CHAPTER 1

Load Testing Procedures: Description and Methods of Interpretation

1. Introduction

Design analysis of pile is required to assess whether the foundation is resistant enough to fulfil its function. This function could be either to carry a particular load, or to settle less than an allowable maximum deformation. The analysis involves generally the calculation of bearing capacity (sometimes split into shaft and base bearing capacities) and the estimation of the possible deformations under working loads. Both points are summarized in the drawing of the load-settlement curve.

Design analysis could also be performed in order to assess the installation feasibility in terms of driving, machine and field aspects. The design methods can be divided in two main groups: the methods based on the fundamental soil properties and the empirical approaches. Most of the pile design methods use a mix of these two main branches. However, the calibration of all these methods remains dependent of the pile type, the pile installation, the pile material, the type of soil and geotechnical conditions.

In order to assess the adequacy of the chosen design method with field conditions, some pile load tests are carried out on specific and representative piles. The first and main preoccupation during these tests is to match as closely as possible the real load history applicable during the foundation life. But this progressive and slow gradation to reach the maximum load level is not convenient with respect to the economical and practical aspects. Several testing methods have been designed to respond to both antagonist requirements: a pile test as close as possible to the actual situation to confirm the design but that can be performed as quickly as possible in order to be efficient and useful.

Two of them, the Static Load Test (SLT) and the Dynamic Load Test (DLT) are involved in this research through the results of two majors test campaigns organised in Belgium during the last 4 years. Since the results of these pile load tests are the reference data of the current research, an explanation of their procedures is required like the main techniques used for their analysis. In this chapter, each proposed testing method is described with a particular attention on the methodology, the measured quantities, the general and main theoretical assessments allowing these tests and the advantages and disadvantages of each.

2. Static Loading Tests

Static pile load testing is the most commonly adopted method of checking the performance of a pile and is often used as reference for the calibration of others pile testing methods.

2.1. Definitions and characteristics

Amongst the procedures developed in this testing category, the maintained load test is commonly used in the general practice. It is characterized by a set of stages where a constant load is applied on the pile head during a given duration. The load is monotonically increased (ΔQ) until reaching a determined load Q_{max} [1] (see Figure I-1). When the maximum test load is reached, the applied load is progressively unloaded to zero with potential different time duration and decrement loads.

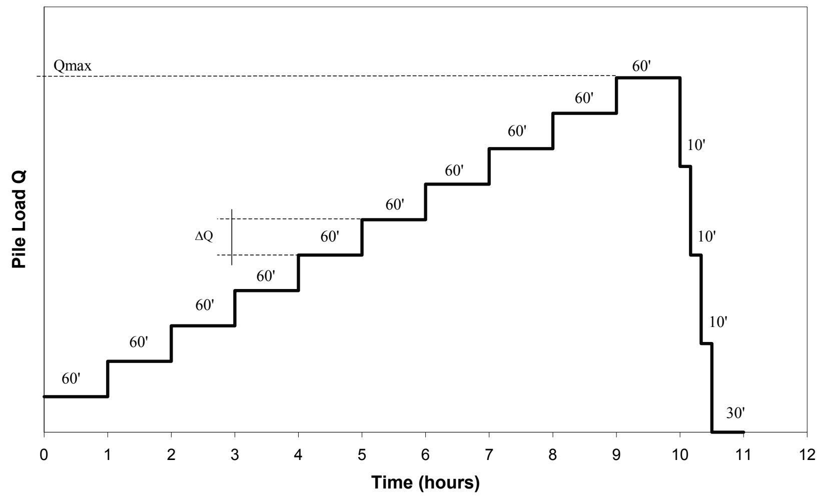


Figure I-1 : Test procedure adopted for the static pile load tests (after Maertens et al., 2001)

If the applied load reaches a value close to the estimated ultimate pile capacity, the load increment can be reduced to avoid a quick failure.

The load is applied by hydraulic jack(s) and kentledge[2], adjacent tension piles or soil anchors are used to provide the reaction for the load applied on the pile (Figure I-2 and Figure I-3).

[1] Either a multiple of the design working load or the predicted ultimate resistance, or another limit imposed by the test schedule

[2] A mass of heavy material placed on a platform constructed on the head of the pile.

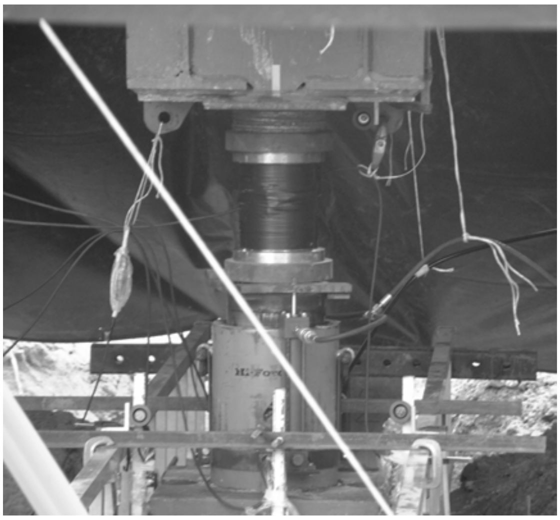


Figure I-2 : Detail of the loading device

Figure I-3 : Steel kentledge

The maintained load test is ruled by official specifications and procedures: the ASTM Test Designation D-1143, the ICP Piling Model Procedure and Specifications (1988 & 1996) or the BS-8004 1986. They give the specification required for the maintained load test including the loading increment schedule and periods of load application.

The ERTC3 (European Regional Technical Committee-Piles, ISSMGE Subcommittee) proposes a document (De Cock et al., 2003) gathering the recommendations for the execution of Axial Static Pile Load Tests (ASPLT). This document contains advice about the preparation, execution, reporting and evaluation of the ASPLT for compression and tension piles. In particular, the paper describes three quality levels in relation with the purpose of the pile load test (from high requirement level (research or design test) to basic requirement (control or acceptance test)). For each category, a summary of the relevant test specifications is given and detailed.

2.2. Measurements

The basic measurements include the pile head displacement and the applied load in function of time. These quantities allow the determination of the pile's performance.

The displacement devices (dial gauges) are fixed on a reference frame and the pile head displacement is generally the average of several distinct values. The measurement can be reinforced and checked with an optical measurement apparatus. The load measurement can be obtained from a load cell or from the pressure of the jack system (De Cock et al. 2003).

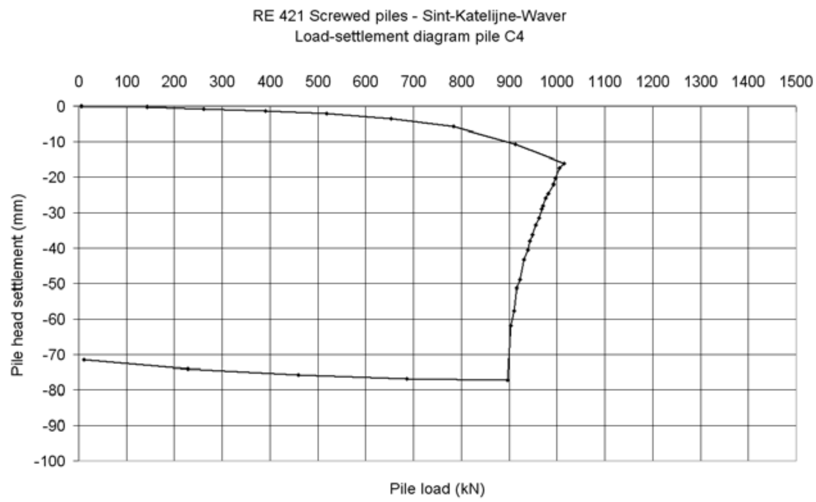


Figure I-4 : Load-settlement curve of a static pile load test (after Huybrechts, 2000)

Figure I-4 shows the pile head displacement measured and averaged vs. the pile head load measurement. This representation is the most used and called the pile load settlement curve.

2.3. Interpretations of the results

According to the amount of measured data and according to the main purposes of the test, the interpretation of the results could be simple, sophisticated, direct or involving considerable post-treatment. All of them concern the analysis of the measurements: displacement and load. Measurement can be analysed separately (vs. time with linear or logarithmic scales allowing the estimation of the creep Resistance Q_c which is deduced by analytical or graphical means as the point of maximum curvature on the creep curve (Figure I-5)) or together (Figure I-4). If the pile is instrumented, (Q_b,s_o) and (Q_s,s_o) plot respectively the base and shaft capacities in function of the pile head displacement.

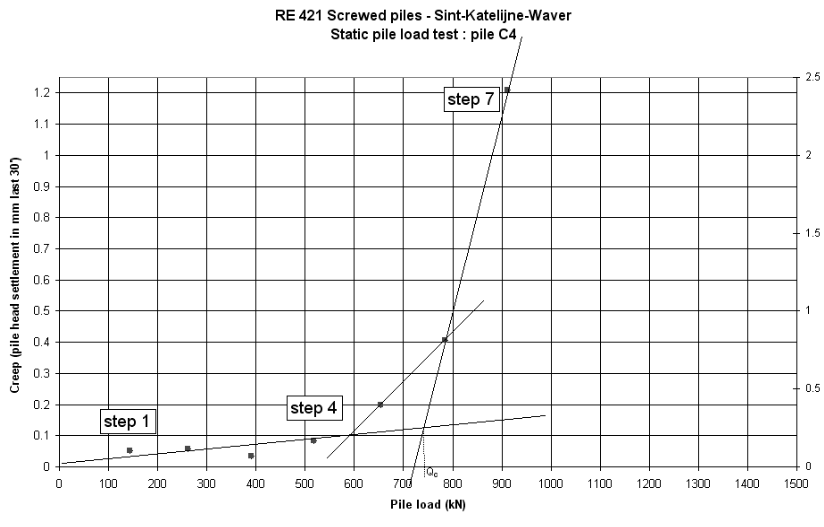


Figure I-5 : Creep curve (load vs. Δso, last 30 min) (after Huybrechts, 2000)

The interpretations of all these curves and figures give a better understanding of the pile/soil interactions and load transfers during the test.

An important operation still remains after drawing and analysing the rough figures: the estimation of the ultimate load, required for the estimation of the working and design loads. There are several approaches, all of them being based on the load settlement diagram:

- The *Chin method* that assumes a hyperbolic relation between the pile head settlement and the mobilized soil resistance expressed by the applied load.

- The *Van der Veen method* that proposes an exponential relation between the base settlement and resistance.

- The *Brinch-Hansen's 80% criteria* that defines the ultimate load as the load for which the pile head settlement is four times the settlement corresponding to 80% of this load. In this configuration, the load settlement diagram becomes a parabola.

Three commonly and simple criteria can be used without an explicit reference to the ultimate resistance but preferably called "conventional rupture load" :

- The load corresponding to a progression of the settlement with no variation of load;
- The Davisson criteria that determines the load as the intersection of the load settlement curve with the straight line of slope $\frac{L}{EA}$ and starting a the settlement s_0 (L: pile length, E: Elastic modulus of the pile material, A: pile section):

$s_o = 3.81\ 10^{-3} + \frac{Diam_{Pile}}{120}$ [SI units];

$$s(Q_{rupt}) = 3.81 10^{-3} + \frac{\phi}{120} + \frac{Q \cdot L}{EA} \qquad \textbf{Equ. I-1}$$

- The load corresponding to a settlement of 10% of the pile base diameter ($Q_{10\%Db}$).

These last conditions consider that the base capacity is fully mobilized for this settlement (the shaft settlement is obviously mobilized for a long time). It happens probably in clay, but not with certainty in sands.

3. The Dynamic Loading Test

3.1. General presentation

Dynamic load testing consists in predicting pile capacity and pile/soil interaction from hammer blows during driving and/or restrike of an instrumented pile. This testing method has been developed owing to the driving analysis improvement emerging during the last 50 years. The computer use and its more and more efficient working capacity, plus the transducers progresses were also pertinent parameters helping the driving analysis development.

The dynamic load testing consists of letting a mass (e.g. 4 tons) fall from a set height upon the pile head (Figure I-6). Transducers (strain gauges and accelerometers) are placed along pile shaft next to the pile head (Figure I-9). A pile/soil model is then used to interpret the dynamic event in terms of an equivalent static load-settlement curve.

The main advantages of this load test are obviously the economical and accurate way to evaluate the hammer-pile-soil system based on force and velocity measurements and to check the integrity and performance of the foundation.

Figure I-6 : Dynamic loading test (crane + mass + pile)

The pile dynamic testing procedure is also accepted and standardized in numerous specifications such as ASTM D4945-89, AASHTO T298, the recommendations for static and dynamic pile tests of the DGGT in Germany (1997) or specifications from many countries (Bein et al., 2000).

The models used in the dynamic pile testing procedure take into account further elements assuming to be representative of the static part, the viscous part, the radiation term of soil reaction and the hysteretic loss of energy through the soil. It is possible to estimate the ultimate resistance of the soil under static conditions but often the soil mobilization during a test does not allow one to have sufficient information to fulfil this requirement. Nevertheless, a good estimation can be done and a conservative value can be given and interpreted as a lower bound. A considerable experience is however required especially to quantify the damping factor often used to take all the mentioned terms (except the static term) into account.

3.2. Definitions and characteristics of the Dynamic Load Test

Different purposes guide the use of dynamic testing, some of them concern the pile driving procedure, others insists on the integrity and the quality of the pile installation and finally, the main goal is guided by the bearing capacity evaluation.

Nowadays, one of the main purposes to use dynamic loading test is the pile bearing capacity evaluation. This is due to the improvements in the pile/soil interaction modelling, due to the computer progress allowing a shorter process and a higher complexity. This is also due to the experience and the numerous comparisons available between static tests and dynamic tests on the same soil and pile configuration. Therefore, the timing of the test becomes a very important parameter, absolutely essential to be integrated. Increase (set-up) or decrease (relaxation) of the pile capacity with time occurs almost always after driving depending of the type of soil and driving. Hence, dynamic testing gives the capacity of the pile at the time of testing. It seems to be appropriate to wait for a few hours (sand) to several weeks (clay) to try and evaluate the long-term capacity through a dynamic test. Using the short-term capacity could result in over- or under-estimate the long term one and it can have dangerous consequences.

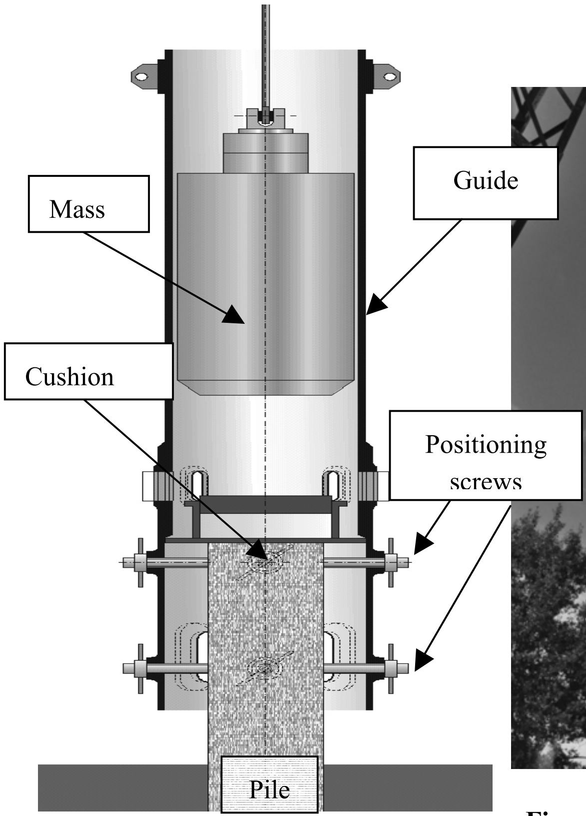
It has to be noted that the dynamic testing capacity estimates the mobilized pile capacity at the time of testing and for the dynamic test soil mobilization because the base resistance is rarely fully activated during a single hammer blow. The more energetic the blow (or higher the ram drop), the closer the bearing capacity estimation to the ultimate resistance.

3.2.1 Test procedure

The basic testing procedure is defined as the drop a of mass on the head of the tested pile where transducers are attached in order to measure the transient signals generated by the impact. The system is composed of a the falling mass, a guide for the correct alignment of the mass and the pile head cushion, a device controlling the energy transfer between the mass and the pile (Figure I-7). A rule of thumb assesses that the ram weight should be 1.5% of the estimated capacity of the tested pile (Goble & Likins, 1996). The ram used for the tests performed at Sint-Katelijne-Waver and Limelette is 4 tons. The scheme of the mass and guide is shown on Figure I-7 and Figure I-8.

The cushion is needed to control the energy coming from the mass to the pile in order to mechanically limit the intensity of the peak stress and lengthen its duration. The material has to be elastic (the cushion can be modelled by a spring and a dashpot in parallel) but should consume as little energy as possible. The goal is to transform the input force signal in a longer and smoother form.

The positioning screwes are used to place the guide of the mass as vertically as possible in the pile axis, so that the pile can be loaded with a purely axially load and not decentralized due to the possible bending and tension stress generated in the section. This device allows also maintaining the guide on the pile when the crane lifts the mass before dropping it.



<u>Figure I-7</u> : Schematic view of the dropping mass and mass guide (after Holeyman, 1992)

<u>Figure I-8</u> : Dropping mass and mass guide

The guide is a metallic tube allowing the mass to drop from a height of about 2 meters vertically on the head pile even if the mass is not exactly upright with the pile.

Before the test, and in function of the goal of the testing session, the transducers are placed on the head pile. Some recommendations express a minimum distance from the pile head and from any change in the pile section dimensions. The minimum distance is generally fixed to 1.5 pile diameter (Goble & Likins, 1996). This distance aims to avoid measurement of local stresses surrounding discontinuities such as corners, angles or section variations.

Two conditions are needed to evaluate correctly the long term capacity:

- The test must be performed on a restrike and during the early blows of the restrike;
- The set per blow must be large enough to ensure significant soil failure during the test.

3.2.2 Data acquisition

The transducers are generally twofold: accelerometers to measure the pile head vertical acceleration and strain gauges to measure the vertical strain in the local section due to the force transmitted by the falling mass.

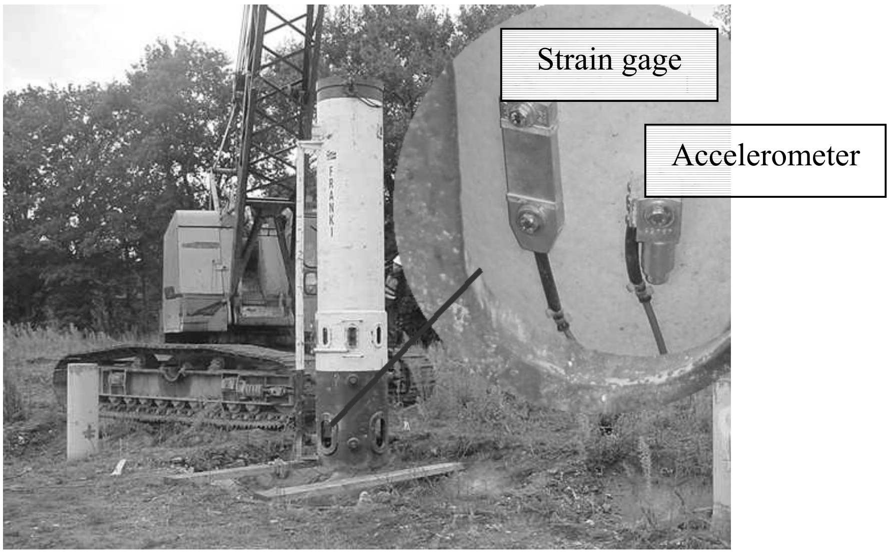


Figure I-9 : Detail of transducers placed on the pile shaft

Figure I-9 blown-up exhibits the detail of the transducers placed on the pile shaft during a dynamic loading test. The figure shows only two sensors, but in reality, the transducers are placed diametrically opposed on the pile section in order to mitigate the influence of bending on the acceleration signal and particularly on the strain measurements. The used data are the average value of both measurements.

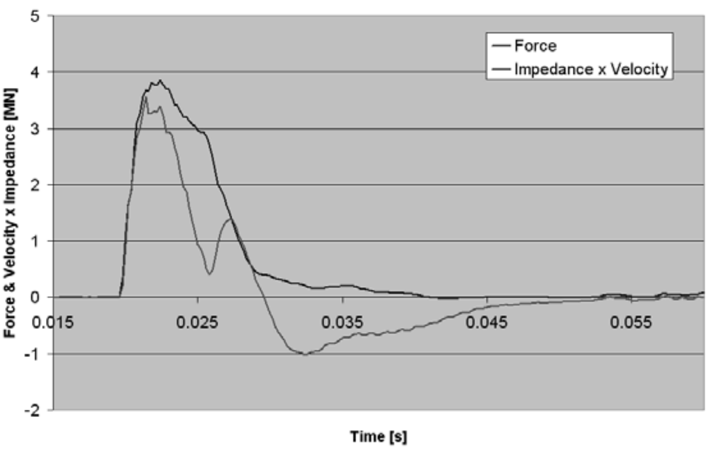


Figure I-10 : Proportionality of the velocity and force signals

Both measured signals can give others very important information after mathematical treatment:

Strain → Stress → Force in the section.

Acceleration → Velocity → pile head displacement.

A next section (chapter 3) will develop the potential errors emerging from this treatment by assessing the values of the elastic modulus E, the section, …

3.2.3 Data interpretation & data analysis techniques

Due to the viscous effect added to the static soil reaction and due to the wave equation phenomenon occurring during a dynamic test event, post treatment and assessments are needed to get the required results. Numerous methods of interpretation have been developed in order to overcome this difficulty. Here after is a short description of some of them.

3.2.3.1 Visual inspection

The visual inspection includes the first inspection of all signals in order to validate their quality. This is not really an interpretation method but an essential step in the interpretation process. The signals must correspond to what is expected: duration, noise, amplitude, final zero value for velocity and force. It is also possible to interpret these defaults or defects as a particular cause. The method remains highly subjective.

3.2.3.2 Driving formulae or energy approach

The main principle of the driving formulae links the called pile dynamic capacity to the measured permanent displacement of the pile (set) at each blow. These formulae are based on the energy balance of the hammer input energy (energy given to the pile) and the work spent by the pile to settle down permanently. The result represents a capacity between the static and dynamic ones since no differentiation is made between the static ultimate resistance of the pile and the dynamic driving resistance.

The mass falls from a determined height, equivalent to a potential energy, to the pile head. The potential energy turns into kinetic energy. This energy is transferred to the pile during impact. The pile translates immediately this kinetic energy into a deformation energy and a pile kinetic energy and turns it out in a penetration work of the pile into the soil.

The penetration work allowing the driving of the pile can be divided into plastic and elastic terms and is limited by the soil reaction mobilization. The energy balance between these elements is given by Holeyman, 1984:

$$\eta_i \eta_c M_m g h = Q_d \left(\frac{s_{el}}{2} + s\right)$$ **Equ. I-2**

Where:

- η_i : impact efficiency [-];
- η_c : falling efficiency [-];
- M_m : ram mass [kg];
- g : gravity acceleration [9,81 m/s²];
- h : dropping height [m];
- Q_d : dynamic soil reaction [N];
- s_{el} : elastic deformation or “quake” [m];
- s : permanent deformation or “set” [m].

The left part of Equ. I-2 corresponds to the transmitted energy (potential energy and efficiency effects) and the right part represents the penetration work based on the modelled displacement/resistance model (Figure I-11).

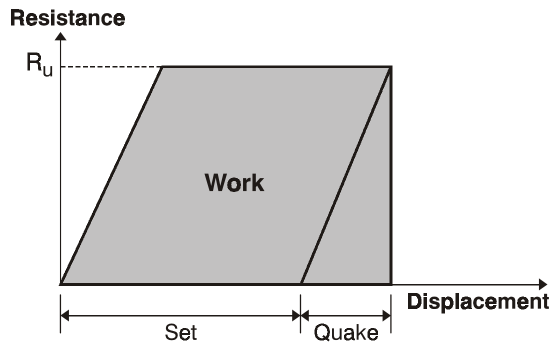


Figure I-11 : Displacement-soil reaction relationship

From Equ. I-2, the static pile bearing capacity is evaluated:

$$Q_{adm} = \frac{Q_s}{SF} \cong \frac{Q_d}{SF}$$ **Equ. I-3**

Where "SF" is a safety factor imposed to the driving formula result. During the driving, the estimated pile capacity can be checked out. If this capacity is assessed, the blow count can be investigated by extracting the permanent set of the driving formulae.

Paikowski and Chernauskas (1992) merge the energy approach with the use of the dynamic signals to evaluate an estimation of the pile resistance. The main principle link the energy transferred to the pile (Enthru) to the work done by the resisting forces along the pile.

The energy transferred to the pile is calculated from the measured dynamic signals instead of the potential energy and the efficiency effects.

$$Enthru = \int_0^{t_\infty} V(t) \cdot F(t) \cdot dt$$ **Equ. I-4**

Where:

- V(t) is the pile particle velocity [m/s];
- F(t) is the force measured close to the pile head [N].

This equation consider only the elasto-plastic energy losses of the pile-soil system and can be regarded as a upper bound of the pile resistance.

3.2.3.3 Signal processing

The signal processing methods work with the measured signals (strain or force, and acceleration or velocity at pile head) by simple operations in order to result in the evaluation of the shaft, base and total resistances. They are all based on the application of the stress-wave theory. The main assessment states that the signal generated by the mass hitting the pile head goes down into the pile towards the pile base. This compression wave travels down the shaft, reaches the base and is reflected back in the direction of the pile head. The time needed for the travelling back and forth is function of the pile length and the wave propagation velocity.

$$T_{\updownarrow} = \frac{2L}{c}$$ **Equ. I-5**

Where:

- T is the time it takes the wave to reach the pile and come back [s];
- L is the pile length (from the transducers position to the pile base) [m];
- c is the wave propagation velocity [m/s].

The form of the reflected wave depends on the base reaction encountered. If the base is free from reaction, the pile is free to deform itself and the force wave that returns back is a tension wave. When the base reaction is infinitely rigid, the force wave is entirely reflected and the upward wave is a compression one identical to the downward wave. The same process generates also particles velocity of opposed sign.

When the pile is considered with the surrounding soil, each soil layer reacts like the base. The incident wave is divided in an upward compression wave (proportional to the soil restraint) and the complementary downward part of the initial wave. Since the main and initial wave is going through the pile to the base, its reflection is expected to arrive at the pile head after a time equal to 2L/c. During this time, all the upward information is only resulting from the shaft soil reaction. Consequently, the shaft reaction can be evaluated by taking the upward wave into account during the instants following the blow; and since all the waves travel at the same propagation velocity (for a homogeneous pile), the arrival time can be used as a indicator of the pile depth.

It seems possible to evaluate the shaft soil reaction during a dynamic load test through the analysis of the upward and downward waves. Moreover, these waves can be extracted from the data measured (force and velocity) at the pile head. The upwards velocity and force waves are function of the pile/soil system, as developed here after.

The Wave Equation

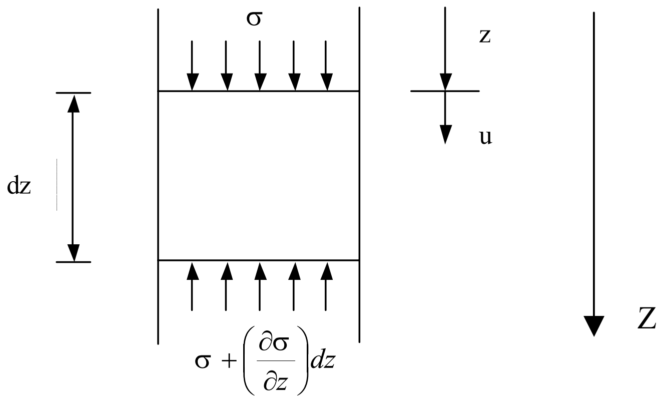
The wave equation theory applied on the pile dynamic test analysis idealizes the pile as a prismatic elastic rod where a wave propagates with a constant velocity depending on the material properties. This wave expresses the travel of a longitudinal strain initiated by a shock on one of its extremities. The main principle expresses that all material does not feel the shock effect immediately but gradually as the wave advances in the rod, like a wave in a cable. The wave is a compressive one if the shock applies a compression on the rod. The motion of each pile particle depends on its position in the rod and regarding the wave position and the time. The wave is considered as one-dimensional because of the large dimension of the length of the pile compared to the diameter. The general referential is Z pointed downwards as shown on figure I-12. The pile is considered as elastic, homogeneous and of constant section A.

Wave Equation Form

Figure I-12 exhibits the section A, positioned at the distance z. Its displacement u_z due to the compression is defined by

$$u_z = u_z(z,t)$$ **Equ. I-6**

In function of time and of the main dimension z.



<u>Figure I-12</u> : Infinitesimal volume of a pile section

Due to the balance of forces applying on an infinitesimal volume plus the inertial effects (<u>Figure I-12</u>) and due to the joint application of Hooke's and Newton's laws, the following equation is obtained:

$$A_p\sigma - A_p\left[\sigma + \left(\frac{\partial\sigma}{\partial z}\right)dz\right] = \rho_p A_p dz \frac{\partial^2 u}{\partial t^2}$$ **Equ. I-7**

$$\left(\frac{\partial u}{\partial z} + \left(\frac{\partial^2 u}{\partial z^2}\right)dz\right)A_p E_p - AE_p\frac{\partial u}{\partial z} = \rho_p A_p dz \frac{\partial^2 u}{\partial t^2}$$

Where :

- A_p is the pile section [m²];
- ρ_p is the volumetric mass of the pile material [kg/m³];
- σ is the axial stress [MPa] and the compression is positive;
- E_p is the elastic modulus of the pile material [MPa];
- dz is an infinitesimal thickness of pile [m];
- $\frac{\partial^2 u}{\partial t^2}$ is the acceleration of the considered section [m/s²].

Simplification of Equ. I-7 gives:

$$\frac{\partial^2 u}{\partial z^2} = \frac{\rho_p}{E_p}\frac{\partial^2 u}{\partial t^2}$$ **Equ. I-8**

And by defining the wave propagation velocity c as :

$$c = \sqrt{\frac{E_p}{\rho_p}} \text{ [m/s]}$$ **Equ. I-9**

Equ. I-8 becomes the fundamental wave equation of the propagation of an elastic compression wave in an elastic media:

$$\frac{\partial^2 u}{\partial z^2} = \frac{1}{c^2}\frac{\partial^2 u}{\partial t^2}$$ **Equ. I-10**

The solution of Equ. I-10 is obtained after substitution. Two fundamental forms compose it:

$$f(z,t) = f_1\ (z-ct) + f_2\ (z+ct)$$ **Equ. I-11**

Where $f_1\ (z-ct)$ and $f_2\ (z+ct)$ are progressive waves propagating in the rod (the pile) without modification with the constant velocity +c (downward wave ↓) and –c (upward wave ↑) , respectively.

The velocity is determined by the material mechanical properties, and the form of the waves (f_1 and f_2) are determined by the initial and boundary conditions.

The functions f_1 and f_2 can be used to describe any behaviour propagating in the pile respecting the elastic propagation assumptions (i.e. force & pile particle velocity). Superposition of both forms respects the general wave equation (Equ. I-10).

It is possible to express the forms of the force waves and the pile particle velocity waves in function of the pile particle displacement:

$$F(z,t) = F^{\downarrow}(z,t) + F^{\uparrow}(z,t) = -E_p A_p \frac{\partial u}{\partial z}$$ **Equ. I-12**

$$\rightarrow\ F(z,t) = -E_p A_p \frac{\partial u}{\partial z} = -E_p A_p \left(\frac{\partial u^{\downarrow}}{\partial (z-ct)} + \frac{\partial u^{\uparrow}}{\partial (z+ct)} \right)$$

$$v(z,t) = v^{\downarrow}(z,t) + v^{\uparrow}(z,t) = \frac{\partial u}{\partial t}$$ **Equ. I-13**

$$\rightarrow\ v(z,t) = \frac{\partial u}{\partial t} = -c\frac{\partial u^{\downarrow}}{\partial (z-ct)} + c\frac{\partial u^{\uparrow}}{\partial (z+ct)}$$

If the geometric conditions impose to work only with one of the two forms, either the downwards waves (semi infinite pile with $z \geq 0$) or the upwards waves (semi infinite pile with $z \leq 0$), Equ. I-12 and Equ. I-13 allow to write:

$$F^{\downarrow}(z,t) = -\frac{E_p A_p}{c} v^{\downarrow}(z,t) \text{ or } F^{\downarrow}(z,t) = I \cdot v^{\downarrow}(z,t)$$ **Equ. I-14**

$$F^{\uparrow}(z,t) = -\frac{E_p A_p}{c} v^{\uparrow}(z,t) \text{ or } F^{\uparrow}(z,t) = -I \cdot v^{\uparrow}(z,t)$$ **Equ. I-15**

Where:

$$I = \frac{E_p A_p}{c} = A_p \sqrt{E_p \rho_p} \text{ [N/ms}^{-1}\text{]}$$ **Equ. I-16**

I is defined as the nominal impedance of the pile, or the damping factor of the dashpot modelling the behaviour of the semi-infinite pile subjected to a force. The force transmitted through a pile section and the pile particle velocity are proportional, if the pile is supposed free of any other forces of solicitations.

Equ. I-14 and Equ. I-15 express that the force and the particle velocity have the same sign when the force is a compression, and have opposite signs when the force is in traction.

It is possible to express $F^{\uparrow}(z,t)$ and $F^{\downarrow}(z,t)$ as functions of F(z,t) and v(z,t):

$$F^{\downarrow}(z,t) = \frac{1}{2}\left(F(z,t) + I \cdot v(z,t)\right)$$ **Equ. I-17**

$$F^{\uparrow}(z,t) = \frac{1}{2}\left(F(z,t) - I \cdot v(z,t)\right)$$ **Equ. I-18**

If initial conditions are imposed to the extremity of the semi-infinite pile (initial force F_0 and initial velocity v_0), each quantity must respect the following statements:

$$v(z,t) = v_0 + v^{\uparrow} + v^{\downarrow}$$ **Equ. I-19**

$$F(z,t) = F_0 + F^{\uparrow} + F^{\downarrow}$$ **Equ. I-20**

Hypothesis of the method of interpretation

As already explained, the force and velocity are proportional to each other by a term known as the pile impedance, but this relationship is only true when there are no soil reactions along and beneath the pile. If soil is present and acts against the pile, corresponding upward and downward force and velocity waves are influenced. The difference of behaviours is not a net measurement of the soil reaction but a consequence of it. It is the result of all the interactions between generated waves along the pile.

By analysing the formulae of $F^{\uparrow}$ and $F^{\downarrow}$ (see Equ. I-17 and Equ. I-18), in case of no soil interaction and in the real situation, the difference represents the soil reaction.

The distribution of the resistance along the pile shaft can be interpreted from the time development of the upward reflected wave up to 2L/c. After this time, all the waves received and considered as upward waves are indeed a mixing resulting of the interaction of several waves. These waves are generally difficult to interpret because they integrate the effects of several depth-and time-dependent variables (Holeyman, 1992).

The shaft friction can be determined if a constant resistance is assumed to be activated during a certain time. Fortunately, the shaft friction is quickly mobilized and even a dynamic load test can fully activate the shaft resistance along the pile. So the rigid-plastic behaviour can be assumed for the modelling of this soil reaction part.

The same assessment cannot be done for the base resistance evaluation because the assumption of reaching the ultimate level of soil resistance cannot be met during a dynamic load test. Consequently, the evaluation of the base resistance term from a dynamic load test using the signal processing analysis is dependent on the assumed hypothesis.

The CASE method (Goble et al., 1975) is based on the signal processing approach. Other methods try to differentiate the base and shaft resistances (Liang, 2003) but all of them assume that all terms use a rigid-plastic behaviour.

CASE method

The CASE method (Goble et al. 1975) is derived from the signal processing theory and results in a single pile capacity either "dynamic", namely taking all the loading rate effects into account, or static, with a the use of an empirical dimensionless factor J_c.

The CASE method uses several severe hypothesis to build up the model allowing the calculation of the total soil reaction from the measured dynamic and kinetic signals: the pile is homogenous, elastic and uniform; the permanent pile displacement is sufficient to fully mobilize the pile shaft and base resistances; the damping (J_c) is only considered to be representative on the pile base; the pile must be short enough in order to ensure that the wave travelling during the 2L/c does not perturb the mobilization of the shaft resistance layers.

The downward and upward travelling waves $F^{\downarrow}$ and $F^{\uparrow}$ are determined from the force and the velocity signals at the section of control marqued with x_1 in Figure I-13 (Equ. I-17 & Equ. I-18).

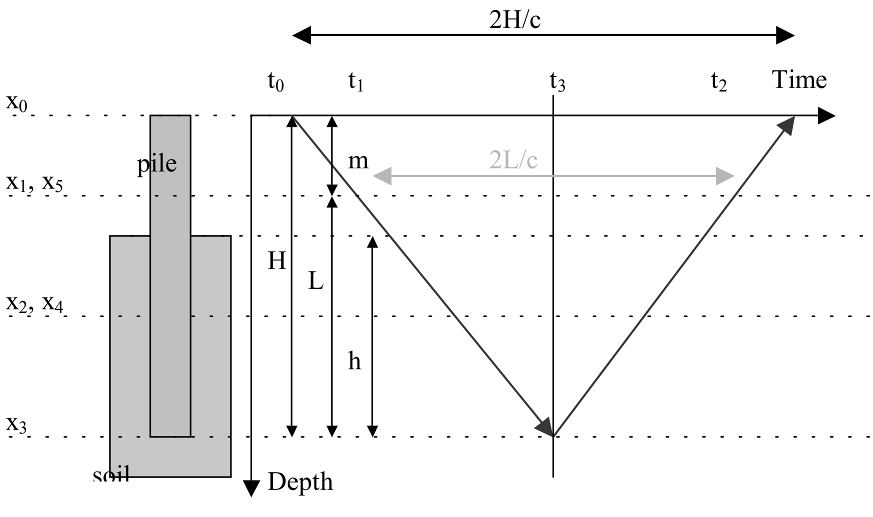


Figure I-13: Description of the propagation wave into the pile (after TNOWAVE manual, 2000)

The driving resistance is calculated from an input/output relation. A wave is introduced to the pile top (input), identified at a time $t_{max} = t_1$ (maximum F_{down}). After one travel, $t = t_{max} + 2L/c = t_5$, part of the wave (F_{up}) returns to the pile top (output). The modification of the wave during this duration is interpreted as the Driving Resistance:

$$R_{tot} = F^{\downarrow}(t_{\max}) + F^{\uparrow}(t_{\max} + \frac{2L}{c})$$ **Equ. I-21**

The total driving resistance R_{tot} is defined as the sum of the downward travelling force wave at a time t_{max} and the upward travelling force wave at a time t_{max} + 2L/c.

To determine the static term from the total estimated resistance, the CASE method assumes that the driving resistance is entirely at the toe and the dynamic driving resistance is assumed to be proportional to the pile toe velocity, calculated from the measured pile head force and velocity. The factor of proportionality is J_c called the "CASE damping constant", and determined by correlation with static load test or from signal matching.

$$F^{\downarrow}(t_{\max}) = F^{\downarrow}_{t1} \text{ and } F^{\uparrow}(t_{\max} + \frac{2L}{c}) = F^{\uparrow}_{t2}$$ **Equ. I-22**

$$R_{tot} = F^{\downarrow}_{t1} + F^{\uparrow}_{t2} = R_{stat} + R_{dyn}$$

$$R_{dyn} = J_c \cdot I \cdot v_{base}$$ **Equ. I-23**

Where I is the pile nominal impedance.

Since

$$v_{base} = \frac{F^{\downarrow}_{t1} - F^{\uparrow}_{t2}}{I}$$ **Equ. I-24**

$$R_{stat} = F^{\downarrow}_{t1}(1 - J_c) + F^{\uparrow}_{t2}(1 + J_c)$$ **Equ. I-25**

$$R_{stat} = \frac{(F(t_1) + I \cdot v(t_1))}{2} \cdot (1 - J_c) + \frac{(F(t_2) - I \cdot v(t_2))}{2}(1 + J_c)$$ **Equ. I-26**

Equ. I-21 can be transformed into:

$$\frac{\left(F(t_{\max}) + F\left(t_{\max} + \frac{2L}{c}\right)\right)}{2} + M \cdot \frac{\left(v(t_{\max}) - v(t_{\max} + \frac{2L}{c})\right)}{\frac{2L}{c}} = R_{tot}$$ **Equ. I-27**

This last result interpreted as the driving resistance is the sum of the average forces applied to the pile head at the times t and (t+2L/c) and the inertial effect of the pile during the period 2L/c considering the average deceleration. (Holeyman, 1984).

Some typical values for the CASE damping constant J_c are given in chapter 2.

The time t_1 can be chosen at any time, but, practically, it must be chosen in order to ensure that the corresponding second time (t_1 + 2L/c) does not correspond to a time where some reaction along the shaft have been inverted. This situation should underestimate the total pile resistance.

3.2.3.4 Numerical models

The numerical models approach the pile/soil interaction by a computer simulation of the event measured during the test. A model is needed to describe the different terms of the soil reaction mobilized: the shaft friction reaction, the base reaction, the viscous and radiation terms of the soil mobilization. The main goal of the technique is to adjust the parameters characterizing the pile/soil model chosen in order to fulfil a criterion based on the measured data issued from the dynamic test. When the criterion is achieved, the model is considered as describing the pile/soil system and the purpose of the test or the design is satisfied.

The criterion commonly used by numerous procedures uses the force and the velocity at the head of the pile in function of time as input and control data of the matching procedure. When the model is processed, both signals (or a combination of both signals) can be calculated if the model is excited with an input corresponding to a dynamic event. The procedure follows the following steps:

Either the force or the velocity[3] is imposed as a stimulus at a boundary of the model;

The model uses this stimulus and calculates the pile/soil response according to the requirements of the operator. Either the velocity (if the force is used as stimulus) or the force (if the velocity is used as stimulus) is calculated in function of the pile/soil model parameters;

The computed signal and the measured signal (non used as a stimulus) are compared and a quality criterion determines the degree of agreement between both signals.

If the matching is sufficient, the procedure is finished and the chosen parameters are considered to describe the pile/soil model for the dynamic test used in the configuration of the chosen model;

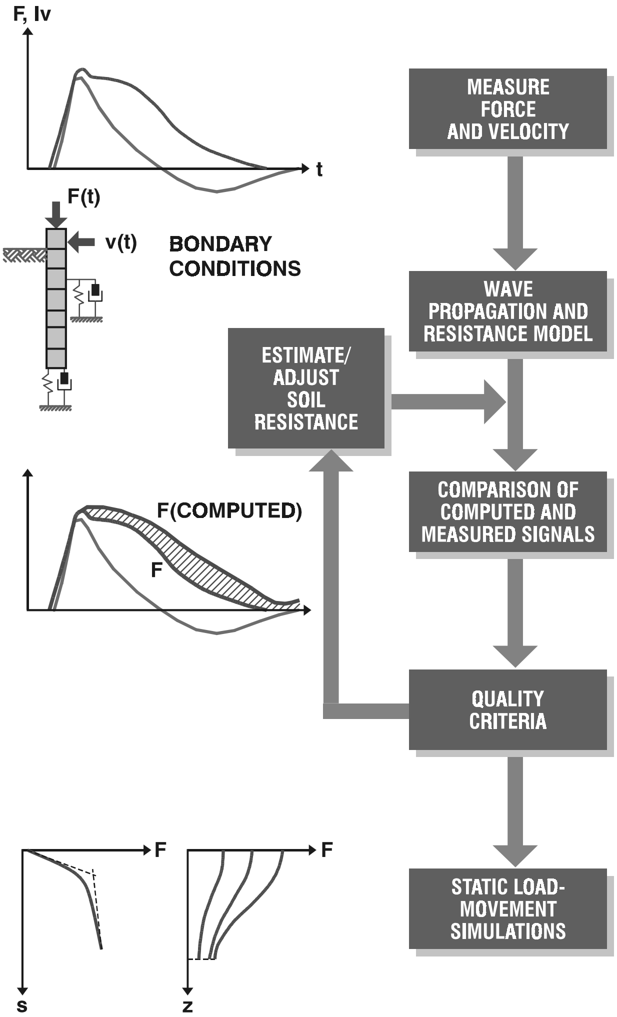


Figure I-14 : NUSUMS procedure (Holeyman, 1992)

If the quality criterion does not fulfil the requirements of agreement, the model parameters are re-estimated and a new loop is launched in order to improve the matching agreement.

3 Or a combination of signals like the downwards force wave

The described procedure is summarized in Figure I-14. All these procedures can be referred to as NUSUMS for NUmerical Simulations Using Measured Signals (according to Holeyman, 1992). Several methods related to this methodology have been developed, CAPWAP® (Rausche et al., 1972) TNOWAVE® (Middendorp, 1987), KWAVE® (Matsumoto et al. 1991), SIMBAT® (Paquet, 1988), the method developed inside the Civil an Environmental Department of the Université catholique de Louvain and called NUMSUM-UCL and a lot of methods without a specific brand names and often corresponding to a scientific research without commercial or wide utilization.

All the methods use the algorithm aiming at minimizing the gap between a measured and a computed signal in order to achieve the matching procedure. But there are some essential differences either in the model describing the pile, the soil or the propagation of the waves in the soil, or the model describing the resolution of the pile/soil interaction:

- The previous methods use either a finite element or a finite difference method in a lumped model or a continuous model based on the differential equation characteristics or still a full continuous description.
- The pile/soil system can be modelled with a classical 1-D longitudinal approach (Smith, 1960; Holeyman, 1984), a construction allowing for the soil behaviour in a near and far field (Deeks and Randolph, 1992, El Naggar, 1992 & 1994), a development of the soil motion in a 1-D radial approach (Holeyman, 1985, 1994, 1996) or a model working with a complete 3-D representation (Chow, 1981).
- The soil interaction is described by many authors (either for shaft or base reaction): Smith (1960) – Holeyman (1984, 1985, 1988) – Randolph and Simons (1985), Randolph and Deeks (1992) – Paquet (1988) – El Naggar & Novak (1992, 1994) – …
- And different loading rate effects on the pile capacity: the viscous term – the radiation term - …
- Each method uses a quality criterion that can be purely mathematical, oriented with statistical data and/or geotechnical guide or purely visual.
- The input and the reference signal can be the raw measured signals (Acceleration and Strain), the same signals after a first treatment (Velocity and Force) or a combination of them (Up and Downward waves).

Examples

TNOWAVE®

TNOWAVE® is based on the method of characteristics, firstly introduced by Voitus van Hamme (1974) and derived from the stress wave theory.

The pile is divided in continuous elements (Figure I-15). Each segment has an uniform section and is modelled with elastic and homogeneous material. The length is equal to the travelling distance of the wave during an incremental time step.

As derived from the wave equation theory and shown by Figure I-15, every soil reaction (base or shaft friction) or impedance sudden change (section, volumetric change) implies the creation of a reflected wave to the incident one. Consequently, at each Δt, the calculations of the propagation of all waves (upward and downward) is done and results in the creation of all reflected waves in function of the soil and pile modelling. The superposition of all waves, the equilibrium and continuity conditions at the boundaries of the segments allow the calculation of the dynamic and kinetic states of each pile segment: forces, velocities, accelerations and displacements are known.

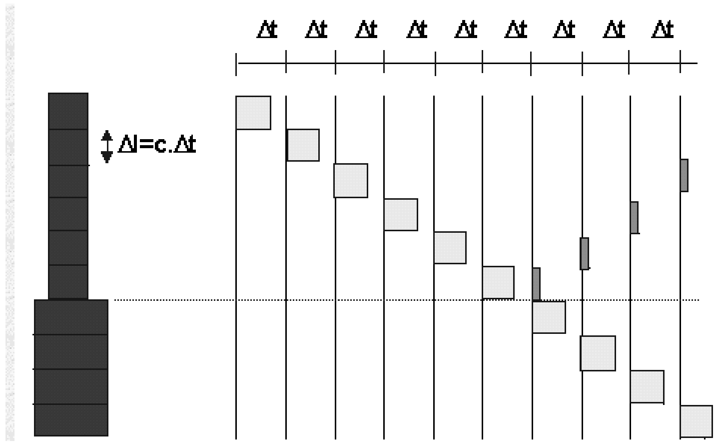


Figure I-15 : propagation of a wave into the TNOWAVE pile discretization (with differentiation) (after TNOWAVE manual, 2000)

The TNOWAVE soil model follows either the approach of Smith 1960, with an elasto-plastic static term and a dynamic term linearly dependent to the velocity, or a more complicated soil model (Randolph and Simons, 1986 and Randolph and Deeks, 1992) allowing for radiation of energy into the soil.

The model allows for loading, and unloading with a possible stiffer unloading. A yield factor allowing the use of an ultimate soil resistance different for the tension and compression domains is also available in the program. The program allows for the analysis of tubular piles and the differentiation of the inside and outside friction. The soil fatigue can also be taken into account. (TNOWAVE help manual, 2000).

CAPWAP®

The CAPWAP® program (CASE Pile Wave Analysis Program) developed by Rausche (1970) is a method allowing to determine soil resistance forces (dynamic and static) and their distribution along the shaft and at the base based on the method of characteristics (like TNOWAVE). This analysis can also determine the soil quake, damping parameter and simulated load –settlement curve for the base and head behaviours.

In each pile element, the superposition of both upward and downward waves yields forces and velocities at the upper and/or lower boundaries of segments. The velocity is integrated into displacement for each pile segment. The pile motion and force states are solved for the chosen description of pile/soil interaction for all the pile elements.

The displacement and velocity of each pile segment are chosen relative to the soil. The static term of the soil reaction is modelled by an elasto-plastic spring and a linear dashpot describes the viscous part of the resistance. The model allows to use either pile head velocity, pile head force or a combination of both measured signals for the signal matching procedure. The model allows for unloading and reloading, stiffer shaft unloading than loading, toe model working with plug and gap between pile and soil (Figure I-16).

A radiation damping can be also added to the basic Smith model in order to account for the energy radiated away from the pile into the soil when the soil reaction is low. The radiation term is function of the pile velocity. A mass and a dashpot are used to replace the rigid soil support of the Smith model. The masses are present in order to express the relative displacement and velocity quantities of the soil adjacent to the pile.

The program models splices and cracks. Internal damping is also modelled by reducing the magnitude of waves travelling into the pile. Finally, a module allows one to take the residual stresses of previous blows into account.

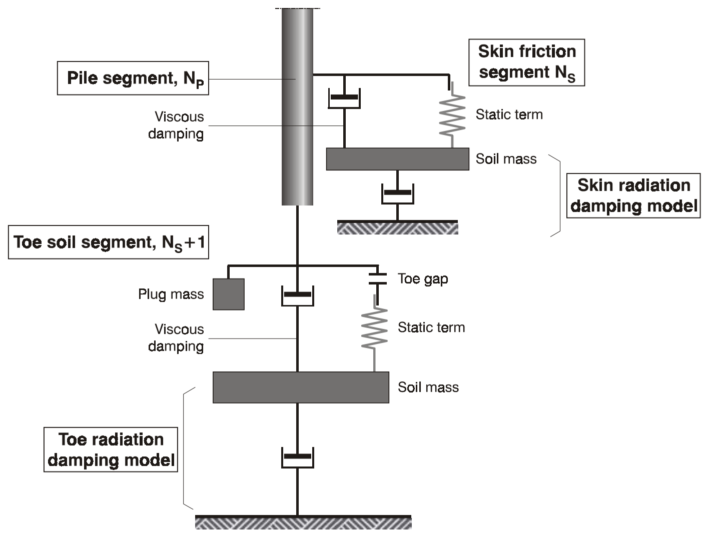


Figure I-16 : CAPWAP extended soil resistance model (after CAPWAP manual, 2000)

SIMBAT®

The SIMBAT® method (Paquet, 1988; Dali & Heritier, 1992; Stain, 1992) is a global procedure used to determine the equivalent static load-settlement curve using a complete sequence of blows of a dynamic load test. In the classical methods, only one blow is used to study the dynamic behaviour and evaluate the static resistance from a dynamic test. SIMBAT uses a sequence of blows in order to transmit different levels of energy.

The SIMBAT philosophy is based on the fact that at high strain rates the ratio "Dynamic" Resistance / "Static" Resistance is higher than at low strain rates. In this way of thinking, a sequence of blows is arranged so that a wide range of strain rate is explored with the aim to project the dynamic resistance versus the penetration per blow for different strain rates (Figure I-17). Where the zero strain rate is reached, the dynamic resistance is supposed to be the static one.

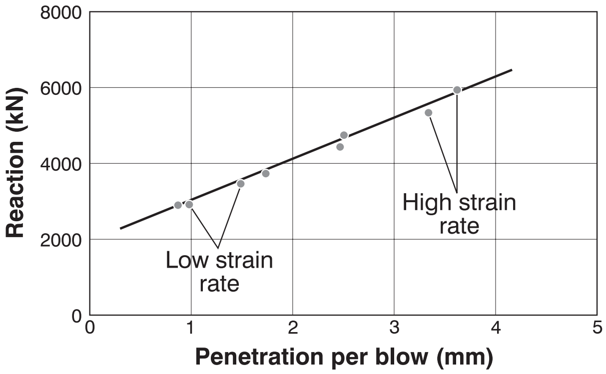


Figure I-17 : Dynamic testing at varying strain rates

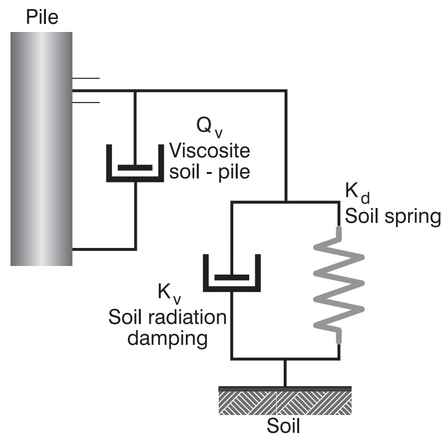


Figure I-18 : SIMBAT shaft model (after Dali & Heritier, 1992)

The dynamic resistance, R_d, is calculated by the classical wave equation theory by using the CASE formula (Equ. I-26) (Goble et al. , 1975). After calculation, a simulation programme can be used to check if rupture has occurred at each point during the return of the wave. If this condition is fulfilled (if the total dynamic resistance calculated from the model corresponds to the CASE one), the failure is assumed everywhere in the pile/soil interface and the model of the pile/soil interaction is validated. If not, the model does not correspond and the soil probably does not reach the ultimate resistance everywhere around the pile. A new simulation with the program must be performed to recalculate R_d and describe the pile/soil model.

The pile soil model adopted for SIMBAT differentiates the interface behaviour than the soil behaviour. The three media (pile, interface, soil) have their own motion and the associated reaction is linked to this motion.

The main methodology is the following:

- Signal acquisition & corrections (several blows);
- Separation of forces (upwards and downwards);
- Measurement of dynamic soil reaction R_{dyn};
- Regression analysis amongst all the blows to deduce the pile/soil system parameters;
- Extracting of the static resistance from the dynamic resistance;
- Take the elastic shortening of the pile into account and plot the load-settlement curve;
- Confirm with a computer simulation using the model explained below to check the hypothesis, and to calculate the distribution of soil resistance. An hyperbolic curve could also represent the total load-settlement computed curve.

KWAVE®

KWAVE®, developed in Japan (Matsumoto & Takei, 1991 takes into account the wave propagation in the soil inside a pipe pile and the influence of the internal shaft resistance on the wave propagation. The pile soil model follows a lumped model (Figure I-19). The soil models for shaft and base follow the models of Deeks and Randolph (1992) and Simons & Randolph (1985).

The values of the spring constants, the radiation damping constants, the dashpot and the soil mass parameters for both shaft and base models are determined from soil test data such as the shear modulus, Poisson's ratio and the soil density (Matsumoto et al, 2000). The process of matching uses the downward stress wave as input for the back analysis resolution.

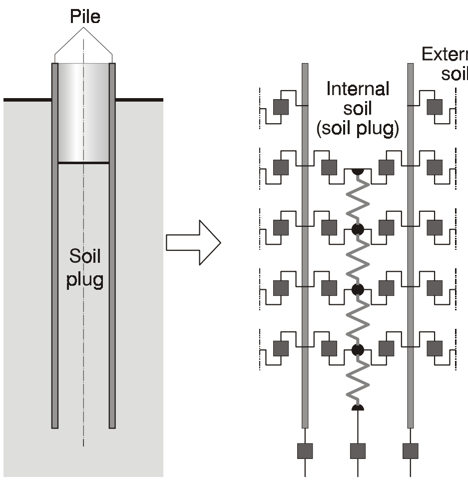


Figure I-19 : KWAVE® pile/soil system for pipe pile (after Matsumoto et al. 2000)

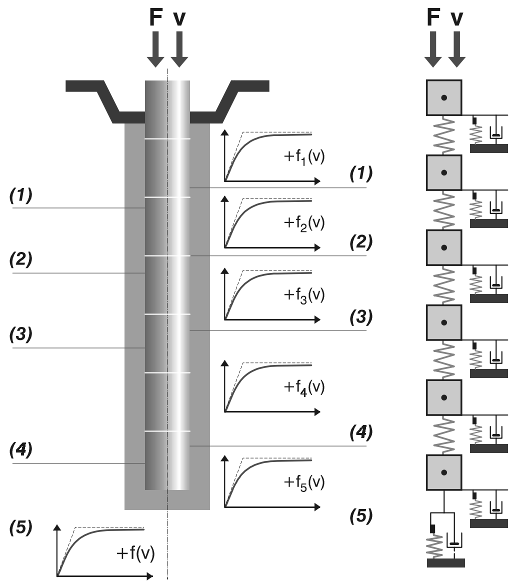


Figure I-20 : Description of the model used in NUSUM-UCL

NUSUM-UCL

The pile and its environment are described with a lumped model such as described in Figure I-20. This model allows a description of the kinetic and dynamic fields either of the pile or the soil in the direct vicinity of the pile during a transient solicitation. Force or velocity can be imposed at the pile head and the relative state of all parts of the model (stresses, forces, displacements, velocities, accelerations, …) is calculated at each time step in an explicit algorithm based on the finite difference scheme. The pile/soil model takes the loading rate effect into account with respect to the radiation damping and the viscous effect. It can consider also a hyperbolic or elasto-plastic static stress-strain relationship either for the base or the shaft modelling. The base and soil models follow the Holeyman approach (Holeyman, 1984) for the of the soil reaction's modelling. The model is allowed to take into account of the slippage between the pile and the soil. The process of the dynamic signal solving is based on an automatic signal matching technique using an algorithm working on the sensitivity of parameters to change towards the best solution.

This model is detailed in chapters 3, 4 and 5 of this thesis, either for the pile/soil model or the optimisation procedure. Slippage is also described in detail. The application of this treatment on the Sint-Kateline Waver and Limelette loading tests will be also proposed in order to point out the loading rate effect.

4. Advantages and disadvantages

There is no test available which can provide unequivocal information on all the design aspects required. A load test, either SLT or DLT, is influenced by many factors (installation, pile type, soil conditions, load history and test, …) but its goal remains the evaluation of the behaviour of the infrastructure in term of safety and settlement during the pile service. None of the tests can be considered as an accurate method but rather as an insight in combination with the test limitations. So, as conclusions to this chapter, it is interesting to gather the advantages and disadvantages of each method in a non-exhaustive list.

4.1. Static Loading Test

4.1.1 Advantages

The static test is considered as the reference test because it is the one that corresponds the most with the way that the load is applied in reality (duration, loading rate and type of loading). Moreover, the static test is generally regarded as the definitive test the one against which other types of tests are compared. These elements are obviously the best advantages of this kind of test.

Another point is that the data obtained are directly interpretable because they are linked to the acceptance criteria (maximum settlement and authorised stiffness and/or design load) and because the main interpretations were created with respect to this kind of test. Consequently, all the other methods tried to predict a response comparable to the load settlement produced by the Static Load Test.

The measurements are generally independent of the pile material properties even if this independence can be called into question (see the disadvantages).

4.1.2 Disadvantages

Because of the similarity of the test to the reality, the time needed to carry it out is necessarily long, and this duration is costly in term of money and contract planning. Moreover, the slow loading rate imposes that the load applied is high enough to get closer to the real load to be applied to the foundation. So the mobilization of this load and of this associated reaction is strongly expensive regarding to the obtained result (one pile tested). Consequently, the higher disadvantages of this loading test motivate the engineers to create other loading tests reliable, rapid, economic, and generating a comparable result.

The reaction supplied for the applied loading (kentledge, reaction piles, ground anchors) generates some associated effects or interaction with pile that perturb the interpretation of the results. These stresses will increase the shaft friction and the base capacity and the pile settlement is reduced and the pile head stiffness is also overestimated. Those interactions can perturb the beam of reference used to measure the settlement.

It should also be noted that there are many procedures for static pile load testing. From Maintained Load Test to Constant Rate of Penetration Test, a broad set of test regimes and procedures are possible. Due to the different loading paths, any pile subjected to the various tests will exhibit a different load-settlement response influencing the conclusions because the results are influenced by loading history (steps and duration).

4.2. Dynamic Loading Test

4.2.1 Advantages

The dynamic test was created with the aim to improve the productivity in term of quality control and design confirmation. These objectives seem to be achieved because the test is accepted as a routine procedure and also used for the estimation of the static load-settlement behaviour to be compared to the corresponding static one.

Since the test need a short time to be organised, performed and analysed, since the used device weighs about 1.5 to 2 % of the mobilized pile capacity, the test can be performed on more piles on site (till 10% of the installed piles) during the same period generating a huge cost saving.

The test is often used to integrity testing giving a method of inspection of the pile profile and of the pile homogeneity. This a direct consequence of the underlying wave equation theory.

The tests can also be performed on very large in diameter and capacity piles that can be tested statically with difficulty. The test can also be applied on piles offshore without difficulty.

4.2.2 Disadvantages

The potential limitations of DLT tests include the fact that the load-settlement behaviour is estimated and not unique (corresponding to a best fit estimation with respects to assumptions made about the pile/soil interaction model). The acceptance criterion is consequently dependent of the assumptions made in term of pile stiffness and soil ultimate resistance.

The high technicality of the test causes a sophisticated phase of interpretation. Without well trained personnel, it is impossible to achieve a correct and proper dynamic test.

As the load is applied very quickly, the time-dependent settlement behaviour are not developed during the test. Besides, the dynamic effect of loading rate generates strongly non linear adding soil resistance that needs an interpretation significantly sensitive to assumptions.

Under impact, a pile is subjected to a complex combination of compressive, tensile and bending forces that may produce much higher stresses than under service conditions. If not accounted for, these stresses can exceed material strength and cause structural damage.

The measurement accuracy is also dependent of either the pile material properties (concrete or steel volumetric mass, wave propagation velocity or dynamic modulus), or the mathematical process needed to evaluate the pile particle velocity and displacement from the measured acceleration. Due to the fact that the pile is struck by a falling mass, the soil is rarely completely mobilized. The dynamic tests can often mobilize the shaft friction but rarely the base reaction. So the prediction of a complete load-settlement curve until large settlements is highly unlikely or must be considered with regards to the assumptions.

Finally, the set-up or relaxation effects and the evolution of the pile bearing capacity with time impose to the test to be performed at a certain time after driving.

5. Conclusions

This chapter aimed to describe the procedures of loading studied in this research, their definitions, their main characteristics and methods of interpretations. It was shown that the pile testing is performed with a wide variety of methods of which the complexity is generally proportional to the rate of the applied loading. the faster is the loading, the more complex is the interpretation and the data management. This complexity is due to the viscous and inertial additive terms that must be withdrawn in order to obtain the equivalent static pile/soil behaviour.

The so-called" dynamic procedures of loading (DLT) are "young" processes and present a large quantity of interpretation methods. Each of them are based on a typical characteristic of the loading procedure: either the propagation of the wave (CAPWAP, …), or the wave equation theory (CASE method). The complexity of the method is often linked to the precision and the diversity of the result (the static pile bearing capacity, the static load settlement curve or the soil reaction distribution along the pile). A general overview of these methods is proposed and each one is described with respects to their main characteristics.

Finally a short critic of the two loading procedures is given and a specific attention is put on their differences, advantages and disadvantages.