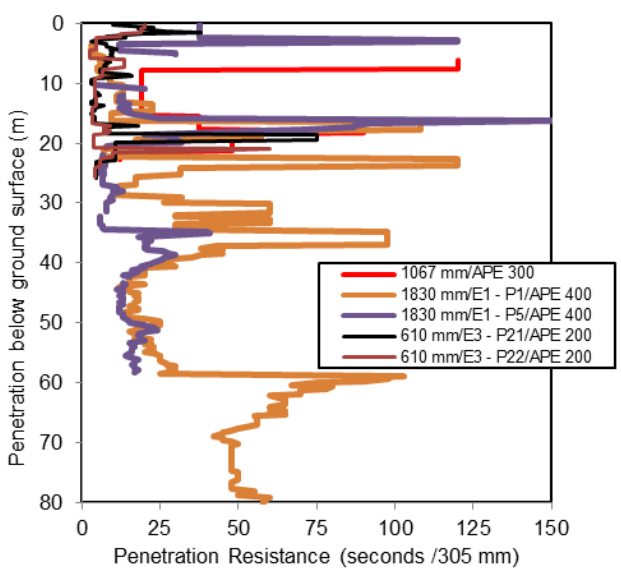
installation started using an APE 400 vibratory hammer. The penetration resistance during installation for three of the nine E1 Pier piles is also shown on Fig. 7. It should be noted that Pile P1 was installed to 80 m using the vibro hammer whereas the others were advanced to only 58 m prior to impact driving.

High strain dynamic testing (HSDT) was conducted at end of drive at Piles P1 and P5 and a series of restrikes tests at Pile P1. Tara (2012) used superposition theory to show the development of shaft resistance with time at Pile P1. As noted by Tara (2012), although the overall shaft resistance at Pile P1 is estimated to be similar to the shaft resistance back calculated from the static load test (SLT) conducted on Pile P5, the distribution of resistance appears to differ significantly. One possible explanation for this difference is the greater depth of penetration using a vibratory driver.

Installation records for 610 mm diameter piles installed at Pier E3 using an APE 200 are also shown on Fig. 7.

It is interesting to note that higher penetration resistances shown in Fig. 7 correspond quite closely with cohesive soil layers identified by the CPT data.

Fig. 7. Pitt River Bridge Pile Installation Records.



CN Fraser River Trestle

CN replaced the former timber trestle approach to the main Fraser River crossing upstream of the Patullo Bridge at the location shown on Figure 4. The timber trestle was supported on driven timber piles with estimated embedments of about 15 m. Work for the replacement trestle was completed in three phases. Piles for the new structure comprised HP 360 x 132 steel piles installed in groups of four to nominally 30 to 34 m. The piles were designed essentially as friction piles in the sand deposit shown on Fig. 6. Piles for the first phase of construction, nearest the main crossing of the Fraser River, were installed using impact hammers comprising both conventional gravity drop hammers and an ICE I 30 diesel. To mitigate vibration induced settlements of the existing trestle during pile installation, sequencing of pile installation had to be carefully controlled to keep the settlements as uniform as possible across the former timber structure. Initially low energy blows were used for pile installation. However, as this resulted in relatively large settlements of the existing trestle, HSDT was conducted to assess pile capacity and the potential effect of hammer energy on trestle settlement. Signal matching analyses conducted on the HSDT results confirmed relatively high soil quakes for the pile-soil system. As high quakes tend to dissipate driving energy and render pile installation more difficult, parametric driveability analyses were conducted to assess the overall benefit of using higher energy blows during installation. The analyses showed that increasing the driving energy would result in an overall decrease in total energy input into the soil. As the amount of vibration induced settlement is believed to be related to the total number of load cycles/energy input (Massarsch 2004), the remainder of the piles in Phase I were installed using impact driving methods with high energy blows. Given the proximity of the work to the existing track, the pile installation work was challenging and proceeded relatively slow as pile driving operations had to stop whenever a train was passing.

In an effort to further reduce construction induced settlements of the existing timber trestle, piles for Phase II were installed using a Resonant Pile Driver Model No. RD 140 (Janes 2012).

Prior to commencing work on Phase III of the trestle replacement project, CN requested that Thurber Engineering Ltd. explore the option of using a conventional vibratory driver to install piles to the design penetration. Part of the assessment involved a review of experience at nearby sites such as Pitt River Bridge. Initial comparisons of steel cross sectional and surface areas and installed lengths of the 610 mm open ended pipe piles to the HP 360 x 132 section suggested that a vibratory driver with an eccentric moment of about 50 kg·m operating in the range of 28 to 30 Hz would likely be capable of advancing the H piles to the design penetration of 30 to 34 m. As a number of vibratory drivers having the above characteristics were readily available in the Vancouver area, it was decided to proceed with a more detailed, driveability analysis.

Driveability Studies

Using available geotechnical information including CPT and SCPT profiling and the results of previous high strain dynamic testing, a driveability assessment was undertaken using the wave equation analysis software program AllWave-PDP by Allnamics-USA. AllWave-PDP is based on the method of characteristics (Middendorp 2004) and has been successfully used to demonstrate the feasibility of installing 22 m diameter piles for the Hong Kong Zhuhai Macau Bridge and Riffgat Windfarm Project (Middendorp and Verbeek 2012, de Neef, Middendorp and Bakker 2013, and Fischer et al. 2013) described earlier.

Characteristics of Vibratory Drivers Used

The vibratory drivers used at the two sites were manufactured by American Piledriving Equipment (APE) and International Construction Equipment, Inc. (ICE). Pertinent information regarding the drivers is summarized in Table 3 below.

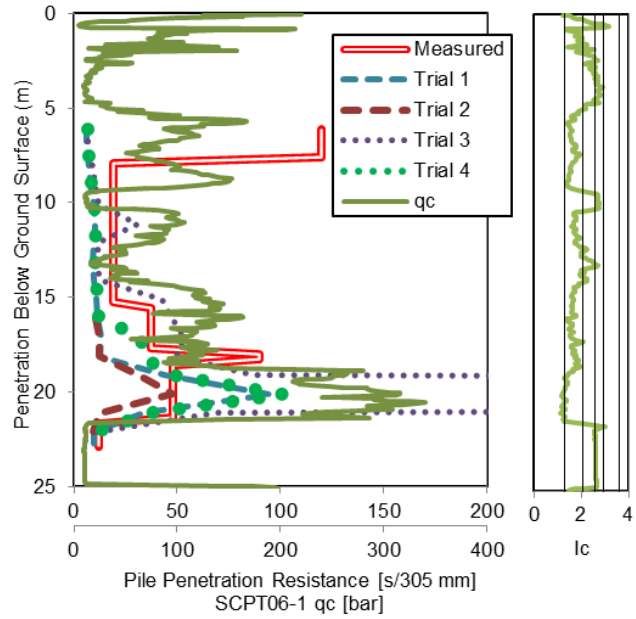
Table 3. Vibratory Driver Details.

Model	Eccentric Moment (kg·m)	Maximum Frequency (vpm)	Centrigual Force (kN)
APE 200	51	1700	1606
APE 300	75	1500	1842
APE 400	150	1400	3203
ICE 44B	51	1800	1789

Calibration at Pitt River Bridge

Driveability predictions were conducted for the 1067 mm indicator pile and the 1830 mm and 610 mm diameter production piles at Pitt River Bridge using AllWave-PDP with data from nearby CPTs to estimate the relevant soil parameters. The software was used to estimate the required time for the pile to penetrate 1 foot (305 mm) which is simply the inverse of the penetration rate and can be compared with blows per unit of penetration typically recorded during impact driving. The results of the driveability predictions (Trials 1 to 4) using an APE 300 for the 1067 mm pile are shown on Fig. 8. Trials 1 and 2 are for shaft and toe fatigue factors of 0.25 and quakes of 2 and 1 mm, respectively. Trials 3 and 4 are for shaft and toe fatigue factors of 0.30 and quakes of 2 and 1 mm, respectively. Also shown on the figure is the CPT tip resistance q_c and I_c. By inspection, the results appear to be quite sensitive to the choice of fatigue factor and quake. Further, it appears that penetration resistance is largely controlled by the predominant soil type as defined by I_c.

Fig. 8. Measured Pile and CPT Penetration resistance for 1067 mm Pile.



The same general methodology was applied to the 1830 mm piles installed at Pier E1. For this case, several trial simulations were conducted with fatigue factors ranging from 0.25 to 0.40. The analyses are labelled Trials 1 to 6 on Fig. 9. Trials 1 and 2 correspond to fatigue factors of 0.25 and quakes of 1 and 2 mm, respectively. Trials 3 and 4 correspond to fatigue factors of 0.35 and quakes of 1 and 2 mm, respectively. Trials 5 and 6 correspond to fatigue factors of 0.40 and quakes of 1 and 2 mm, respectively. By inspection, the simulation is quite good to about 25 m with a typical fatigue factor of 0.25 reflecting the presence of predominantly sandy material whereas the simulation is better between 25 and 40 m using a fatigue factor of 0.40 reflecting the presence of predominantly silty materials.

The same general methodology was also applied to the 610 mm piles installed at Pier E3 where a greater overall thickness of silt ($I_c \geq 2.6$) was present above 25 m depth. Also, the CPT data on Fig. 5 and 9 shows a relatively dense zone with q_c approaching 30 MPa and an I_c of less than 1.31 between 21 to 24 m. For this case, several trial simulations were conducted with fatigue factors of 0.25 to 0.65. The analyses are labelled Trials 1 to 6 on Fig. 10. Trials 1 and 2 correspond to fatigue factors of 0.25 and quakes of 1 and 2 mm, respectively. Trials 3 and 4 correspond to fatigue factors of 0.45 and quakes of 1 and 2 mm, respectively. Trials 5 and 6 correspond to fatigue factors of 0.65 and quakes of 1 and 2 mm, respectively.

Fig. 9. Measured Pile and SCPT Penetration resistance for 1830 mm Piles E1-P1 and E1-P5.

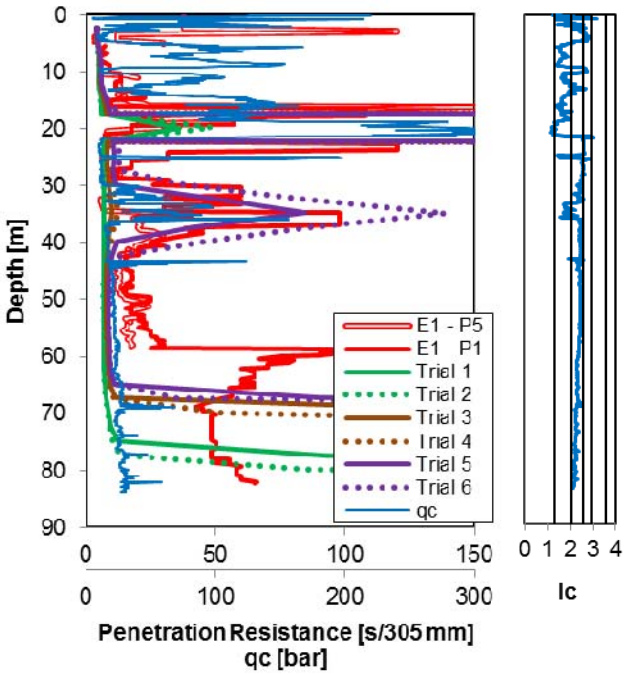
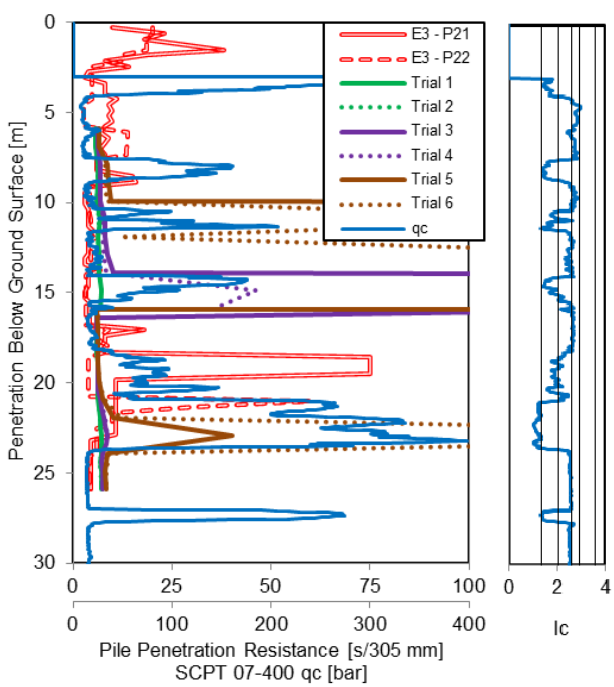


Fig. 10. Measured Pile and SCPT Penetration resistance for 610 mm Pile E3-P21 and E3-P22.



By inspection, the simulation is quite good above 18 m with a more typical fatigue factor of 0.25 but requires the use of a larger fatigue factor below. Although fatigue factor selection is primarily affected by the soil type as identified by I_c, these simulations suggest that some other phenomenon such as plugging or partial plugging may have been controlling the pile behaviour. It is also possible that the presence of dense zones affected the simulation outcome. Rodger and Littlejohn (1980) noted that, where high point resistance is anticipated, preference is to use a "low frequency" vibro hammer with "large displacement amplitude".

Driveability Prediction for CN Fraser River Trestle

The calibration results at Pitt River Bridge for 610, 1067 and 1830 mm diameter pipe piles suggested that installation of HP 360 x 132 piles to nominally 34 m would be feasible at the CN Trestle site using an APE 200 or equivalent, especially given the predominantly soil type at the site. Trial simulations were performed using AllWave-PDP. To assess the robustness of the proposed installation method, simulations were performed with fatigue factors of 0.25, 0.30 and 0.35 for both shaft and toe. Quakes were maintained at 1 mm based on the modelling results for the Pitt River piles. Trial 1 was for a fatigue factor of 0.25, Trial 2 for 0.30 and Trial 3 for 0.35. The results are summarized in Table 4.

Table 4. Estimated HP 360 x 132 Pile Penetration Resistance.

Depth [m]	Installation Time/Pile Segment [s]				
	Trial 1	Trial 2	Trial 3	Average	Pile 56
Beta	0.25	0.30	0.35	N/A	N/A
0.0 - 13.7	199	202	204	408	120
13.7 - 29.0	295	300	304	357	240
29.0 - 34.6	108	131	138	303	180

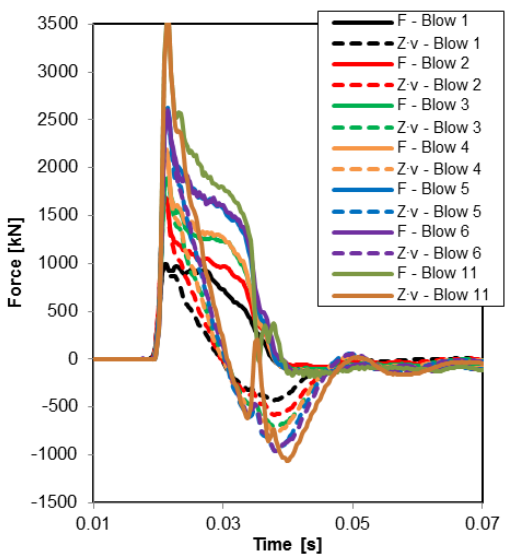
Production Piling at CN Fraser River Trestle

Pile installation for Phase III began in early 2013. All piles were advanced in 3 segments to a design penetration of about 34 m using an ICE 44B (see relevant characteristics given in Table 3). Review of the installation records suggests that the total installation time typically ranged from 12 to 29 minutes (18 minutes on average) using the vibratory hammer whereas, at 10 blows/305 mm on average, impact driving would require about 25 to 30 minutes using a diesel impact hammer and double to triple that using a gravity impact hammer. The penetration resistance for a typical pile (Pile 56) is also shown in Table 4. Unfortunately, only the total installation time for each of the 3 pile segments was recorded. As a consequence, the resolution of the measured penetration resistance is relatively coarse. Nonetheless, by inspection the predicted and measured penetration resistances compare reasonably well.

High strain dynamic testing was conducted at 8 to 12 days after completion of pile installation using a 38 kN drop hammer. The measured force (F) and

impedance times velocity (Z·v) traces and of the upwave [U↑ = (F - Z·v)/2] for Pile 56 at 12 days after installation are shown on Fig. 11 and 12, respectively. Blows 1 and 2 were seating/calibration blows with drop heights of 0.3 and 0.6 m each. Blows 3 and 4 were for 1.2 m drops, Blows 5 and 6 for 1.8 m drops and Blows 7, 8, 10 and 11 were for 2.4 m each. Blow 7 was not included in the figures due to space restriction and Blow 9 (a 3.0 m drop) due to data quality issues. While it may difficult to visualize what is going on with the shaft resistance in the F and Z·v plots on Fig. 11., it should be quite obvious in the U↑ plots in Fig. 12 that, from Blow 5 onward, the shaft resistance remains relatively unaffected by additional blows.

Fig. 11 Measured F and Z·v from 12 day test at Pile 56.



Signal matching analyses were conducted using the software program AllWave-DLT by Allnamics-USA. The analysis used the results of CPT11-1 as input for evaluating Blow 5 initially. The Blow 5 soil model was then used as a basis for evaluating the other blows applied to Pile 56. Fig. 13 summarizes the results of the signal matching analyses in the form of equivalent load-movement response of a static load test. By inspection, the maximum resistance is obtained with Blow 5 and subsequent blows show very similar maximum resistances and stiffnesses.

Fig. 14 and 15 provide plots of F and Z·v and of U↑, respectively, for Pile 60 which was installed by impact driving in Phase I. Blows 1 to 6 are shown with Blow 1 a calibration blow using a 0.6 m drop height and the remaining blows comprising 1.8 m drops. Similar to Pile 56, signal matching analyses were conducted and the results are summarized on Fig. 16.

Fig. 12. Upwave U↑ from 12 day test at Pile 56.

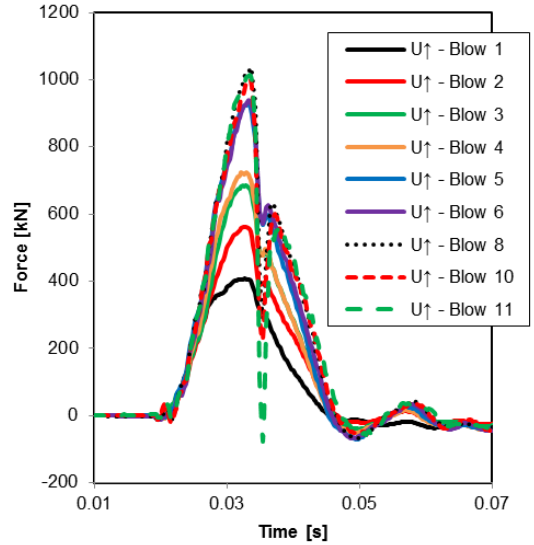


Fig. 13. Static Load Test Simulation for Blows 1 to 8, 10 and 11 at Pile 56.

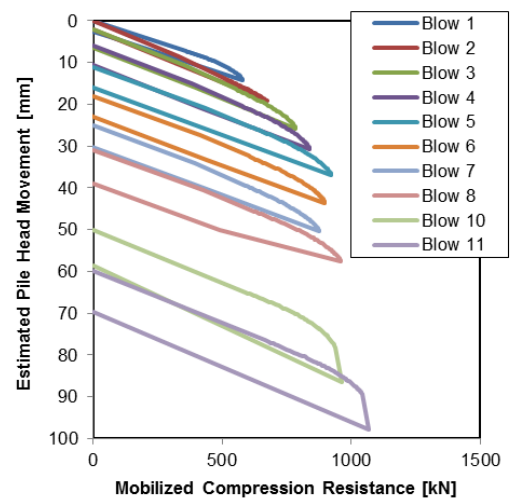
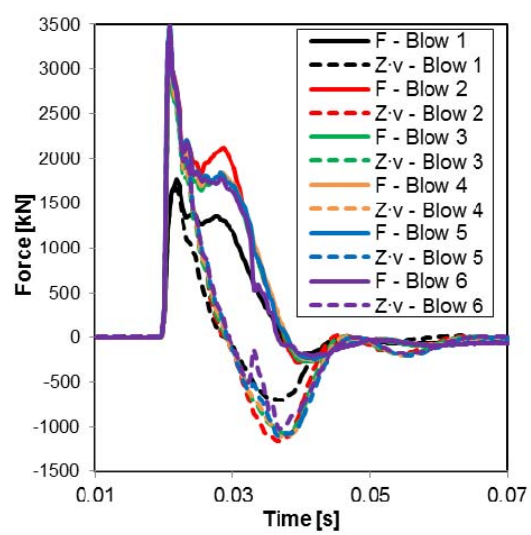


Fig. 14. Measured F and Z·v from 11 day test at Pile 60.



From the upwave plots on Fig. 15 it is apparent that the greatest mobilized resistance would be expected for Blow 2 with a significant loss of resistance with subsequent blows as demonstrated on Fig. 16.

Fig. 15. Upwave U↑ from 11 day test at Pile 60.

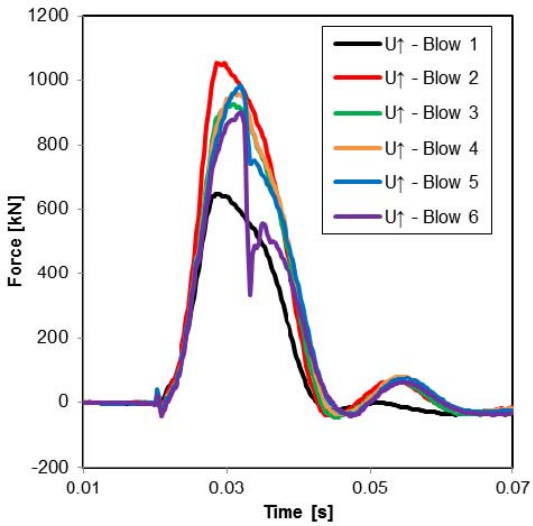
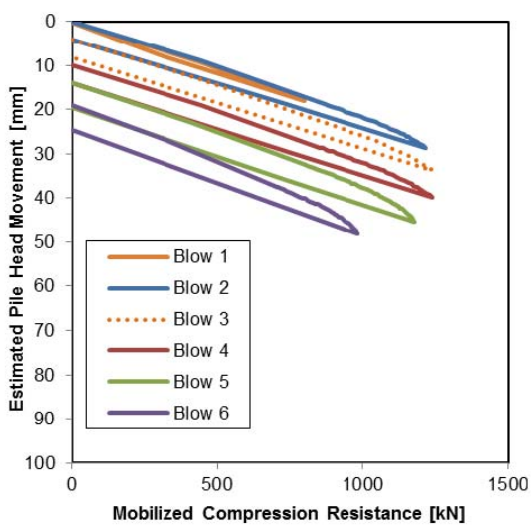
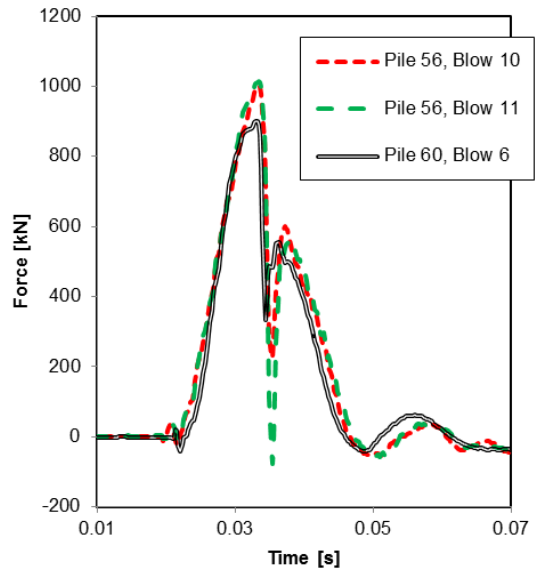


Fig. 16. Static Load Test Simulation for Blows 1 to 6 at Pile 60.



By inspection, the shaft and toe resistance of the impact driven pile appears to decrease much more rapidly than the vibro driven pile. The 20% reduction is primarily due to loss of shaft resistance and this reduction corresponds reasonably well with the loss expected based on Fig. 2. Further, the soil resistances represented by the measured upwaves tend to become similar for both impact driven piles and vibratory driven piles with increasing number of blows as shown in Fig. 17 where Blows 10 and 11 on Pile 56 are plotted with Blow 6 on Pile 60. While we expect to see the driving resistance decrease with the application of multiple blows as in the case of recommencing driving following splicing, the rapid loss in resistance is significant. Further, the fact that this behaviour is not apparent at Pile 56 is worth exploring. As the signal matching analyses suggest that both Piles 56 and 60 had similar overall vertical compression resistances and stiffnesses near the end of the testing sequence, this would suggest that consideration should be given to using vibratory methods to advance piles in the deltaic soils such as those encountered in the Fraser River Delta near Vancouver.

Fig. 17. Comparison of Upwave at Vibro and Impact Driven Piles 56 and 60, respectively.



Conclusions

This paper demonstrates that vibro hammers provide an effective means of installing open ended steel pipe and H piles in deltaic deposits in and around the Vancouver area. The driveability simulations also indicate that the installation process can be modelled reasonably well and the vibro hammer selection process can be refined with selection of appropriate fatigue factors. Additional testing, analysis and calibration is required to develop fatigue factors for our local conditions. The results of high strain dynamic testing on H piles at the trestle site suggest that, at large displacements, mobilized resistance of both impact and vibro driven piles is comparable.

Only rudimentary monitoring of pile installation time per unit of depth was performed during advancement of piles referenced herein. To better understand the vibratory installation process, more attention must be paid to monitoring the actual installation process. One simple improvement would be to carefully monitoring rate of penetration. Viking (2006) provided recommendations for a simple

optical sensor that can be attached to the pile for monitoring movement amplitude during installation. The next level of monitoring that should be considered if we want to develop a better understanding of the installation process is to apply Vibratory Driving Analysis (VDA), which involves instrumenting the pile with strain gauges and accelerometers (similar to Pile Driving Analysis) as was done at Riffgat.

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