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VIBRATIONS INDUCED BY VIBRATORY PILE-DRIVING

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ABSTRACT

Within the framework of the French National Research project on vibratory pile-driving an extensive field test research of sinking by vibrating was carried out in spring 2003 to study the behavior of various types of piles. The test site is located in the airfield of Merville, North of France. The subsoil consists of dense Flanders clay. All these piles were instrumented. The vibrations transmitted to the ground were recorded continuously. The paper presents the experimental attenuation curves of particle velocities as well as a finite element model to analyse wave propagation in the ground.

Keywords: Vibratory driving, Wave propagation, Finite element model

INTRODUCTION

One of the objectives of the French National Research project on vibratory pile-driving, controlled by the IREX organization (French Institute for Civil Engineering Research and Experimentation) and funded by the French Ministry of Public Works, Transport and Housing and by private companies was to study the environmental impact of the building sites of Vibratory pile-driving and more particularly to study the wave propagation into the ground in order to quantify the attenuation of the movement (particulate speeds).

It was decided to reactivate the LCPC research site in the airfield of Merville, North of France.
In the "Merville project", 13 piles and sheet-piles were installed between March and April 2003 (Fig. 1). All these piles were instrumented.

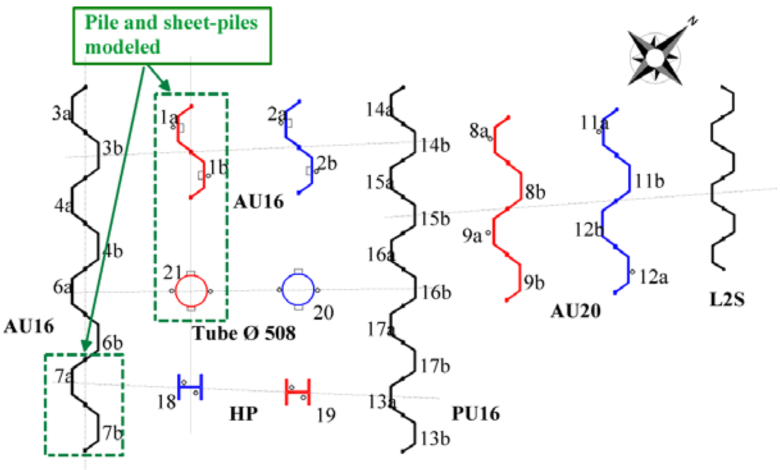

Figure 1. General layout of the Merville site

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To vibratory drive the piles into the dense Flanders clay an additional mass with the ICE 815 vibrator (table 1) had to be used.

Table 1. ICE 815 vibrator main characteristics (with clamps)

Maximum frequency	1600 rpm / 26 Hz
Eccentric moment	46 kg.m
Centrifugal force	1250 kN
Maximum amplitude	26 mm
Vibrating weight (without clamps)	3550 kg
Additional weight	4000 kg
Total weight (for AU 16)	12110 kg
Total weight (for tube 508 mm)	11360 kg

For better apprehending the attenuation of particulate velocities, a finite element simulation was carried out with software CESAR-LCPC. Two sheet-piles and one open-ended tube has been studied and modeled for this communication.

SITE INVESTIGATION

The Flanders clay is a marine clay deposit of Ypresien (beginning of the Eocene tertiary). The total thickness of the original formation is greater than 250 m. Later erosion has removed this formation to leave a typical layer of 40 m of Flanders clay.
The natural soils encountered are quite uniform over the whole site of Merville. The near surface soils, up to 2.2 m depth, consist of silt with a hardened crust of some 50 cm due to humidification and desiccation cycles. The Flanders clay is then found down to 42 m. Its geotechnical properties increase linearly with depth.
The level of the water table in the silty layer fluctuates rapidly between 1.5 m and 1.9 m below ground, according to the seasons and the rainfall. It is difficult to establish a water table level in the Flanders clay, a very impermeable soil with a strongly micro-cracked structure. The ground level is located at 17.8 m above the sea level. The stratigraphic subdivision is as follows :

- from 0 to 2.2 m approx. low plasticity silt affected by water table fluctuation,
- from 2.2 m to 42 m depth, Flanders clay of Ypresien,
- from 42 m to 84 m sand and clay horizons from Landenien,
- finally, chalk bed of Senonien and Turonien.

The principal PMT, SPT and seismic results are reported in table 2 (Rocher-Lacoste et al 2003) (Ferber and Abraham, 2003).

Table 2. Geotechnical results

Level	Depth	Ground	p_l* (MPa)	E_M (MPa)	q_c(MPa)	ρ(kN/m^3)	E(MPa)	ν
C1	00 to 2.2m	limon	0.25 –0.6	5	1	16.2	80	0.483
C2	2.2 to 30m	Flanders clay	0.75 –1.8	25	2 - 5	18.5	125	0.496

FIELD INSTRUMENTATIONS

During vibratory pile-driving, all the piles were monitored with a continuous record of :

- the penetration rate,
- the acceleration and the stresses at the top and at the pile toe (Sieffert and Rocher-Lacoste 2007),
- the vibrations transmitted to the ground,
- the uplift load applied by the crane, only in case of vibratory driving.

The vibrations transmitted to the ground were measured by three geophones (Input/Output SM-6, 3D) installed at 5 m, 10 m and 15 m from the pile in the North-East direction.
All the sensors were connected to an acquisition carte, 32 channels, of the National Instrument NI6071E type. The software was developed under LabView by FUGRO France.

EXPERIMENTAL RESULTS

Five AU16 sheet pile and one open-ended tube were vibratory driven on April 2003. The vibrodring of the sheet piles was stopped at approximately 7 meters, in 17 to 31 minutes. The steel open-ended tubes, ∅508 mm, was vibratory driven the April 7, 2003 at 9.40 m, in 44 minutes. The driving record of the AU16 sheet piles and of the steel open-ended tube, ∅508 mm is shown on figure 2.

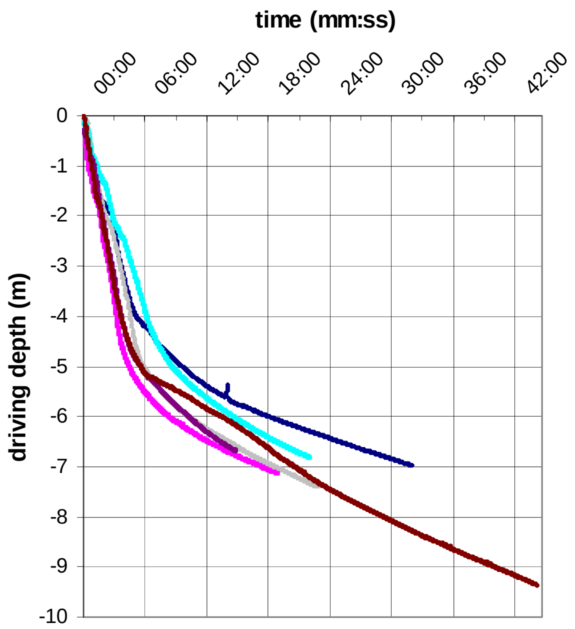

Figure 2. Vibratory driving record to sheet piles and open-ended tube

The maximum penetration rate reached was around 1.4 m/min from 0 to 4 m. Between 4 and 5 m, the penetration speed rapidly decreased to 20 cm/min. At the end, the penetration was only 10 cm in one minute, near the refusal criteria of 5 cm/min. The penetration rate of the sheet piles AU 16 type, number 1 and 7 and of the steel open-ended tube, ∅508 mm is shown on figure 3. These elements will be the object of the finite element model.

It was possible to define the working frequency (Figure 4) of the ICE 815 vibrator with some sensors, which were connected.

During the vibratory pile-driving the working frequency of the vibrator decreased from 25 to 19 Hz at the end of the penetration. In the same time, the penetration rate was lower.
When the penetration was stopped, a decrease of 25 % of the frequency between the beginning and the end of vibratory pile-driving was observed.

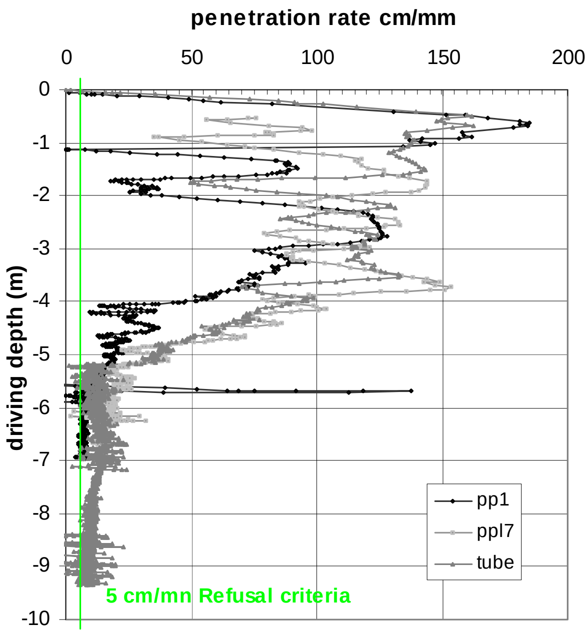

Figure 3. Vibratory driving penetration rate

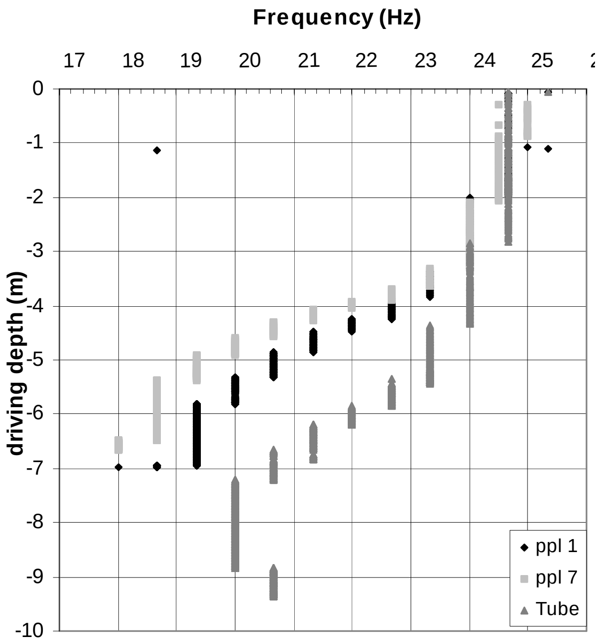

Figure 4. Vibratory frequency

For transient signals, caused by a hydraulic hammer, as well as for continuous and periodic signals generated by a vibrodriver, the measurement system must include sensors (e.g. 3D geophones) that produce electrical signals proportional to the vertical and horizontal vibrations, along with a signal acquisition and processing system. Vibrations can then be estimated by following the various recommendations found in the literature, with an emphasis however on in situ measurements.

Standards and regulations have been applied in a number of foreign countries. The figure 5 (Akle and Rocher-Lacoste, 2006) (Gonin et al. 2006) presents a comparison of various rules adopted abroad with respect to the French 1986 circular for continuous vibrations, applicable to vibratory pile-driving.

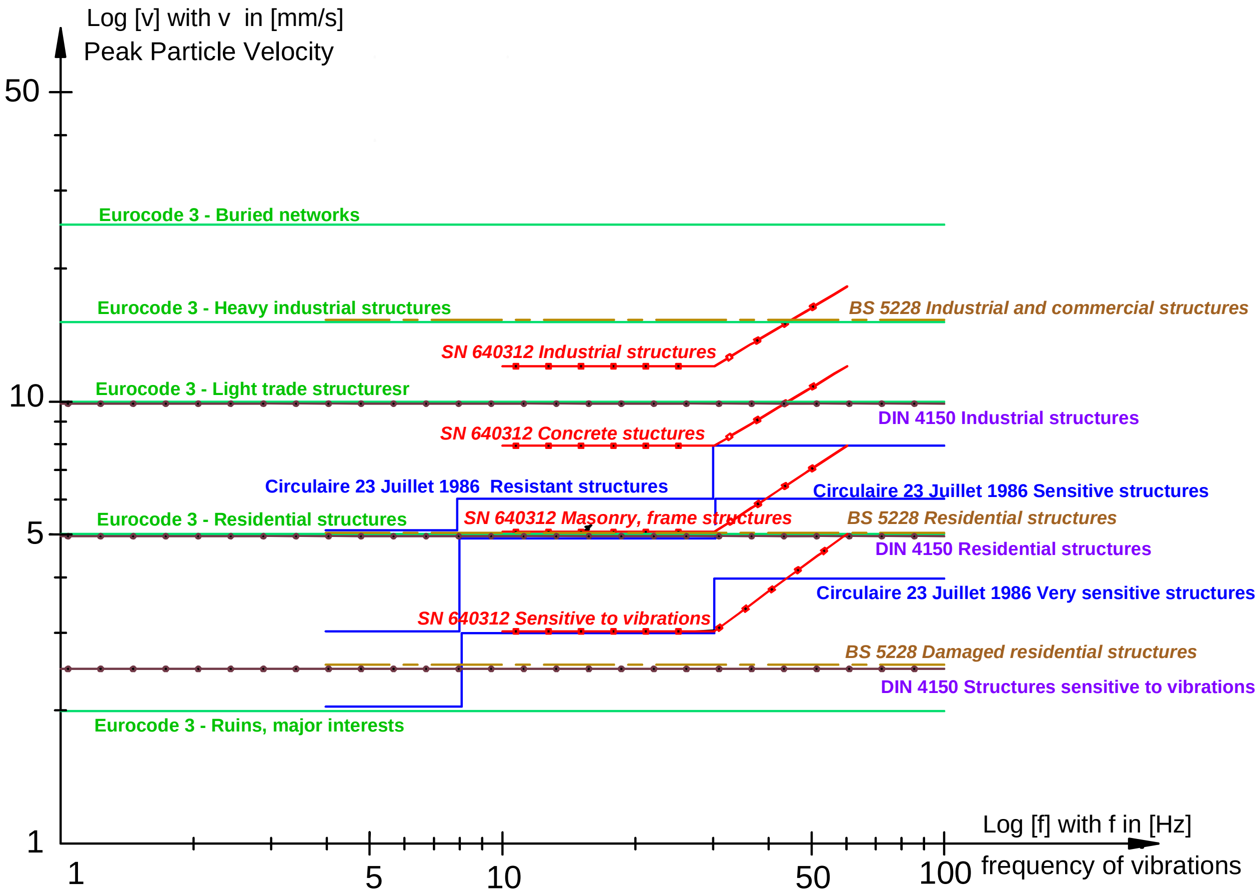

Figure 5. Comparison of the various standard-defined thresholds for continuous vibrations

As an initial approach, an observation of the entire series of attenuation curves leads to stating that the particle velocities measured with all transverse geophones are more reliable than those measured on the other geophones: radial and vertical.

For the pairs of AU16 sheet piles and the Ø508 tube, the particulate velocities recorded are higher at the end of the driving operation for the vibratory-driving installation after a 4-meter penetration. One explanation for this rise in particulate velocity lies in comparing attenuation curves with penetration speed curves (Figure 3) and vibrodriver frequency curves (Figure 4), which decrease with depth in a Flanders clay sample.

According to both the German standard (DIN 4150) and Swiss standard (SN 640312), the interpretation of particle velocities (Figure 6) transmitted to the ground, for frequencies between 18 and 25 Hz, would enable driving the sheet piles studied into Flanders clay at a distance of 5 m from a resistant building. This distance reaches 10 m per the French Environment Ministry circular, for a driving depth up to 4 m. Beyond 4 m, the frequency and penetration speed drop, the wave propagation speed in the ground increases significantly and necessitates driving at distances in excess of 15 m.

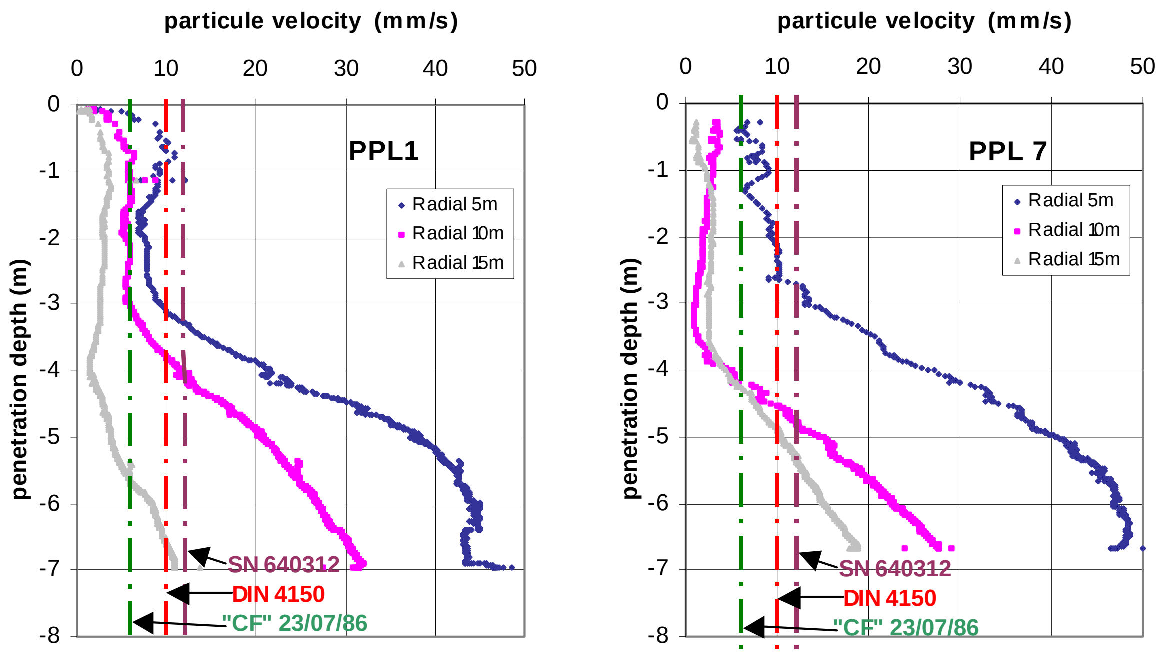

Figure 6. Pairs of vibratory-driven AU16 sheet piles - Maximum particulate velocity

MODELING OF THE WAVE PROPAGATION IN THE GROUND

Model by finite elements

In an attempt to better understand this propagation phenomenon for worksite vibrations, a 2D finite element model was developed using the CESAR-LCPC computation code in linear dynamics (module “DYNI”) (Humbert et al., 2005). An axisymmetric model type was selected. Only the two pairs of vibratory-driven AU16 sheet piles (ppl 1 and ppl 7) and the open-ended tube were examined.

Ground characteristics

When you analyze the geotechnical results of Merville site, you can defined a model with two soil horizon (linear, elastic and isotropic behavior). The mechanical characteristics are defined in table 2. Mechanical characteristics standards were taken for the steel of the pile:

- Density: ρ (kg/m3) = 7800
- Young Modulus: E (Mpa) = $2.1\ 10^5$
- Poisson's ratio: ν = 0,33

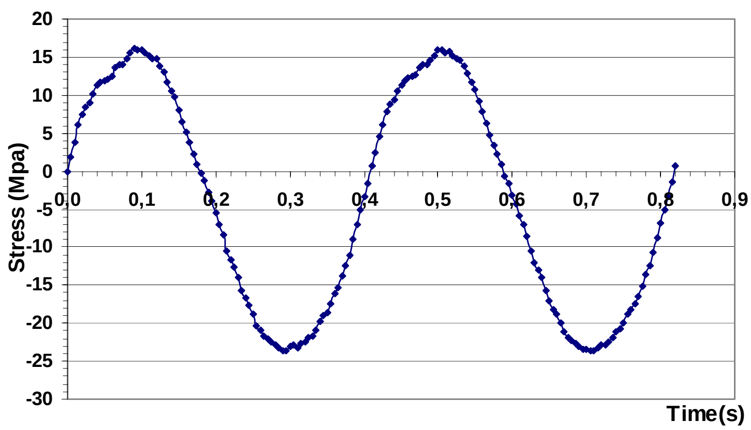

Figure 7. Two periods of the stress measured at 2.5 m depth on the tube

Stress type

The model is excited by a pressure applied at the head of the steel section. It is the result of the measurements taken by the strain gauges positioned at the head of pile during the implementation. Figure 7 represents an example of the stress applied to the piles, it is two periods of the strain gauge measured to the modeling depth.

Definition of the grid

The maximum depth of the pile was of 9.4 m. In order to observe the wave propagation well and to avoid the reflexions on the edges of the model, the field with a grid is 50 m broad on 30 m of depth. The solid mass of ground is modeled with triangular elements of solid mass with 6 nodes, the pile with quadrangular elements with 8 nodes. The grid of finite elements is thus composed of 11574 nodes is 5557 surface elements. A fine grid is defined in the area of ground close to the pile and the surface of the ground. Conditions in displacements are imposed on the borders of the grid: a horizontal displacement no one (U = 0) on the with dimensions ones and with a vertical displacement no one (v = 0) at the base of the model (figure 8).

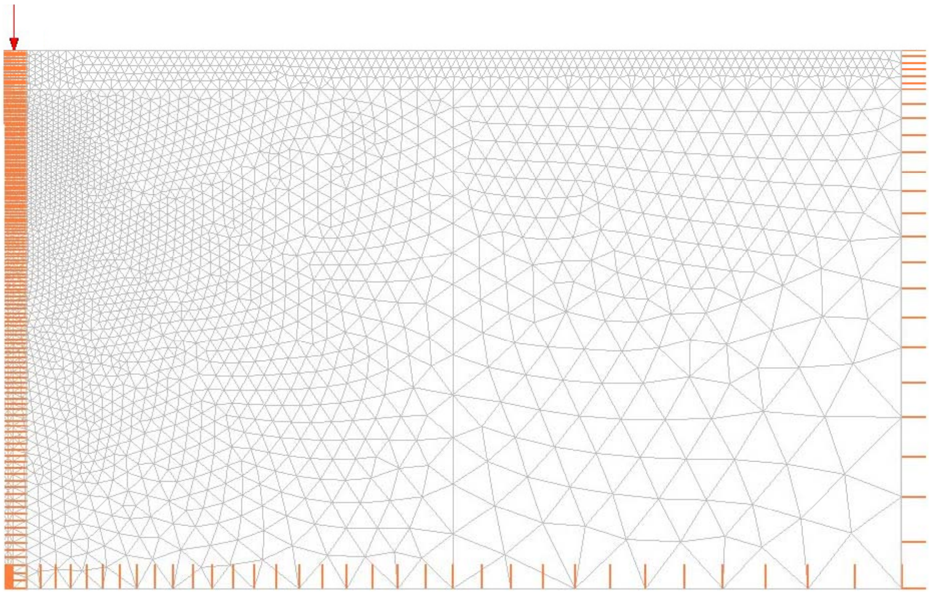

Figure 8. Model of calculation in axisymmetric deformation (grid and boundary conditions)

For this study, only various moments of the vibratory pile-driving of the tube were studied : depths 2.5, 4.5, 6.5 and 8.5 m for the tube and depths 2, 3, 4, 5 and 6 m for the sheet piles.

With open-ended pile, the mass of ground inside is taken into account in calculation. The stress is applied only to the steel section of the pile.

To be able to make an axisymmetric 2D model with a sheet pile it was necessary to transform this one into an equivalent circular pile, modeling is then similar to open-ended tube.

Calculation of the curves of attenuation

To a depth of 2,5 m for the open-ended pile, the figure 9 shows a comparison (calculation/measurement) of the curves of velocity attenuation according to the distance compared to the source. The calculated curves are higher than the experimental results but qualitatively rather close.

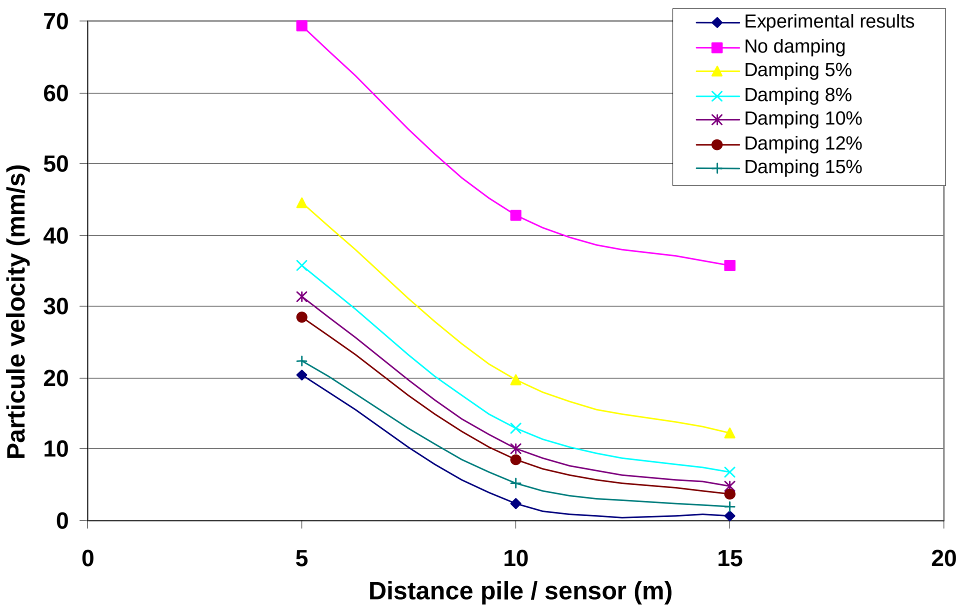

Figure 9. Curves of vertical velocities attenuation to 2,5 m depth (open-ended pile)

Characterization of damping

A way to model damping numerically consists in using the formulation suggested by Rayleigh. It consists in building a matrix of damping [C] proportional to the matrices of mass [M] and rigidity [K] in the following form:

$$[C] = a.[M] + b.[K] \tag{1}$$

This construction leads to an attenuation which is expressed as the sum of a term proportional to the frequency and a term inversely proportional to the frequency, is:

$$Q^{-1} = \frac{a}{\omega} + b.\omega \tag{2}$$

It was shown that the damping of Rayleigh is equivalent to the Maxwell rheological model (Semblat 1997). This equivalence is valid for the moderate values of the damping coefficient (ξ< 20%), i.e. when the approximation $Q^{-1} = 2\xi$ is justified and that material dispersion is weak.

To determine the values of the Rayleigh coefficients, the method consists in setting a damping coefficient ξ and to vary the variables a and b in order $Q^{-1} = 2\xi$ to have in the frequency band which interests us, 20Hz to 25Hz in our case. The results obtained are synthesized in table 3 and figure 10.

Table 3. Values of the Rayleigh coefficients to various damping coefficients

Damping coefficient (ξ)	2%	5%	8%	10%	12%	15%
Coefficient to the matrix of mass (a)	3.2	6.6	10	14	16	20
Coefficient to the matrix of rigidity (b)	1.22 10^{-4}	3.78 10^{-4}	6.31 10^{-4}	7.13 10^{-4}	8.9 10^{-4}	11.1 10^{-4}

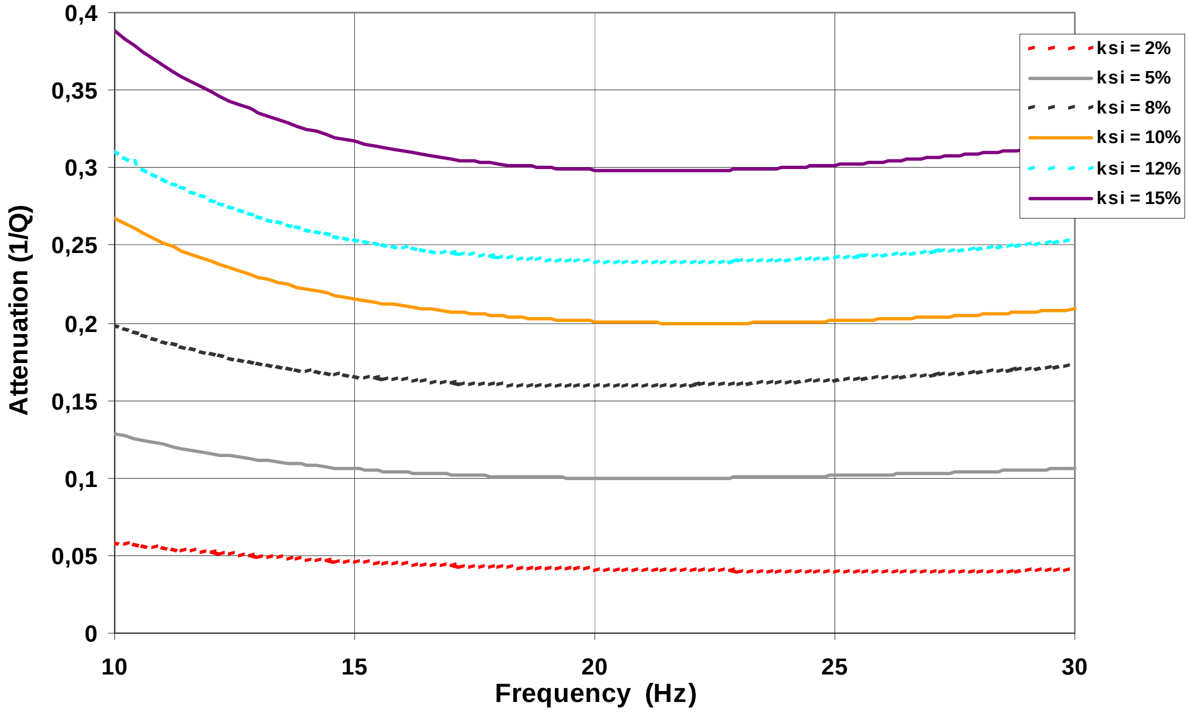

Figure 10. Curves of attenuation associated with the Maxwell rheological model

The results obtained, for different value from the damping coefficient, are represented on figure 11 to the sheet pile number 7.

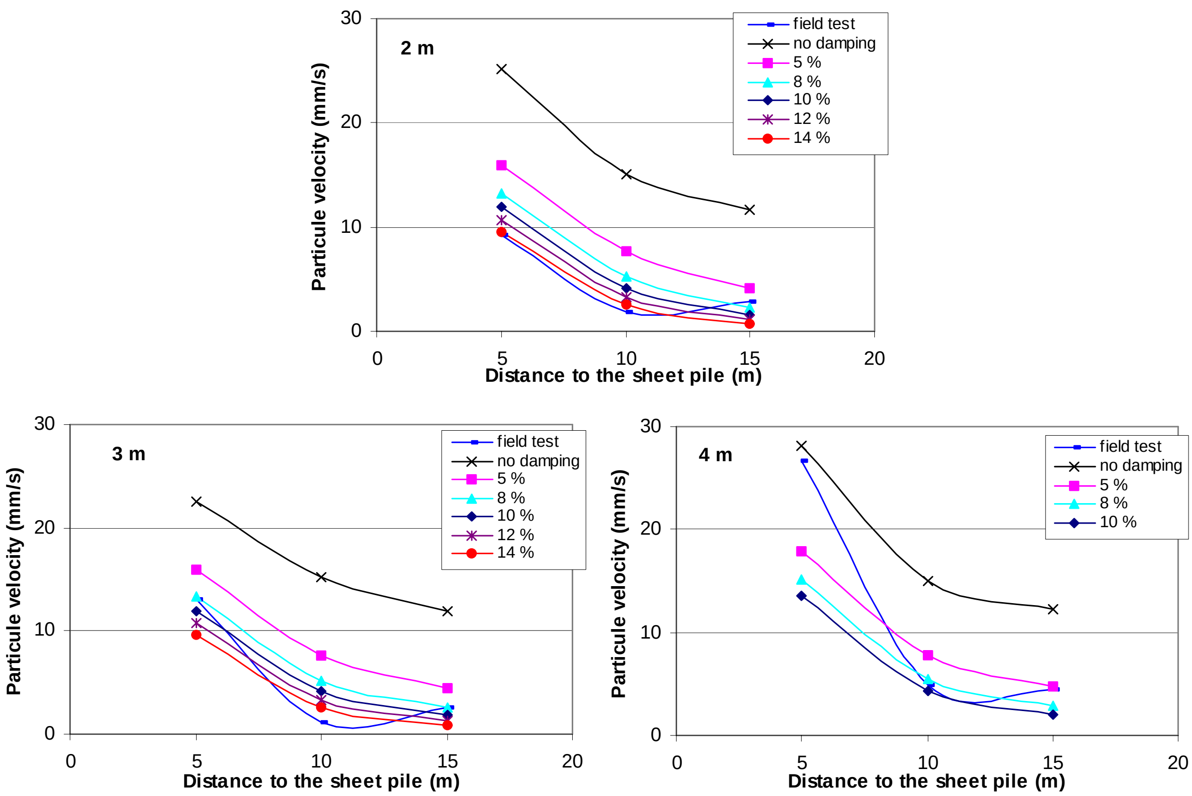

Figure 11. Curves of attenuation radial speed particulate experimental and numerical

Now, it is possible to compare the results obtained with the sheet pile number 7 (for example) for the radial component of the movement, to various damping coefficients and various penetration depths.

The analysis of the curves of attenuation, experimental and numerical, makes it possible to note that the curves obtained from numerical modeling with a damping coefficient ξ 12% or 14% are close the particulate speeds measured during vibratory pile driving, for a penetration depth of 2 meter. The same result are observed for the sheet pile (number 1) and the open-ended tube (number 21). For a penetration depth of 3 m, it is more delicate to conclude. Because the best agreement is obtained for ξ = 8% for the radial component at 5 and 15 m of the pile and ξ = 14% to 10 m. Also, for the vertical component ξ = 14% are most satisfactory. For a penetration depth of 4 m, the modeling without damping is close to the experiment to 5 m, where it is necessary ξ = 8% to 10 and 15 m, for the radial component. For the vertical component, the modeling is close to the field result with ξ = 10%.

Table 4. Values of damping coefficients to the open-ended tube 21

Distance to the open-ended tube	Component speed	Depth 2.5m	Depth 4.5m	Depth 6.5m	Depth 8.5m
5 m	Vertical	15 %	5 %	0 %	0 %
	Radial	12 %	5 %	-	0 %
10 m	Vertical	15 %	10 %	5 %	2 %
	Radial	15 %	5 %	0 %	0 %
15 m	Vertical	15 %	10 %	5 %	5 %
	Radial	12 %	12 %	5 %	2 %

Table 5. Values of damping coefficients to the sheet pile 1

Distance to the sheet pile	Component speed	Depth 3m	Depth 4m	Depth 5m
5 m	Vertical	8 %	5 %	0 %
	Radial	14 %	5 %	-
10 m	Vertical	14 %	14 %	8 %
	Radial	8 %	5 %	-
15 m	Vertical	8 %	5 %	2 %
	Radial	8 %	12 %	-

Table 6. Values of damping coefficients to the sheet pile 7 (Rocher-Lacoste et al. 2005)

Distance to the sheet pile	Component speed	Depth 2m	Depth 3m	Depth 4m	Depth 5m
5 m	Vertical	14 %	14 %	10 %	8 %
	Radial	14 %	8 %	0 %	-
10 m	Vertical	14 %	14 %	-	5 %
	Radial	14 %	14 %	8 %	0 %
15 m	Vertical	12 %	14 %	8 %	2 %
	Radial	12 %	8 %	5 %	0 %

From these tables (4, 5 and 6), we observe that the damping coefficient that we will have to use to model the wave propagation decrease according to the depression depth of the pile:
ξ = 14 to 15% to 2 and 2.5 m of penetration depth,
ξ = 0 to 5% to a penetration depth of 5 to 8.5 m.

This could be due to the mechanical characteristics of the Flandres clay which grow with the depth. Indeed, the pressure limits pl grows linearly with the depth : 0.75 MPa to 4 m and 1.8 MPa to 16 m and the toe resistance qc : 2 MPa to 4 m and 5 MPa to 16 m.

CONCLUSIONS

From the point of view of French standardization, in the Flandres clay of Merville, the limit to vibratory pile-driving tubes or sheet piles would be 15 m for the resistance structures but for a penetration depth not exceeding 4 to 5 m.

The results generated with the 2D finite element model on the ground wave propagation study are satisfactory. Uncertainties remain however and further calibrations are necessary. The introduction of damping into the numerical model (Rayleigh formulation) has enabled us to obtain an even closer approximation of reality. The lack of experimental measurements beyond 15 m is unfortunate and would have served to better define the damping coefficients. On the whole, this modeling set-up has yielded quite satisfactory results for small driving depths, which are representative of the majority of urban building projects.

To Merville site, numerical modeling using the finite element method showed that the damping coefficient decrease of 15% to 0% according to the depth of depression of the pile.

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REFERENCES

Akle E., Rocher-Lacoste F. (2006) "Vibration de chantier : point sur la normalisation," Proc. TRANSVIB 2006. Paris, ISBN 2-7208-2466-6.

Eurocode 3, "Calcul des structures en acier, Partie 5 : Pieux et palplanches, Annexe C : Mouvements du sol associés aux pieux et palplanches," AFNOR. Europe, 1998.

Ferber V., Abraham O., "Apport des méthodes sismiques pour la détermination des modules élastiques initiaux : application au site expérimental de Merville " 2003

Gonin H., Rocher-Lacoste, Sieffert J.-G et al, "Vibratory pile-driving - Technical guide", ENPC, 282, 2006, ISBN 2-85978-423-2

Humbert P., Fezans G., Dubouchet A., Remaud D., "CESAR-LCPC, un progiciel de calcul dédié au génie civil," Bulletin des Laboratoires des Ponts et Chaussées, n° 256-257, 2005.

Rocher-Lacoste F., Borel S. and Gianeselli L., "Comparative behaviour and performances of impact and vibratory driven piles in stiff clay", Proc. Cyclic Behaviour of Soils and Liquefaction, Balkema, 533-540, 2004, ISBN 90-5809-620-3

Rocher-Lacoste F., Ben Romdhane A., Semblat J.F., "Etude des vibrations dans le sol pendant la mise en œuvre par vibrage de palplanches dans l'argile des Flandres," Proc. Conférence Internationale GeoCityNet, Lille, 2005.

Semblat J.F., "Rheological interpretation of Rayleigh Damping," Journal of Sound and vibration n° 206 (5), pp 741-744, 1997.

Sieffert J.-G., Rocher-Lacoste F., "Vibratory pile-driving analysis of mearures and countermeasures with Braxuus," 4th International Conference on Earthquake Geotechnical Engineering, June 25-28, Thessaloniki, 2007.