Pile Installation Effect on Mobilized Side Shear Resistance in Fraser River Deposits

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ABSTRACT

The friction fatigue phenomenon correlates the effect of the installation method on the degradation of mobilized side shear resistance, with vibro-hammers having been observed to result in further degradation of the side shear resistance in comparison with impact and jacked installation methods as reported by White (2005). In the Lower Mainland of British Columbia, Canada, nearshore structures constructed along the Fraser River are generally supported on open-ended pipe piles that are either partially or fully installed using vibro-hammers. This paper will review the results of high-strain dynamic testing on open-ended pipe piles installed in the Fraser River sediments to assess the effect of the installation method on the predicted mobilized side shear resistance.

Keywords: High-strain dynamic testing, friction fatigue, Fraser River Sediments, open-ended pipe piles, pile setup

1 INTRODUCTION

Pile installation, completed by means of either impact or vibro-driving, has been observed to result in a degradation of the side shear resistance. The phenomenon, first termed as friction fatigue by Heerema (1980) has since been the center of several studies and publications. For instance, White (2005) compiled several pile loading test results carried out in silica and carbonate sands to demonstrate the relation between side shear resistance degradation and the applied number of cycles during pile installation as shown in Figure 1.

Driven pile installation in Fraser River sediments, in the Lower Mainland of British Columbia, Canada typically includes vibro-driving the piles to refusal, or to a maximum embedment typically required for constructability reasons. The vibro-driving is then followed by impact driving to the required pile toe embedment.

In this paper, we will examine the effect of the installation method on the mobilized side shear resistance in open-ended pipe piles by revisiting the high-strain dynamic test (HSDT) results completed for the Pitt River bridge project along with another project recently completed further downstream along the Fraser River. The approximate site locations are highlighted in Figure 2.

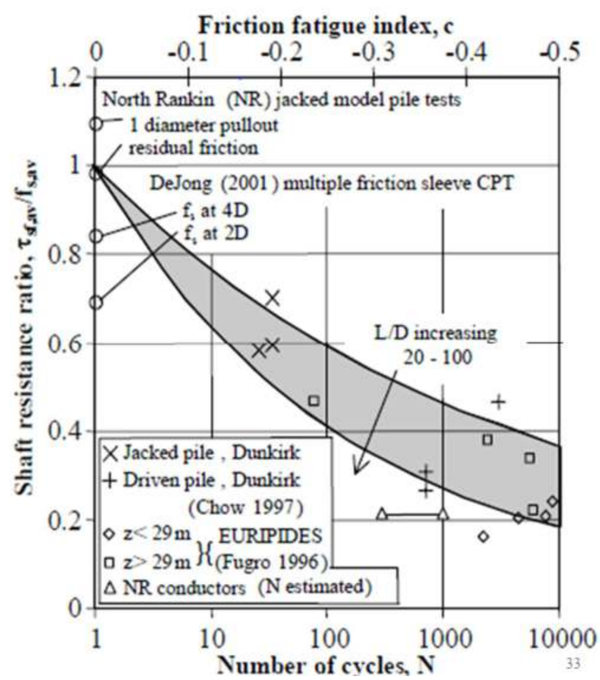

Fig. 1. Degradation of side shear resistance vs. Number of cycles (from White, 2005).

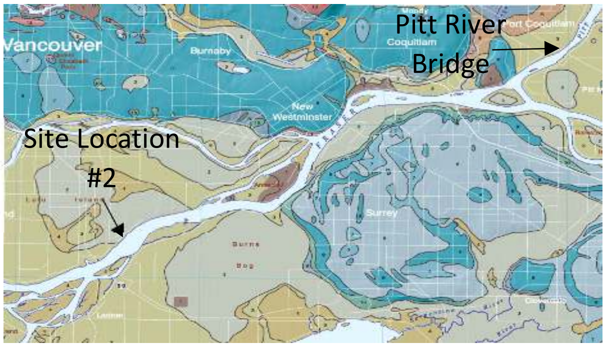

Fig. 2. Projects' location.

2 PITT RIVER BRIDGE PROJECT

The Pitt River bridge is a 380 m long, cable-stayed multi-lane bridge. The project included the completion of full-scale, top-down static and high-strain dynamic loading tests during the pre-bid and construction stages, and high-strain dynamic testing during production. Further information on the geologic conditions, and full-scale and dynamic pile testing can be found in Tara (2012).

2.1 Geotechnical Conditions

The subsurface conditions at the site generally comprised an interbedded unit of silt, clay and sand of variable thickness over a thick deposit of silty clay extending to nominally 100 m depth. These materials overlie very dense Pleistocene deposits of glacial till or drift and inter-glacial sediments. Figure 3 shows the soil profile determined from CPT/SCPT tip resistance (q_t) and water content (w_n). The higher water contents between nominally Elev. -40 m to -60 m are likely due to the presence of higher organic content in this zone.

While ground water was generally encountered at shallow depth and was influenced by the water level in Pitt River, deep drilling and CPT pore pressure dissipation tests in granular zones and glaciated deposits revealed artesian conditions at depth.

2.2 Installation Procedure of HSDT Piles

Pier E1 comprised 1,830 mm x 25.4 mm, open-ended pipe piles that were installed initially using a vibro-driver followed by an APE D180-42 diesel impact hammer. Piles P1 and P5 at pier E1 were installed in a similar fashion and subsequently subjected to HSDT. The major difference in installation, however, was that the vibro-driver was used to advance pile P1 to 81 m embedment while P5 was only advanced to 59 m embedment. Figure 4 below summarizes the driving sequence for the two piles. The downtime for splicing for the two piles is not presented in the Figure.

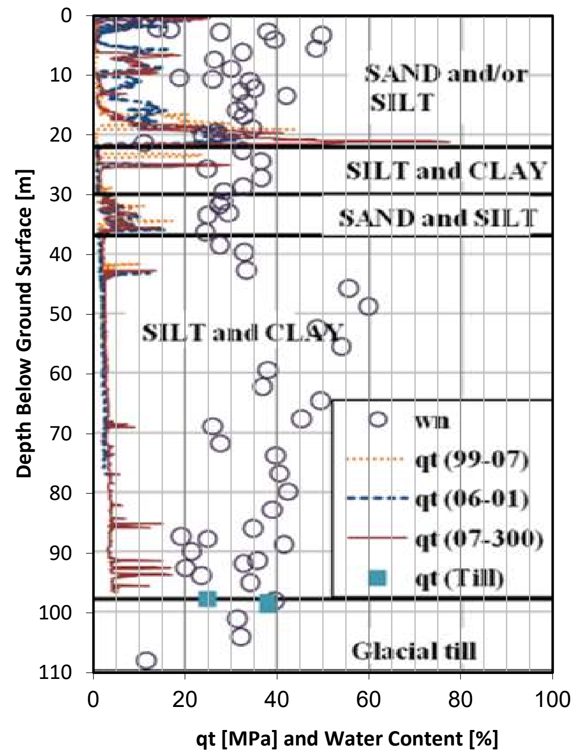

Fig. 3. Subsurface Conditions and CPT Tip Resistance at Pitt River Bridge.

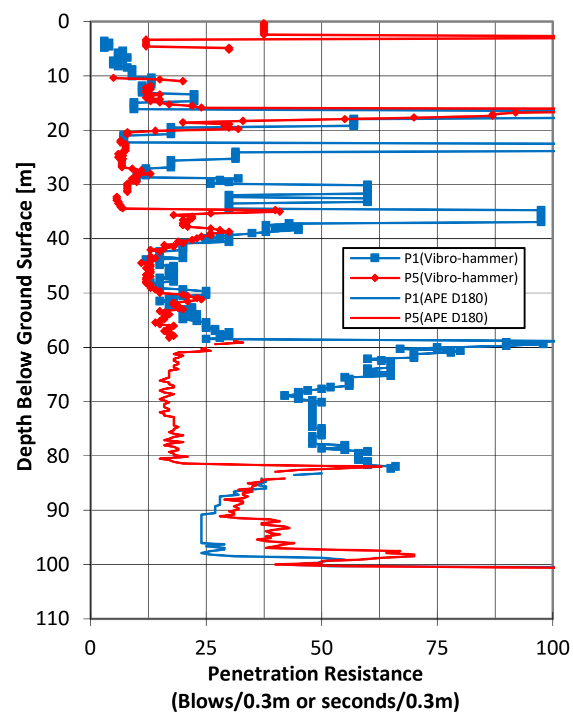

Fig. 4. Recorded Penetration Resistance during Installation of Piles P1 and P5 at Pier E1, Pitt River Bridge.

2.3 HSDT Results

Figure 5 summarizes the predicted side shear resistance from the HSDT results at end of drive. During testing, the maximum transferred energy was 278 kJ for both piles and the recorded permanent sets were 1.4 mm and 1.2 mm for piles P1 and P5, respectively. While the permanent sets were relatively similar for both piles, they are below the minimum recommended permanent set of 2 mm to fully mobilize the side shear resistance per ASTM D4945-17. Note that PDI's training notes (2017) recommend a permanent set of not less than 2.5 mm and not exceeding 8 mm.

In general, the mobilized side shear resistance in the upper 58 m is similar in both piles. Between 58 m and 81 m, the increase in side shear resistance in P5 is about double that of pile P1. This noted difference is believed to reflect the difference in the installation methods within this zone.

Between 81 m and 92 m depth, the slopes of the side shear resistance curves are similar, which is to be expected given that the installation method and recorded penetration resistance (i.e. blows/0.3 m) were similar. Below about 94 m depth, there is a marked difference in the mobilized side shear resistance where the piles are inferred to be embedded in the till-like material. This is likely attributed to a variation in the density/consistency within the till-like material rather than the installation procedure.

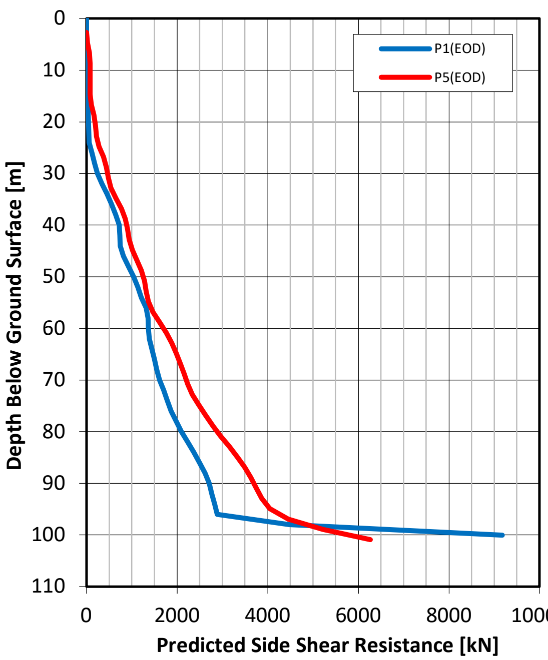

Fig. 5. Predicted Side Shear Resistance at End of Drive of Piles P1 and P5 at Pier E1 at Pitt River Bridge.

Restrike testing of Pile P1 was completed 51 days after end of drive. Figure 6 compares the predicted side shear resistance at end of drive and 51-day restrike. The maximum transferred energy was 278 kJ and 275 kJ at end of drive and restrike, respectively. The recorded permanent sets were 1.4 mm and 1 mm during end of drive and restrike testing, respectively.

Unlike Figure 5 where a marked difference in the slopes of the side shear resistance curves was observed following the switch to impact driving, the slope of the side shear resistance curve from the restrike test is constant from about 60 m depth which suggests that the installation method has no effect on the long-term mobilized resistance.

While it is unlikely that the side shear resistance was fully mobilized during both tests given the low permanent sets observed, it is clear that the side shear resistance during restrike testing was mobilized even less, particularly when comparing the computed side shear resistance values in the till-like material below 94 m depth. As such, the data presented in Figure 6 is considered insufficient to draw any conclusion on the impact of the installation method on the long-term mobilized side shear resistance.

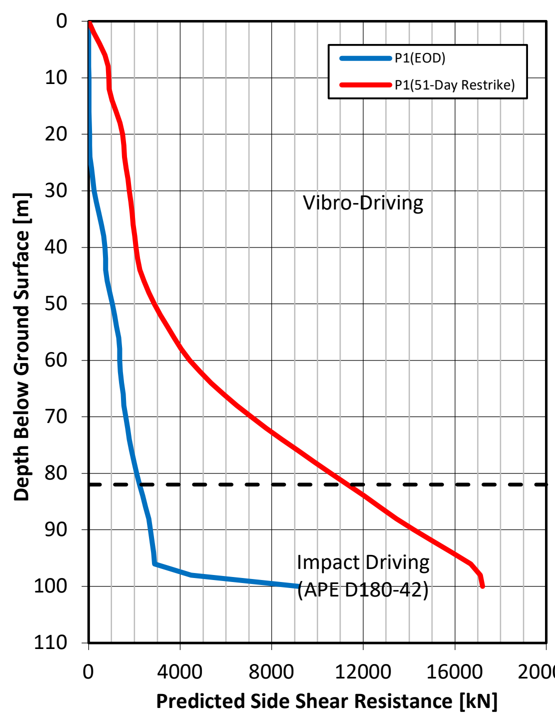

Fig. 6. Predicted Side Shear Resistance at End of Drive and at 51 Day Restrike of Pile P1 at Pier E1, Pitt River Bridge.

3 OTHER PROJECT IN FRASER RIVER DEPOSITS

Thurber completed HSDT on another project located further downstream along the Fraser River as shown in Figure 2. The project comprised the installation of 1,778 mm x 30 mm and 1,219 mm x 25.4 mm open-ended steel pipes in the Fraser River. The subsurface conditions nearshore generally comprised a sequence of Fraser River sand with trace to some fines that was encountered at mudline to about Elev. -5 m, a relatively thin layer of firm to stiff silty clay, over an interbedded unit of silt sand, sandy silt, silty clay and clayey silt extending to Elev. -100 m depth. The

CPT/SCPT tip resistance profile and water content measurements are plotted in Figure 7.

Three 1,778 mm x 30 mm piles (1 to 3) and two 1,219 mm x 25.4 mm piles (4 and 5) were installed adjacent to the CPT soundings and subjected to HSDT. The piles were initially installed using a vibro-driver, with piles 1 to 3 subsequently driven using an IHC S280 hydraulic hammer as shown in Figures 8, and piles 4 and 5 subsequently installed using a Pileco D100 diesel hammer as shown in Figure 9. Penetration resistance (i.e. seconds/0.3 m) during vibro-driving was not recorded. The downtime for pile splicing is not presented in the figures.

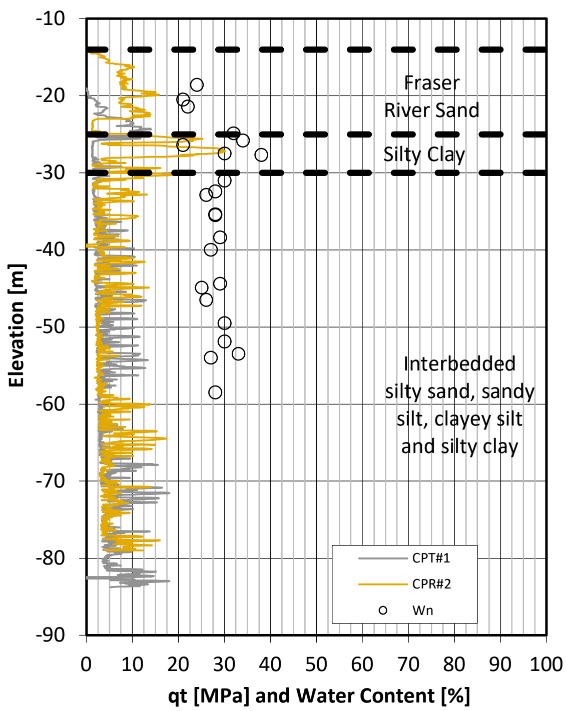

Fig. 7. Subsurface Conditions and CPT Tip Resistance.

Figure 10 summarizes the predicted side shear resistance at end of drive and beginning of restrike testing for piles 1 to 3. During testing, the maximum transferred energy varied between 177 kJ and 274 kJ. The recorded permanent sets varied between 3 mm and 5.5 mm, which is an indication that the side shear resistance was fully mobilized during testing of all piles.

Figure 11 summarizes the predicted side shear resistance for pile 4 at days 3 and 10 following end of installation, and at end of drive and 7 days after installation for pile 5. During testing, the maximum transferred energy varied between 102 kJ and 119 kJ.

The recorded permanent sets were 3.6 mm and 9.1 mm for the first round of testing of piles 4 (3-day restrike) and 5 (end of drive), respectively. For the second round of testing, the measured permanent sets were 1.9 mm and 2.1 mm for piles 4 (10-day restrike) and 5 (7-day restrike), respectively.

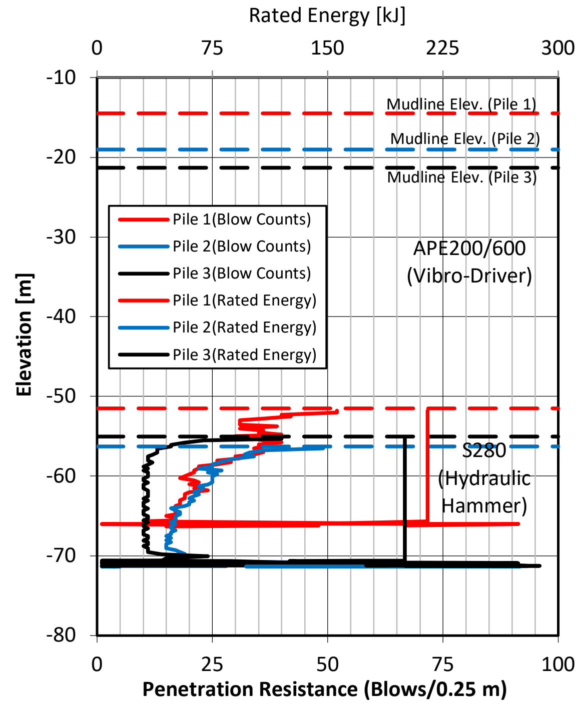

Fig. 8. Recorded Penetration Resistance for Piles 1 to 3 (1,778 mm x 30 mm).

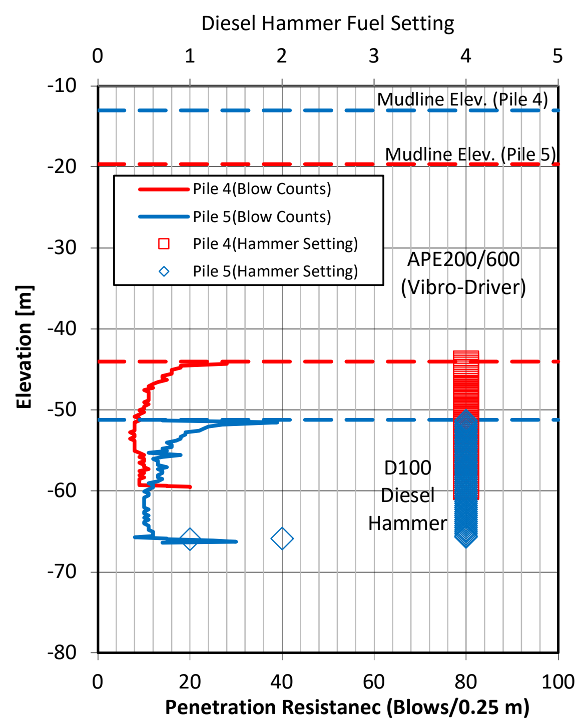

Fig. 9. Recorded Penetration Resistance for Piles 4 and 5 (1,219 mm x 25.4 mm).

By examining Figures 10 and 11, two general observations can be made. First, the mobilized side shear resistance at about or nominally above the depth where impact driving was initiated is markedly greater than the mobilized resistance above. The only exception is Pile 5 where there is no marked change in the slope of the side shear resistance curve at end of drive. A possible explanation is that the reliability of the signal matching analysis decreases where the measured permanent set exceeds 8 mm as suggested in PDI (2017) training notes.

Second, there is a marked increase in the observed setup in side shear resistance, about or nominally above the depth where impact driving was started. The only exception is Pile 4 where nominal setup was noted between the two tests. A possible explanation is that pile has experienced some setup given that the first round of HSDT was completed three days after end of drive as opposed to the other piles where the first round of testing was completed at end of drive. A second possible explanation is that the side shear resistance was not fully mobilized during the 10-day restrike test based on the recorded permanent set of only 1.9 mm, which is less than the recommended minimum of 2 mm in ASTM 4945-17.

Both observations suggest that the installation method of open-ended pipe piles has an effect on both the short and long-term, mobilized side shear resistance.

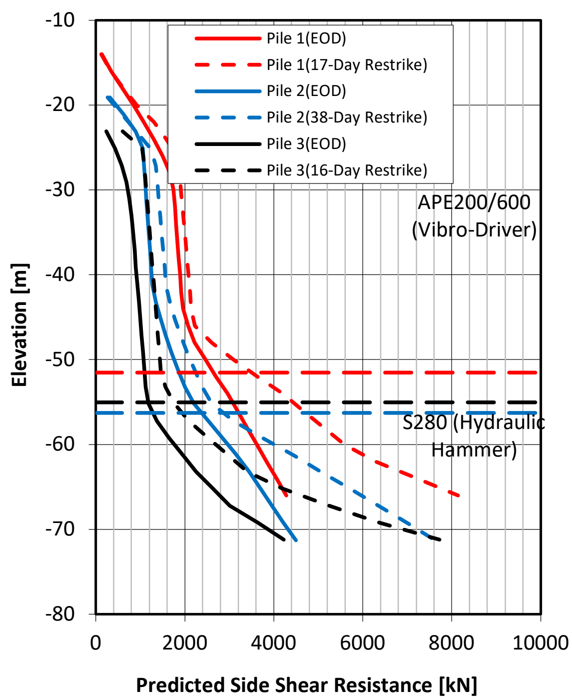

Fig. 10. Predicted Side Shear Resistance for Piles 1 to 3 (1,778 mm x 30 mm).

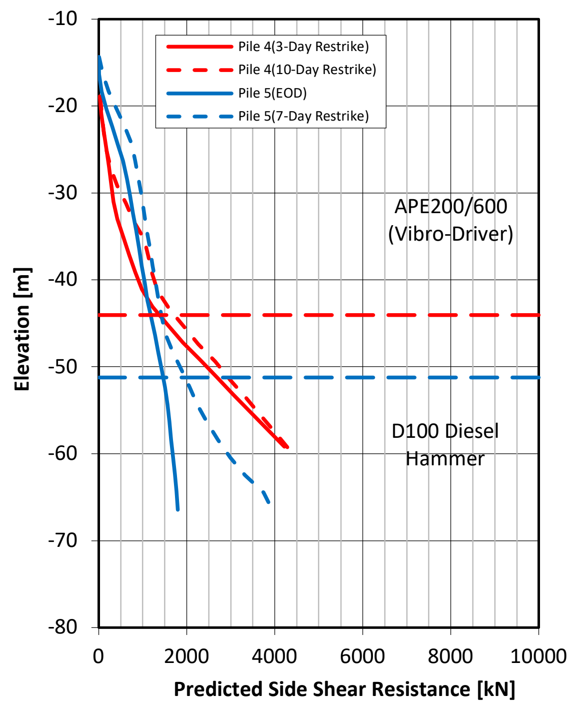

Fig. 11. Predicted Side Shear Resistance for Piles 4 and 5 (1,219 mm x 25.4 mm).

4 CONCLUSIONS

Friction fatigue was observed at two sites with differing subsurface conditions as described above. The HSDT results suggest that effect of friction fatigue is more pronounced where vibro-drivers are used to install the piles in comparison to impact hammers. Further, the effects of friction fatigue appear to affect both the short and long-term, mobilized side shear resistance of open-ended pipe piles.

The wait period between splicing of pipe sections, which could vary from hours to weeks for certain projects, is potentially another variable that has the potential to affect the mobilized side shear resistance. This was not included in our current assessment.

Finally, consideration should be given to limiting the use of vibro-drivers in soils that are prone to friction fatigue such as Fraser River sediments, particularly where the approval of the production piles is based on the mobilized resistance estimated from end of drive HSDT results.

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