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VIBRATORY PILE-DRIVING. ANALYSIS OF MEASURES AND COUNTERMEASURES WITH BRAXUUS

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ABSTRACT

Penetration tests were performed as part of the French Vibratory Pile-Driving Project. In the same time, the simplified prediction software BRAXUUS has been developed within the project scope. This software needs the knowledge of the soil-pile interaction laws. Relations are established between the static cone resistance and the interaction parameters deduced from the measured accelerations and stresses at the top and at the toe of the pile. It is shown in this paper that the elastoplastic laws obtained for the site of Montoir can be used for other sandy sites like Zeebrugge and Loon-Plage. On the other hand, the viscoelastic laws developed for the site of Merville can be used for other clayey sites like Limelette.

Keywords: Vibratory driving, dynamic soil-pile interaction laws, penetration prediction, CPT

INTRODUCTION

Vibratory driving is commonly applied:
- in granular soils: sands, silts and gravels, of relatively low or medium density,
- in cohesive soils featuring weak shear strength (soft soils) and relatively little plasticity,
- in sandy-clayey soils with only slightly-consolidated or relatively thin clay layers.

Vibratory equipment manufacturers and profile manufacturers have derived various empirical methods to facilitate vibrodriver selection. Most of these empirical methods do not account for the mechanical characteristics of the soils being traversed during vibratory driving operations.

At the present time, several software applications are available to theoretically select the equipment best adapted for installing piles into the soil by means of vibratory-driving. They were all adapted from software developed for impact-driving. They are based on the wave propagation equations but are not able to give continuously the penetration rate as a function of the penetration depth. Designed specifically for vibratory driving, the BRAXUUS software (Sieffert and Grézis, 2006) is based on a simplifying rigid-body hypothesis. Two soil-pile interaction laws are proposed to the user: an elastoplastic law and a viscoelastic law. These laws were derived from the interpretation of measurements performed on site.

The elastoplastic model is a 6-parameter model: 3 plastic plateaux, 2 quakes, and 1 displacement before the starting point of the elastic loading phase. It is shown in Figure 1 (left) illustrating a cycle from the start of the downward motion of the pile, and consists of:
- "a" for the upper loading plastic plateau,
- "b" for the lower loading plastic plateau,
- "c" for the lower unloading plastic plateau,

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- "q_1" for the loading quake,
- "q_2" for the unloading quake,
- "d" for the starting point of the elastic loading phase.

The viscoelastic model is a 3-parameter model: 1 stiffness term, 1 damping term, and a mean stress. The model is described in figure 1 (right). It consists of a linear elastic spring of stiffness K and a damper of characteristic C. A force F_{mean} independent of time, is applied to the base of the spring which itself is not fixed during a cycle but has a permanent instantaneous displacement $u_{per}(t)$. The base of the damper, on the other hand, is fixed. So the parameters are:
- "K" for the stiffness term,
- "$C\omega$" for the damping term,
- "a" for the mean stress.

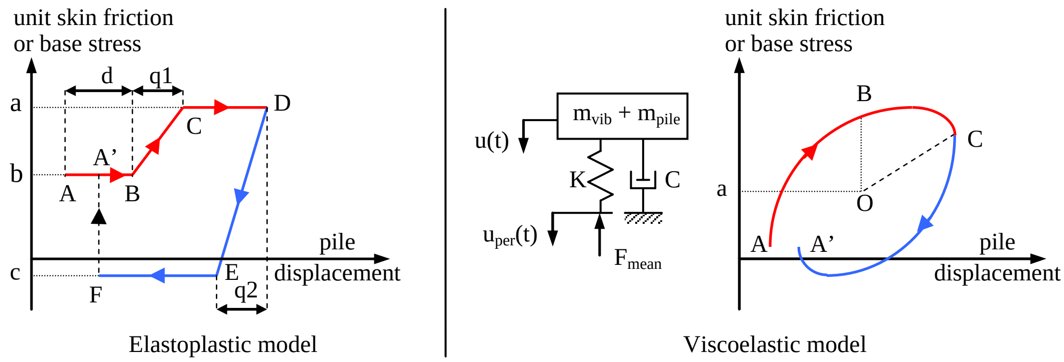

Figure 1. Elastoplastic and viscoelastic laws

All recorded signals (accelerations and stresses at the top and at the pile base) on the occasion of the field tests are periodic. Consequently, all forces applied to the pile (force generated by the vibrodriver, soil resistance, and so on) can be separated into two components: a "continuous" component (slowly variable with time and penetration depth) and an alternative component whose mean value over one cycle is zero. Because the excitation force and the inertial force have no continuous component, the balance of: the continuous components of the toe and the shaft resistances, the weight of the system vibrator-clamp-pile and the uplift load applied by the crane is continually zero (Sieffert and Borel, 2004). Therefore the analysis presented in this paper concerns only the alternative component.

Our purpose here is to propose in a first time a law linking the soil-pile interaction parameters to the static cone resistance through the knowledge coming from the Montoir and Merville full-scale field tests. The CPT was chosen as the most usual and representative in-situ soil investigation test. In a second time, we will check this law can be used to predict successfully the penetration rate in other sites.

KNOWLEDGE FROM THE MONTOIR FIELD TEST

General information

The new container terminal TMCD 4 was built in 1999 in Montoir-de-Bretagne on the estuary of the Loire River, close to the Saint-Nazaire Bridge. A previous paper (Borel and Durot, 2002) describes in detail the field tests carried out on a closed-end tube. It gives information on the site investigation, in particular the CPT profile (figure 2, left) and the statigraphic description of the soil (figure 2, middle). The vibratory driving was monitored using continuous record of the penetration rate (figure 2, right), the accelerations and the stresses at the top and at the pile base and the uplift load applied by the crane retaining the vibrator. The pile was a closed-ended iron tube (length 32 m, external diameter 339 mm) and was installed using an ICE 815 vibrator (frequency: 1330 rpm, excitation force: 896 kN). The refusal was observed at 17.8 m.

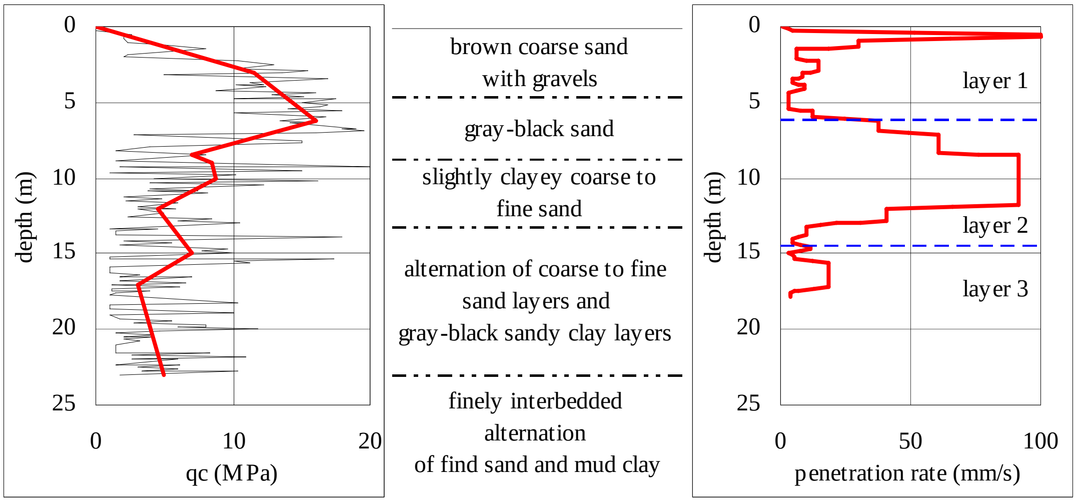

Figure 2. CPT and measured penetration rate (Montoir)

From the measured accelerations and stresses at the top and at the pile base, it was possible to propose soil-pile interaction laws for the base resistance and for the shaft resistance. A previous paper (Sieffert and Borel, 2004) describes the method and the results. It was shown found these laws were elastoplastic laws as shown on figure 1 (left) throughout the driving. We recall these results concern only the alternative component of the base resistance and of the skin friction force. Figure 3a gives the results obtained from this investigation for the 6 parameters of the model: "a", "b", "c","q_1","q_2" and "d" as defined previously. On this figure, we can see distinctly a first change of behaviour about 6 m depth, and a second about 14.5 m depth. We distinguish therefore experimentally 3 layers (figure 2, right). These layers do not correspond exactly to those obtained by the soil investigation (figure 2, middle). The boundaries of these layers correspond more or less successively to a large increase and a large decrease of the penetration rate (figure 2, right).

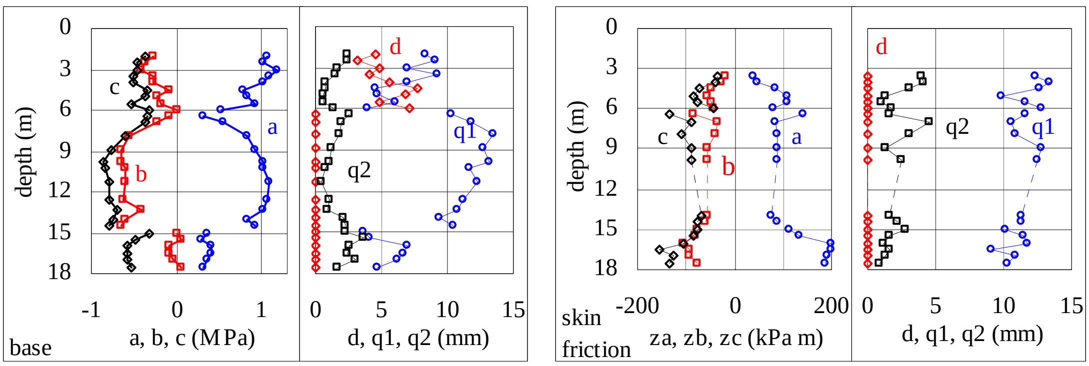

Figure 3a. Elastoplastic parameters versus depth (Montoir)

Parameter laws versus static cone resistance

As said previously, our purpose is to describe the elastoplastic parameters directly as a function of the cone resistance qc. Consequently we have simplified the variation of qc with the depth using linear functions. Then we have ruled out the depth between the cone resistance and the elastoplastic parameters. The results concerning the base resistance of the pile are given on figure 3b.

It is clear the behaviour in layer 3 differs from the behaviour in layers 1 and 2. A possible explanation comes from the nature of the soil layers. Layers 1 and 2 are more or less homogeneous sandy soils. Layer 3 is an alternation of sand and sandy clay layers (see figure 3, middle). We are looking here for a law adjusted specially to granular soils. For this reason the proposed modelling of the elastoplastic parameters is limited to layers 1 and 2. The proposed models for "a", "b", "c", "q1" and "q2" are linear laws (see figure 3b) notwithstanding the results scattering particularly for "a", "q1" and "q2". No modelling is proposed for "d" because the values in layer 1 are very different from the values in layer 2 (and in layer 3).

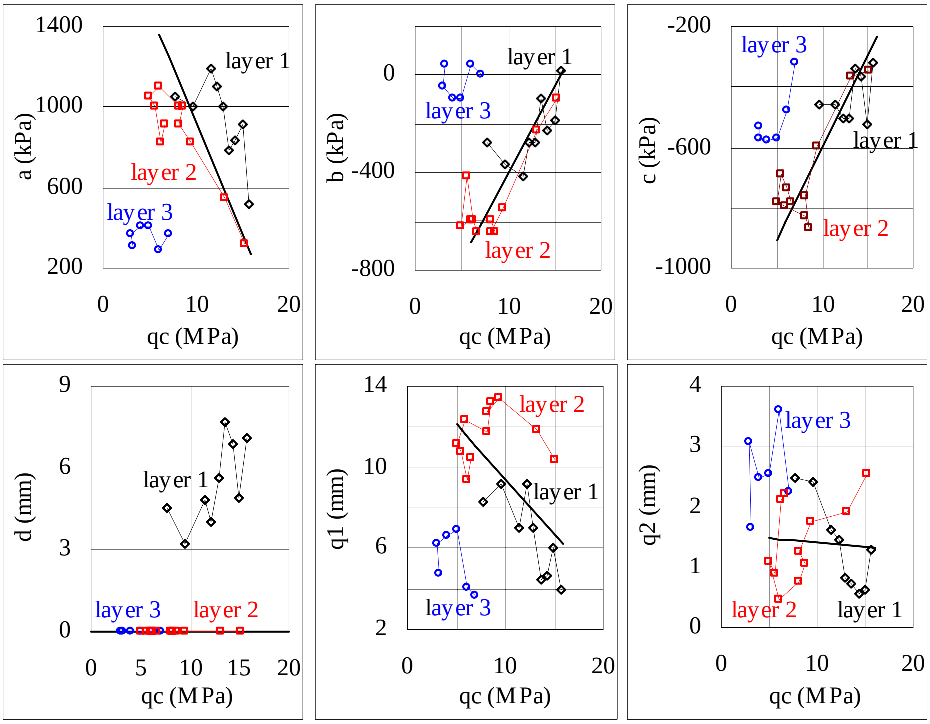

Figure 3b. Elastoplastic parameters versus cone resistance (Montoir)

The results concerning "a", "b", "c" may seem surprising: the upper loading plastic plateau decreases and the lower loading plastic plateau and the unloading plastic plateau increase in the same time when the cone resistance increases. In other words, the loop size decreases when the static cone resistance increases. These results concern only the alternative component of the base resistance as said previously. On the other hand, the "continuous" component increases with the cone resistance as displayed in the "Vibratory pile-driving – Technical guide" (Gonin et al, 2006). Due to the scattered experimental results, the law concerning the unloading quake has to be considered prudently and only as a general information.

COUNTERMEASURES AT THE SITES OF ZEEBRUGGE AND LOON-PLAGE

General information - Site of Zeebrugge

Vibro-driving tests were performed during the construction of the Dock Albert II in Zeebrugge (Belgium, North Sea). The information concerning the soils is confined to a CPT profile (see figure 4, left). The first meter is a recent deposit resting on saturated sands.

Two double sheet piles (Arcelor AZ36) were installed using an ICE 35RF vibrator (frequency: 2000 rpm, excitation force: 1535 kN) and a clamp ICE 200TU. Both sheet piles AB1 and AV1 have the same length (21.0 meters) and of course the same cross-sectional area of steel (331.4 cm^2).

Concerning the vibratory driving, we have at our disposal only the recorded penetration rates. They were obtained from the recorded time every 50 cm penetration depth (see figure 4, right). The first three meters are not very significant because of the results are influenced by the verticality adjustment of the sheet-piles. After these first three meters, the penetration rate can be considered as constant (until 10 m penetration depth for the sheet pile AB1 and 17 m penetration depth for the sheet pile AV1), with a mean value, in these ranges, about 60 mm/s for the sheet pile AB1 and about 72 mm/s for the sheet pile AV1. We have no information to explain the difference between the results obtained for identical sheet piles driven with the same vibrator in the same soil.

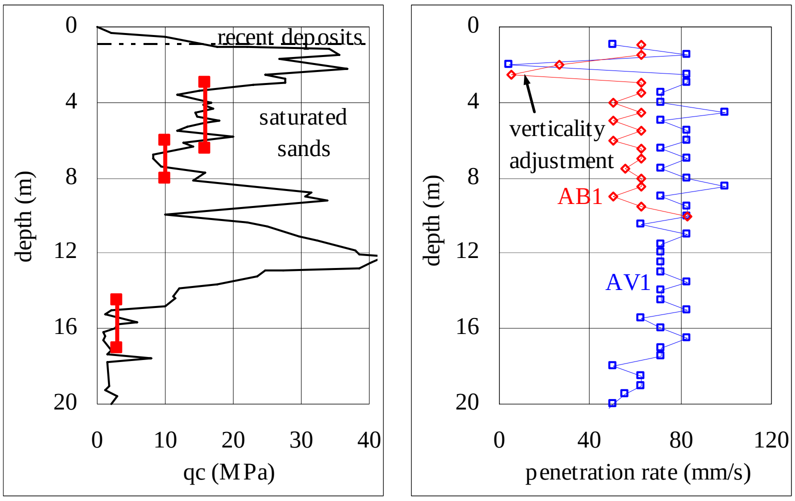

Figure 4. CPT and penetration rate - Zeebrugge

General information - Site of Loon-Plage

Vibro-driving tests were performed in 1999 in Loon-Plage close to Dunkerque (France, North Sea). The information concerning the soils is limited to a CPT profile (see figure 5, left). The soil consists of a mixed silts and sand layer and a fine grey sand layer resting on medium grey sand.

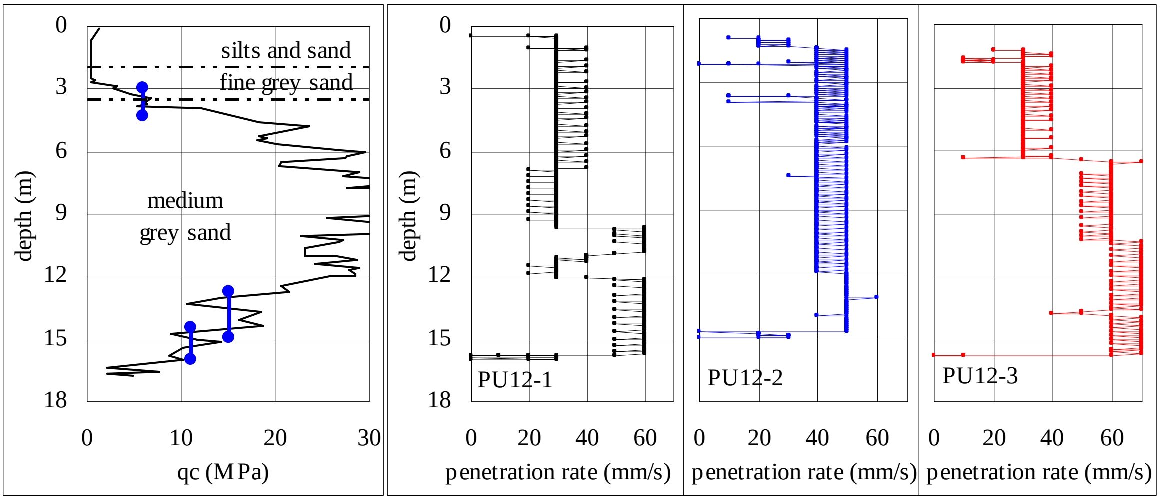

Figure 5. CPT and penetration rate - Loon-Plage

Three double sheet piles (Arbed PU12) were installed using an ICE 815C vibrator (frequency: 1570 Hz, excitation force: 1243 kN) and a clamp ICE 160TU. The three sheet piles have the same length (18 meters) and the same cross-sectional area of steel (168 cm^2).

Like for the site of Zeebrugge, we have at our disposal only the recorded penetration rates. They were obtained from the recorded penetration every second (see figure 5, right). The penetration rates can be considered as constant from 2 to 6 m depth (mean value 35mm/s for PU12-1and PU12-3, 45 mm/s for PU12-2:); from 7 to 9 m depth (mean value 25 mm/s for PU12-1, 45 mm/s for PU12-2, 55mm/s for PU12) and from 12 to 14.5 m depth (mean value 45 mm/s for PU12-1, 50 mm/s for PU12-2, 55mm/s for PU12-3). We have here also no explanation for the difference between the results obtained for same sheet piles driven with the vibrator.

Methodology and results

Neither acceleration nor stresses at the top and at the base of the sheet piles were recorded during the vibro-driving in Zeebrugge and in Loon-Plage. Consequently the direct determination of the soil-pile interaction laws is not possible. The way chosen here is to assume:

- the soil-pile interaction laws are elastoplastic laws because the soil is primarily sand like in Montoir,
- the parameters of these laws have the same value for the tube at Montoir and the sheet piles at Zeebrugge or at Loon-Plage when the cone resistance qc is the same in both sites.

From the observed constant penetration rates, it can be also assumed the shaft friction is negligible in Zeebrugge as well as in Loon-Plage.

With aid of the program BRAXUUS, we searched for the values of the elastoplastic parameters as close as possible to those obtained at Montoir using the linear laws (parameters versus qc) described above. The calculations were performed for three values of the cone resistance qc:

- Zeebrugge: 5 MPa (observed from 3 to 6.5 m depth), 10 MPa (observed from 6 to 10 m depth) and 16 MPa (observed from 14.5 to 17 m depth). The solutions were searched for a penetration rate in the range 65-70 mm/s,
- Loon-Plage: 6 MPa (observed from 3 to 4.5 m depth), 11 MPa (observed from 14.5 to 16 m depth) and 15 MPa (observed from 12.5 to 15 m depth). The solutions were searched for a penetration rate in the range respectively 35-45 mm/s, 55-65 mm/s and 50-65 mm/s.

It was certainly interesting to have also calculation for higher value of qc, for example 25 or 30 MPa. But the proposed laws from the results at Montoir site are limited to 15 or 16 MPa.

The calculations were performed supposing the parameter "d" (starting point of the elastic loading phase) is systematically nil. This assumption reduces the number of parameters. On the other hand, as said above, one cannot conclude reasonably for this parameter from the experimental data recorded at Merville site.

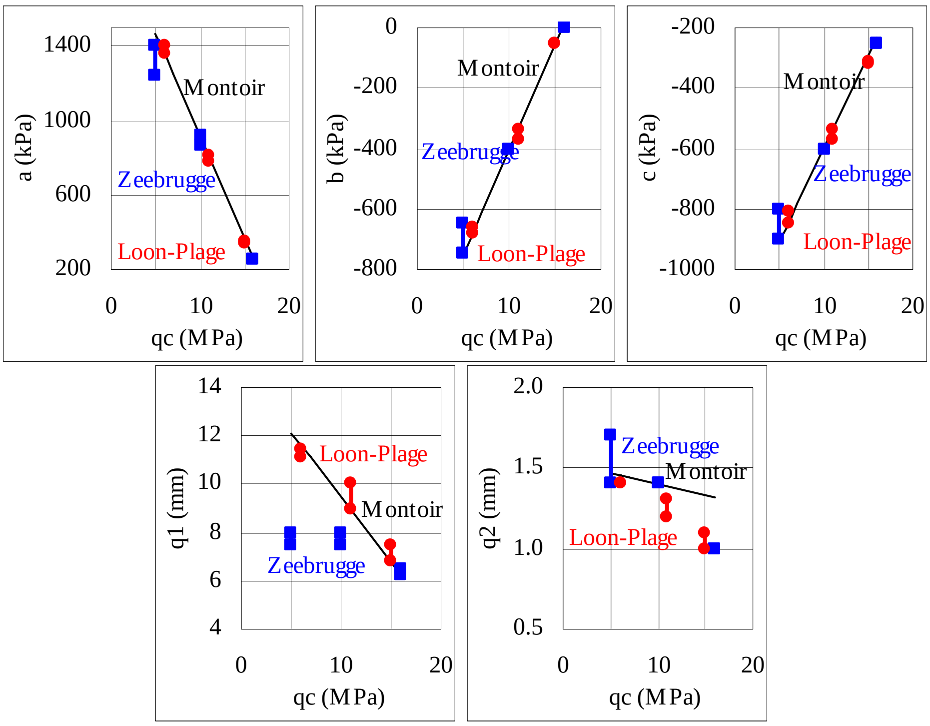

Figure 6. Elastoplastic parameters

The results are given on figure 6. Because the elastoplastic model is a 6-parameter model (more precisely 5 here) and because the solutions were searched in a range of penetration rate values, the solution is not unique. That explains the results presented on figure 6 in form of ranges and not in form of points. The results relative to Zeebrugge are characterized by blue lines limited by squares, and those relative to Loon-Plage by red lines limited by circles. In a general point of view, we see the obtained ranges are in the same time small and close to the proposed linear laws for Montoir.

One point is very important in this analysis: the upper loading plastic plateau "a" cannot be reached if the value of the loading quake "q_1" is too large. The theoretical value of the amplitude (peak to peak) of the alternative displacement for the free handing system is 10.2 mm for Montoir, 6.7 mm for Zeebrugge and 12.4 mm for Loon-Plage. At these values, we can add the value corresponding to the "permanent" displacement during one cycle, namely approximately 2 mm at Montoir, 1.5 mm at Zeebrugge and at Loon-Plage. If the aim is to reach effectively the upper loading plastic plateau "a", the loading quake "q_1" must be limited approximately to 12 mm at Montoir, 8 mm at Zeebrugge and 13 mm at Loon-Plage. Consequently it is not surprising that the results concerning Zeebrugge are significantly below the model established for Montoir.

Conclusions

- In the range qc = 5 to 16 MPa, the proposed laws for the value of the three plastic plateaus versus qc are globally verified in both sites of Zeebrugge and Loon-Plage.
- In the same range, the proposed law for the loading quake versus qc is also globally verified, but it must be limited in order to reach the upper loading plateau.
- The proposed law for the unloading quake versus qc was established from scattered value. It has certainly to be modified: a better law is perhaps a linear function going through the points (5, 1.5) and (15, 1).

KNOWLEDGE FROM THE MERVILLE FIELD TEST

General information

In the airfield of Merville, North France, the LCPC has performed for several decades many tests on this site: driving and vibro-driving on open base tube and sheet piles and static loading tests. A previous paper (Rocher-Lacoste and Borel, 2004) describes in detail the soil conditions of the site: it is made of silt up to 2.2 m depth resting on Flanders clay (marine clay deposit of Ypresien). This paper gives also information on the site investigation, in particular the CPT profile (see figure 7, left). A law in two parts is sufficient to describe the variation of the cone resistance qc versus depth: a constant value from 0 to 2.2 m and a linear function below. The soil is more homogeneous than in the site of Montoir.

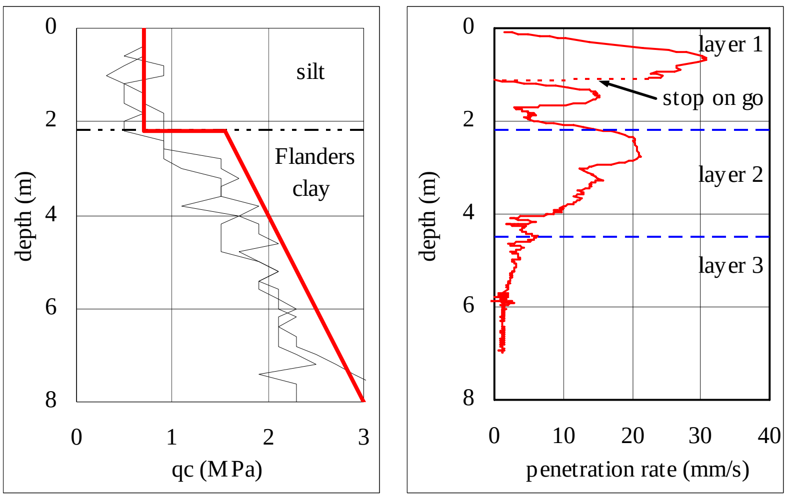

Figure 7. CPT and penetration rate - Merville

The vibratory driving was monitored using continuous record of the penetration rate (see figure 7, right), the acceleration and the stresses at the top and at the pile base and the uplift load applied by the crane retaining the vibrator. Owing to record problem, the penetration was stopped at about 1 m depth during half an hour and was continued after solving the problem. The pile was a double sheet pile AU16 (length 13 m, steel surface at the base 269.3 cm^2) and was installed using an ICE 815 vibrator (frequency: 1481 rpm at driving beginning and 1115 rpm at refusal). The refusal was observed at a depth of 7 m only.

Parameter laws versus static cone resistance

Using the same methodology as for Montoir, it was possible to propose viscoelastic soil-pile interaction laws for the base resistance and for the shaft resistance from the measured acceleration and stresses at the top and at the pile base. Here again, we consider only the alternative component. Figures 8 give the results obtained from this investigation for "K" (stiffness term) and "Cω" (damping term). Figures 8 (left) concern the values of the both parameters given as force per displacement unit, versus the penetration depth: the interpretation is easier in this form. On these figures, we can see distinctly a first change of behaviour at 2.2 m depth, and a second at about 4.5 m depth. The second change of behaviour is probably due to a mass of soil participating at the sheet pile movement. We distinguish therefore experimentally 3 layers (see figure 7, right).

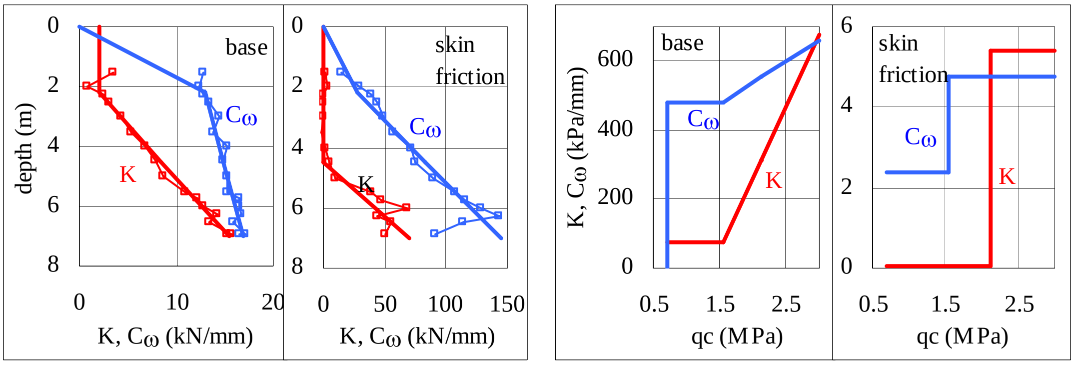

Figure 8. Viscoelastic parameters - Merville

Figures 8 (right) concern the modelling of the both parameters given as stress per displacement unit, versus the static cone resistance qc. It is clear both parameters "K" and "Cω" relative to the alternative components are increasing with the cone resistance. Contrary to the site of Montoir, the size of the stress-displacement loop increases with the cone resistance.

The "continuous" component increases also with the cone resistance as displayed in the "Vibratory pile-driving – Technical guide" (Gonin et al, 2006).

APPLICATION AT THE SITE OF LIMELETTE

General information - Site of Limelette

Limelette is situated in Belgium, near Louvain-la-Neuve. The information concerning the soil consists only in a CPT profile (see figure 9, left). Below a first layer (0 to 1.5 m), the soil consists successively of loam (1.5 to 3.5 m), silty clay (3.5 to 6.5) and quaterny sandy clay (below 6.5 m). The static cone resistance versus depth can be described with a law in three parts: constant (2 MPa) from 0 to 3.5 m, constant (3 MPa) from 3.5 to 5.8 m, and linear below 5.8 m (see figure 9, left, red line).

A double sheet pile was installed using a vibrator (frequency: 2000 Hz, excitation force: 1535 kN) and a clamp. The sheet pile is 20 meters long and has a cross-sectional area of steel of 311.4 cm^2 and a developed perimeter of 439 cm.

Like for the sites of Zeebrugge and Loon-Plage, we have at our disposal the recorded penetration rate. It was obtained from the recorded penetration depth every second (see figure 9, right, black curve with indication "measured"). The penetration rate is more or less constant until 4 m depth, and then decreases until the refusal depth (6.4 meters).

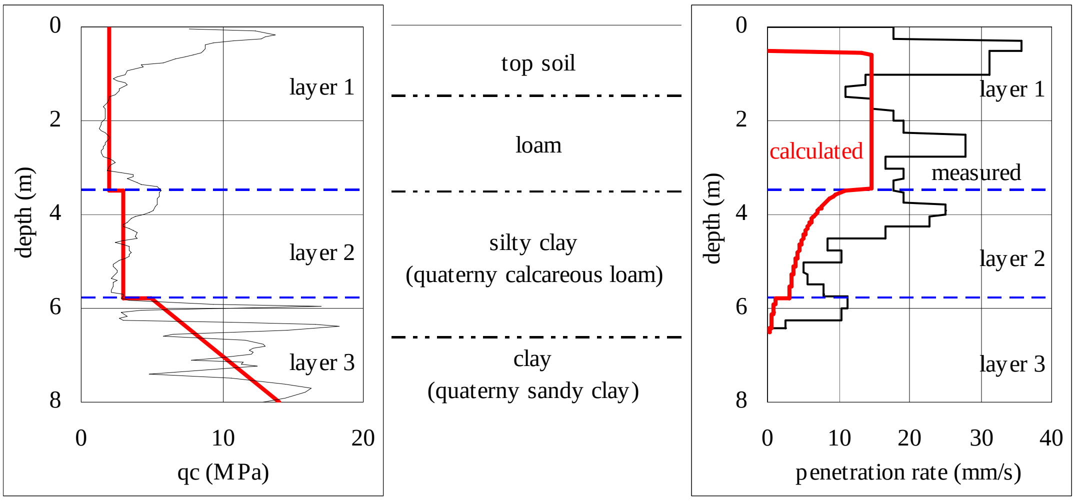

Figure 9. CPT and penetration rate - Limelette

Neither acceleration nor stresses at the base of the sheet pile were recorded during the vibro-driving in Limelette. Consequently the direct determination of the soil-pile interaction laws is not possible. The way chosen here is to assume:

- the soil-pile interaction laws are viscoelastic laws because the soil is a fine cohesive soil like in Merville,
- the parameters laws versus qc defined at Merville site can be extended at Limelette site.

Using the modelling of the parameters versus qc (figure 8, right) and the modelling of qc versus depth (figure 9, left), we obtain the parameters law versus depth for the site of Limelette (see figure 10).

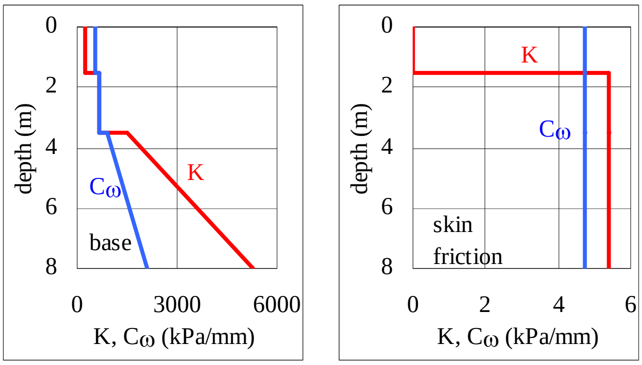

Figure 10. Viscoelastic parameters - Limelette

The penetration rate was recalculated using these laws. From the observed more or less constant penetration rate above 3.5 meters; it was assumed the shaft friction is negligible in the first layer. Figure 9 (right, red curve with indication "calculated") shows the penetration rate calculated with this information and these assumptions. The general form of the "calculated" curve is the same as the form of the "measured" curve. The calculated values are lower than the measured values. But in the same time, the calculated refusal depth is 6.5 m, very close to the measured value.

Conclusion

The parameter laws establish for Merville, extended to Limelette and introduced into the software BRAXUUS give satisfactory results concerning the penetration rate and the refusal depth. Of course, BRAXUUS cannot describe all details of the measured curve:

- the cone resistance law is a simplified law comparing the measured values,
- the interaction law parameters are constant in each layer.

CONCLUSION

At the present time, our data base is limited to two sites: one concerns a sandy soil (Montoir), the other a clayey soil (Merville). The recorded accelerations and stresses at the top and at the toe pile give access to the pile-soil interaction laws: a 6-parameter elastoplastic law for the sandy soil, and a 3-parameter viscoelastic law for the clayey soil are proposed. For both laws we were looking for modelling the values of the constitutive parameters versus the static cone resistance which is an usual soil investigation test. In spite of an important scattering of some results, a modelling of the parameters versus qc was proposed. Some results concerning the elastoplastic parameters are very surprising: the size of the soil-structure interaction loop (stress versus displacement) decreases when the cone resistance increases. On the other hand, the size of the soil-structure interaction loop concerning the viscoelastic law increases with the cone resistance as expected. We have no physically explanation for this observed phenomenon which concerns only the alternative component. We remind that, in the same time, the "continuous" component increases with the cone resistance for both types of soil.

The proposed modelling for the elastoplastic law was verified by countermeasure on two sandy sites (Zeebrugge and Loon-Plage). The results are very satisfactory: they are close to the proposed model. Nevertheless the loading quake has to be limited regarding the amplitude of the alternative displacement of the free hanging system and the "permanent" displacement. In the same time, it seems the unloading quake has to be a little modified. The proposed modelling for the viscoelastic law has allowed the calculation of the penetration rate curve (Limelette) which is very satisfactory comparing with the measured curve. Consequently, this software is able to predict correctly the penetration in sandy soils and in clayey soils. Of course, the software BRAXUUS cannot describe all details of the measured curve.

All analysis and calculations presented in this paper concern only the alternative component of the forces. Therefore it is not necessary to know the uplift load applied by the crane to the vibrator. That is an advantage for the prediction of the pile penetration into the soil, because this load versus time or penetration depth cannot be known in advance. It is only sure this load is approximately the weight of the system vibrator-clamp-pile at the penetration beginning, and nil at refusal depth. One can be surprised that this parameter does not interfere directly in the vibro-driving process.

Our aim in the future is to develop our data base by introducing the largest possible experimental results from field tests on instrumented vibro-dived piles; in order to verify - or to modify – the modelling proposed in this paper.

ACKNOWLEDGEMENT

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