

## Behaviour of impact- and vibratory-driven piles in stiff clay during installation and static loading

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### ABSTRACT

This paper summarises the results of an extensive field test carried out in the framework of the “French National Research Project on Vibratory Driving” and the ProfilArbed Research Programme, to investigate and compare the behaviour and performance of eight impact- and vibratory-driven piles. The test site is located in Northern France. The subsoil consists of dense Flanders clay. The piles were driven using an ICE815 vibrator and an IHC70s impact hammer to the same depth. All these piles were instrumented with accelerometers and strain gauges positioned at the top and bottom of the piles. Penetration rate, uplift load applied by the crane, vibration transmitted to the ground, operating pressure, oil flow at the vibratory power-pack, and energy per blow for the hammer were continuously recorded. The paper presents the main results obtained from double sheet piles (AU16 type) driven to 7 m depth, a small wall of two double sheet piles (AU20 type) driven to 8 m depth, HP bearing piles driven to 10.2 m, and steel open-ended tubes of 508 mm diameter driven to 9.4 m depth. After a delay of 6 to 8 weeks, the piles were statically loaded to failure. The piles were instrumented with removable extensometers, which made it possible to measure the mobilisation of shaft friction and toe resistance. The measured bearing capacity was significantly lower for the vibratory-driven piles (around 35%) and it is consistent with the results obtained by the Laboratoire des Ponts et Chaussées de Paris at other sites.

### INTRODUCTION

At the airfield of Merville, Northern France, the Laboratoire des Ponts et Chaussées de Paris (LPCP) has established a research site since last thirty years. In the framework of the “French National Research Project on Vibratory Driving” and ProfilArbed S. A., it was decided to select the Merville site in order to study the behaviour of the dense Flanders clay during static loading by driving various steel piles. The field site is flat and easily accessible.

To carry out the study, it was decided to reuse an reaction system consisting of three pairs of L2S type sheet piles impact driven to 16 m. For this purpose, six pairs of sheet piles (AU16 type and AU20 type), two HP-bearing piles (HP400\*213), and two open-ended steel tubes of 508 mm diameter were installed between March and April 2003 (Fig. 1). All these piles were instrumentally monitored.

In order to drive the vibratory piles into the dense Flanders clay an additional mass with the ICE815 vibrator (Table 1) had to be used.

It was decided to test a new steel pile (AU type) from ProfilArbed S. A. and a new IHC anvil for the AU steel pile (Table 1). To carry out the static loading test, it was necessary to install two new reaction walls. The first was made out of five pairs of sheet piles (PU16 type) while the second contained four pairs of sheet piles (AU16 type). All the steel piles were installed from 0 m to around 7.5 m depth with the

ICE815 vibrator (Fig. 2) and from 7.5 to 12.75 m depth with the IHC70s hammer (Fig. 3).

### SITE INVESTIGATION

The Flanders clay is a marine clay deposit of Ypresian (early Eocene) age. The total thickness of the original formation is greater than 250 m. Subsequent erosion has removed this formation to leave a typical layer of 40 m thickness (Fig. 4).

The natural soils encountered are quite uniform over the whole site of Merville. The near-surface soil, up to a 2.2 m depth, consists of silt with about 50 cm thick hardened crust due to humidification and desiccation cycles. The Flanders clay is then found up to a 42 m depth. Its geotechnical properties vary linearly with depth.

The water table in the silty clay layer fluctuates frequently between 1.5 and 1.9 m below ground, depending on the season and amount of rainfall. It is difficult to establish a water table in the Flanders clay, a very impermeable soil with a strongly micro-cracked structure. The ground level is located at an altitude of 17.8 m. The stratigraphic subdivisions in the study area are as follows.

- From 0 to about 2.2 m depth: low-plasticity silt affected by water table fluctuations;
- From 2.2 m to 42 m depth: Flanders clay of

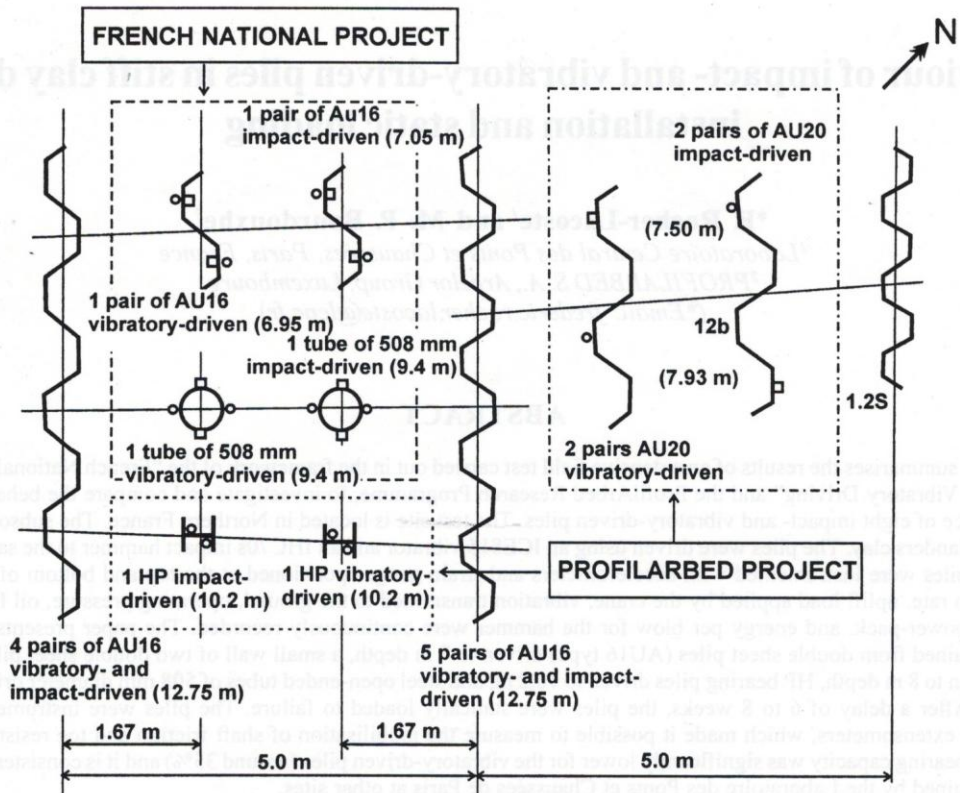


Fig. 1: General layout of the site

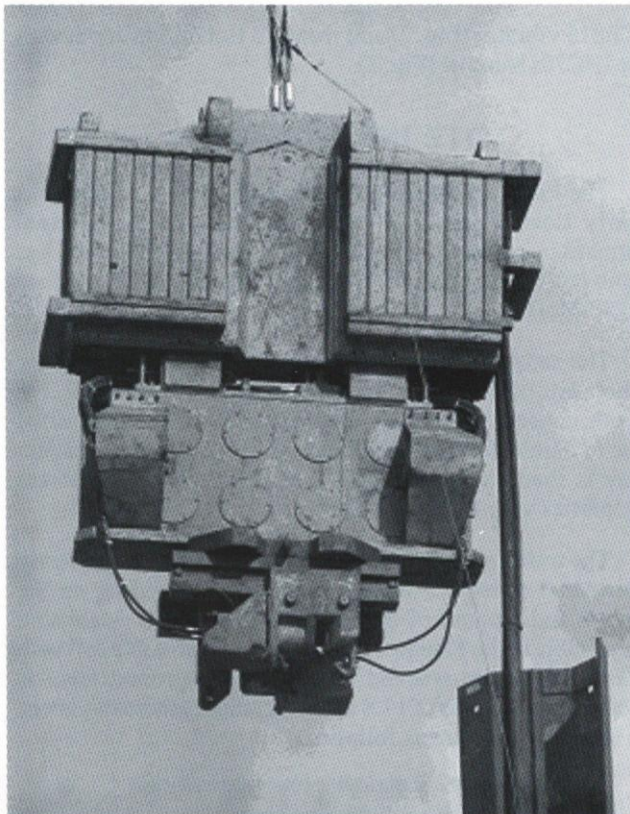


Fig.2: ICE815 vibrator with the additional mass

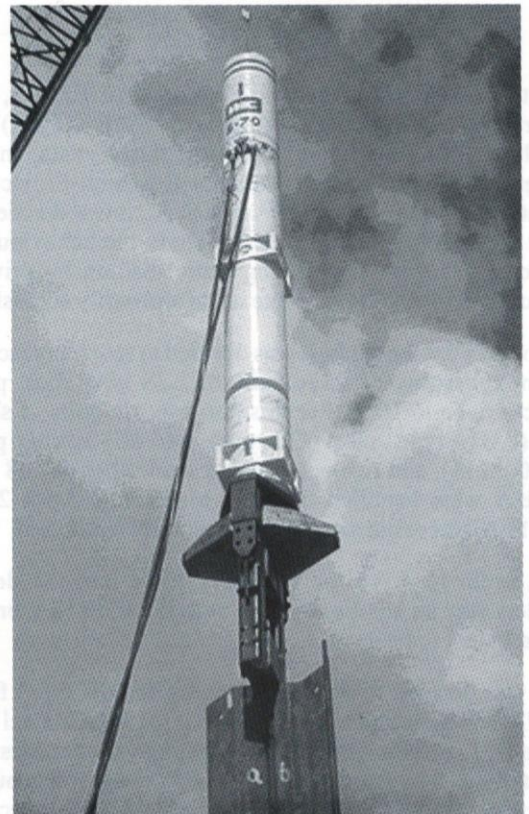


Fig. 3: IHC 70s hammer with the new AU anvil

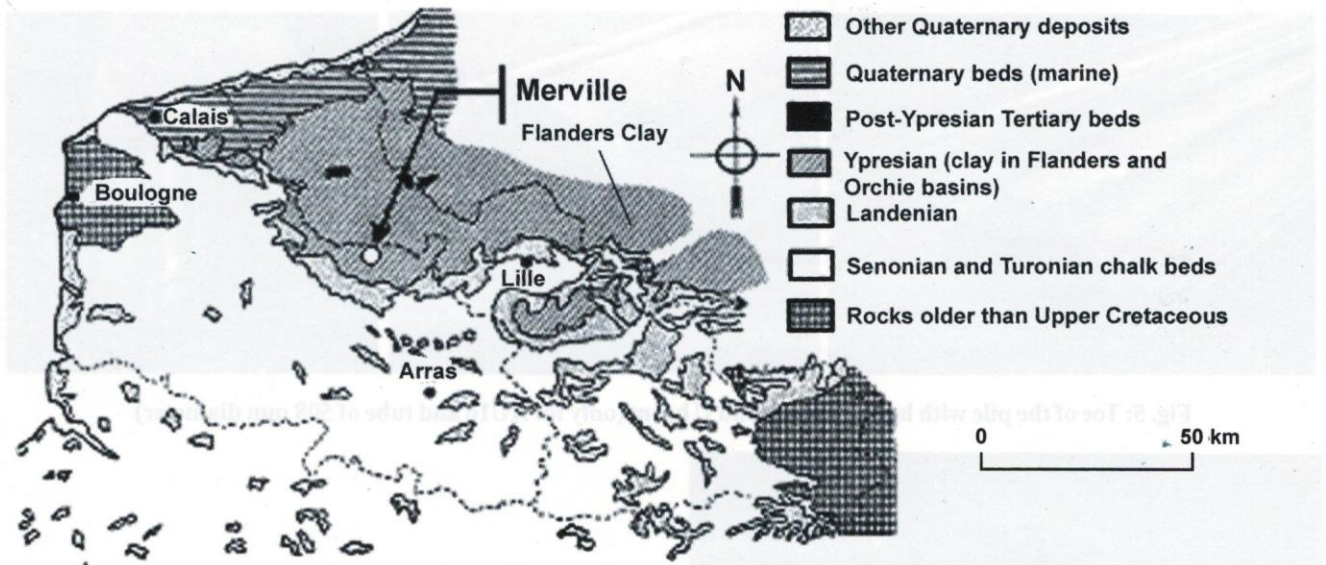


Fig. 4: Geological map of North France showing the distribution of Flanders clay

Table 1: ICE 815 and IHC 70s main characteristics

ICE 815 vibrator		IHC 70s hammer	
Maximum frequency	26 Hz	Ram weight	3500 kg
Eccentric moment	46 kg.m	Maximum blow rate	50 blow/min
Centrifugal force	1250kN	Max. energy per blow	70 kN.m
Maximum amplitude	26 mm	Min. energy per blow	2 kN.m
Vibrating weight (without clamps)	3550 kg	Total weight (for AU)	10910 kg
Additional weight	4000 kg	Total weight (for Ø 508 mm)	10390 kg
Total weight (for AU)	12110 kg	Total weight (for HP)	11660 kg
Total weight (for Ø 508 mm)	11360 kg		
Total weight (for HP)	10950 kg		

Ypresian age;

- From 42 m to 84 m: Landenian sand and clay horizons;
- Below 84 m: Senonian and Turonian chalk bed.

The penetrometer test (PMT) and standard penetration test (SPT) results are given in Table 2.

Table 2: Geotechnical results

	PI* (Mpa)	q <sub>c</sub> (Mpa)
Silt (0 to 2.2 m)	0.25 to	1
Flanders clay (at 4 m)	0.75	2
Flanders clay (at 16 m)	1.8	5

### INSTRUMENTATION OF PILES

In both cases of vibratory and impact driving, the piles were monitored with a continuous record of:

- penetration rate,
- acceleration and stresses at the top and at the pile toe,
- vibration transmitted to the ground,
- uplift load applied by the crane (only in case of vibratory driving),
- oil pressure and the outflow from vibratory power pack, and
- energy per blow and blow count of the impact driver.

Two additional steel tubes were welded to receive the signals from the LPC removable extensometers during the static loading test (Bustamante and Doix 1991). Two extra steel U-beams were also welded to the opposite ends to receive the records of accelerometers and strain gauges from the toe. The positions of various accessories are shown in Fig. 5.

### OBSERVATION DURING INSTALLATION

The first batch of piles was vibratory driven (Fig. 6) and the second one was impact driven. We present only the observations on the AU16 type sheet piles and the open-ended steel tubes.

#### Vibratory driving

An AU16 type of sheet pile was vibratory driven to a depth of 6.95 m on 2 April 2003 in 31 minutes. The toe accelerometers recorded the vibration at an interval of 30 minutes. The open-ended steel tubes of 508 mm diameter were vibratory driven to a depth of 9.40 m on 7 April 2003 in 44 minutes.

The driving records of the sheet pile and open-ended tube are shown in Figs. 7 and 8 respectively. During vibratory driving, the penetration rate reached a maximum of around

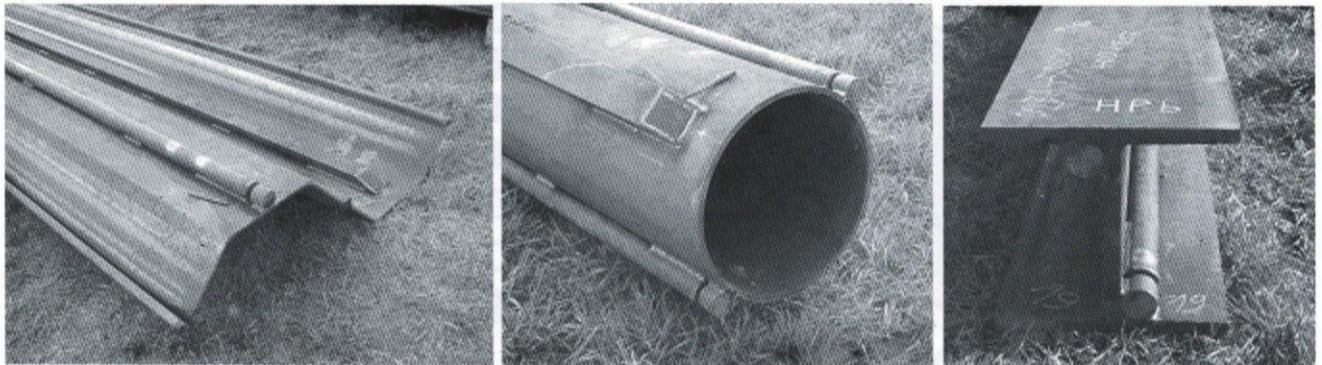


Fig. 5: Toe of the pile with housing tubes and U beam (only for AU16 and tube of 508 mm diameter)



Fig. 6: Driving the pile

1.4 m/min between 0 and 4 m depths. Between 4 and 5 m depths, the penetration speed rapidly decreased to 20 cm/min. At the end, the penetration rate was only 10 cm/min, close to the refusal value of 5 cm/min.

The uplift load applied by the crane is shown in Fig. 9. When the penetration rate started to decrease, the uplift also decreased. The crane operator just maintained the uniform load (around 10 kN) of the ICE815 vibrator.

It was possible to measure the working frequency (Fig. 10) of the ICE815 vibrator with the help of a connected

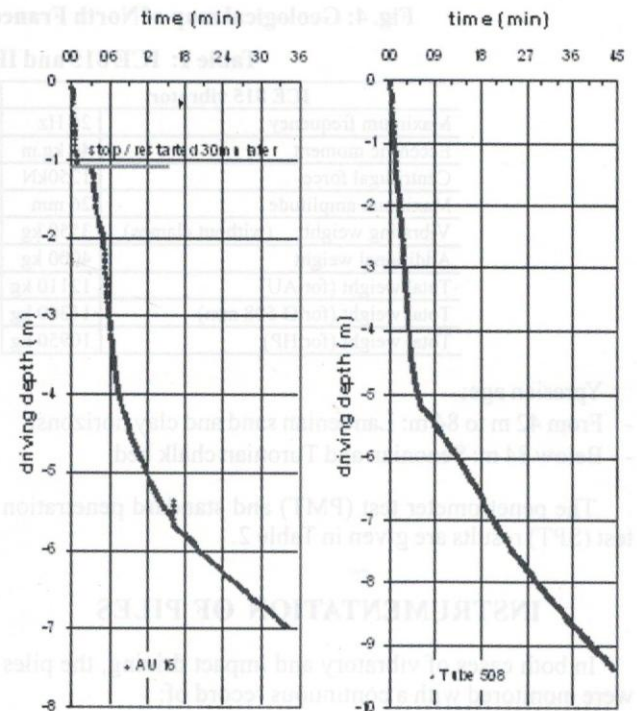


Fig. 7: Vibratory driving records of sheet piles and tubes

sensor. At a depth of about 4 m, the pile was into the Flanders clay with a limiting pressure of 0.75 MPa. During the vibratory driving, the frequency decreased from 25 to 19 Hz at the end of the penetration. At the same time, the penetration rate also decreased.

When the penetration was stopped, a decrease of 25% in the frequency between the beginning and the end of vibratory driving was observed.

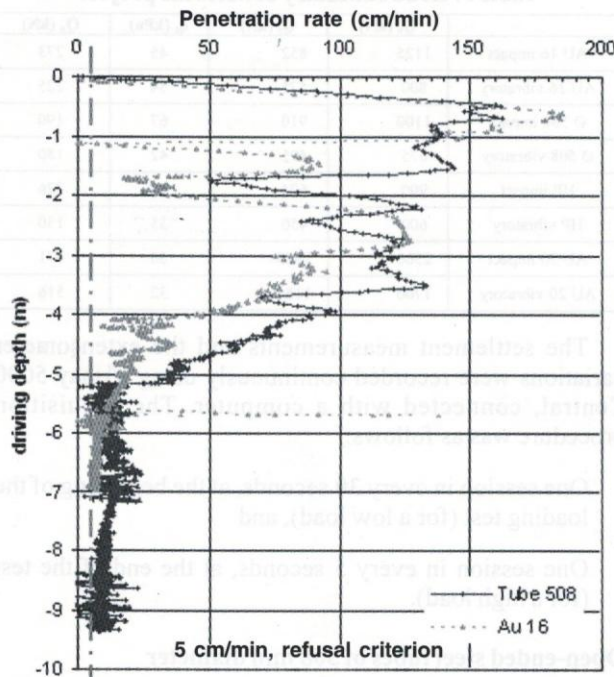


Fig. 8: Penetration rate of vibratory driving

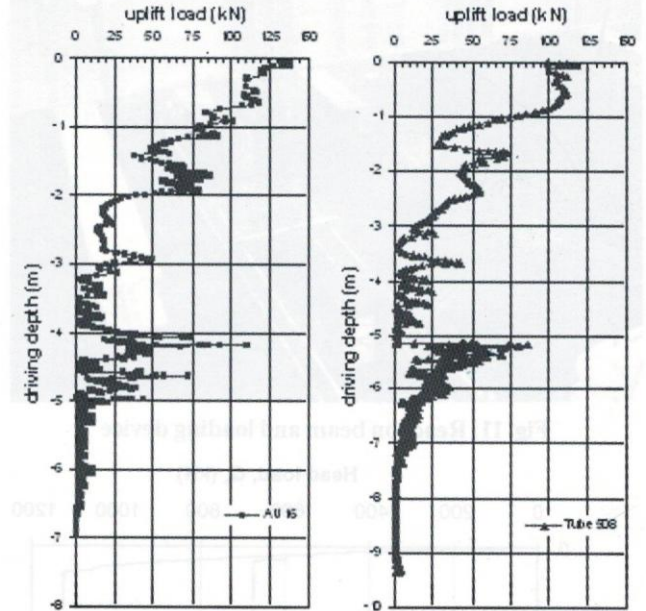


Fig. 9: Uplift load of the crane

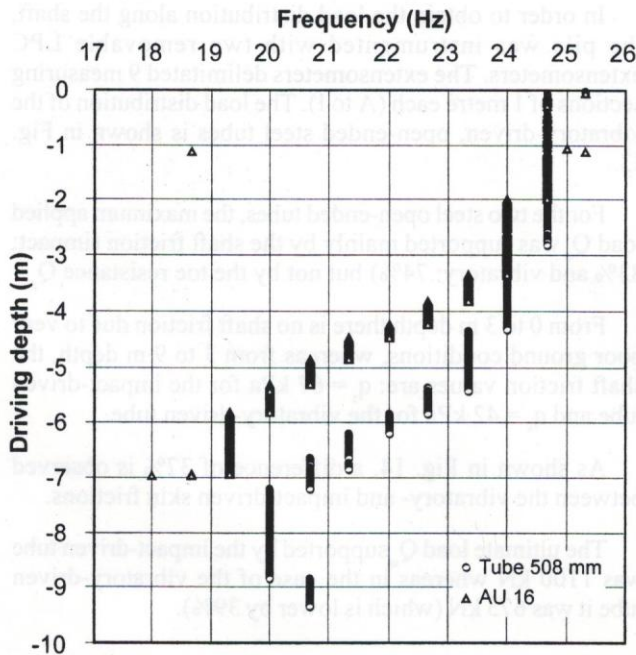


Fig. 10: Plot of driving depth against vibration frequency for tubes and piles

#### Impact driving

The AU16-type sheet pile was impact driven on 3 April 2003 to a 7.05 m depth in 13 minutes. The steel open-ended tube of 508 mm diameter was impact driven on 8 April 2003 to a depth of 9.40 m in 15 minutes. It is necessary to add to this time the vibratory driving installation time of the pile from 00 to 1.5 m.

The IHC S70 hammer was operated with the following procedure:

- Driving was started at an energy level of 10 kNm.
- When the penetration rate was lower than 25 cm for 40 blows, the energy level was increased to 20 kNm.
- The driving was repeated until a maximum energy level of 70 kNm was reached.

The rise of the energy level from 10 to 20 kNm was observed at a 5 m penetration depth for the AU16-type sheet pile and at a 7 m penetration depth for the open-ended steel tube of 508 mm diameter.

The refusal limit was not reached with the impact-driving method because the impact penetration was stopped at the vibratory-driving level. A target of this project was to study the behaviour of two similar piles installed with the two different methods.

#### STATIC LOADING TEST

The piles were loaded in compression to failure, according to the Maintained Load procedure. Each loading



Fig. 11: Reaction beam and loading device

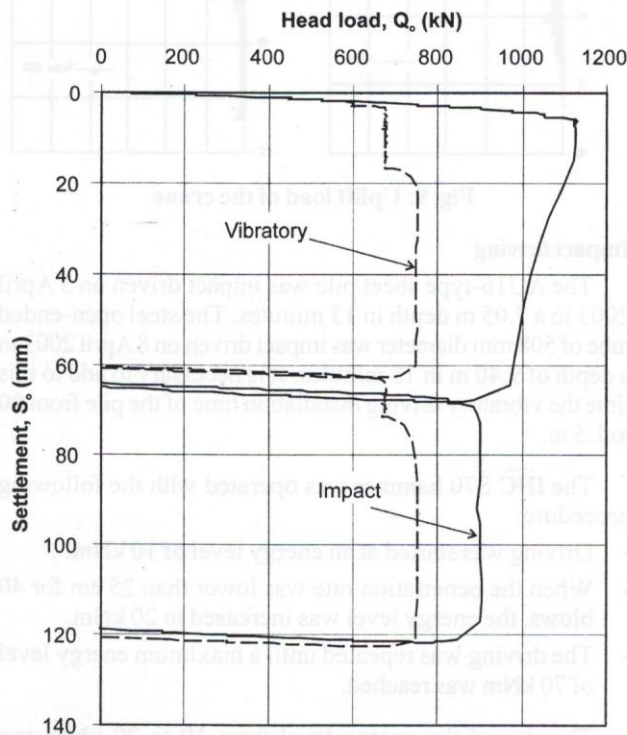


Fig. 12: Load-settlement curves for open-ended steel tubes

step was applied for 30 minutes. The load was checked with a 2500 kN loading cell and the displacement was measured with 4 linear potentiometers linked to reference beams. The loading frame consisted of a reaction beam supported by 2 reaction sheet walls (Fig. 11).

All the piles were instrumented with removable LPC extensometers, which recorded the load distribution along the shaft and at the base. The extensometers defined 7 measuring sections on the sheet piles of AU type, 9 sections on the open-ended steel tubes, and 10 on the HP-bearing piles.

Table 3: Load summary of Merville project

	$Q_u$ (kN)	$Q_s$ (kN)	$q_s$ (kPa)	$Q_p$ (kN)
AU 16 impact	1125	852	45	273
AU 16 vibratory	800	575	34	225
Ø 508 impact	1100	910	67	190
Ø 508 vibratory	675	495	42	180
HP impact	900	674	50	226
HP vibratory	600	450	35	150
AU 20 impact	2500	1829	50	671
AU 20 vibratory	1700	1184	32	516

The settlement measurements and the extensometer variations were recorded continuously using Vishay 5000 Central, connected with a computer. The acquisition procedure was as follows:

- One session in every 30 seconds, at the beginning of the loading test (for a low load), and
- One session in every 5 seconds, at the end of the test (for a high load).

**Open-ended steel tubes of 508 mm diameter**

An impact-driven open-ended steel tube was loaded in compression on 22 May 2003 after a delay of 44 days while the other one was vibratory driven on 3 June 2003 after a delay of 57 days. Their load settlement curves are shown in Fig. 12. The ultimate resistance (i.e. the load for which the pile head movement exceeds 10% of the tube diameter) is presented in Table 3.

In order to obtain the load distribution along the shaft, the pile was instrumented with two removable LPC extensometers. The extensometers delimited 9 measuring sections of 1 metre each (A to I). The load distribution of the vibratory-driven, open-ended steel tubes is shown in Fig. 13.

For the two steel open-ended tubes, the maximum applied load  $Q_u$  was supported mainly by the shaft friction (impact: 83% and vibratory: 74%) but not by the toe resistance  $Q_p$ .

From 0 to 3 m depth there is no shaft friction due to very poor ground conditions, whereas from 3 to 9 m depth, the shaft friction values are:  $q_s = 67$  kPa for the impact-driven tube and  $q_s = 42$  kPa for the vibratory-driven tube.

As shown in Fig. 14, a difference of 37% is observed between the vibratory- and impact-driven skin frictions.

The ultimate load  $Q_u$  supported by the impact-driven tube was 1100 kN whereas in the case of the vibratory-driven tube it was 675 kN (which is lower by 39%).

**AU16 type sheet piles**

Compressive loads were applied to an AU16 type impact-driven sheet pile on 20 May after 46 days of delay and to

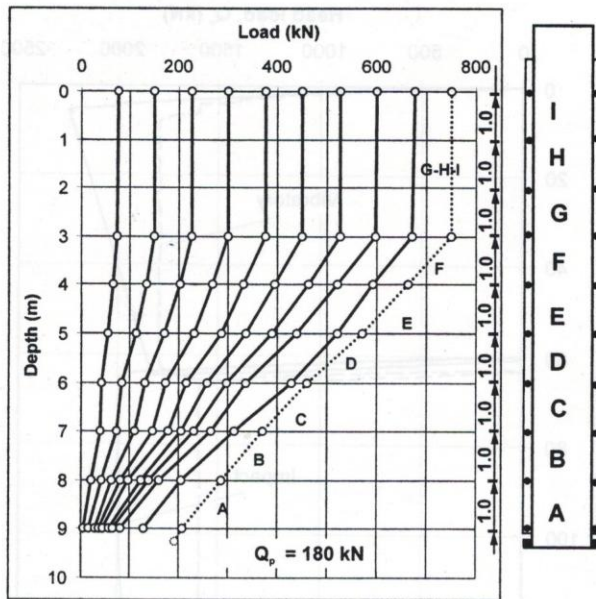


Fig. 13: Plot of load distribution versus depth for the vibratory-driven open-ended steel tubes

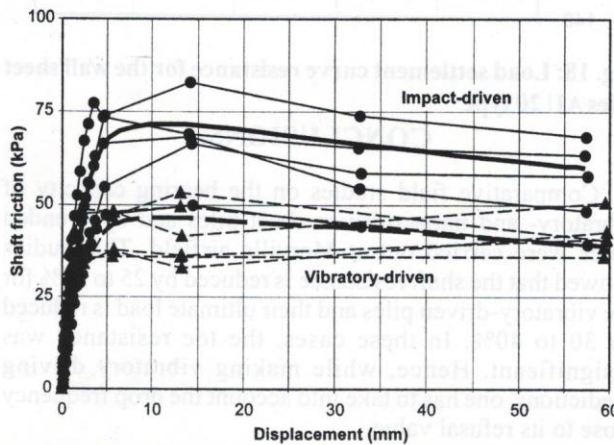


Fig. 14: Mobilisation of shaft resistance along the steel open-ended tubes of 508 mm diameter

another AU16 type vibratory-driven sheet pile on 21 May 2003 after 48 days of delay.

The plots of settlement versus load for the AU16 type sheet piles are shown in Fig. 15. The ultimate resisting load for which the pile head movement exceeds 10% of the diameter, is presented in Table 3. The ultimate load  $Q_u$  supported by the impact-driven sheet pile was 1125 kN and it was 800 kN (i.e. 30% less) for the vibratory-driven sheet.

For the two sheet piles of AU16 type, the maximum applied load  $Q_u$  (75%) was supported mainly by the shaft friction and the toe resistance  $Q_p$  (25%) played only a minor role. No shaft friction is recorded from 0 to 3 m depth because of the very poor ground. From 3 to 7 m depth, the shaft friction values are:  $q_s = 45$  kPa for the impact driven AU16 sheet pile and  $q_s = 34$  kPa for the vibratory-driven Au16 sheet pile.

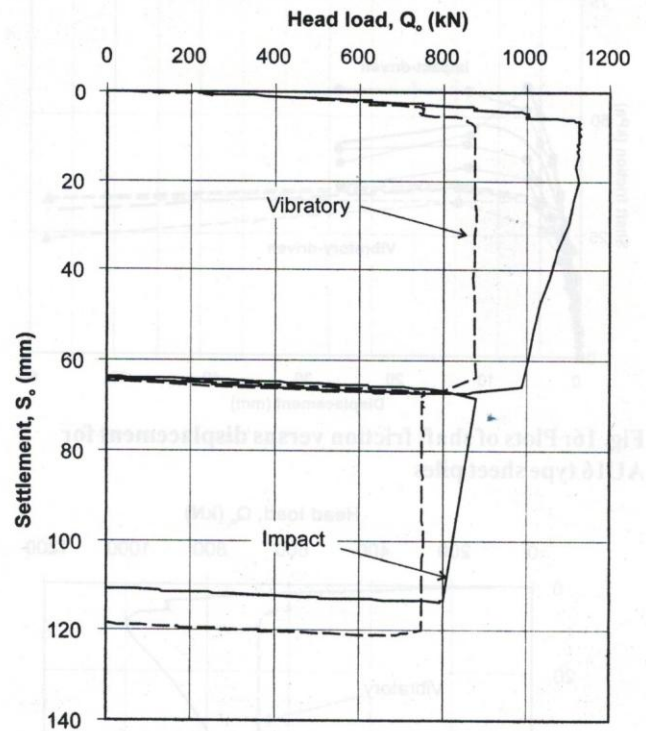


Fig. 15: Plots of settlement against load for AU16 type sheet piles

As shown in Fig. 16, a difference of 25% between the vibratory- and impact-driven skin frictions is observed.

#### HP-bearing piles

An impact driven HP-bearing pile was loaded in compression on 4 June 2003 after a delay of 60 days and another HP-bearing pile was vibratory driven on 3 June 2003 after a delay of 62 days. Their load–settlement curves are shown in Fig. 17 and the ultimate resistance is presented in Table 3.

The ultimate loads  $Q_u$  supported by the impact-driven HP pile and vibratory-driven sheet were 900 kN and 600 kN (i.e. 33% less) respectively. As in the case of AU16 type sheet piles, for the two HP piles, the maximum applied load  $Q_u$  (75%) was supported mainly by the shaft friction and the toe resistance  $Q_p$  (25%) played a minor role. From 0 to 3 m depth there is no shaft friction because of very poor ground conditions. The shaft friction values from 3 to 10 m depth are:  $q_s = 50$  kPa for the impact-driven HP and  $q_s = 35$  kPa for the vibratory-driven HP.

#### AU 20 type sheet piles

The AU20-type impact-driven sheet piles were loaded in compression on 11 June 2003 after a delay of 74 days and similar other sheet piles were vibratory driven on 12 June 2003 after 76 days of delay. Their load settlement curves are shown in Fig. 18 whereas the ultimate resistance is presented in Table 3. The ultimate loads  $Q_u$  supported by the impact-

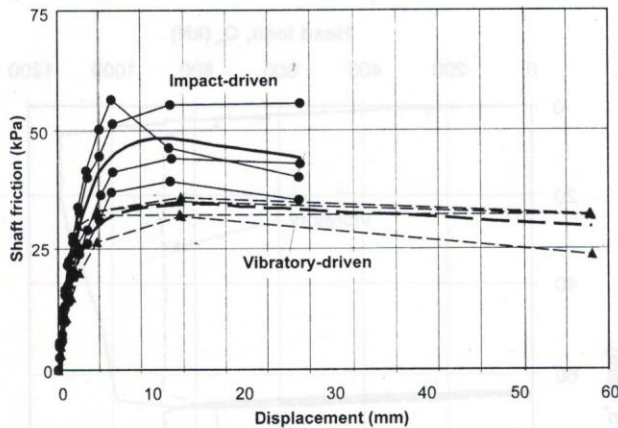


Fig. 16: Plots of shaft friction versus displacement for AU16 type sheet piles

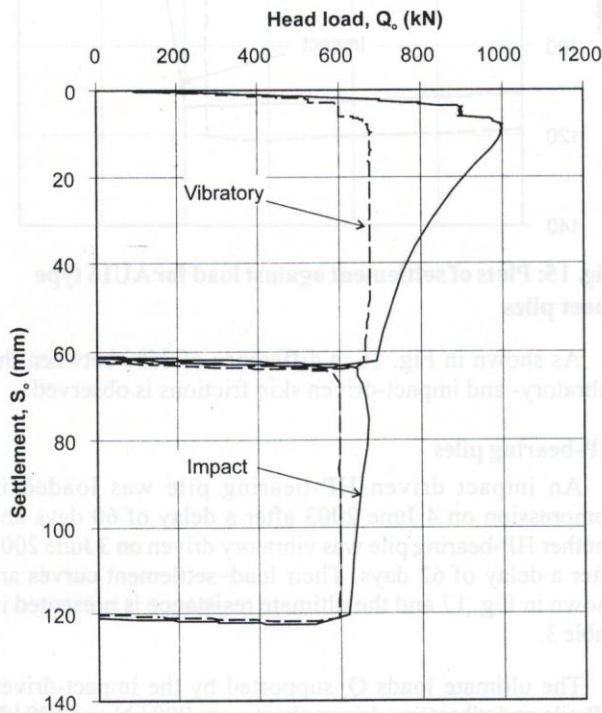


Fig. 17: Plots of settlement against load for HP-bearing piles

driven sheet piles and the vibratory-driven sheets were 2500 kN and 1700 kN (i.e. 32% less) respectively.

For the two pairs of AU20 sheet piles, the maximum applied load  $Q_u$  was supported mainly by the shaft friction (70%) and not by the toe resistance  $Q_p$  (30%). From 0 to 3 m there is no shaft friction, because of very poor ground conditions. From 3 to 8 m depth, the shaft friction values are:  $q_s = 50$  kPa for the impact-driven AU 20 piles and  $q_s = 32$  kPa for the vibratory-driven AU20 piles.

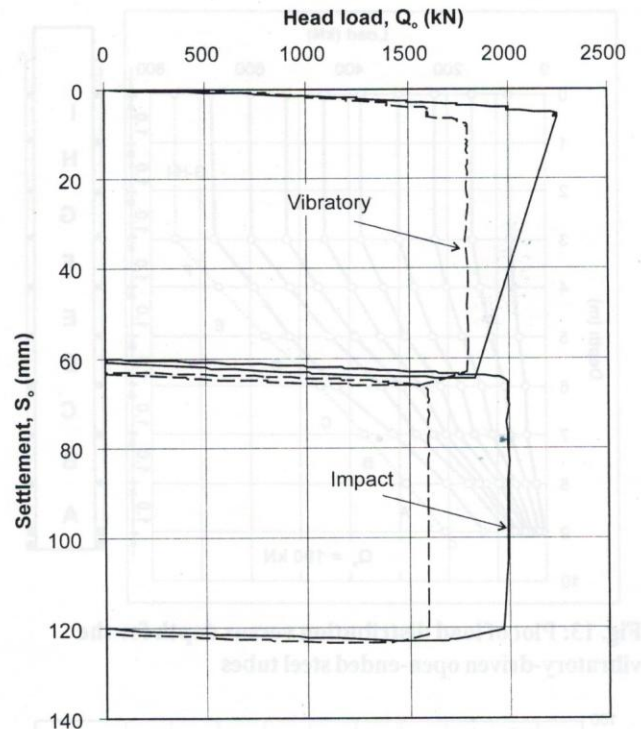


Fig. 18: Load settlement curve resistance for the wall sheet piles AU 20 type

### CONCLUSIONS

Comparative field studies on the bearing capacity of vibratory- and impact-driven sheet piles and open-ended tubes were carried out at Merville airfield. The studies showed that the shaft resistance is reduced by 25 to 35% for the vibratory-driven piles and their ultimate load is reduced by 30 to 40%. In these cases, the toe resistance was insignificant. Hence, while making vibratory driving predictions, one has to take into account the drop frequency close to its refusal value.

### ACKNOWLEDGEMENTS

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