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Toe resistance during pile vibratory penetration
Résistance à la base lors du vibrofonçage d'un pieu

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ABSTRACT : The purpose of this article is to compare the behaviour of soil observed in a laboratory experiment, which simulates the vibratory driving of a pile with results of the Vibrive II model (Holeyman 1993). In the laboratory experiments, a model sheet-pile was driven in dry sand using a vibrator. The toe and shaft resistances were measured continuously during driving. These values are compared with the static resistance of the soil evaluated using a cone penetration test (CPT). The Vibrive II model predicts the penetration speed of a pile during vibratory driving. The behaviour of the soil is simulated with a visco-elasto-plastic model. The toe and shaft resistances calculated using this model are compared with the measured values from the laboratory experiment. Based on this comparison, recommendations are made for improvement of the Vibrive II model.

RÉSUMÉ: Le but de cet article est de comparer l'observation du vibrofonçage d'un pieu en laboratoire avec les résultats du modèle numérique Vibrive II (Holeyman 1993). Le pieu expérimental est foncé dans du sable sec au moyen d'un vibrateur. Les résistances du pieu, à la base et au frottement latéral, sont mesurées en continu durant la vibration. Ces valeurs sont comparées à la résistance statique du sol mesurée au moyen d'un test de pénétration (CPT). Le modèle Vibrive II calcule la vitesse de pénétration d'un pieu lors de son vibrofonçage. Le comportement du sol est simulé par un modèle visco-élasto-plastique. Les résistances à la base et au frottement latéral calculées par le modèle sont comparées aux mesures expérimentales. A partir de cette comparaison, des recommandations sont données pour la modification du modèle numérique.

1 LABORATORY EXPERIMENT

The principle of the vibratory driving experiment is illustrated on Figure 1. The tests were performed on a reconstituted soil specimen in a cylindrical container with a diameter of 0.625 m and a height of 1.50 m. The sand used in the experiments is a dry bruxellian sand (d_{50} = 0.17 mm and Cu = 2.3). The samples were constructed in two steps : first, the dry sand was pluviated into the container and, second, compactive effort was applied to the sample, using vibratory methods, to reach the desired relative density.

Following placement of the dry sand specimen, a CPT test was performed including measurements of the local skin friction. Results of the CPT testing indicate the sand samples were relatively homogeneous. The average cone resistance measured for each test was approximately 2.0 MPa, which corresponds to a loose sand (Dr=65%).

The model pile consisted of 2 steel U-profiles welded together to form a 'long oval section'. The section has a full area of 3200 mm² and a perimeter of 240 mm. The mass of the sheet-pile is 15.9 kg. The pile was equipped with 6 strain gages (2 towards the top, 2 in the middle and 2 towards the toe). The applied load and soil reaction to the pile were directly evaluated using those

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6 strain gages. The vertical acceleration of the pile at the top and the penetration of the pile into the soil were continuously monitored (1 kHz sampling frequency).

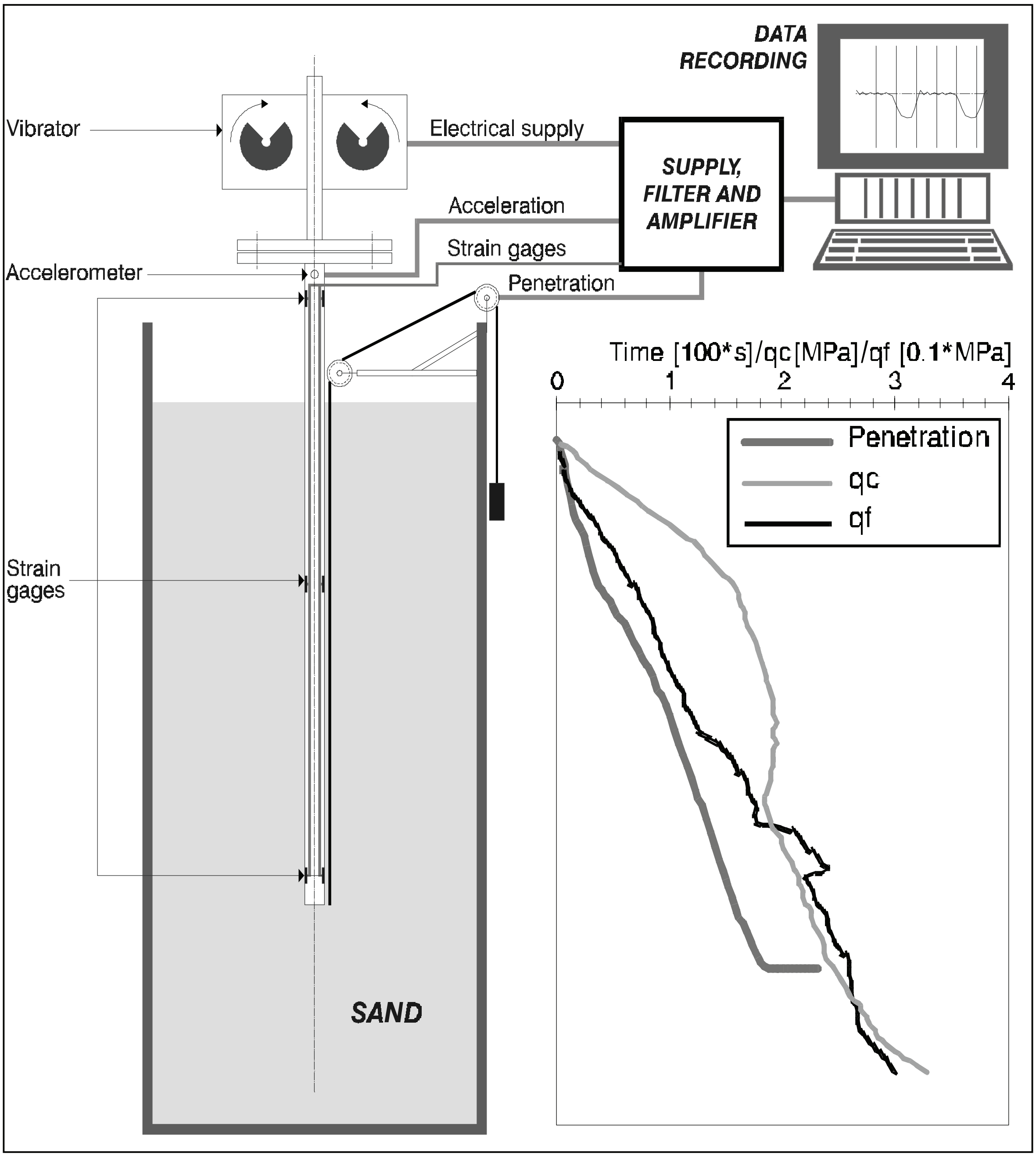

Figure 1. Experimental set-up

The model pile was driven by two electric vibrators, which applied a cyclic vertical load to the top of the pile. The influence of different eccentric moments, Me (from 0.071 kg.m to 0.3085 kg.m), and frequencies, υ (50 Hz and 25 Hz), were analysed.

2 EXPERIMENTAL RESULTS

The strain gages were positioned on the pile such that the measured soil-pile interaction could be described as two different types of forces: toe resistance and shaft resistance. Figure 2 shows the measured pile acceleration and soil resistance values plotted versus time.

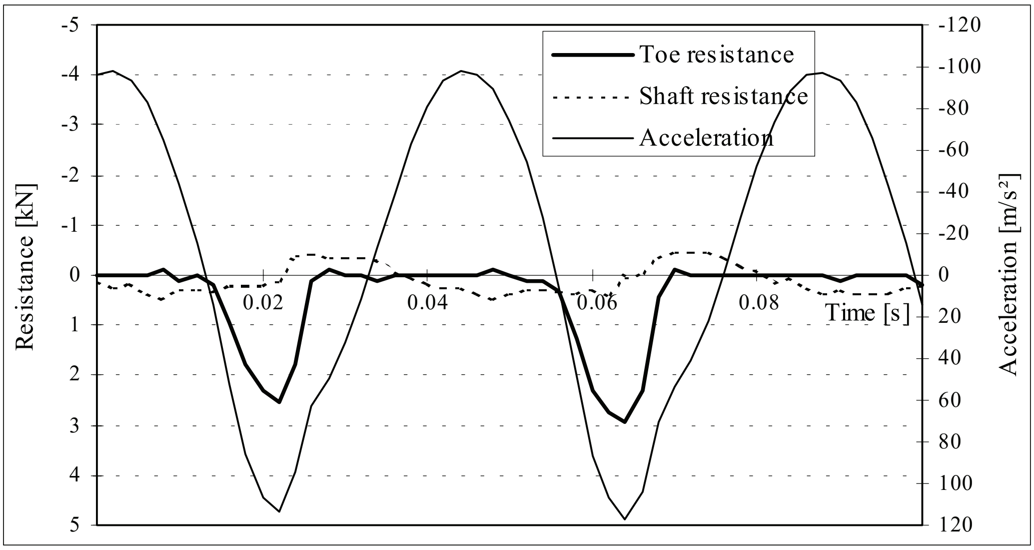

Figure 2. Measured values of pile acceleration and soil resistances
(Penetration of 0.7 m, Me = 0.1542 kg.m and υ = 25 Hz)

A more insightful way to characterise the penetration behaviour is offered however by plotting the penetration speed versus soil resistance measured during CPT testing, as shown in Figure 3. The total shaft static resistance is the estimated value calculated by multiplying the local shaft resistance by the total shaft surface area of the pile in contact with the soil. The data shown in Figure 3 is labelled with the ratio of total shaft static resistance and toe static resistance values, η, at the considered depth. The data was obtained from 7 different experiments (i.e. sample preparation and pile driving) combining 4 sets of the vibratory parameters Me and υ applied in different sand specimens.

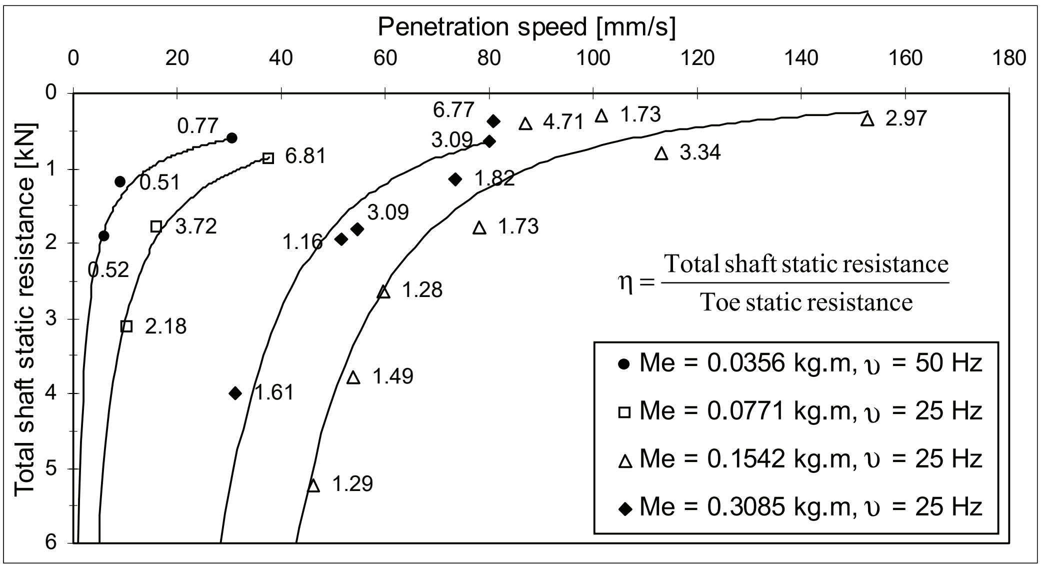

Figure 3. Measured penetration speed as a function of the total shaft resistance

These 4 lines corresponding to the 4 sets of vibratory parameters show that for a given shaft resistance the largest eccentric moment does not necessarily lead to the maximum penetration speed. For a given frequency of 25 Hz, the maximum penetration speeds are obtained with an eccentric moment of 0.1542 kg.m.

3 MODEL

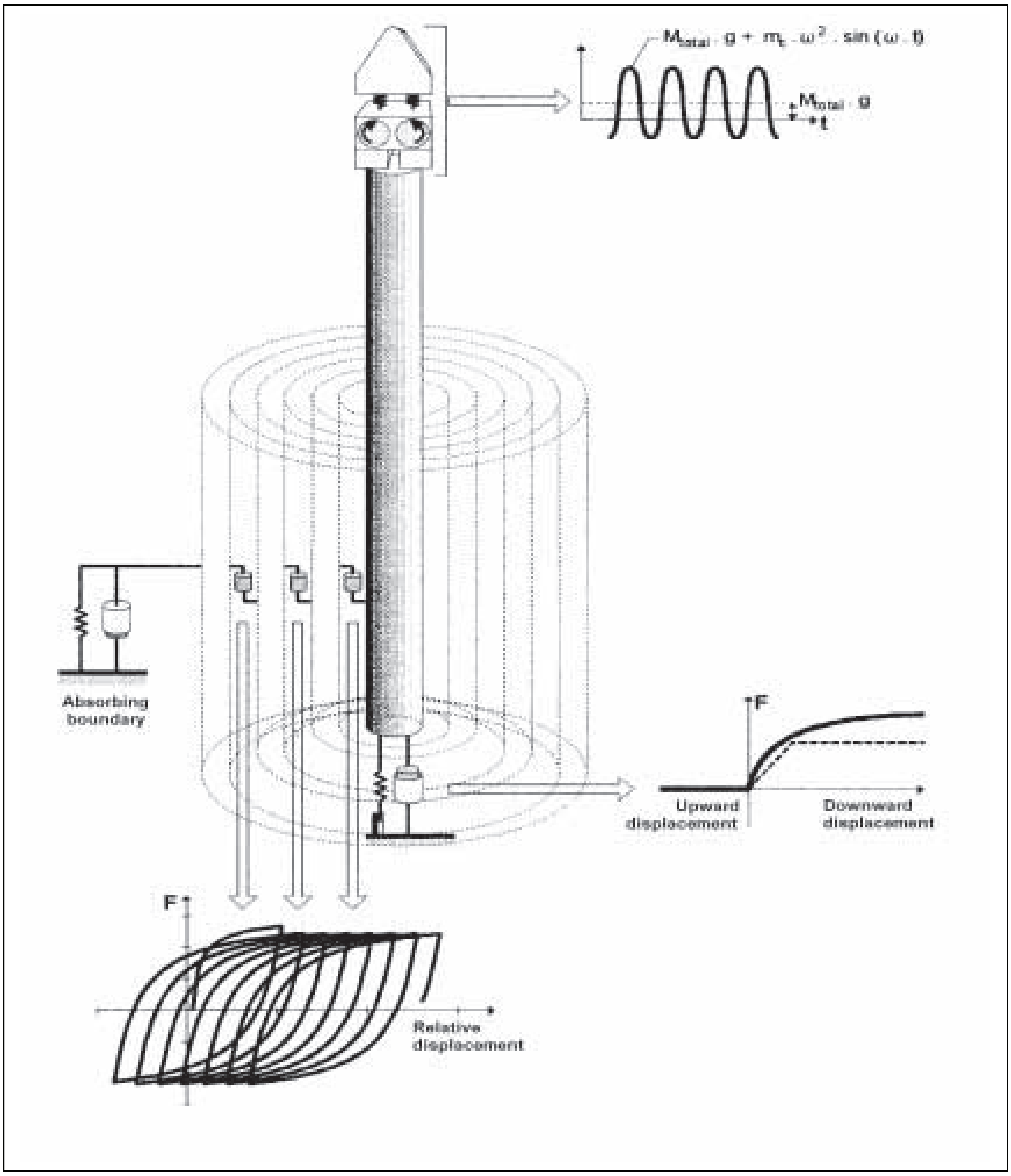

Figure 4. Geometric model

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The Vibrive II model (Holeyman 1993) is based on the analysis of the dynamic behaviour of a cylinder embedded in a semi-infinite medium. The pile is represented by a rigid mass and the soil is represented by a stack of discrete concentric rings that have their own individual mass (Fig. 4) and transmit forces to their neighbouring rings. The shear force-displacement relationship between successive rings is calculated based on an elasto-plastic stress-strain relationship.

The model uses the hyperbolic law (Kondner 1963) to describe the static behaviour of soil and the two Masing's laws (Masing 1926) to represent the hysteresis observed during a cyclic loading. The toe resistance of the pile is modelled by the combination of a spring, a damper and a slider. Vibrive II computes the different parameters needed for these models using correlations with CPT test data.

The movement of the pile and the rings are calculated from the time integration of the law of motion: i.e. the acceleration resulting from the net unbalanced loads acting on each element is double integrated in order to estimate the displacement of each element.

4 COMPARISON OF EXPERIMENTS WITH BASIC MODEL

Figure 5 presents the toe resistance, the shaft resistance and the acceleration values calculated using Vibrive II for the same conditions of the laboratory experiment presented on Figure 2.

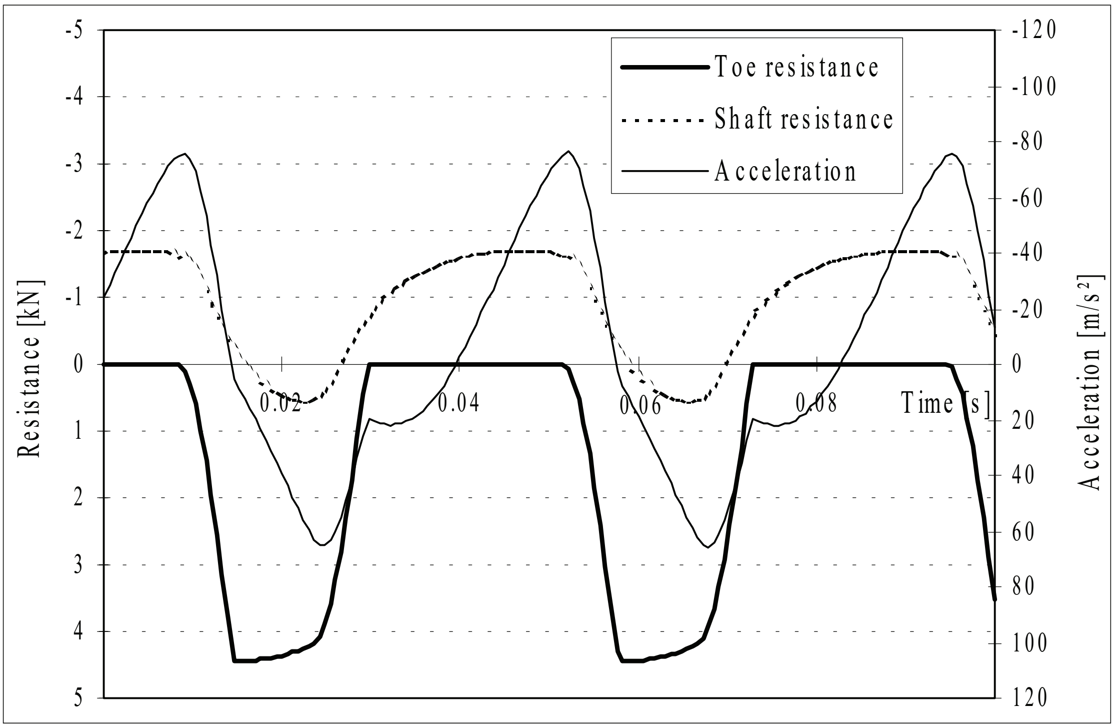

Figure 5. Calculated values of acceleration and soil resistances
(Penetration of 0.7 m, Me = 0.1542 kg.m, υ = 25 Hz, Toe resistance = 1.7 MPa, Friction ratio = 1.42 %)

Several observations can be made regarding the results of the Vibrive II model (Fig. 5) and the laboratory experiments (Fig. 2):

- The calculated values of the maximum toe resistance are approximately 2 times larger than the measured values. In the model, the toe resistance is mobilised in each cycle (i.e. when the direction of the movement changes) when the acceleration begins to decrease. However, the experimental results show that the toe resistance initiates when the sign of the acceleration changes.

- For the shaft resistance, timing of mobilisation is very different. In the model, the maximum value corresponds to the maximum acceleration but the measurements show a delay between these two extremes. The analysis of the experimental results point out the importance to also take into account of the instantaneous speed of the pile.

- The evolution of the acceleration is quite similar. However, the values are very different (100 m/s^2 measured vs. 60 m/s^2 calculated). This phenomenon is a direct consequence of the two first observations.

The penetration speed measured when the model pile had a penetration of 0.7 m is 49 mm/s. But the model predicts a refusal at this deep with the same vibrator. To show the influence of the toe resistance on the calculated speed, another calculation was run with zero toe resistance. The calculated penetration speed was 300 mm/s. Taking into account this strong influence, two modifications are proposed in the following section.

5 MODEL IMPROVEMENTS

To decrease the maximum reaction force at the pile toe, the characteristics of the slider at the toe should be changed. In the model, as soon as given "quake" is reached, the force becomes limited implying that the soil can not provide more resistance. The maximum force is given by: $F_b = \Omega.q_c / L$ where Ω is the surface of the pile toe and q_c is the cone resistance measured at this deep during the CPT test. The denominator L expresses the degradation from the static value resulting from the cycle nature of loading.

A second problem was noted when the pile goes down again from one cycle to another. The model introduces a toe resistance as soon as the pile penetrates. On the other hand, measurements show that the pile is able to move downwards some distance below its cycle apex without mobilising much resistance. This behaviour tends to indicate that the gap formed below the pile toe during the withdrawal phase is not completely closed when the pile reverses its motion.

More insight into the mobilisation of the toe resistance can be obtained by plotting the toe resistance as mobilised by the toe penetration, as shown in Figure 6. Figure 6b was obtained from the experiments, where the pile penetration was evaluated by double integration of the acceleration signal over time. In spite of that double integration, the error is less than 0.4 mm/s for the penetration speed and less than 0.036 mm for the penetration. This figure represents the evolution of the toe resistance in function of the pile penetration when the model pile had a penetration of 0.9 m (Me = 0.1542 kg.m and υ = 25 Hz).

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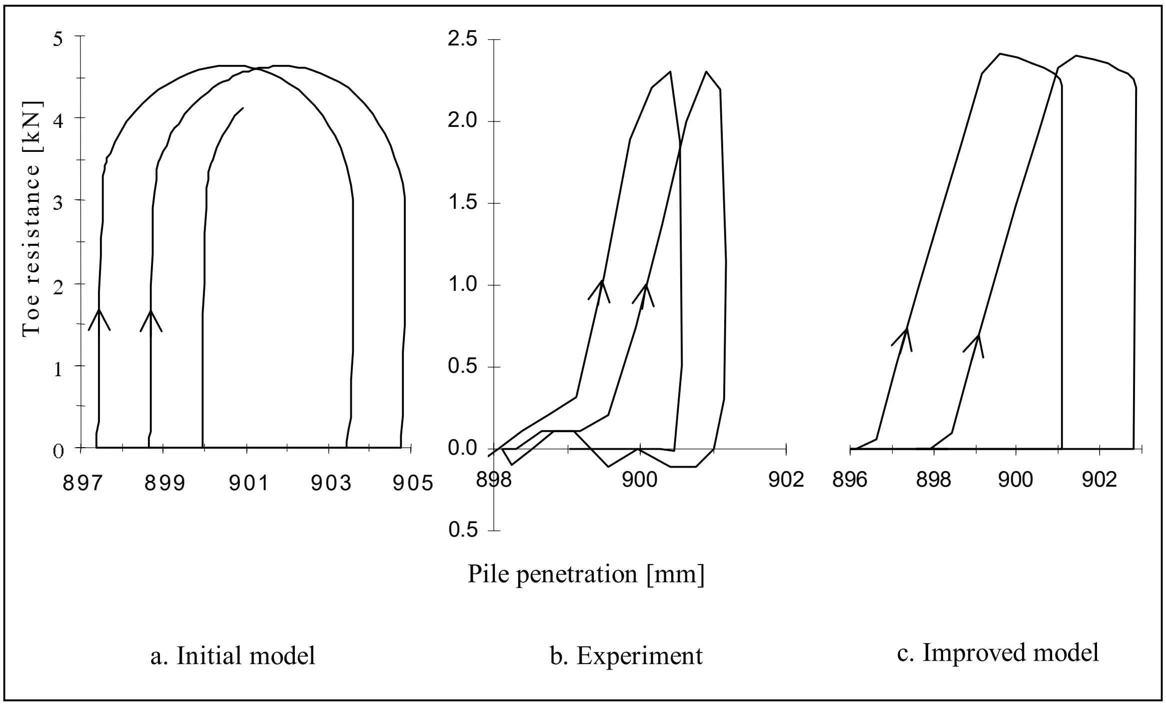

Figure 6. Mobilization of the toe resistance as a function of the pile penetration

Figure 6b shows that:

- the maximum toe resistance for a cycle does not correspond exactly with the maximum penetration for this cycle

- during the descent of the pile, the increase of the toe resistance is nearly linear. The value of this resistance seems to tend to a same maximum for each cycle.

Figure 6c shows the mobilisation of the calculated toe resistance after these two improvements were implemented in the programme code. The curves have similar patterns and amplitudes. To have a best result, the characteristics of the slider, the damper and the spring have been modified too.

6 CONCLUSIONS

The experimental set-up presented has permitted to compare the Vibrive II numerical model to experimental results of the penetration speed of a model-instrumented pile. Tests results have shown that the larger eccentric moment does not lead to the maximum penetration speed for a given shaft resistance.

The strain gages placed on the pile have allowed the authors to perform a detailed study of the soil reaction mobilised at the toe of the pile. Two modifications to the program code have been made to improve the realism of the modelled load-settlement curve at the toe pile. The first one consisted in decreasing the maximum force the soil can mobilise. The second one allows for the opening and closing of a gap below the pile toe. These two modifications have reduced the influence the toe resistance in the calculation of the penetration speed.

7 ACKNOWLEDGEMENTS

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