Penetrability and Drivability of Piles - Belgian National Draft Report

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1. Introduction.

The northern part of Belgium is constituted by loose tertiary and quaternary formations, slightly bended to the north. The tertiary layers are essentially constituted of over-consolidated sands and stiff fissured clays. The quaternary layers are essentially constituted of pleistocene sands, silts and gravels. Along the Scheldt River and tributaries and at the location of former river beds and creeks, they are covered by alluvial deposits consisting of fluviatic silts or peat.

In the southern part of Belgium, the primary formations predominate, the recent layers subsisting only under the form of fragments spared by the erosion or as fillings of dissolution surfaces in the secondary chalk formation.

In the northern part, most piles are based in the dense sand layers within the pleistocene formations, or in the tertiary sands and clays. In the southern part of Belgium, piles are usually based onto firm rock, whose characteristics depend on the location, but usually shales. All these features call for piles foundations, whith an average depth comprised between 8 and 18 m. In large cities, deeper piles are more frequent, especially when the stratigraphy has been under the strong influence of a river.

Among the driven piles used in Belgium, the driven cast-in-situ pile is very popular. It can be provided with an expanded or an overexpanded base, (Franki System); its shaft can be made of rammed concrete or vibrated concrete, the latter being more frequent. Of significant use is also the precast concrete pile, usually pre-stressed and imported from the Netherlands. Driven steel tubes or steel profiles are still marginal. Wooden piles are no longer used.

The profession is organized as follows : the owner appoints an architect and/or a consultant, depending on the nature of the work. They have the responsibility for the design. The project is elaborated to a stage when documents are established for contractors to price the job. The job is usually attributed on the basis of an open bid, the cheaper bidder securing the job. Construction of the piles is sub-contracted to a specialist, who is responsible for the execution and who can introduce alternatives, depending on the specificity of the tendering documents.

Practically no standard exist in the field of geotechnics in Belgium, even less so in the piling sector. A standard for pile foundations is under draft since 1977. Up to now the drafts relative to the soil investigation and the loading tests have been released.

The present national report covers the main topics encountered in the field of driveability of piles in Belgium : design of piles, prediction of blowcount diagram, prediction of driving stresses, driving equipment, determination of final set, recent researches and environmental aspects.

2. Design of Piles

2.1 From CPT tests

In Belgium a very large experience exists with the design of piles starting from the results of CPT-tests. In general, mechanical CPT tests are performed with a mechanical cone type M4 and with deduction of the total side fiction Q_{st} from the total penetration resistance.

For the calculation of the ultimate bearing resistance at the base of a driven pile the scale effect is introduced following the method of De Beer (1971, 72). In fig. 1 the curve $q_{b,DB}$ gives the variation with depth of the ultimate bearing resistance at the base of driven piles with a diameter of 300 mm, resp. 600 mm, as deduced from the diagram of the cone resistance q_c with the method of De Beer.

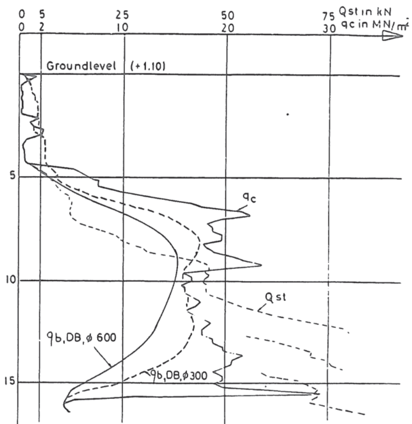

Fig. 1 CPT test and $q_{b,DB}$

From the results of static loading tests it has been deduced that near to the upper and lower boundaries of the bearing layer the calculated value $q_{b,\ DB}$ corresponds more to the limit load (settlement of the pile base approximatively equal to 2,5 % of the base diameter) and more to the conventional rupture load (settlement of the pile base equal to 10 % of the base diameter) at large depth in that layer. (De Beer, 1984). When sufficient data is available, the trend is now to introduce a corrective factor to take into account the particular geometry of the base and the casting procedure.

The ultimate bearing capacity resulting from skin friction is in most cases deduced from the total side friction Q_{st} measured in the CPT test, assuming proportionality with the perimeters. When possible, a factor is introduced to take into account the nature of the pile skin and the installation procedure.

Normally a safety factor of 2 is applied to the calculated ultimate bearing resistance at the base and a safety factor between 2 and 3 to the calculated bearing capacity by side friction.

The allowable bearing capacity can also be obtained from the ultimate bearing capacity, predicted from the results of CPT tests, by applying partial safety factors, one covering the problem of large deformations, and two others, covering the dispersion, resp. for the base and lateral bearing capacities (De Beer et al,1981).

2.2 From DPT, SPT and PMT tests

The design of piles can also be done by means of the pressuremeter characteristics : the Menard modulus E_M and the limit pressure p_l. The bearing capacity is function of the values of p_l around the base of the pile and along the shaft. The settlement of the top of the pile will be given by formulae into which the values of E_M are introduced (see Menard (1963, 1975) and Gambin (1963)).

Otherwise, the limit pressure p_l can be found by using correlation formulae between the cone resistance q_c and p_l :

$$\frac{q_c}{p_l} = \begin{cases} 9 & \text{for sand} \\ 6 & \text{for silt,} \\ 3 & \text{for clay.} \end{cases} \qquad (1)$$

Some correlations have also been established with dynamic penetrations tests (DPA, DPB, DPL and SPT). The total dynamic specific energy

$$E_d = \frac{Mgh}{A.s} \qquad (2)$$

with M : mass of the hammer,
g : acceleration due to gravity,
h : height of the fall,
A : area of the base,
s : penetration per blow

gives the q_c values by $q_c = \eta_d \cdot E_d$ with (3)

η_d = 0.60 to 0.70 for penetrometer DPL in sand,
0.40 to 0.50 for SPT in sand,
0.12 to 0.15 for clayey soils;

and the p_l values by :

$$p_l = f_{p_l} \cdot E_d \text{ with } f_{p_l} = \frac{\eta_d}{(q_c/p_l)} \qquad (4)$$

3. Prediction of blowcount diagram

The blowcount diagram is usually predicted by the contractor in order and select the driving equipment and to assess his production beforehand.

Two main procedures can be used :

1) determine the bearing capacity features of the member along the driving depth and from there determine the set by various methods,
2) scale the results of a dynamic in-situ penetration test to the size of the pile and hammer energy.

3.1. From bearing capacity features

On the basis of the methods exposed in chapter 2, the bearing capacity of the driven member can be estimated at any depth. The two main methods to relate this bearing capacity to the set are the use of driving formulae and a wave equation analysis. The first one is often used for most of the cases whereas the second one, which requires more work and computer facilities is performed for special cases. However, due to the shortcommings of the driving formulae, the present trend is in favour of the wave equation analysis.

3.1.1. Driving formulae

To show the various formulae used in Belgium, it is interesting to present the energy equation relating the driving resistance Q_D to the plastic set s in the general form (Sørensen and Hansen (1957))

$$\eta_i \ \eta_c \ Mgh = \frac{1}{2} Q_D \ s_{el} + Q_D \cdot s \qquad (5)$$

in which η_i is the efficiency of impact
η_c is the efficiency of drop
s_{el} is the transient displacement or "elastic" rebound.

The four most popular driving formulae are the ones of Eytelwein (or dutch formula), the danish formula, the Hiley formula and the Delmag formula (after Crandal). Their parameters are summarized in the following table :

	η_i	η_c	s_{el}
Dutch	$\frac{1}{1+\mu}$	1	0
Danish	1	0.7 - 1	$\left(\frac{2\ \eta_c\ Mgh\ L}{A_p E}\right)^{1/2}$
Hiley	$\frac{1+e_r^2\mu}{1+\mu}$	0.75 - 1	$C_1 + \frac{Q_D\ L}{A_p E} + C_3$
Delmag	$\frac{1}{1+\mu}$	1	$0.6\ 10^{-3}\ L$

with L = length of pile (m)
A_p = Section of pile (m^2)

E = Modulus of deformation of pile [MPa]
M_p = Mass of pile [Mkg]
μ = M_p/M

C_1 and C_3 = Hiley constants obtained from tables [m] ($0 < C_1 + C_3 < 12\ 10^{-3}$ m)
e_r = Coefficient of restitution [-]

Results of these four formulae differ and experience has shown that some are better suited for certain applications. The Delmag formula is used when a Diesel hammer is considered. Better estimates are obtained when using the actual working energy at 44 blows/min rather than the rated energy. The Dutch formula is used for precast piles, but more often in the reverse application (chapter 6) with a high factor of safety. The Hiley formula is used by those who have enough judgement or experience to choose the right values of C_1, C_3 and e_r. The Danish formula has shown to give rather reliable results, even for bottom driving of Franki tubes.

3.1.2 Wave equation analysis

The algorithm suggested by Smith is certainly more satisfactory, provided one has a rational way to choose the so-called "quake" and "damping" in each soil layer.
In that respect, Holeyman (1984) has suggested to model the soil behaviour at the base and around the shaft in a more physical way. This physical model enables one to make a clear distinction between the geometrical damping, the viscous damping, the hysteretic damping and the velocity dependency of the ultimate strength. An hyperbolic stress-strain relationship is integrated to the geometry of the base or of the shaft so that the behaviour of each layer is governed by 3 characteristics : the ultimate unit strength, the initial tangent modulus and the velocity dependency of the ultimate strength. The ultimate strength is obtained from CPT tests, with some reduction factor to take into account the geometry of the base, the dynamic nature of the driving (liquefaction) and the friction wear. The initial tangent modulus is obtained either from available methods used in earthquake engineering or from a correlation with the cone resistance. The velocity dependency law of the ultimate strength is the one suggested by Gibson and Cooyle (1968), of the type

$$\tau = \tau_s \, (1 + v^{0.2}) \quad \text{with :} \tag{6}$$

τ ultimate strength at velocity v,
τ_s ultimate strength at velocity 0,

except that it is adapted to the reference speed v_r of the penetration test (v_r= 2 cm/s).
An advantage of the wave-equation analysis is to handle the diesel hammer action in a satisfactory way. A side product of such an analysis is also the prediction of dynamic stresses which is not given by driving formulae.

3.2 From dynamic penetration tests

The principle of the prediction lies in the comparison of two specific energies E_d :
the first one for the pile, the second one for the penetrometer.
But the specific energy which is to be considered is the energy working on the top of the penetrometer E_t. One has : $E_t = \eta_t \cdot E_d$
η_t (smaller than 1) measures the efficiency after the shock.

$$E_t = \frac{\eta_t \cdot Mg \cdot h}{s} = K_p/s \tag{7a}$$

The blowcount diagram is obtained by means of the following equation :

$$K_{pile}/s_{pile} = K_{pen}/s_{pen} \tag{7b}$$

The various penetrometers have for K the following values in MJ/m (with η_t= 1)

for the DPA $\dfrac{1 \times 63.5 \times 0.75 \times 9.81}{30 \times 100} = 0.156$

for the DPR $\dfrac{1 \times 63.5 \times 0.75 \times 9.81}{20 \times 100} = 0.234$

for the DPL $\dfrac{1 \times 10.0 \times 0.50 \times 9.81}{(10 \text{ or } 5 \text{ or } 4) \times 100} = 0.490$ ($A=10 cm^2$)
$= 0.980$ ($A=5 cm^2$)
$= 1.226$ ($A=4 cm^2$)

for the SPT $\dfrac{1 \times 63.6 \times 0.76 \times 9.81}{(20.3 - 7.9\, p) \times 100} = 0.264$ ($p=0.9$)
$= 0.360$ ($p=0.6$)

p being the ratio of the height of the sample inside the sampler to the penetration of the sampler.

The test pile of ESOPT II (Amsterdam - 1982) had

$$K_p = \frac{(0.9 \text{ or } 0.6) \times 1200 \times 2.6 \times 9.81}{625 \times 100} = 0.441 \; (\eta_t = 0.9) \; = 0.293 \; (\eta_t = 0.6)$$

The real blowcount diagram is compared in figure 2 with the blowcount diagram obtained by means of the DPB test with respectively η_t= 90 % and η_t= 60 %.

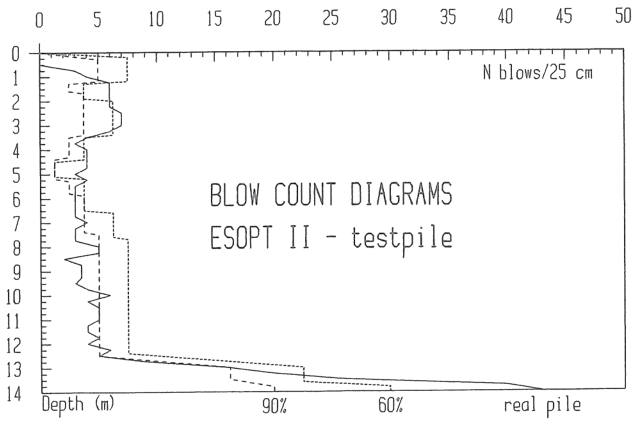

Fig. 2 : ESOPT II Blowcount diagrams

4. Prediction of driving stresses

Knowledge of the dynamic stresses during driving is critical to the driving of concrete precast piles and thin steel casings. The stresses in compression and in traction are predicted by three methods : empirical formula, analytical and numerical solution of the wave equation.

4.1 Empirical formula

It is proposed (Wallays, 1984) to introduce in the Belgian standards draft for pile foundations a limitation for the maximum compression stress σ'_b in the concrete of R.C. precast piles. σ'_b is assessed from a semi-empirical formula (Fellenius 1973).

$$\sigma_b = 30\sqrt{h_e} \quad (8)$$

σ_b' in N/mm^2, h_e in m being the equivalent drop height.

For a drop hammer actuated by rope and winch, h_e

$$h_e = \eta_c h \quad (9)$$

The efficiency η_c of the driving depends on the type, the characteristics and the condition of the used equipment. From the litterature (Brinch Hansen, 1960) the average value $\eta_c = 0{,}7$ can be used in first approximation when the equipment is well appropriated to the job and is in a medium condition.

On site, η_c can be deduced from a large enough series of measurements h_i and t_i of the drop height h and the elapsed time t from the formula :

$$\eta_c = \frac{1}{n} \sum_{1}^{n} \frac{2h_i}{gt_i^2} \quad (10)$$

For a diesel hammer, h_e is given by the approximate formula :

$$h_e = 1{,}22\, t_{d+u}^2 - 0{,}1 \quad (11)$$

The needed time t_{d+u}, in sec, for the drop and the upward motion of the hammer is given by :

$$t_{d+u} = \frac{60}{N} \quad (12)$$

N being the driving frequency in number of drop per min. Formula (11) is based on the assumptions that the drop time is equal to the upward motion time and that the sum expressed in drop height of the shock and pile penetration durations is equal to 0,1 m.

The maximum compression stress σ_b' must not be larger than the critical one $\sigma_{b,cr}'$

$$\sigma_b' \leqslant \sigma_{b,cr}' \quad (13)$$

$\sigma_{b,cr}'$ is given by :

$$\sigma_{b,cr}' = \alpha . \beta . \gamma . R_{bk}' \quad (14)$$

R_{bk}' is the characterstic value of the compression concrete strength determined on 150 mm diameter cylindrical samples.

α is a multiplying factor taking into account that the value of the concrete resistance is larger in case of shock than in standard test (De Kezel, 1979, Mainstone, 1975).

β is a reduction factor taking into account the repetition of the stressing during driving.

γ is a reduction factor taking into account the accepted value for the rupture probability.

For driving up to 2000 hard blows, the values $\alpha = 1{,}25$ $\beta = 0{,}8$ are appropriate so that in this case

$$\sigma_{b,cr}' = \gamma\, R_{bk}' \quad (15)$$

Usually the value $\gamma = 0{,}9$ or $\gamma = 0{,}8$ is accepted. They approximately correspond to one rupture for respectively 80 or 700 piles.

4.2 Analytical solution of the wave equation

The problem of the hammer of mass M impacting with velocity v a tube or pile of impedance I trough a cushion with a stiffness coefficient k can be solved analytically in the case of an infinitely long pile. The impedance of the pile is defined by : $I = A_p\sqrt{E\rho}$, ρ being the specific mass of the material of the pile. v is obtained by the formula $v = \sqrt{2\eta_c gh}$. Formulae expressing the maximum stress proposed by Hirsh (1966) have been rearanged in a simple chart (Holeyman 1984) reproduced in fig. 3. It gives a non-adimensional form of the maximum compression ($F_{max}/Iv = \sigma_{max}/\sqrt{E\rho}$) as a function of the impedance ratio, another adimensional factor defined by : $\eta_I = 2I/\sqrt{kM}$.

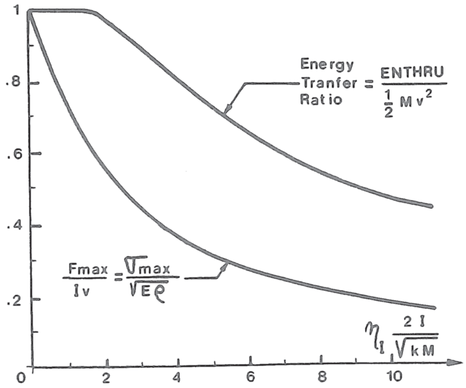

Fig. 3 Adimensional stress and energy transfer vs impedance ratio (semi-infinitely long pile).

Therefore the maximum impacting stress can be obtained from this chart with the following input data : M, v, k, I. In this chart is also given the energy transfer from the hammer into the infinitely long pile : this ratio of the transferred energy (Enthru) to the actual kinetic energy ($Mv^2/2$) only depends on the impedance ratio η_I. It can be related to the maximum value of η_i in equation (5). For tension forces, the method suggested by Hirsh (1966) is used.

4.3 Wave equation analysis

Since a wave-equation numerical simulation gives at various points of the pile, the force as a function of time, it is easy to obtain both maximum compression and maximum tension forces. The reliability of the results is more easily achieved for compression than for traction. In the latter case, one must be careful to use small pile segments. The introduction of a visco-elastic behaviour for the pile material has been found a convenient way to shave off some unrealistic peaks, leaving the significant ones unaltered, specially in tension. One is now aware that the driving system must be carefully modelled and in the case of driving Franki tubes, the tube head must be perfectly modelled. For the driving system, essential features need be known : the velocity of impact and the cushion stiffness. The first one requires measurements performed on specific hammers whereas the second one can be assessed by compression tests.

5. Driving equipment

Depending on the way the hammer is actuated, one can distinguish the following types most often used in Belgium :

- winch operated hammers,
- diesel hammers,
- hydraulic hammers.

The winch operated hammers are most often used to bottom drive the heavy tubes leading to the construction of an expanded base. In that case no helmet is required since the internal drop hammer is hitting the gravel or dry concrete plug which cushions the blow. The mass of the hammer varies between 2000 and 5000 kg and the height of fall between 1 and 12 m. These "free fall" hammers can also be used to top drive precast piles or tubes for cast in-situ piles. In that case, a helmet is used to cushion the blow which has a drop height usually of 1.5 m for tubes and 0.9 m for concrete piles. The efficiency η_c of the free fall depends on the winch mechanism and its relative inertia respective to the mass of the hammer. Typically η_c varies between 0.65 and 0.85.

The diesel hammers are used to drive precast piles and tubes for cast-in-situ concrete piles. Their striking mass usually varies between 2200 and 5000 kg whereas their actual drop height varies between 1.6 and 2.4 meters, thus significantly lower than the theoretical one given by the manufacturer (usually 3.2 m). Their selection is done according to the manufacturers' recommendations, taking into account either the mass of the driven member, or a combination of the length of the member and its driving resistance. In Belgium, since the length of driven piles is rather uniform around 16 m, the type of hammer depends on the section of the precast pile or of the driven tube. For piles .30x.30 m, one uses a 2200 kg hammer whereas for tubes up to Ø 0.45 m a 3500 kg hammer and for piles up to Ø 0.70 m, a 5000 kg hammer. It is clear that the final concrete section is in relation with the expected bearing capacity, such that the only governing parameter is truly the section of displaced soil.

More recently have appeared hydraulic hammers, which basically do the same job as diesel hammers, with the suggested relative advantages : control of drop height, no stalling in weak layers, regularity of operation. Their mass varies between 3 and 6 tonnes, and the drop height is typically 1-1.5 m. The efficiency of the free-fall depends on the hydraulic set up : it can vary between 0.8 and 1.0.

The peak energy transferred to the pile is still a portion of the actual kinetic energy of the hammer upon impact. Referring to the chart of fig 3 valid for a semi-infinitely long pile, one can select the cushion stiffness in order to get the best transfer without damaging the pile. More detailed charts for piles of finite length can also be consulted (Holeyman 1984). In practice, the ratio of the peak energy transferred to the pile to the actual kinetic energy of the hammer is on the order of 0.8 to 0.9. The energy finally transferred to the pile can however still be lower than the peak energy. In order to increase this energy transfer to the pile, some developments have also taken place in the field of helmets. Among the most effective ones, is the prestressed-type, which allows a reduction of the peak stress. This in turn allows to use higher drops, increasing therefrom the driving efficiency.

6. Determination of final set

6.1 Driving criterion

In Belgium, the following driving criterion is recommended :
A certain number of CPT tests are previously performed at the verticals where certain piles are to be driven. The allowable bearing capacity and the level of the pile base are deduced from the results of these CPT tests (De Beer, 1971/72; De Beer et al., 1977).

During driving the 1st pile at the very location or at least in the close vicinity of a CPT test, the set is measured at the proposed level. This set is then imposed within a narrow margin when driving the neighbouring piles. Driving is thus to be continued until the pile is placed in the same layer as the test pile and at such a depth that the same set is obtained. For calculating the set or the penetration per blow, or the mean value over the last 10 or 25 cm or the mean value of 5 consecutive observations of 10 blows is taken.

When none of the CPT tests has been performed at the very vertical or at least in the close vicinity of one of the piles to be driven, the allowable bearing capacity and the level of the pile base are deduced from the results of the less favourable CPT test.
If necessary, and based on the results of the previously performed CPT tests, the building site is divided into different areas and for each area, use is made of the CPT test with the less favourable results in the considered area. During driving the first and all successive piles the same procedure as described is recommended.

6.2 Deduction of bearing capacity from driving measurements

The deduction of the static bearing capacity from dynamic measurements is controversial because of two principles :
- the dynamic loading behaviour is not necessarily representative of the static one,
- the displacement induced during driving is much smaller than what is recognized to yield significant data about the bearing capacity of a pile.

In spite of these arguments, some deductions are made using available methods based on the wave equation. The first type is the application of the "Case" method formula. Studies in Belgium of this method (Holeyman 1984) tend to show that the result depends strongly on the shape of the impacting force diagram (role of helmet) and on the level of energy. It is found that the "Case" bearing capacity increases with the maximum temporary displacement. This confirms the conventional character of the results obtained with this method, on top of the empirical nature of the choice of the "Case Damping" constant.

More universal is the dynamic analysis with an imposed boundary condition with the aim to match the other calculated boundary feature with the measured one. This method allows in theory to find one or more solutions which satisfy the observed data. In practice however, it is time consuming to find exactly the resistance distribution along the pile and approximate solutions are settled for when no improvement can be achieved. For this procedure, it appears that the significant parameters that can be found with a reasonnable degree of reliability are the ultimate skin friction and the loading curve at the base, up to the mobilized load (Holeyman, 1984). The ultimate failure load, if required, is then a matter of extrapolation as in the case of a loading test not carried-out to failure. This kind of analysis, conducted in the case of dense sand at the base and compared to load tests has shown that compared to the static case, the dynamic base behaviour is similar whereas the ultimate skin friction is reduced.

7. Recent Researches

7.1. Steel H-piles in dense sand and stiff fissured clay

Two test programs on steel H-piles with different kinds of laggings and enlargements have been performed at Kallo into a very dense sand and at Kontich into a stiff fissured clay, (De Beer et al., 1981; De Beer et al., 1982).

The piles were hot rolled wide flange beams from the American W series (W 14 x 16 x 142). They were driven by an open end Diesel hammer D36 developing a theoretical maximum energy of 115 kNm per blow and having a drop weight of 36 kN. The pile cap made of welded steel had a high density polyethylene fill and weighed 3 kN.

From the measurements with the saximeter it could be deduced that the ratio ε between the actual energy E_a developed by the Diesel hammer and the theoretical energy E_{th} given by the manufacturer varied between 0.36 and 0.75 at Kallo and between 0.40 and 0.90 at Kontich : the lower values correspond to easy driving when the pile started to penetrate into the soil, while the highest values correspond to the end of driving when high resistances were encountered. The difference between the highest values of ε at Kallo and at Kontich depended essentially on the maintenance of the hammer.

The ratio between the actual Energy E_a and the energy transferred to the pile E_p given by the "Pile Driving Analyzer" varied from blow to blow between 0.30 and 0.40 which seem to be usual values for this kind of cap and cap-fill.

The obtained results indicate how dangerous it is to make use of the theoretical values of the energy E_{th} in the dynamic formulae for the determination of the bearing capacity of piles.

As the actual energy $E_a = \varepsilon \cdot E_{th}$ and the energy transferred to the pile E_p have been measured in a direct way, the well-known Hiley driving formula can be used to calculate the dynamic resistance of the pile. This analysis leads to the following conclusions for the two sites :

a) dense sand at Kallo test site.

When the energy really transferred to the pile E_p is introduced in the Hiley formula, values obtained are somewhat lower than from the "static resistance" given by the Pile Driving Analyzer with the Case Method using a damping factor $J_c = 0.1$.

On the contrary, when using the theoretical and even the actual energy, the Hiley formula gives too high resistances. Furthermore, when introducing the theoretical energy E_{th} in the formula of Hiley, the ratio between the predicted resistance and the "static resistance" by the Case Method is not a constant, making it very difficult to define an adequate factor of safety.

From static load tests it could be deduced that the values obtained by the Case Method and by the Hiley formula based on the transferred energy, are very close to the values of the limit load deduced from the static loading test results by the criterion of Davisson.

b) stiff clay at Kontich test site.

Introducing a damping factor $J_c = 0.5$, which seemed appropriate because of the high content (55 %) of clay particles (<2μm), a static resistance was obtained with the Case Method which was very small with regard to the dynamic resistance calculated with the formula of Hiley.

A CAPWAP analysis on a blow recorded during normal driving indicated that for short delay static capacity predictions, a damping factor of 0.17 should have been adopted.

By redriving some piles it could be observed that the damping factor drops from 0.5 to 0.17 during driving, indicating that no valuable static capacity prediction can be made with dynamic methods for piles driven in clay by analyzing blows recorded during straight driving.

In order to predict correctly the bearing capacity of piles in stiff fissured clay, the Case Method has to be applied to redriven piles.

7.2. Comparative tests on bored and driven piles at Kallo.

Following a proposal of the National Committee on Pile Foundations an extensive research program was performed by the Ministry of Public Works at Kallo. Static loading tests were performed on two driven and two bored piles with a diameter of 600 mm, installed at a depth of ca 5 m in a dense sand layer, and at 11 m underneath the original soil level (De Beer, 1984).

The driven piles consisted of heavy steel tubes ∅ 600 mm, e = 40 mm, closed at their end by a heavy steel plate with renforcements. The piles were driven with a Hera 5700 Diesel-hammer. During the installation of the piles the number of blows necessary for each penetration of 10 cm was registered. Furthermore the elastic rebound of the pile was registered continuously and acceleratins, velocities and strains were continuously recorded at different levels within the pile.

The obtained blow-count diagrams are given on fig. 4. In this figure the result of the CPT test performed at the spot of pile A is also given. The result of the CPT-test performed at the spot of pile B is given on figure 1. During the installation of pile A the driving was interrupted at a depth of 9,68 m for 12 min 30 sec. and at a depth of 10,52 m for 4 hours. During the installation of pile B the driving was interrupted at a depth of 11,02 m for 4 hours.

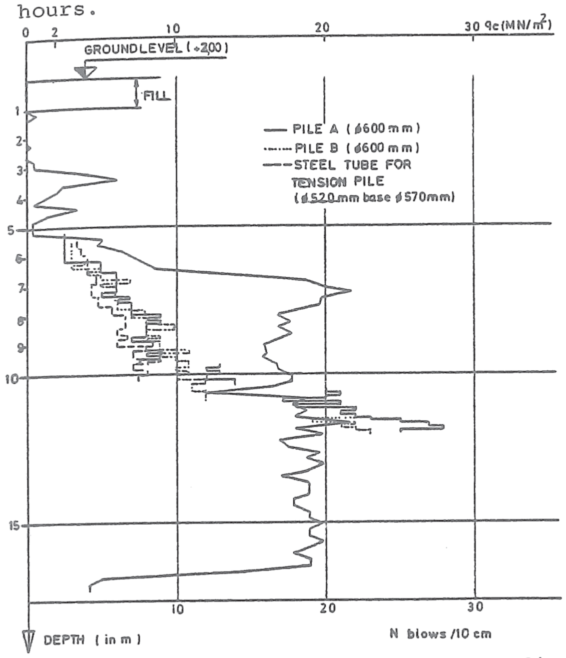

Fig. 4 : CPT test and blow count diagrams at Kallo.

For the execution of the static loading tests Franki piles were installed at a distance of 5,80 m from the test piles. Therefore a steel tube with an outer diameter of 520 mm and closed at the bottom with a plate with a diameter of 570 mm and a thickness of 30 mm was driven until a depth of 10,30 m. The blowcount diagrams registered for the tension pile is also given on fig. 4.

8. Environmental problems

8.1 Noise and vibration.

In Belgium, no specific standard exists in order to limit the noise and vibration levels caused notably by pile or sheet pile driving. Nevertheless, the communal authorities, which by law are responsible for preserving order, use their prerogative in view that in the residential areas the inhabitants are not excessively disturbed by noise and vibrations. It is observed that inside or around inhabited areas, driving is progressively disappearing to the advantage of other techniques.

When pile foundations must be installed in the vicinity of technical equipments sensitive to vibrations, the consulting engineer or the owner usually specifies particular appropriate solutions. For example, for the construction of a relatively large and piled building located in the immediate vicinity of a computerized signal box, the Belgian National Railways had specified the critical vibration levels, the use of two types of piles, cheaper driven piles and more expensive special bored piles, and the continuous measurement during piling of the vibration level inside the box. The installation began with the more remote driven piles so that the boundary from which special bored piles were used has been determined experimentally with the job progressing.

8.2 Heave of clayey or plastic silty soils

The Belgian National Railways is extremely sensitive to the phenomenon of ground heave in the neighbourhood of pile foundations next to the track. One has cast a formula by means of certain hypotheses giving the vertical displacement of a point with polar coordinates (r, θ) caused by an isolated pile with radius R driven into clayey or plastic silty soils (see fig. 5)

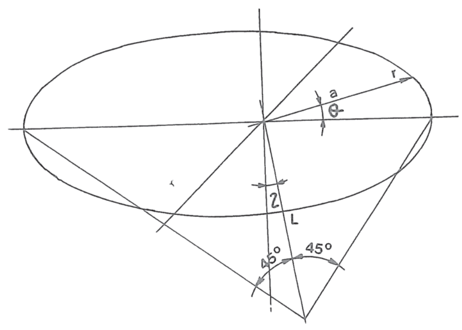

Fig. 5 Soil displacement mechanism.

The assumptions are as follows :
- the soil is considered incompressible under the influence of dynamic actions
- the volume of soil displaced in the dihedral angle $d\theta$ by the pile at each blow is evenly distributed at the surface onto an area defined by the intersection of the initial surface and a cone whose generating lines form an angle of 45° respective to the axis of the pile.
- the zone of influence to which a given point has to belong in order to heave can be assessed on the sole basis of the rake angle ι and the driven length L.

The zone of influence resulting from these hypotheses is described by :

$$r \leqslant a = \frac{\sqrt{2}.L}{2\cos(\pi/4-\beta)} \qquad (17)$$

with $\beta = \cos^{-1}(\sin\iota \,.\, \cos\theta)$
With $\alpha = a/R$ and $\lambda = L/R$, the soil heave Δh is given by :

$$\Delta h = \frac{R\cos\iota(1+\sin 2\beta)(\lambda - \sqrt{2}.\alpha\cos(\pi/4-\beta))}{\sin 2\beta\ \sqrt{2}.\alpha\ \lambda\cos(\pi/4-\beta)} \qquad (18)$$

In the case of a vertical pile, the soil heave is reduced to :

$$\Delta h = R.\frac{\lambda-\alpha}{\alpha.\lambda} \qquad (19)$$

Table I : Δh measured [cm]/Δh calculated [cm].
Vertical profile $\iota° = 0°$ R = 0.20 m

L / a [m]	2.4 m	6.0 m	10.1 m
1.0	2.35/2.38	3.20/3.32	3.55/3.66
2.0	0.30/0.34	0.80/1.34	1.45/1.61

During jacking of vertical rectangular profiles in the tertiary Boom clay at Kontich, (De Beer et al, 1977), the soil heave has been measured around the profile for various values of the penetration depth L.
These values are compared in table I with the ones resulting from the formula (19).

In the case of a pile group, the effects due to each pile are added. For a bridge at Quevy, piles had to be driven in the vicinity of an existing track (see piles 1 to 8 in fig. 6). Heave was measured at benchmarks located on the track and labelled A to D in fig. 6. The top 7 m of the soil profile consists in clayey silt and clay layers. The adjacent piles with an equivalent diameter R = 0.37 m are raking at $\iota = 30°$ towards the track.

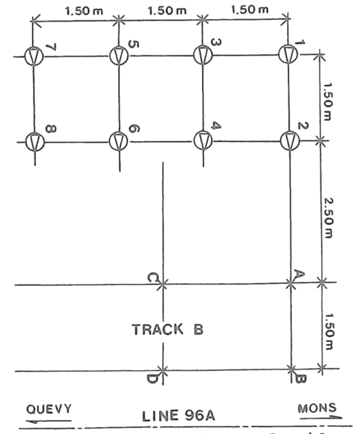

Fig. 6 Plan view of piles and benchmarks.

Table II compares the measured heave with the calculated one, for 3 phases of the driving operations.
In this case of a foundation with raking piles, one observes some tendency to overestimate the soil heave at short distance from the pile(s) and to underestimate it at large distances. The agreement between measured and predicted heave is rather satisfactory, considering the simplicity of the hypotheses.

Table II : Δh measured [cm]/Δh calculated [cm].
Raking piles at Quevy $\iota = 30°$ R = 0.37 m

Driven piles	Benchmarks A	B	C	D
1	0,50/0,91	-	-	-
1 & 2	3,00/4,32	1,70/0,91	-	-
1 to 8	-	-	>12,0/11,74	>8,0/2,56

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