Deep Foundations on HP Piles

STATE-OF-THE-ART

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Introduction

The objective of this handbook is to discuss briefly the state of the art in assessing the bearing capacity, drivability, load testing, and the execution of deep foundations by means of HP steel piles.

The engineer designing this type of foundation must inquire into and follow up on site the following questions which will be discussed in detail in this publication:

- subsoil investigations must provide sufficient geotechnical information to calculate the bearing capacity of HP piles and enable a decision to be made on how to install them;
- the design of the HP piles. The calculation of the bearing capacity based on French regulations, and a broad general view of methods used in other countries are given.
- piling forecasts. Driving forecasts, which nowadays simulate driving by the use of computer software, allow driving curves to be drawn, as well as providing advice on the choice of the driving system and an estimate of the stresses in the steel section;
- checking the pile installation by means of driving records or by instrumentation of the pile being driven, as well as by static and dynamic tests;
- the common methods of piling by impact or vibration.

1. Geotechnical investigations

The hydrology and geotechnics of the site must be ascertained before a foundation can be designed.

Subsoil investigation procedures vary in different countries because of the techniques used and the problems to be solved. However, certain common elementary criteria help to achieve a rational foundation design and to ensure safe installation.

The engineering design of steel piles, whether driven or vibrated, includes both the dimensioning of the pile and the choice of pile driver as well as the piling method, and is justified by:

- an assessment of the maximum axial bearing capacity of the pile,
- a piling forecast, based on the resistance of the soil to the penetration of the pile,
- an analysis of the pile behaviour under axial and possibly lateral loads.

To arrive at these estimates, the geotechnician must know the precise:

- stratigraphy throughout the site,
- type of ground penetrated,
- physical and mechanical properties of the soils in place.

The information should be available for a depth greater than the embedment of the pile so that the soil quality is known throughout the pile length as well as beneath its toe.

Continuity of the subsoil investigation is especially important for driven or vibrated piles. The presence of a hard layer only a few decimetres thick, undiscovered by the soil investigation, can cause great difficulty in driving, such as damage to the pile toe or premature refusal.

A well-designed geotechnical investigation should include two types of borehole: cored holes and holes drilled for in-situ tests. The number and spacing of the holes depend obviously on the size of the site and its geological complexity.

The cored holes are useful for identifying and describing the ground. Undisturbed samples are taken and submitted to laboratory tests. Their physical properties are measured (water content, plasticity index, grain size, etc.), as well as other parameters (cohesion and angle of friction) and certain other mechanical properties useful for estimating soil strength and deformability. All this information is written into the borehole record or log (Fig. 1).

The in-situ tests yield some direct parameters of deformation and/or resistance of the soil in place, in order to estimate loading variables and build up a model of the behaviour under transverse loading of a deep foundation. This information is helpful and often indispensable for the design and calculation of a deep foundation because, combined with the tests on samples, any disturbance to the soil caused by drilling for and transport of the samples can be disregarded.

Although no ideal, universal in-situ test exists, certain tests are suitable for resolving particular problems in given soils. Most in-situ tests have standardized procedures which must be followed strictly.

The static penetration test (CPT or Cone Penetration Test) is the most suitable for the engineering design of driven piles but is less suitable for compact gravels, calcareous marls or other hard cemented soils. On the one hand, it provides a continuous record of the soil resistance; on the other hand, as it simulates the driving of a pile into the ground, it provides information about two parameters – the toe resistance q_c and the shaft friction Q_{st}. These can be used directly in calculating the loadbearing capacity of the pile (Fig. 2).

The dynamic penetration test not only provides more effective penetration but also produces a continuous record of resistance. However, the information obtained – the dynamic toe resistance R_p – is less easy to use directly in foundation design.

The SPT (Standard Penetration Test) provides a semi-continuous record of the number of blows N needed to drive a standard core barrel of 30 cm diameter into the ground (Fig. 2). It is widely used in the USA, less in Europe. Other tests have only limited use (for example, the vane test in soft clays) or are even more limited in application (such as the Marchetti dilatometer or the selfboring pressuremeter).

The pressuremeter produces two resistance parameters – the limit pressure p_l and the creep or flow pressure p_f, and a deformation parameter – the pressuremeter modulus E_M (Fig. 3). Rigorously careful testing is needed to obtain pressuremeter parameters of high quality. These data can be used directly for calculating the bearing capacity of piles, but as the tests are discontinuous, the results cannot be guaranteed as being representative in the case of stratigraphically variable soils. Continuous recording of the drilling parameters partly overcomes this handicap and should be systematically performed.

The pressuremeter test is the most widely used in-situ test in France, but is seldom used abroad.

2. Engineering design

2.1. Bearing capacity of HP piles

The ultimate bearing capacity of piles in the most general way is:

$$Q_u = q_p \cdot A_p + \Sigma f_{si} \cdot A_{si}$$

in which

Q_u = ultimate capacity or capacity at failure. It is theoretically defined as the axial load applied at the head of the pile, which causes it to sink an unlimited amount. In practice, it is the axial load which causes the pile head to sink by at least one tenth of its diameter or by a tenth of the width of an HP pile.

A_p = cross section of the pile at its toe,

A_{si} = lateral contact surface of the pile with the soil in layer i,

q_p = the unit soil resistance under the pile toe,

f_{si} = the unit shaft resistance in layer i.

The permissible load is deduced from the ultimate bearing capacity by introducing safety factors. Two types of regulation are applied in different countries and different types of structure:

- regulations based on permissible stress: the permissible stress is determined from the ultimate bearing capacity by applying an overall safety factor (API Standard RP2A, or British Standard 8004);
- regulations based on partial safety factors for load and soil resistance: the combinations of loads with weightings appropriate to the structure are compared with the values of weighted soil resistances (Fascicule 62 – Titre V, Eurocode 7).

The methods of calculating the ultimate bearing capacity do not provide any idea of the safe load and can therefore be investigated independently of the relevant regulations. However, we should remember that while certain methods are accepted by most of the regulations, others are less widely recognized.

HP piles have particularly complex geometry. For this reason it is hard to determine the precise rupture surface and thus to assess the values of A_p – the effective area of the point of the pile, and A_s – the lateral contact surface between pile and soil.

An HP pile penetrates the soil:

- either as a knife blade: in this case the steel enters the soil by simply shearing it and the rupture surface develops in the steel-soil interface;
- or by the formation of a soil plug: in this case a mass of compressed soil within the flanges of the steel sticks to the pile during driving; the rupture surface develops following a complex line depending on soil-steel as well as soil-soil friction. We should say that the pile is completely plugged when the rupture surface is equivalent to the rectangle circumscribing the pile. In any intermediate situation we would say that the pile is partly plugged.

The same sort of behaviour is noted when an HP pile is loaded statically to failure. But there is no direct relation between driving behaviour and long term behaviour; for example, a pile can be driven unplugged , but after some time may develop a plug.

Plug development depends on three factors:

- the type of loading: static, dynamic, compressive, tensile;
- the type of soil: eg dense sands favour plug formation;
- pile geometry: plugs develop more easily on piles with a narrow web.

Several experimental series have been run on actual piles in diffferent soils. This programme, supported by the EC [European Community], has brought better understanding of these phenomena.

Thus the French regulations for Government contracts (Fascicule 62 – Titre V, réf. [1]) recommend considering the pile (Fig. 4):

- as unplugged when calculating shaft friction,
- as partly plugged when calculating the toe load by applying to the whole plug cross section a reduction coefficient of 0.50 for clays and 0.75 for sands.

Generally speaking, we strongly recommend consideration of the various probable kinematics of failure in each case, but using only the most unfavourable results for the solution.

The values of the unit stresses q_p and f_s are functions of the type of soil under consideration, of the state of stress at the level considered and of the re-shaping occasioned by installing the pile.

When in-situ test results are obtainable from the CPT or pressuremeter, the load and skin friction can be calculated according to the French Fascicule 62 – Titre V (ref. [1]). A more detailed calculation method, based on CPT test results, is shown in Appendix 1. The assumptions concerning toe cross section and perimeter are shown in Fig. 4. This method also allows for the assessment of the capacity of piles with laggings attached at a given level.

A method of calculation using the SPT is proposed by Meyerhof mainly for granular soils. This method is based on a correlation between the penetrometer toe resistance and the number of blows N in the SPT.

$q_c = 400 \cdot N$ (kPa) for sands.

The ultimate bearing capacity in kN is given by:

$Q_u = 400 \cdot N \cdot A_p + 2 \cdot N' \cdot A_s$

in which

A_p = cross-section of pile (m^2),

A_s = shaft area of pile (m^2),

$$N = \frac{N_1 + N_2}{2}$$

where:

N_1 = the smaller of the values of N averaged over 2 diameters under the pile toe, calculated or measured in the immediate neighbourhood of the pile toe.

N_2 = average of N over 10 diameters under the pile toe.

N' = average of N along the pile shaft.

The value $2 \cdot N'$ comes from a correlation with the CPT values:

$$f_s = \frac{q_c}{200} = 400 \cdot \frac{N'}{200} = 2 \cdot N'$$

For saturated fine sands with values of $N > 15$, the following reduced value of N (N_{red}) is substituted for N:

$N_{red} = 15 + 0.5 \cdot (N - 15)$

If, however, only laboratory soil test results or inadequate in-situ test results are available, we give below some guidelines for values obtained from international technical literature that are widely accepted. Of course, these values should not be substituted for those imposed by the regulations. The geotechnician should also make use of his judgement and experience as an engineer.

In clays the toe load and the unit skin friction at failure can generally be expressed by:

$q_p = N_c \cdot C_u$

$f_s = \alpha \cdot C_u$

in which

C_u = undrained cohesion of the layer under consideration,

N_c = bearing capacity factor taken as equal to 9,

α = steel-soil adhesion factor.

In the case of driven steel piles, the steel-soil adhesion factor under static load is generally between 0.5 and 1: this diminishes with increase of the undrained cohesion. The curves of the adhesion factor as a function of the undrained shear strength C_u currently recommended in international regulations, are given in Fig. 5. Assuming soil-soil shear as can exist throughout the length of a plugged or partly plugged HP pile, the factor α is obviously equal to 1 throughout the surface of rupture concerned.

In sands and gravels it is generally accepted that below a certain critical depth expressed as a multiple of the diameter or of the width of the foundation, the toe load and the unit shaft friction increase up to what is termed an ultimate value.

The ultimate toe load and skin friction values increase with the soil grain size and its density. Five main classes are usually considered and Table 1 summarizes the American regulations (according to API RP2A, 1989, ref [2]). The values are close to those of Fascicule 62 – Titre V (ref. [1] [3]), and they are summarized in Figure 6.

Class	Type of soil and its density	Ultimate base resistance, qpl, (MPa)	Ultimate skin friction, fsl, (kPa)
I	Very loose sand Silt and loose sand – Medium dense silt	2	50
II	Loose sand Silt and medium dense sand – Dense silt	3	65
III	Medium dense sand – Silt and dense sand	5	80
IV	Dense sand – Silt and very dense sand	10	100
V	Dense gravels – Very dense sand	12	120

Table 1:
Ultimate values for base resistance and skin friction in siliceous granular soils (according to API RP2A, 1989)

The availability of in-situ test results (especially the static penetrometer base resistance, q_c), together with the experience of the geotechnician, ensure the right decision on whether to use or to change these values whenever necessary.

In the American regulation for offshore piling API RP2A, the ultimate values from Table 1 are applicable when the depth is more than a critical depth, ie around 20 to 30 m. For depths between 0 and the critical depth it is necessary to interpolate linearly for q_p and f_s between the value 0 at the surface and the ultimate values of q_{pl} or f_{sl}. Knowing that this critical depth is about 20 times the smallest dimension of the piles generally used for offshore foundations, it would be legitimate to consider limit values for HP piles to be at depths of 5 to 8 m.

This practice differs from the French regulation (Fascicule 62 – Titre V), which considers that for any given soil the ultimate values apply from the surface, provided that the foundation is deep. A foundation is considered deep when its penetration depth D_e, as defined in Fascicule 62 – Titre V, is more than five times its width.

In unsaturated soils it can be advisable to neutralize or reduce the friction for a certain depth from the head of the pile so as to bring into consideration the separation of the soil within the flanges during driving. This reduction is made whenever needed for a height equal at the most to five times the smallest dimension of the pile.

In rocky soils wherever driving is possible (marl, chalk), the level of unit friction has a limit as for sands. The ultimate values recommended in Fascicule 62 – Titre V (ref [1]) are shown in the diagram in Figure 6.

The different methods of calculation described in this section are illustrated by simple examples in Appendix 2.

2.2 Pile load tests

It may be essential during the early conceptual stage of the foundation system to perform static load tests and for the following reasons:

- contracts and regulations: several regulations make a static test virtually obligatory where foundation design calculations have to be approved,
- engineering factors: a load test may be needed when
  - the mechanical properties of the soil are not known or uncertain,
  - the available methods of calculation for the bearing capacity give divergent results,
  - the structure has a large number of piles, thus calling for optimized design.

In the case of large structures or those with significant variations in soil properties, several tests may be contemplated.

A detailed geotechnical investigation can never be superseded by preliminary tests. On the contrary, the purpose of these tests is to obtain accurate geotechnical values for the parameters controlling pile behaviour including the resistance to, and the displacement of the piles.

To be useful, load tests must be carried out early enough in the project conception stage for their results to be used to optimize the project. They should be representative, which means that they should be made on a pile of the same type and dimension as those proposed for the foundations, and the pile installation method should be the same.

The static load on the pile should be such as to apply similar loads (tension, compression, etc.) to those carried by the final structure. It is always essential to keep strictly *to an accepted operating method.*

Where preliminary static tests could optimize foundation costs, for instance with numerous piles and variable stratigraphy, it can be helpful to use strain gauges or extensometers. The measurement system should make it possible to estimate the load distribution between toe and shaft as well as the distribution of friction along the shaft. This sort of information reinforces validity and enables the test results to be extrapolated with considerable reliability to other piles in the structure.

2.3. Design stresses in the pile

The stress in the steel under static load should be limited to a fraction of the steel's elastic limit σ_e.

Fascicule 62 – Titre V (ref. [1]) defines the design stress, called the design yield stress σ_{ed}, as:

$$\sigma_{ed} = \frac{1}{1.25} \cdot \sigma_e$$

for the ultimate limit state (ELU), and $\sigma_{ed} = \sigma_e$ for the combined accidental limit state.

This limitation is less conservative than that normally allowed in Anglo-Saxon regulations for open tubular steel piles driven offshore or in harbour work, for which

$$\sigma_{all} = \frac{1}{1.5} \cdot \sigma_e$$

for the ELU combination.

Recent tests in Hong Kong have confirmed that HP piles can carry stresses of this magnitude. It follows therefore, that apart from piles driven to rock, the bearing capacity of HP piles is generally limited by the soil resistance and not by the steel properties.

2.4. Corrosion

For underground exposures, it has been shown by extracting steel elements that „the type and amount of corrosion observed on the steel pilings driven into undisturbed natural soil, regardless of the soil characteristics and properties, is not sufficient to significantly affect the strength or useful life of pilings as load-bearing structures" (ref [8]). For practical purposes, a corrosion rate below 0.005 mm/year can be assumed here. Moderate corrosion may occur on piles exposed to fill soils above the water table or in the water table zone. In non aggressive fills, corrosion rates between 0.005 and 0.010 mm/year may occur.

For piles installed in fresh, uncontaminated water, a corrosion rate of 0.01 – 0.02 mm/year can generally be assumed for asessing long term loss of thickness.

The parts of the piles exposed to the atmosphere are generally subject to low corrosion. Additionally, these areas can be inspected regularly and protective measures taken if necessary. Corrosion rates below 0.01 mm/year are valid in normal circumstances whereas values up to 0.03 mm/year may be reached in polluted industrial atmosphere.

The risk of corrosion is more serious, however, for piles in direct contact with polluted and/or sea water. Here, long term corrosion rates may vary in average from 0.10–0.15 mm/year in the main attack zones (splash zone, slightly below mean low water level) depending on local conditions. Below these critical zones, corrosion is generally less than 0.08 mm/year.

The commonest way of dealing with corrosion in areas of slight to moderate corrosive attack is to allow for an increase of steel stress at the end of life time or to provide an extra thickness of steel.

In very special and rare cases, when the corrosion risk is considered high enough in relation to the life of the structure, several anti-corrosion measures may be foreseen, such as:

- the application of a coat of paint or galvanizing, etc.;
- cathodic protection by induced current or sacrifical anode, etc.;
- wrapping the vulnerable upper part of the pile in concrete or impervious mortar. It can be assumed that a continuous mortar or cement grout coating, 5 cm thick, is a good protection if it consists of at least 500 kg cement per cubic metre and the water: cement ratio is below 0.5.

One special case of corrosion can be induced by electric currents in the soil. Alternate current does not allow for the creation of stray effects. Only direct current is critical. As an example, steel structures next to railways operating under direct current may be influenced in areas where the rail is negative against the soil. Here, the structure becomes anode and the current exits in this zone, subjecting the steel element to electrolytic corrosion. Protection against this type of corrosion is derived by isolating the electric source and/or the electric conducts against the soil. Soils with high resistivity and the separation of the rails from the soil by isolating ballast as well as a good conductivity of the rails themselves offer a passive protection against stray currents. Active protection can be achieved by polarized drainage which consists in deviating intentionally the direct current from the railway to the steel structure. Precise procedures can be found in national railway regulations such as ref [9].

2.5. Driving forecasts

The purpose of a pile driving forecast is to:

- decide on the pile material and the means (diesel, hydraulic) of installing the piles, as well as the size of pile driver;
- check the agreement between, on the one hand, the energy needed to overcome the soil resistance and, on the other, the integrity of the pile; the steel cross-section must be adequate to allow enough energy to be transmitted to the toe, and the dynamic stresses must not be so large as to damage the head or the toe;
- establish criteria to decide on when to stop driving; this concept will be discussed in more detail later.

The driving forecast must form part of the engineering design of the foundations. This idea may be relatively new for civil engineering on land but is common practice offshore. Forecasting methods are by now sufficiently well tested to be treated systematically and in detail.

An investigation of pile driving includes three distinct but complementary stages, illustrated by Figure 11:

- estimation of the soil resistance to driving (Fig. 11a),
- simulation of the hammer-pile-soil system using computer programs (Fig. 11b),
- combination of the two previous stages into curves which form the driving forecasts (Fig. 11c).

A detailed description of the successive stages of the driving forecast is given in Appendix 3 of this report.

2.6. Refusal criteria for pile driving

An engineering design of pile driving generally includes decisions on refusal criteria.

In the current state of knowledge of the static and dynamic behaviour of subsoil, one cannot guarantee the static load capacity of a pile on the basis of its dynamic behaviour alone. The idea of a refusal criterion should therefore be handled carefully and the assumptions expressed in exact terms.

Generally speaking, the contractual embedment of a pile is assessed from the static resistance of the soil. Such data depend on the results of the geotechnical investigation and previous load tests. It is clear that these data are obtained at the precise locations of drill holes or piles under test.

A driving forecast as described in Section 2.5. should be undertaken in parallel, to estimate the maximum number of blows called for by these data.

At or near a drill hole, pile driving should continue until the pile penetrates to the depth required by the static calculations. If the expected maximum number of blows is exceeded before the required penetration, this would be an indication of dynamic behaviour which is „better" than forecast. On this basis and in case of premature refusal, the pile behaviour should be re-analysed in the light of the new information obtained by driving.

The conclusions may lead either to final acceptance of the pile or to renewed driving with a heavier hammer.

Refusal criteria are needed for large contracts and for piles located away from boreholes or where there is a lack of the necessary geotechnical information. These criteria should allow projected penetrations to be adapted to the particular site conditions, allowing for variations in stratigraphy and the mechanical behaviour of the ground. Based on data from driving nearby boreholes, the criteria should include factors permitting to stop the driving, such as the set per blow, the energy supplied by the hammer, and the minimum acceptable penetration.

In this way, the refusal criterion as described separates the idea of static loadbearing capacity from that of resistance of the soil to driving.

2.7. Pile-driving formulae

Driving formulae are semi-empirical expressions which, for a given hammer and pile, relate the set of the pile under a hammer blow to the dynamic resistance in the soil. Refusal is defined in the same way as the penetration of the pile under the hammer blow; it includes an elastic part and a plastic part corresponding to the true downward movement of the pile in the ground. The dynamic resistance is the resistance by which the soil opposes the downward movement of the pile. It is a function of the downward speed of the pile and should not be confused with the static resistance of the pile.

As a matter of principle, the purpose of such a tool should be firstly, to determine the size of hammer needed to drive the piles and secondly, to determine from what level of penetration per blow the expected resistance of the pile is reached and thus to establish a refusal criterion in its traditional sense. In reality, as discussed in Section 2.6., the idea of refusal criteria must be treated with care because of the fundamental difference between the dynamic resistance to driving and the long-term static resistance of the pile.

There are many piling formulae: the Dutch formula, Crandall's formula, the Delmag formula (for diesel hammers only), and Hiley's formula. By way of example, Hiley's formula is often used and is moreover not confined to any particular type of hammer:

$$R_d = \frac{f \cdot E_r}{s + \frac{1}{2}(C_1+C_2+C_3)} \cdot \frac{W_r + e^2 \cdot W_p}{W_r + W_p}$$

in which:

R_d = total dynamic resistance to driving,

E_r = nominal energy of the hammer (data from the manufacturer),

f = efficiency of the hammer,

s = set of the pile per blow,

C_1, C_2, C_3 = elastic compression of the dolly, the pile and the soil respectively,

W_r = weight of the ram,

W_p = weight of the pile,

e = coefficient of recovery.

Although the formula seems sophisticated, it should be considered with caution. Obviously driving formulae are simple to use but their rationale is incomplete.

Driving formulae should not be used to establish refusal criteria unless the properties of the soil and the hammer have been confirmed by a previous pile test.

Driving formulae can be useful for checking the size of a hammer or can be used during driving to compare one pile with another.

3. Monitoring pile driving

Pile driving is inspected normally by the content of driving records for each pile, which should contain:

- the pile number and its location,
- the pile description,
- the hammer description:
  - type, size, weight,
  - weight of the ram,
  - type of dolly,
  - maximum stroke of ram,
  - nominal energy of the hammer;
- the driving data:
  - driving speed, blows/mn (diesel hammer),
  - number of blows per 25 or 50 cm penetration,
  - stoppages or incidents during driving.

The number of blows per unit of penetration should preferably be recorded throughout the driving. If not it must at least be recorded at the approach of the design driving depth and just before driving is stopped.

This minimum information, required by current regulations, is not always enough for true quality control of the installation of the piles.

On large jobs or on sites where piling may be risky, some piles should be instrumented.

3.1. Dynamic measurements during pile driving

The instrumentation and monitoring of piles during driving, is based on:

- instruments on the piles which measure the accelerations and stresses in the pile head at each blow of the hammer (Fig. 12a);
- an analysis in real time of the data from high-quality programs which indicate essentially (Fig. 12b):
  - the maximum stresses in the pile,
  - the true energy delivered to the pile by the ram,
  - the total soil resistance under each hammer blow;
- a detailed analysis in due course of the recordings using simulation programs which provide information about the behaviour of the pile-soil system during driving.

Systems of this kind provide the engineer in charge at any moment with information which is fundamental for determining (Fig. 12c):

- the risk of damage to the pile when driving through hard or heterogeneous layers;
- the operation of the hammer and its true efficiency compared with the value used in the forecast;
- the increase in the dynamic resistance during driving.

Information about these data on site is extremely useful in the event of premature refusal. It provides the geotechnician with what he needs for:

- estimating whether the refusal is due to stronger soil resistance than expected or to faulty operation of the hammer;
- deciding whether to accept the pile at the level of refusal or to put expensive replacement procedures into operation, ie ordering a more powerful hammer, strengthening the pile by injecting cement grout or welding on reinforcing plates, and so forth.

In practice, the use of pile monitoring is justified in the following instances:

- the need to pass through an identified layer of stiff soil (sandstone, calcareous marl and the like);
- the risk of premature refusal, with insufficient performance data on the pile hammers available;
- inadequate geotechnical information and related uncertainty of final static bearing capacity of the piles;
- a site with local variations in stratigraphy or mechanical properties of the soil;
- a site with a very large number of piles.

The number and layout of the instrumented piles depend obviously on the problems to be solved and the size of the foundation contract.

3.2. Static load tests

One or more static tests can be contractually imposed and should verify a posteriori that the piles installed have a bearing capacity at least equal to that required by the contract.

The load tests can be performed:

- on piles specially designed for the purpose, which will therefore be tested to failure.
- on foundation piles which are part of the final structure. In this case the static test need not continue until failure. Indeed, the application of a test load not exceeding 150% of the permissible load is enough to evaluate correctly the performance of a pile.

Most piles are not instrumented and measurement of the displacement of the pile head in relation to the applied load may be sufficient, but it is important that the loading procedure is followed strictly.

3.3. Dynamic load tests

The use of static loading tests on piles is an expensive, laborious job, the main problem being to find a reaction device.

It is therefore particularly desirable to find a dynamic load test which gives information about the static load capacity of the pile. This is difficult because there is no direct relation between dynamic and static load capacity.

The method in most common use was developed in the USA and is known as the Case method. It is based on measurements on the pile head of two time-based variables (Fig. 12a):

- the impact force,
- the velocity of the particles in the pile.

The instrumented testing during driving and dynamic testing of the pile after driving do not have the same purpose and take place at different times of pile installation:

- dynamic measurements during pile driving aim to supply the information needed to guarantee the quality of the installation;
- dynamic load testing occurs after the installation of the pile in a redriving stage and requires only a small number of hammer blows. Its purpose is to estimate resistance to driving after the soil has recovered. This value can help when assessing the bearing capacity of the pile.

The total resistance of the soil to penetration of the pile is given by:

$$R(t) = \frac{1}{2}[F(t) + F(t + \frac{2L}{c})] + \frac{1}{2} Z[V(t) - V(t + \frac{2L}{c})]$$

in which,

$F(t)$ = force measured at instant t,

$V(t)$ = particle velocity at instant t,

L = total length of pile below gauges,

c = wave speed in steel,

Z = impedance of the pile

$= \frac{mc}{L}$ where m is the pile mass.

The function $R(t)$ has a maximum R, which is the total maximum resistance under the impact.

It is accepted that the total resistance R consists of:

- one component R_u independent of the particle velocity, which corresponds to the static resistance (for $v = 0$) of the pile at the moment of impact,
- and a second component R_d which is a function of the particle velocity, which corresponds to the dynamic resistance and is expressed as:

$$R_d = j \cdot \frac{mc}{L} \cdot v_b$$

in which,

j = damping factor,

v_b = particle velocity at the pile toe.

The value R_u is given by:

$$\mathbf{R_u = R - R_d}$$

It is a function of the damping factor chosen. This factor can vary in the range $0.1 < j < 0.8$ (s/m) according to the soil type.

The use of the Case method involves:

- instrumenting the pile head and treating the data with equipment and methods very similar to those used for the instrumentation in monitoring pile driving;
- using a driving system able to develop enough energy to displace the pile toe.

Representative results are obtained only with extreme rigour in the test procedure and the interpretation of data:

- the method provides a value for pile resistance at zero speed (R_u) at the moment of test. Experiments suggest that this resistance closely represents the static capacity of the pile at that moment;
- the static resistance at the moment of testing is not necessarily representative of the long-term static bearing capacity of the pile, especially in clays, in which recovery after driving may take several months. The date of the test must therefore be chosen carefully.

In effect, several tests at different intervals of time may be needed:

- it is also advisable to test at variable energies (stroke of ram) so as to determine the value of the damping factor j.

Generally speaking, it is not advisable to base the verification of a whole site on the dynamic method alone. However, on a large site involving several static tests the method may, after calibration with a static test, save outlay on the possible subsequent tests.

4. Installation of HP piles

HP piles are normally installed by impact or by vibration:

- impact driving usually involves diesel or hydraulic hammers,
- vibration may be by either hydraulic or electric vibrators.

These two techniques may be used either separately or in conjunction with each other; the pile can be started by vibration, then impacted to its final position.

4.1. Impact driving

The two most widely used pile drivers are diesel and hydraulic hammers.

4.1.1. Diesel hammers

These are the usual hammers for piling on land. They work with an internal combustion engine whose piston is the ram. The explosion of the gas mixture releases enough energy to lift the ram which falls again under its own weight and strikes the helmet through the dolly. The helmet transmits the blow towards the pile.

Although competitive from the viewpoint of outlay cost and maintenance, this type of hammer has a low efficiency due to the energy losses at each of its many intervening parts. The efficiency of the hammer is the ratio of the effective energy transmitted to the pile by the stroke of the ram, divided by the ram's incident energy. Instrumentation tests during driving have shown that the efficiency of diesel hammers rarely exceeds 45% and often falls to between 30 and 35%.

4.1.2. Hydraulic hammers

Hydraulic hammers first appeared some ten years ago in offshore work where their achievements in underwater piling ensured their success. For some years they have developed rapidly on land and in harbour work.

These are double-acting hammers; the hydraulic pressure lifts the ram, then accelerating it down towards to the anvil.

The type of steel/steel impact and the existence of only one intervening unit (the anvil) reduces energy losses and results in very high overall efficiency. Instrumentation during driving has shown that the efficiency is generally above 75%. In addition, these hammers have a wide range of use because hydraulic power is easily adjusted; also the internal instrumentation measures the kinetic energy of the ram instantaneously.

The minimum values for penetration recommended by the hammer manufacturer are between 1 and 2 mm/blow and, exceptionally, 0.5 mm/blow for a series of 100 blows.

API RP2A allows driving to continue to 800 blows/0.30 m or 300 blows/0.3 m on five successive heats of 0.30 m each.

The maximum driving rates are stated by the hammer manufacturers. They are generally between 35 and 55 blows per minute for diesel hammers and 45 to 60 blows per minute for hydraulic hammers.

4.2. Vibratory pile driving

Vibration applied to the head of a steel pile during driving reduces the soil-to-shaft friction. This is the principle of vibratory hammers.

The vibrations are generated by an even number of eccentrics (generally 2 or 4), revolving at the same speed in opposite directions so as to counteract the horizontal component of the resulting centrifugal force and to generate only a vertical sinusoidal force (Figure 13).

Loss of friction under the effect of vibrations is a characteristic of sands and gravels; this method is therefore efficient in soils of this kind. Penetration rates reaching 5 m/mn can be obtained in loose saturated sands. For general efficiency reasons, the penetration rate should be kept higher than 50 cm/mn. Under this limit the reduction of friction rapidly gets unsufficient. For safety reasons, the manufacturers of vibratory hammers generally recommend that this rate should not be exceeded.

In soft cohesive soils, vibration is also sometimes used but it involves considerable remoulding of the soil, and consequently a loss of bearing capacity for the piles.

Vibration is not recommended in stiff clays in which it is very inefficient.

Low- or medium-frequency electric or hydraulic hammers developing a centrifugal force of 15 to 460 tonnes, and high-frequency hammers (of some 40 Hz) developing from 40 to 280 tonnes are available.

High-frequency vibrator performance generally compares favourably with that of the much heavier low-frequency vibrators, and the high-frequency tools generate vibrations that are quickly damped by the soil and are therefore less harmful to surrounding buildings .

4.3. Vibration followed by impact driving

Pile instrumentation during vibro-driving has not been used much to date and there is as yet insufficient data to connect refusal under vibration with the dynamic resistance of the soil.

Consequently, impact driving after vibratory driving is generally necessary in order to obtain a better estimate of the pile bearing capacity.

APPENDIX 1

Determination of the H-pile bearing capacity from the results of CPT tests

Determination of the resistance of a bearing surface

Notations

The unit ultimate resistance q_r below the base of a steel pile or below the base of a lagging can be written

$$q_r = \frac{\alpha_e \alpha_b}{s} q_{r,DB} \qquad [1]$$

be:

- $q_{r,DB}$: unit base resistance calculated following the method described by de Beer (1985), in function of the equivalent diameter D_e from the bearing surface A

$$D_e = \sqrt{\frac{4A}{\pi}} \qquad [2]$$

ARBED owns a computer program for the determination of $q_{r,DB}$.

- α_b: reduction factor

$$\alpha_b = 1 - \beta\left(\frac{D_e}{d} - 1\right) \qquad [3]$$

(d = diameter of testing rod)

in sands $\alpha_b = 1.0$

in stiff fissured clays, $\beta = 0.01$ with $D_b = D_e$

- α_e: escaping factor considering the fact that a lagging is not always attached over the whole length of the pile above the bearing surface, thus enabling the soil to escape above this surface. The escape of soil gives a lower soil displacement, compared to a full and constant section pile and so a lower unit resistance below the bearing surface.

$$\alpha_e = \frac{1}{\sqrt[4]{1 + \left[\frac{D_e}{D_{e,H}} - 1\right]\left[1 - \left(\frac{\chi_{ow}}{\chi}\right)^2\right]}} \qquad [4]$$

be: $D_{e,H}$ = equivalent diameter of the section A_H of the H-beam

$$D_{e,H} = \sqrt{\frac{4A_H}{\pi}} \qquad [5]$$

χ = perimeter of the bearing surface of lagging

χ_{ow} = part of perimeter of the bearing surface which is equipped with a jacket having a height of at least $3D_e$.

- s: shape factor of bearing surface

$$s = \frac{1.3}{1 + 0.3\frac{b}{l}} \qquad [6]$$

b = width of bearing surface

l = length of bearing surface ($l \geq b$)

Case of a bearing surface without plug formation

- Considering the toe of a bare H-beam, following relationships are given

$A = A_H = A_b$

$\alpha_e = 1.0$

Because of the special shape of a H-section, the factor s has to be assumed on the safe side by introducing

$s = 1.3$

consequently

$$q_r = \frac{\alpha_b}{1.3} q_{r,DB} \qquad [7]$$

- In case of a lagging at the base of the pile, $A = A_b$ represents the bearing surface of the lagging and the total resistance $Q_{r,b}$ is given by

$$Q_{r,b} = q_{r,b} A_b \qquad [8]$$

- In case of a lagging with a bearing surface A_l at a higher level than the pile base, the total resistance $Q_{r,l}$ below this lagging is given by

$$Q_{r,l} = q_{r,l}\,(A_l - A_b) \qquad [9]$$

(for $q_{r,b}$ or $q_{r,l}$, exchange in formula [1] q_r by $q_{r,b}$ or $q_{r,l}$.)

Case of a bearing surface with partial plug formation (granular soils)

Tests in dense sand showed that, during driving, compacted soil forms a plug between the flanges of the H-section. For computation purposes, the following dimensions of the plug may be assumed

$$\text{for } a < d' < 2a : e = \frac{a}{4} \qquad [10]$$

$$\text{for } d' \geq 2a : e = \frac{1}{2}\frac{a^2}{d'} \qquad [11]$$

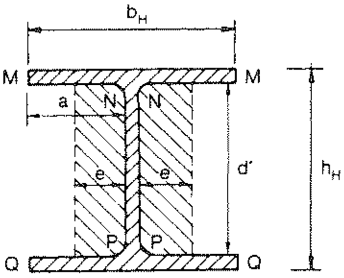


Fig. 1

The bearing surface A_p in case of plug formation can now be written

$$A_p = A_H + 2\,ed' \quad [13]$$

The unit base resistance $q_{r,DB}$ has to be determined in function of the equivalent diameter $D_{e,p}$ with

$$D_{e,p} = \sqrt{\frac{4A_p}{\pi}} \quad [14]$$

For small values of e against d', the factor s has to be assumed on the safe side by introducing

$s = 1.3$

The total resistance can now be written

$$Q_{r,p} = q_{r,p}\,A_p \quad [15]$$

In case of a lagging with a bearing surface A_l at a higher level than the pile base, the total resistance $Q_{r,l}$ below this lagging is given by the formula [9] with $A_b = A_p$.

Case of a bearing surface with soil adherance between the flanges of the H-section (cohesive soils)

The soil plug is assumed to fill the whole space MNPQ between the flanges of the H-beam (fig. 1).

Therefore, the bearing surface A_a of the H-beam may be written

$$A_a = b_H\,h_H \quad [16]$$

The equivalent diameter $D_{e,a}$ is given by

$$D_{e,a} = \sqrt{\frac{4A_a}{\pi}} \quad [17]$$

The determination of the total base resistance $Q_{r,a}$ and $Q_{r,l}$ is done by using formulae [8] and [9] and by introducing

$Q_{r,b} = Q_{r,a}$
$A_b = A_a$
$q_{r,b} = q_{r,a}$

Determination of skin friction from the results of CPT tests

Case without plug formation

For a driven full displacement pile, the unit lateral friction is given by the formula

$$f_s = \alpha_s\,f_{s,CPT} \quad [18]$$

for a steel shaft:

$\alpha_s = 0.65$ in stiff fissured clays
$\alpha_s = 1.0$ for all other soils

$f_{s,CPT}$ can be deduced from the diagram of the total friction resistance Q_{st}. Underneath the depth, at which the total thrust of the sounding apparatus has been reached, the friction is artifically decreased with a friction reducer. For estimating $f_{s,CPT}$ here, one refers to the measured cone resistance q_c.

The general shape of a Q_{st} diagram is shown on fig. 2.

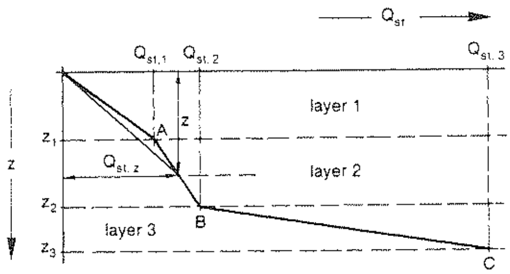


Fig. 2

The unit side friction in layer 1 is:

$$f_{s,CPT,1} = \frac{Q_{st,1}}{\pi d\,(z_1 - 0)} \quad [19]$$

In layer 2, Q_{st} increases much slowlier than in layer 1. In such cases, a unit side friction is considered, which varies with the depth z below the soil surface. At the lower level of layer 2, the unit side friction is

$$f_{s,CPT,2} = \frac{Q_{st,2}}{\pi d z_2} \quad [20]$$

At a depth z, such that

$z_1 < z < z_2$

one considers

$$f_{s,CPT,z} = f_{s,CPT,1} + \left(f_{s,CPT,2} - f_{s,CPT,1}\right)\frac{z - z_1}{z_2 - z_1} \quad [21]$$

In layer 3, the unit side frictiion is

$$f_{s,CPT,3} = \frac{Q_{st,3} - Q_{st,2}}{\pi d\,(z_3 - z_2)} \quad [22]$$

For the estimation of the unit side friction $f_{s,CPT}$ from q_c measurements, the following empirical relationships are proposed:

- in cohesionless quartz sands

$$f_{s,CPT} = \frac{q_c}{200} \text{ for } q_c \geq 20\text{ MPa} \quad [23]$$

$$f_{s,CPT} = \frac{q_c}{150} \text{ for } q_c \leq 10\text{ MPa} \quad [24]$$

For intermediate values of q_c, the value of $f_{s,CPT}$ is calculated by linear interpolation between the value $q_c/200$ and $q_c/150$.

- in cohesive layers, q_c and $f_{s,CPT}$ follow a relationship which is a function of the rigidity index and thus of the value of q_c.

$f_{s,CPT}$ in function of q_c is shown in the table below

q_c (MPa)	.075	.2	.5	1	1.5	2	2.5	3	>3
$f_{s,CPT}$ (kPa)	5	10	18	31	44	58	70	82	$\frac{q_c}{36.6}$

In case of a steel H-pile without lagging, the soil displacement is less important than for a full section pile. Therefore, the introduction of a reduction factor is required

$$f_{s,H} = \frac{\alpha_s\, f_{s,CPT}}{1.10} = \frac{f_s}{1.10} \qquad [25]$$

In case of a H-pile with lagging, the soil is strongly displaced and the value $f_{s,l} = f_s$ is introduced for the computation of the friction over the height of the lagging. Above the lagging, the friction is strongly decreased over the part of the perimeter, where a protrusion exists. For that part of the beam, a unit friction of 20 kPa is considered. Over the part where no protrusion exists, the reduced side friction in layer i is determined by

$$f_{s,i}^{red} = \alpha_s \left[f_{s,CPT,i} - (f_{s,CPT,i} - 20)\left(1 - \frac{\chi'}{\chi}\right) \right] \qquad [26]$$

with χ = perimeter of the bearing surface of lagging
χ' = total length above lagging over which no protrusion exists.

By multiplying the values of $f_{s,i}$ with the corresponding lateral surfaces for which they are valid, the individual skin friction resistances $Q_{s,i}$ are obtained. The sum of these individual resistances gives the total skin friction resistance $Q_{s,r}$ of the pile.

Case with partial plug formation (granular soils)

When, during driving, a plug is formed between the flanges from a certain depth on, the soil is strongly displaced over the height of the plug and formula [18] is used.

The height of the plug is assumed to be

$$l_p = 10\, D_{e,p} \qquad [27]$$

Above the plug, the soil between the flanges is strongly released; consequently, the unit friction $f_{s,i}^{int}$ of the soil between the flanges in layer i has to be reduced

$$f_{s,i}^{int} = \frac{1}{2}\alpha_s\, f_{s,CPT,i} \qquad [28]$$

$$f_{s,i}^{ext} = \alpha_s\, f_{s,CPT,i} \qquad [29]$$

The friction above the plug in layer i is now determined as follows:

$$Q_{s,i}^{int} = f_{s,i}^{int}\, \chi_i^{int}\, L_i \qquad [30]$$

$$Q_{s,i}^{ext} = f_{s,i}^{ext}\, \chi_i^{ext}\, L_i \qquad [31]$$

with χ^{int} = $2h_H$ (h_H = height of section)
χ^{ext} = $2b_H$ (b_H = width of section)
L_i = height of considered soil layer

Case with soil adherance between the flanges (cohesive soils)

For the determination of the unit lateral friction between soil and steel, formula (25) is used. Along the line MQ on fig. 1, the friction soil to soil is determined by using formula [25] with α_s = 1.0. For piles with lagging, the unit friction on the lagging is given by formula [18].

Above lagging, the unit friction $f_{s,i}^{int}$ at the part of the perimeter where a protrusion exists, is given by

$$f_{s,i}^{int} \begin{cases} \leq 20\ \text{kPa} \\ \leq \alpha_s\, f_{s,CPT,i} \end{cases} \qquad [32]$$

At the part of the perimeter above lagging, where no protrusion exists, the unit friction $f_s^{ext,\,red}$ may be expressed by formula [26].

Determination of the load/movement behaviour of a driven H-pile from the results of CPT tests

Assumptions

The aim of the proposed method is the prediction of an upper safe limit of the movement of a single pile under the working load. The following general data are helpful:

1. The rupture load around the pile base of a driven pile is only reached when the movement of the pile base is equal or larger than 10% of the equivalent pile diameter.

$$s_{b,r} \geq 0.10\ D_e \qquad [33]$$

2. The static load tests show that for loads which are much smaller than the rupture load, there is a quasi linear relationship between loads and movements. It should however be erroneous to deduce from this linearity that the soil is a material characterized by a unique and constant modulus of elasticity.

3. The mantle friction is mobilized for much lower values of the relative movement than the base resistance. In most cases the mantle friction of a driven pile is totally mobilized for a relative movement of the base in the order of 0.01 D_e.

When the variaton of Q_b is drawn versus the relative movement at the base s_b/D_e, one gets in reality a curve OPQQ' on fig. 3, with a quasi linear part OP, a curved part PQ and eventually a quasi linear part QQ'.

The variation of the mantle friction Q_s versus s_b/D_e is as shown by the curves ABC or ABC'. The part OA depends on the residual load and on the elastic deformation of the pile. For sake of simplicity and safety, the real curve ABC is replaced by the broken line OEE'.

It is assumed that the predicted mantle rupture load $Q_{s,r}$ is obtained for 0.01 D_e or 0.01 b (b = width of H-section), whichever is larger.

For the base resistance, the real curve OPQQ' is replaced by the curve ORSS'. It is assumed that the predicted rupture load $Q_{b,r}$ corresponds to a relative movement of the base s_b equal to 10% of the equivalent diameter D_e.

It is further assumed that there is a linear behaviour for all loads Q_b smaller than

$$Q_{b,ref} = \frac{Q_{b,r}}{3} \qquad [34]$$

Between the points R and S, a fluent curve is drawn which is tangent to both straight lines OR and SS'.

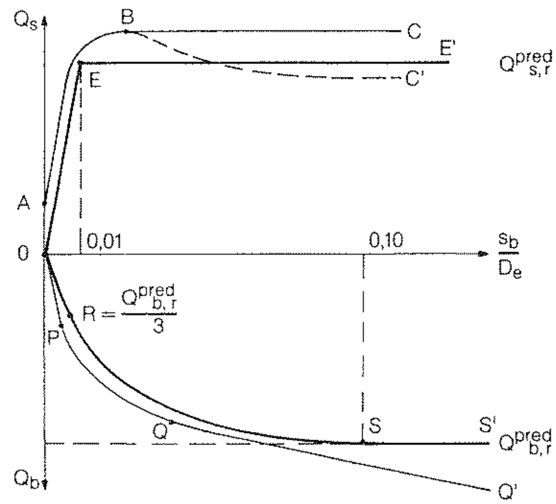


fig. 3: **Mobilization of base resistance Q_b and skin friction Q_s**

For determining the linear relationship OR, two different approximate ways are at disposal:

1. The first way is to assume the soil underneath the base as an homogeneous, isotropic, linearly elastic material characterized by a constant modulus of elasticity E_s and a constant Poissons ratio μ.

For the case of an infinitely rigid circular loading with a section A, the settlement s_b is given by

$$q_b = K_s\ s_b \qquad [35]$$

$$\text{with } K_s = 1.13 \frac{E_s}{1-\mu^2} \cdot \frac{1}{\sqrt{A}} \qquad [36]$$

A relationship between E_s and q_c in case of virgin compression is

$$E_s \geq 3/2\ q_c \qquad [37]$$

For sand, one finds in the literature μ = 0.3; for clay with immediate settlement at constant volume μ = 0.5 and after final consolidation μ = 0.1.

In case of driven piles, the settlement related to the subsequent static loads do not correspond to a virgin loading, but to reloading.

Under reloading conditions, the deformation modulus E'_s is about 10 times larger than the modulus of virgin loading in case of pure quartz sands, and about 3 times in case of clays. Consequently for driven piles:

$E'_s \geq 15\, q_c$ for sand [38]

$E'_s \geq 4.5\, q_c$ for clay [39]

However, these formulae should only be applied if the values of q_c are constant over the so called compressive thickness underneath the level of the pile base.

2. In the second and recommended way it is admitted that, for loads on the base smaller than one third of the rupture load of the soil, the settlement due to lateral deformation of the soil can be neglected. So, for the load $Q_{b,r}/3$, the settlement can be computed in an oedometric way. In case of virgin loading of the soil, the settlement $s_{b,ref}$ under the reference load $Q_{b,ref}$ is given by

$$s_{b,ref} = \sum_{0}^{z_c} \delta h_i = \sum_{0}^{z_c} \frac{h_i}{C_i} \ln \frac{\sigma'_{z,i} + p'_{t,i}}{p'_{t,i}} \qquad [40]$$

in which:

- C_i = constant of compressibility of the layer with a thickness h_i
- $\sigma'_{z,i}$ = the effective stress increase in the layer i caused by the load on the pile base
- $p'_{t,i}$ = the original effective stress in the layer i

The stress increases $\sigma'_{z,i}$ are not calculated by the law of Boussinesq, but by the law of Buisman, which gives larger stress concentrations. These stress increases are computed along the vertical of the singular points. The location of the singular circle

$$R_S = 1/2\, R_e \sqrt{2} \qquad [41]$$

and the variation of the influence factor $i = \sigma'_z / q_b$ versus the relative depth z/D_e are given on fig. 4.

In case of sand: $C_i \geq 2\, q_{c,i}/p'_{t,i}$ [42]

In case of clay: $C_i \geq 1.65\, q_{c,i}/p'_{t,i}$ [43]

In case of reloading (driven piles), C_i has to be replaced in the formula [40] by the recompression constant A_i. For pure quartz sand:

$A_i \geq 10\, C_i = 20\, q_{c,i}/p'_{t,i}$ [44]

$A_i \geq 3\, C_i = 5\, q_{c,i}/p'_{t,i}$ [45]

For other soils, for instance clayey sands, glauconitic sands, calcareous sands, samples should be taken from the considered layer and brought in the laboratory at about their field density. In an oedometer test, the sample is submitted to a loading and reloading cycle, and the factors A_{oed} and C_{oed} are determined.

One calculates further

$$A_i = \frac{A_{oed}}{C_{oed}} C_i \qquad [46]$$

For glauconitic sands, calcareous sands and clayey sands

$$A_i \geq \frac{A_{oed}}{C_{oed}} 2 \frac{q_{c,i}}{p'_{t,i}}; \qquad [47]$$

for sandy clays

$$A_i \geq \frac{A_{oed}}{C_{oed}} 1.65 \frac{q_{c,i}}{p'_{t,i}} \qquad [48]$$

The formulae [47] and [48] are written as an inequality. In the choice of the values to be introduced in the computations, attention should be paid to the two following facts:

1. in a heterogeneous layer, the deformations are not governed by the mean deformability parameters, but by the properties of the lesser deformable parts.

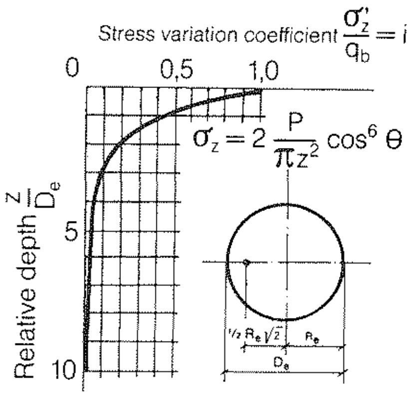


***Fig. 4:* Variation of the stress variation coefficient i vs. depth foll. Buisman**

2. in anisotropic layers, when the deformability in the horizontal direction is less than in the vertical direction (overconsolidated layers), the values of A_i are larger than given by the equality of the formulae [47] and [48].

The compression of the soil is considered to be limited to the so called „compressed thickness z_c", which extends to a depth where:

$$\frac{\sigma'_z}{p'_{t,i}} = \frac{i\, q_b}{p'_{t,i}} \simeq 0.1 \qquad [49]$$

Movement of the pile head

As one disposes over a relationship between the movement at the base s_b and the load at the base Q_b and also over a relationship between s_b and the load Q_s taken by mantle friction, one gets a relationship between s_b and a total load Q with

$$Q = Q_b + Q_s \qquad [50]$$

The following step is to determine the movement of the pile head, with the relations

$$s = s_b + s_e \qquad [51]$$

$$s_e = \frac{Q_b\, L}{E_p\, A} + \frac{1}{E_p\, A} \Sigma\, Q_{s,i}\, L_i \qquad [52]$$

with:
- s_e = elastic shortening of the pile
- L = length of the pile
- $Q_{s,i}$ = friction force in the pile at the level of the partial length L_i.

It is supposed that at all depths

$$\frac{Q_{s,i}}{Q_{s,r,i}} = \frac{Q_s}{Q_{s,r}} \qquad [53]$$

Finally, the load /movenment curve Q = f (s) can be drawn.

Comments on the proposed procedures

1) With the values of q_r and f_s, deduced in the indicated way from the results of the CPT tests, it becomes possible to make a prediction of the bearing capacity of a steel H-pile with and without lagging.

   As beforehand one ignores, whether or not a plug is formed (granular soils), or whether or not adherance occurs (cohesive soils), the calculations are successively made in the hypothesis of the absence and of the formation of a plug or adherance. For the sake of safety, the lowest value, obtained from the two computations, is used. For non conventional types of laggings, appropriate assumptions have to be made.

   It is important to note that the proposed calculation method is predicting the conventional rupture load of the pile.

   In order to determine the allowable working load of the pile, appropriate safety factors have to be applied on the calculated base resistance and skin friction.

2) The predicted load/movement behaviour of a single H-pile under an axial compression load may be compared to the measured results of a static load test. As implied however by the nature of the problem, no analysis performed on a single pile can be regarded as generally valid for a whole foundation project because, even on one site, the pile behaviour varies from one location to another. Consequently, the proposed procedure relies mainly on empirical relationships, allowing for a crude and safe prediction. It is also clear that the predicted movement of a single pile does not necessarely correspond to the movement of the foundation as a whole. Here, a realistic assessment has to be made on the group effect of the designed structure.

References

DE BEER, WEBER (1985). Belgian Geotechinical volume published for the 1985 golden jubilee of the international society for soil mechanics and foundation engineering, p. 9–25. Leuven: Ceuterick.

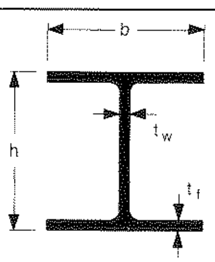
WEBER, DE BEER (1988). Prediction of the load/settlement behaviour of steel H-piles from the results of CPT tests. Proceedings of ISOPT-1. Penetration testing 1988 volume 2, p. 1043–1049.

APPENDIX 2

Examples of methods for calculating the bearing capacity of HP piles

Example 1 - Test with the Ménard pressuremeter

Calculation of the bearing capacity of a HP 360 × 109 pile using the pressuremeter method of Fascicule 62 - Titre V (ref [1]).



HP 360 x 109

h = 346.4 mm
b = 370.5 mm
$t_w = t_f$ = 12.9 mm
G = 109 kg/m
A = 138.9 cm²
A_{tot} = 1283 cm²
P = 2.123 m

The log of the pressuremeter test is given in Figure 3 (Pressuremeter A).

- Soil conditions:

  from 0 to 3 m = clayey silt (γ' = 6 kN/m³)
  from 3 to 18 m = soft to stiff clay (γ' = 7 kN/m³)

- ultimate resistance under the pile toe at z = 15 m embedment

  – Equivalent ultimate pressure
  depth of pile in the bearing layer:
  h = 12 m
  a = 0.5 m
  b = inf{a, h} = a = 0.5 m

$$p^*_{le} = \frac{1}{b + 3a} \cdot \int_{D-b}^{D+3a} p^*l(z)dz$$

with D = 15 m
that is

$$p^*_{le} = \frac{1}{2} \int_{14.5}^{16.5} p^*l(z)\,dz$$

according to Table 1: p^*_{le} = 1.614 MPa.

z (m)	p_l (MPa)	p_0 (MPa)	p^*_l (MPa)
14.5	1.89	0.194	1.69
15.0	1.58	0.203	1.37
16.0	2.00	0.216	1.78
16.5	2.00	0.225	1.77

***Table 1:* Results of pressuremeter tests – pressuremeter A (Fig. 3)**

– bearing capacity factor
$k_p = 1.5$

– reduction factor for the toe section:
$\rho_p = 0.5$

– total area of the pile:
A = 0.1283 m²

From which:

$$Q_{pu} = \rho_p \cdot A \cdot q_u = \rho_p \cdot A \cdot k_p \cdot p^*_{le}$$
$$= 0.5 \cdot 0.1283 \cdot 1.5 \cdot 1.614$$

$\mathbf{Q_{pu}}$ **= 155 kN**

- ultimate resistance from skin friction

– unit ultimate friction

from 0 to 5 m = p^*_l = 0.3 to 0.8 MPa (clayey silt)
curve Q_1 : q_s = 30 kPa

from 5 to 8 m = p^*_l = 0.9 to 1.2 MPa (clay)
curve Q_2 : q_s = 60 kPa

from 8 to 15 m = p^*_l = 1.5 to 1.8 MPa (clay)
curve Q_2 : q_s = 75 kPa

– reduction factor for the pile perimeter:
$\rho_s = 1.0$

– developed perimeter of the HP pile:
P = 2.123 m

From which:

$$Q_{su} = \rho_s \cdot P \cdot \int_0^{15} q_s(z)\,dz$$

$$= 1.0 \cdot 2.123 \cdot (5 \cdot 30 + 3 \cdot 60 + 7 \cdot 75)$$

$\mathbf{Q_{su}}$ **= 1815 kN**

- ultimate compressive load on an isolated HP pile:

$$Q_u = Q_{pu} + Q_{su}$$

$\mathbf{Q_u}$ **= 1970 kN**

- allowable working load:

$$Q_{all} = \frac{Q_c}{1.4} = \frac{0.7 \cdot Q_u}{1.4} = \frac{Q_u}{2}$$

$\mathbf{Q_{all}}$ **= 985 kN**

Example 2.1. - Static penetration test (CPT)

Calculation of the bearing capacity of a HP 360 × 109 pile using the static penetrometer method of Fascicule 62 - Titre V (ref [1]).

The log of penetrometer tests is given in Figure 2 (CPT-B).

- Soil conditions:

  from 0 to 3.4 m = soft clay ($\gamma' = 6$ kN/m^3)

  from 3.4 to 6.0 m = clayed sand ($\gamma' = 7$ kN/m^3)

  from 6.0 to 16.2 m = compact sand ($\gamma' = 10$ kN/m^3)

- ultimate resistance under the pile toe at the depth z = 14 m

  – equivalent toe resistance at z = 14 m

  depth of pile in resistant layer:

  h = 8 m

  a = 0.5 m

  b = inf{a, h} = a = 0.5 m

$$q_{ce} = \frac{1}{b+3a} \cdot \int_{D-b}^{D+3a} q_c(z)dz$$

in which D = 14 m

that is

$$q_{ce} = \frac{1}{2} \cdot \int_{13.5}^{15.5} q_c(z)\,dz$$

according to Table 2: q_{ce} = 15.08 MPa.

z (m)	q_c (MPa)
13.5 to 15.5 15.5	16 to 14.2 14.2

Table 2: **Results of CPT tests – CPT-B (Fig. 2)**

– bearing capacity factor

  k_c = 0.50 mean of sands B and C)

– reduction factor for the toe cross-section:

  $\rho_p = 0.75$

– total area of pile:

  A = 0.1283 m^2

From which:

$$Q_{pu} = \rho_p \cdot k_c \cdot q_{ce} \cdot A$$
$$= 0.75 \cdot 0.5 \cdot 15.08 \cdot 0.1283$$

$\mathbf{Q_{pu} = 725\ kN}$

- ultimate resistance from skin friction

– unit ultimate friction

  from 0 to 3.4 m : $q_s = 0$

  from 3.4 to 14.0 m : $q_s = q_c/300$

z (m)	q_c (MPa)	q_s (kPa)
3.4–6.0 6.0–9.5 9.5–10.5 10.5–14.0	3.5 16.0 22.2 15.6	11.7 53.5 74.0 52.0

Table 3: **Determination of the unit skin friction – CPT-B (Fig. 2)**

– reduction factor for the pile perimeter:

  $\rho_s = 1.0$

– developed perimeter of the HP pile:

  P = 2.123 m

From which:

$$Q_{su} = \rho_s \cdot P \cdot \int_0^{14} q_s(z)\,dz$$

$= 1.0 \cdot 2.123 \cdot (11.7 \cdot 2.6 + 53.3 \cdot 3.5 + 74.0 \cdot 1.0 + 52.0 \cdot 3.5)$

$\mathbf{Q_{su} = 1004\ kN}$

- ultimate compressive load on an isolated HP pile:

$$Q_u = Q_{pu} + Q_{su}$$

$\mathbf{Q_u = 1729\ kN}$

- allowable working load:

$$Q_{all} = \frac{Q_c}{1.4} = \frac{0.7 \cdot Q_u}{1.4} = \frac{Q_u}{2}$$

$\mathbf{Q_{all} = 865\ kN}$

Example 2.2. – Static penetration test (CPT)

Calculation of the bearing capacity of a HP 360 × 109 pile using the static penetrometer method described in appendix 1.

The log of the CPT test is the same as for example 2.1. and is given in figure 2 (CPT-B).

- Soil conditions:

See example 2.1. For computation purposes, the assumptions concerning granular materials are considered over the embedment depth of the pile.

The values $q_{r,DB}$ are calculated via computer following the method described by de Beer (ref [15]) in function of the equivalent diameter of the pile. The results are shown in figure 2.

- Case without plug formation

$$D_e = \sqrt{\frac{4 \cdot 0.01389}{\pi}} = 0.133\ \text{m}$$

The unit base resistance $q_{r,DB}$ is given in figure 2. At 14 m depth $q_{r,DB}$ = 14.52 MPa.

For the bare H-pile in sands, $\alpha_e = \alpha_b = 1.0$ and the shape factor s = 1.3.

– unit ultimate base resistance at z = 14 m

$$q_r = \frac{\alpha_e \cdot \alpha\, b}{s} q_{r,DB}$$

$$= \frac{1.0 \cdot 1.0}{1.3} \cdot 14520 = 11169 \text{ kPa}$$

– ultimate base resistance

$Q_{r,b} = q_r \cdot A = 11169 \cdot 0{,}01389$
$Q_{r,b} = 155$ kN

– skin friction

The unit skin friction is assessed from the q_c values for sand.

The average q_c value over the depth of 14 m is
$q_c = 10.77$ MPa

and the empirical relation for the unit skin friction becomes

$$f_{s,CPT} = \frac{10770}{153.5} = 70.2 \text{ kPa and}$$

$$f_{s,H} = \frac{70.2}{1.10} = 63.82 \text{ kPa}$$

From which

$Q_{s,r} = f_{s,H} \cdot P = 63.82 \cdot 2.123 \cdot 14$
$Q_{s,r} = 1897$ kN

- Case with partial plug formation

The width e of the plug is

$$e = \frac{a}{4} = \frac{1}{4} \cdot \frac{(b-t_w)}{2}$$

e = 4.47 cm

The partially plugged base area becomes

$A_p = A + 2e\,(h-2 \cdot t_f)$
$= 138.9 + 2 \cdot 4.47\,(34.64 - 2 \cdot 1.29)$
$A_p = 425.5 \text{ cm}^2$

$$D_{e,p} = \sqrt{\frac{4 \cdot 425.5}{\pi}} = 23.3 \text{ cm}$$

The length of the plug is

$l_p = 10\, D_{e,p}$
$l_p = 10 \cdot 23.3 = 233$ cm

The perimeter of the partially plugged pile is

$P_p = P - 4 \cdot e = 2.123 - 4 \cdot 0.0447$
$P_p = 1.944$ m

unit ultimate base resistance at z = 14 m

from figure 2: $q_{r,DB} = 14.36$ MPa

$$q_{r,p} = \frac{\alpha_e \cdot \alpha_b}{s} q_{r,DB}$$

$$= \frac{1.0 \cdot 1.0}{1.3} \cdot 14360$$

$q_{r,p} = 11.046$ kPa

– ultimate base resitance

$Q_{r,p} = q_{r,p} \cdot A_p$
$= 11.046 \cdot 0.04255$
$A_{r,p} = 470$ kN

– skin friction

Over the height of the plug, the average q_c value is
$q_{c,p} = 14.94$ MPa

and the empirical relation for the unit skin friction becomes

$$f_{s,CPT,p} = \frac{14940}{174.7} = 85.52 \text{ kPa}$$

Above the plug, the average q_c value is
$q_c = 9.913$ MPa

and the empirical relation for the unit skin friction becomes

$$f_{s,CPT} = \frac{9910}{150} = 66.07 \text{ kPa}$$

from which

$Q_s = f_{s,CPT,p} \cdot P_p \cdot l_p + f_{s,CPT} \cdot (z-l_p)\,(0.5 \cdot 2 \cdot h + 2 \cdot b)$
$= 85.52 \cdot 1.944 \cdot 2.33 + 66.07\,(14-2.33)\,(0.5 \cdot 2 \cdot 0.3464 + 2 \cdot 0.3705)$
$= 387.4 + 838.4$
$Q_s = 1226$ kN

- The assumption with partial plug formation gives the lowest capacity. Following a proposal from Prof. de Beer, the allowable working load can be calculated as follow:

$$Q_{all} = \frac{1}{1.4} = \left(\frac{Q_b}{1.5} + \frac{Q_s}{1.3}\right)$$

$$= \frac{1}{1.4} = \left(\frac{470}{1.5} + \frac{1.226}{1.3}\right)$$

$\mathbf{Q_{all} = 897\ kN}$

Example 3 – Standard penetration test (SPT)

Calculation of the bearing capacity of a HP 360 × 109 pile, using Meyerhof's method based on SPT tests.

The SPT test log is given in Figure 2 (SPT-B).

- Soil conditions:

  from 0 to 3.4 m = soft clay ($\gamma' = 6$ kN/m³)

  from 3.4 to 6.0 m = clayey sand ($\gamma' = 7$ kN/m³)

  from 6.0 to 16.2 m = compact sand ($\gamma' = 10$ kN/m³)

- ultimate resistance under the pile toe at z = 14 m of embedment

– total area of the pile:
  $A = 0.1283 \text{ m}^2$

– reduction factor for the toe section:
  $\rho_p = 0.75$

– equivalent diameter at the toe:

$$D_e = \sqrt{\frac{4 \cdot \rho_p \cdot A}{\pi}}$$
$= 0.350$ m

$2D_e = 0.70$ m $\quad 10D_e = 3.50$ m

– $N_1 = \inf \{N_{(14m)}, N_{(14m + 2De)}\}$
$= \inf \{48, 43\} = 43$

– $N_2 = \text{average} \{N_{(14m)} \longrightarrow N_{(14m + 10De)}\}$
$= 47$

– $N = \dfrac{N_1 + N_2}{2} = 45$

For saturated fine sands with N > 15, N is replaced by the reduced value:

$N_{red} = 15 + 0.5 (N - 15) = 30$

$q_u = 400 \cdot N_{red} = 12000$ kPa

$Q_{pu} = \rho_p \cdot A \cdot q_u$
$= 0.75 \cdot 0.1283 \cdot 12000$

$\mathbf{Q_{pu} = 1155}$ **kN**

- Ultimate resistance from skin friction:

$- N' = \dfrac{\Sigma N_i \cdot z_i}{\Sigma z_i}$

where

z_i = thickness of layer i (or distance between 2 measurements)

N_i = mean of N_{SPT} in layer i

that is

N' = 27.33 between 0 and 14 m.

For fine saturated sands with N' > 15, we substitute the reduced value:

$N'_{red} = 15 + 0.5(N' - 15) = 21$

– mean ultimate unit friction:

$q_s = 2N'_{red} = 42$ kPa

– reduction factor for the pile perimeter:

$\rho_s = 1.0$

– developed perimeter of the HP pile:

P = 2.123 m

Using the same hypothesis as example 2.2., that is q_s = 0 between 0 and 3.4 m, we obtain:

$Q_{su} = \Sigma (q_{si} \cdot h_i) \cdot \rho_s \cdot P$
$= (14 - 3.4) \cdot 42 \cdot 1 \cdot 2.123$

$\mathbf{Q_{su} = 945}$ **kN**

- Ultimate compressive load on an isolated HP pile:

$Q_u = Q_{pu} + Q_{su}$

$\mathbf{Q_u = 2100}$ **kN**

- Allowable working load with a safety factor of 2:

$Q_{all} = \dfrac{Q_u}{2}$

$\mathbf{Q_{all} = 1050}$ **kN**

This value is more optimistic than the value obtained by the CPT method in examples 2.1. and 2.2.

It will be noted that the equation $N = \dfrac{q_c}{400}$ is in general approximate.

A more realistic equation can be obtained from the soil grain size distribution (not available in the present case).

Example 4 – Laboratory tests

Calculation of the bearing capacity of a HP 360 × 109 pile, using the API RP2A method (ref. [2]) based on geotechnic parameters from laboratory tests.

The log of the parameters is that of borehole B2 as given in Figure 1.

The calculation method proposed by API RP2A is for tubular piles. Therefore we have used the geometric characteristics of a tubular pile which is equivalent to the HP pile in question.

There are two hypotheses:

– The soil sticks to the H pile: the H pile is modelled on a closed tubular pile with the same cross-section at the toe as the H pile. The equivalent diameter is thus defined by:

$D_e = \sqrt{\dfrac{4A}{\pi}} = 0.404$ m

where

A = the total pile section = 0.1283 cm²;

– The soil does not stick to the H pile: the pile is modelled on a tubular pile of the same steel cross-section with the same frictional surface (per linear metre). The diameter D_e and the wall thickness t of the equivalent tube are given by:

$P = \pi \cdot D_e + \pi \cdot (D_e - 2t)$

$A = \pi \cdot \dfrac{D_e^2}{4} - \pi \cdot \dfrac{(D_e - 2t)^2}{4}$

where

P = total pile perimeter = 2.123 m

A = steel section of the HP pile = 0.01389 m²

We obtain:

D_e = 0.3510 m

t = 0.0131 m

The limiting wall friction in the clay beds was calculated by using the two methods for calculating the adhesion coefficient, given respectively in the versions 1986 and 1992 of API RP2A (ref. [2]) (see Fig. 5).

The results are given as follows:

Fig. 7: Unit skin friction

Fig. 8: Unit toe resistance

Fig. 9: Bearing capacity in compression, using API RP2A, 86

Fig. 10: Bearing capacity in compression, using API RP2A, 92

The ultimate bearing capacity Q_u for each depth is defined as the smaller of the two values obtained for cases where the soil either sticks to the steel or does not.

The allowable bearing capacity Q_{all} is deduced from the ultimate bearing capacity by using a safety factor of 2.

For example, at z = 20 m we obtain:

API RP2A-86: $Q_u = 1960$ kN

$Q_{adm} = 980$ kN

API RP2A-92: $Q_u = 1948$ kN

Qadm = 974 kN

cause of difficulties in piling (premature refusal, damage to piles). Methods of investigation aimed at providing a continuous soil or ground profile should be systematically researched. The static penetrometer is the most suitable tool because it also provides parameters of direct use in dimensioning the foundation. Failing this, either dynamic penetrometer testing or the continuous recording of drilling variables should be considered. Representative soil samples for the appropriate laboratory tests should be insisted upon.

A realistic appreciation of the dynamic behaviour of soil is critical to the development of a driving forecast. The geotechnician has a prime role in stage 1. His experience of driving problems and his knowledge of procedures are usually the best guarantees of a reliable result.

The results of stage 2 depend heavily on hypotheses about the true efficiency of the hammer used. Considerable progress has been made in the last few years on this subject, thanks to instruments which can directly measure the energy transmitted to the pile.

APPENDIX 4

FIGURES

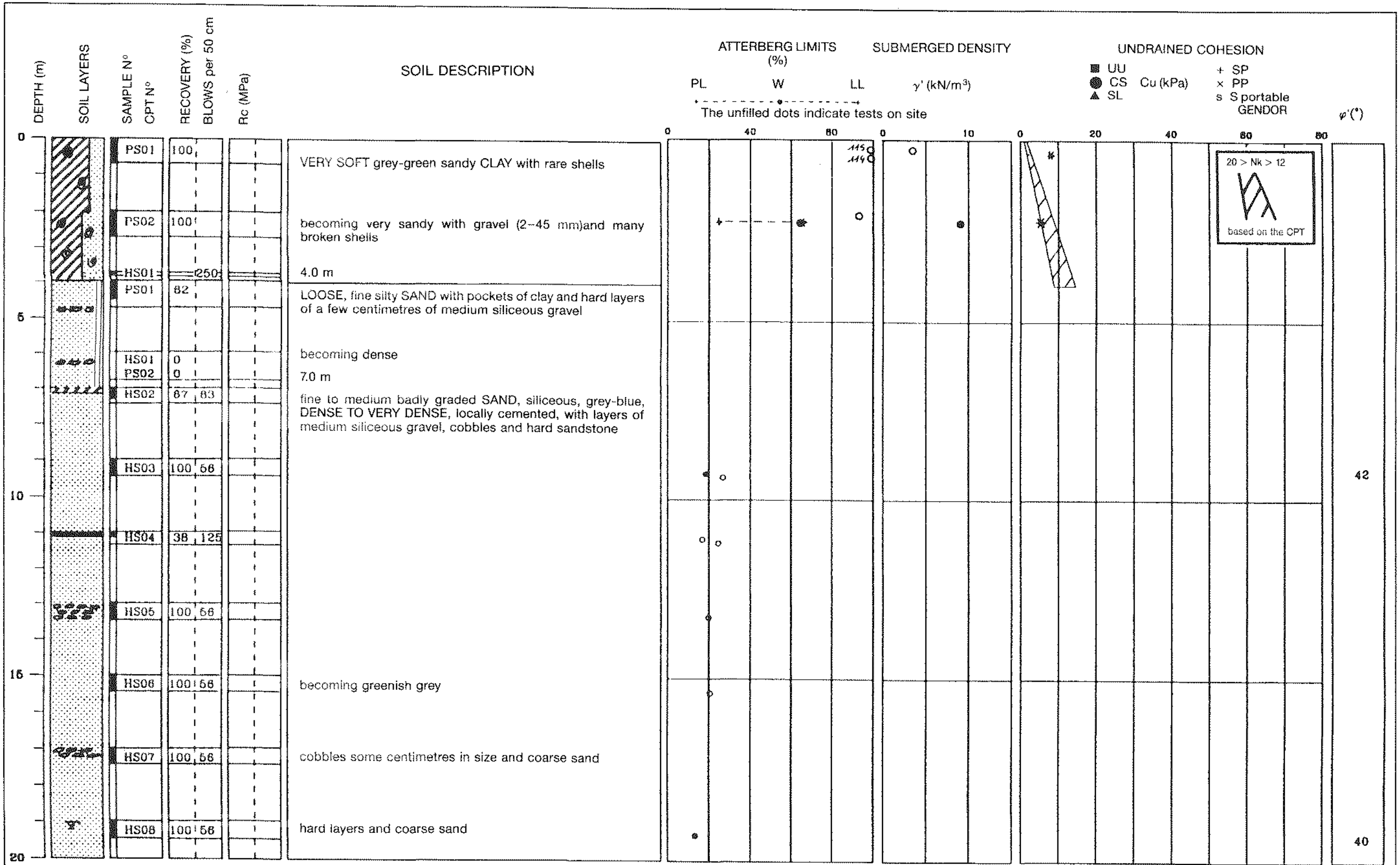


Figure 1: EXAMPLE OF A BOREHOLE DRILLING LOG

APPENDIX 3

Forecast of pile driving

The driving forecast should be part of the engineering design of the foundations. Well-tested forecasting methods now exist to deal with the subject systematically and in detail.

The foundation design stage uses soil data from the geotechnical investigations and loads carried by the structure to decide on the number of piles, the steel cross-section, its thickness, the embedment of piles in the ground and the type of steel. The checking of the bearing capacity of the piles is an important part of this stage, in conformity with the regulations in force.

The driving forecast is made before the beginning of site work, and is based on the soil data available and the geometry of the piles proposed during the foundation design stage. Its purpose is to:

- define the procedures and the materials to be used: to choose the hammer type (diesel, hydraulic) as well as size and number, etc.;
- check whether the power required to overcome the soil resistance matches the integrity of the pile (cross-section adequate for transmission of the energy to the toe, and the level of dynamic stress, etc.);
- establish „refusal criteria" or procedures to be applied in case of premature refusal (drilling beneath the toe, etc.). The concept of such a criterion is discussed below.

Any driving study should include two separate but complementary stages, followed by a synthesis stage.

◆ Stage 1: Determination of soil resistance to driving

The SRD, soil resistance to driving, has to take account of the mechanical properties already determined (geotechnical reconnaissance, pile type, steel cross-section with or without lagging, etc.) as well as of special phenomena such as the possible formation of a soil plug.

In practice this involves a range of values and defining minimum and maximum resistances to continuous driving, as well as a consideration of the consequences of soil recovery after driving has been halted (see Fig. 11a).

Calculation of the SRD involves the geotechnician's knowledge and experience. The procedures used are inspired by the recommendations of Toolan et al (1977), Stevens et al (1982) and Puech et al (1990). Obviously they gain in effect when backed by sound information about the soil in question, as well as long experience of the driving method in this soil, and furthermore when verified against properly documented cases from practice.

◆ Stage 2: Simulation of the behaviour of the hammer-pile-soil system

This simulation is made with tested numerical programs available on microcomputers and based on the theory of unidimensional propagation of stress waves. The programs are derived from the solution of the wave equation (WEAP, TTI, BATTPILE, etc.) or from the theory of characteristics (ADIG, TNOWAVE, etc.).

The result is stated in the form of a system-specific relationship between the soil resistance to driving and the number of blows needed to drive the pile 50 cm (SRD-N/50 cm curves, see Fig. 11b). These curves relate to several depths of embedment and several hammer-specific values of efficiency, as well as the distribution of the resistance between the toe and wall friction. An additional estimate is obtained of the maximum compressive and tensile dynamic stresses in the pile during the passage of the stress wave.

◆ Stage 3: Graphs for driving forecasts

The comparison of the results of the simulation with those from the resistance curves produces curves giving the expected number of blows per 50 cm as a function of the penetration (Fig. 11c) and are used to evaluate the drivability. It is generally accepted that actual refusal occurs between 250 and 350 blows per 50 cm, depending on the hammer.

It is advisable to choose a hammer which, in continuous driving, keeps the number of blows per 50 cm below 150; experience shows that the reliability of current forecasting methods decreases for high values of N. For clays with rapid recovery, this criterion could be lowered to permit effective recommencement of driving after a stoppage of several hours (breakdown or change of hammer, etc.).

◆ Stage 4: Reliability of the driving forecasts

The reliability of a driving forecast depends basically on the following three conditions.

Fundamental is the significance of the information about the location and type of soil layers. It is essential that after the soil investigation the geotechnician should be able to build up the continuous stratigraphic profile, including soil mechanical properties. The presence of a hard layer – even a thin one such as sandstone or limestone – not discovered comprehensively by the drilling, is the main

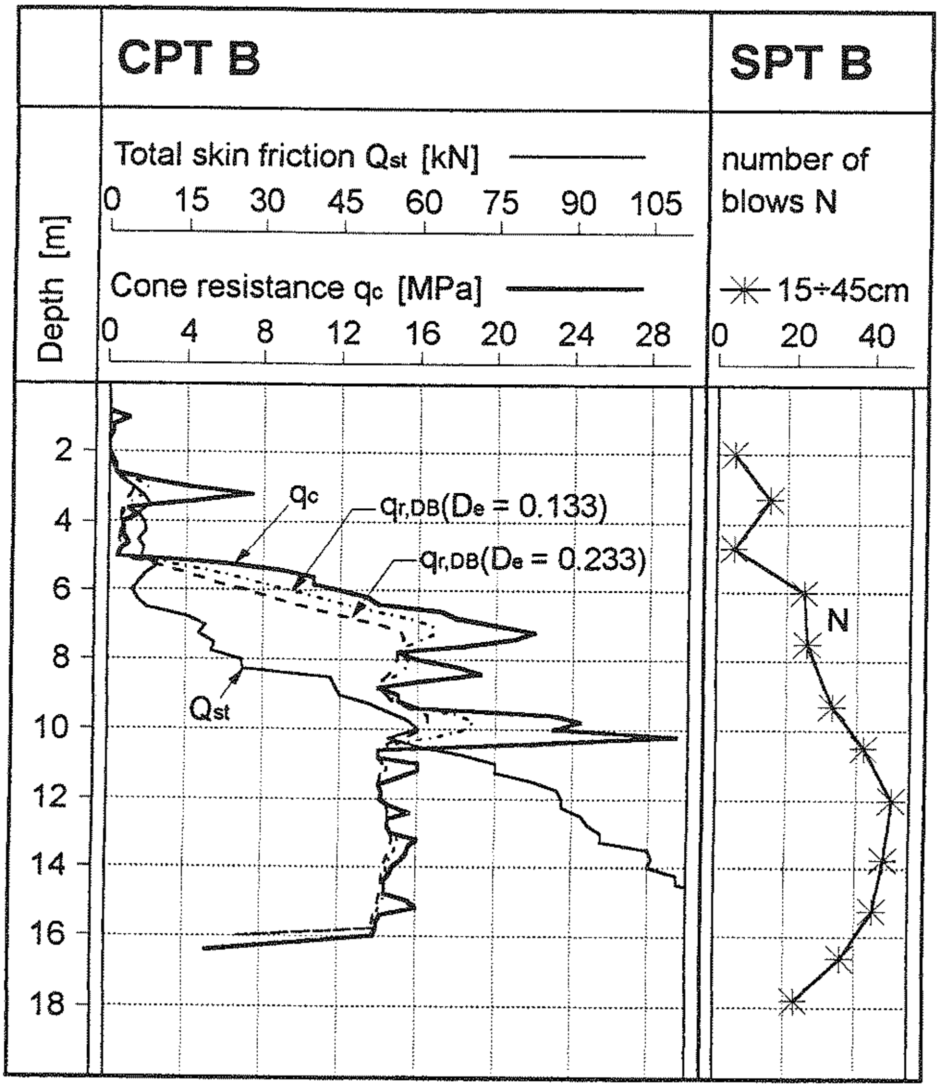


Figure 2: RESULTS FROM A CPT AND SPT TEST

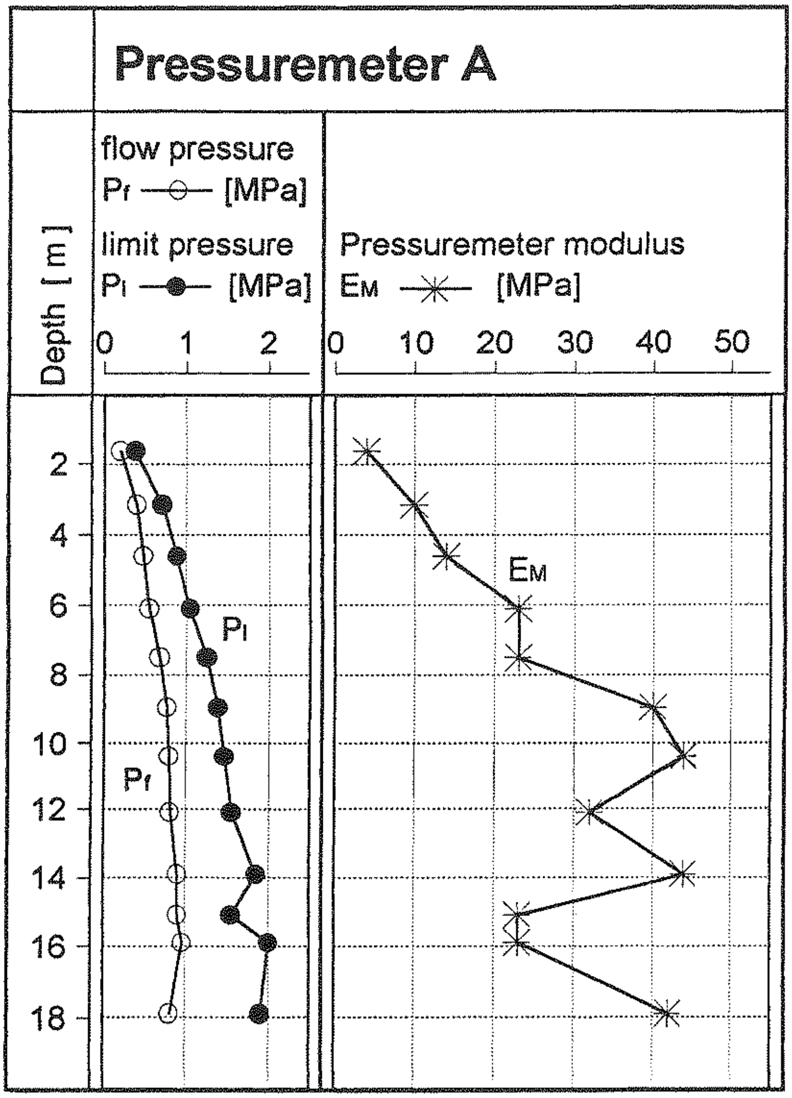


Figure 3: RESULTS FROM A PRESSUREMETER TEST (PMT)

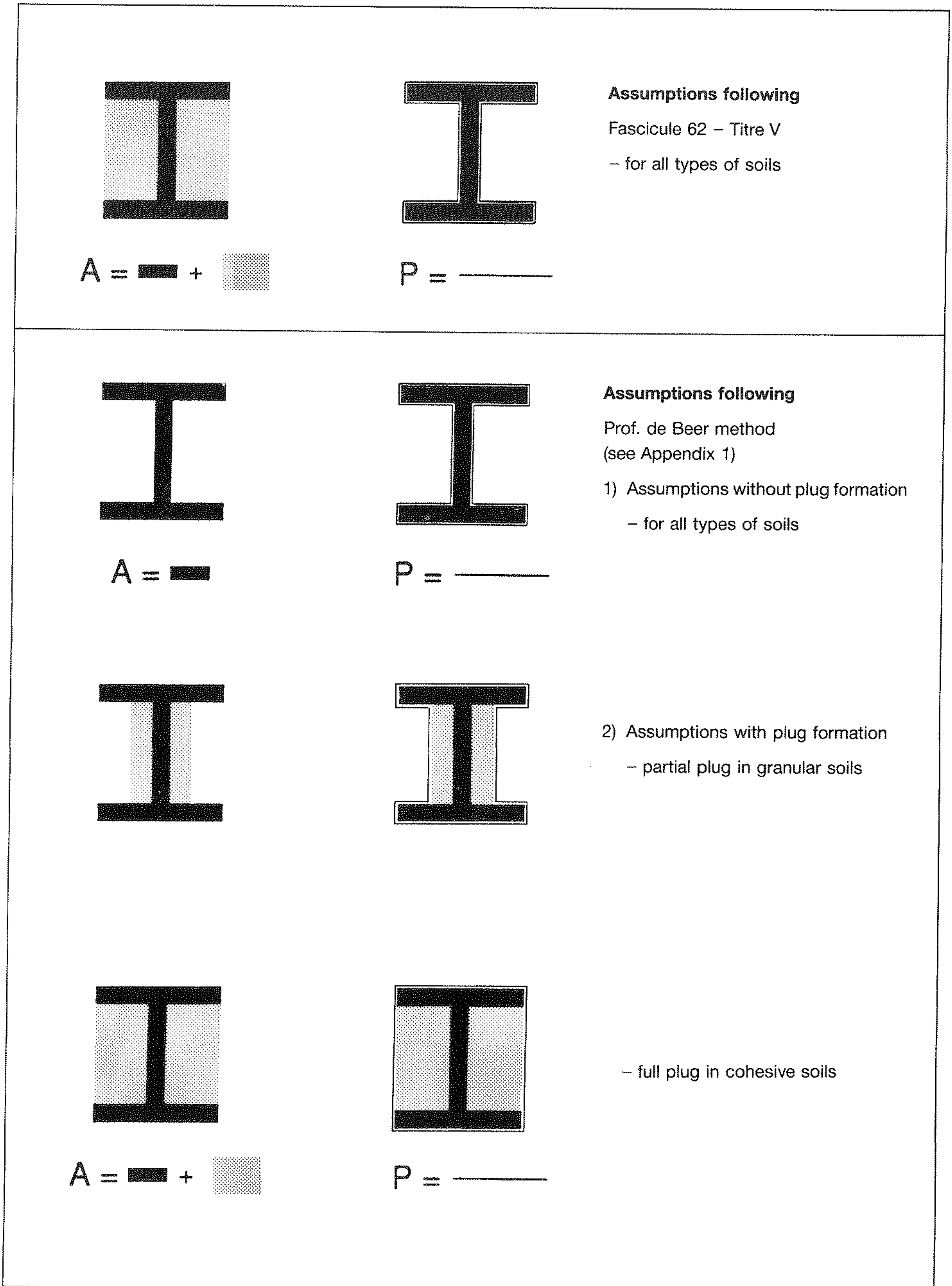
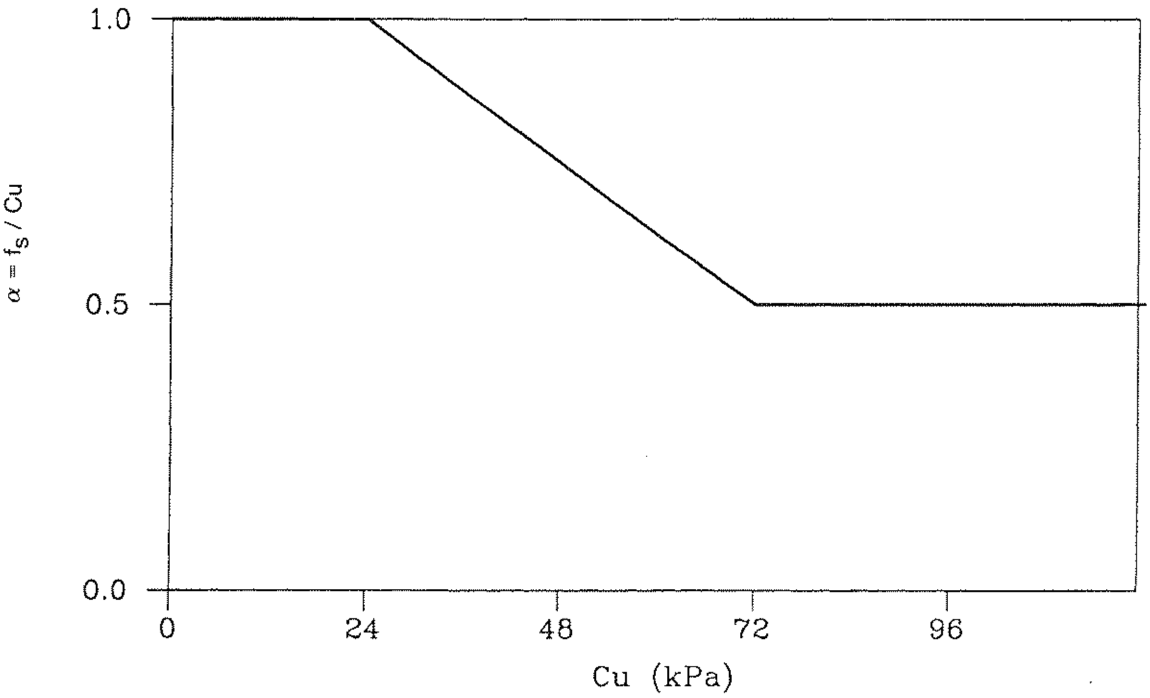


Figure 4: ASSUMPTIONS CONCERNING TOE CROSS SECTION
AND PERIMETER OF THE H-PILE (CPT methods)

API RP2A - 14th edition (1984)



API RP2A - 20th edition (1992)

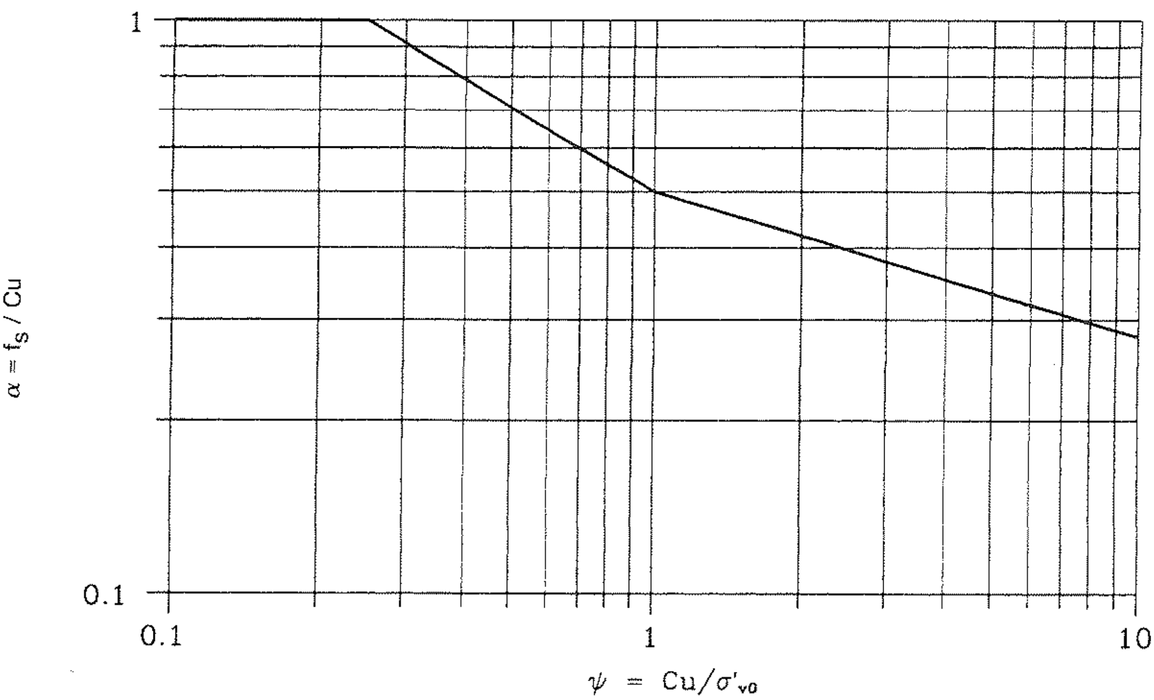
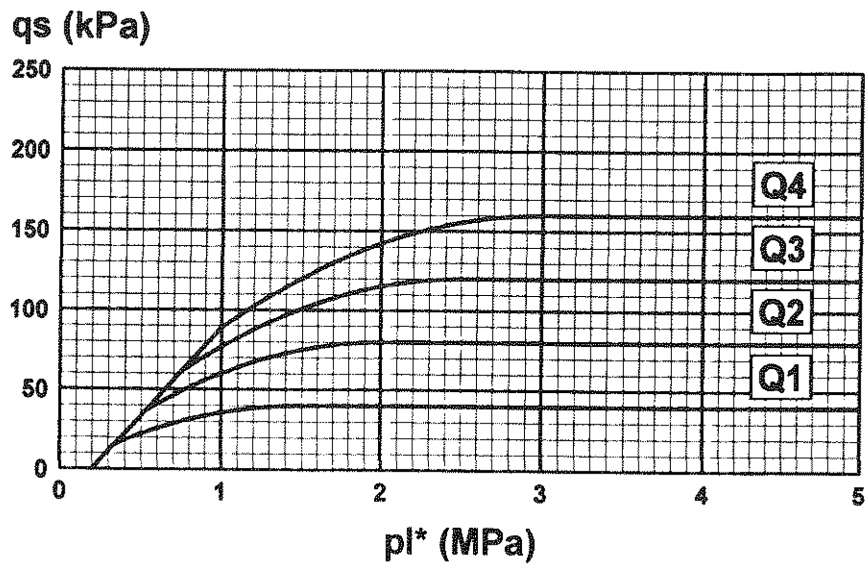


Figure 5: SOIL-PILE ADHESION COEFFICIENT α (API RP2A ref. [2])



Curves of ultimate unit friction along the pile shaft

Type of soil		Curve
CLAYS-SILTS	A	Q1
	B	Q2
	C	
SAND-GRAVELS	A	Q2
	B	
	C	Q3
CHALKS	A	Special study for each case
	B	
	C	
MARLS	A	Q3
CALCEREOUS MARLS	B	Q4
ROCK		Q4

Figure 6: CHOICE OF CURVES FOR DETERMINING q_S
(Fascicule 62 – Titre V. ref [1])

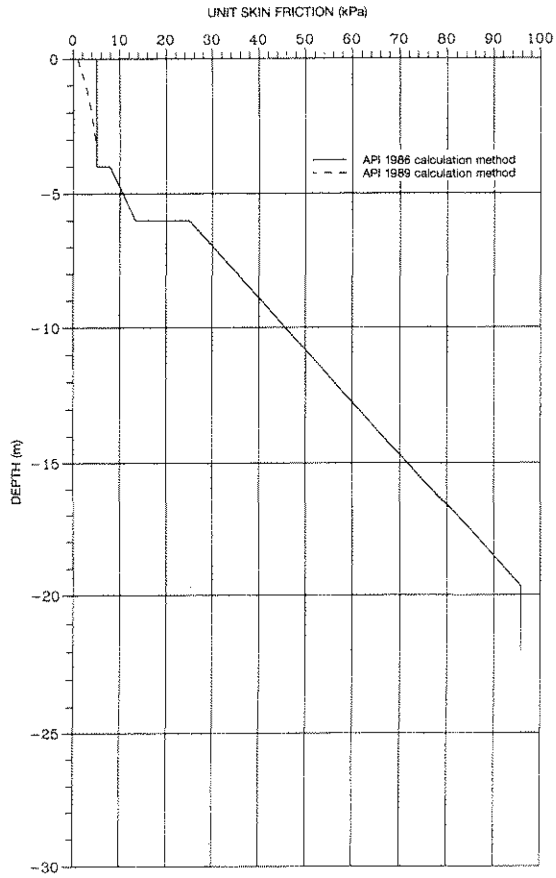


Figure 7: UNIT SKIN FRICTION (HP 360 × 109 PILE)

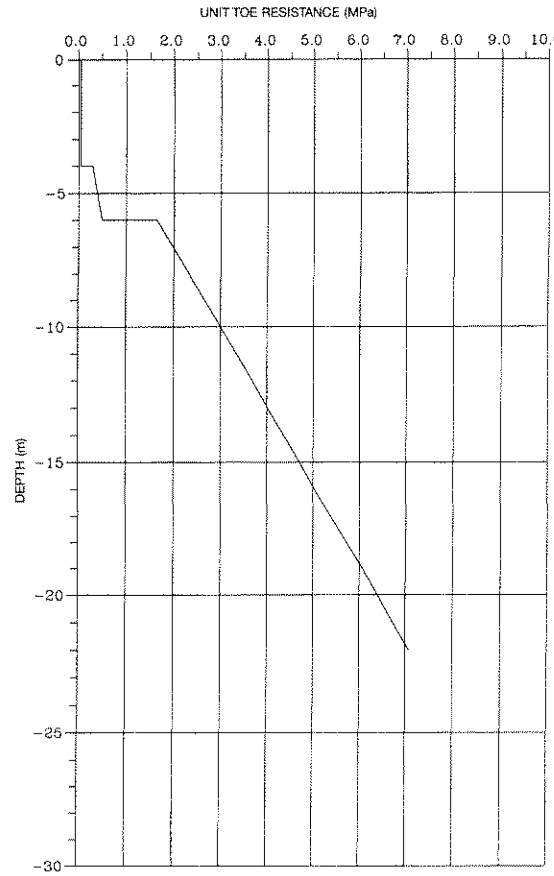


Figure 8: UNIT TOE RESISTANCE (HP 360 × 109 PILE)

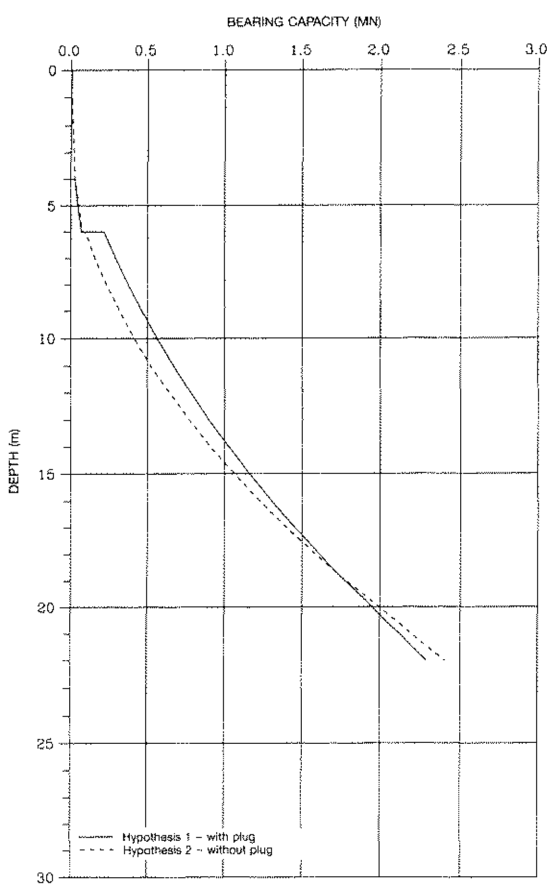


Figure 9: BEARING CAPACITY (HP 360 × 109 PILE) USING API RP2A-86

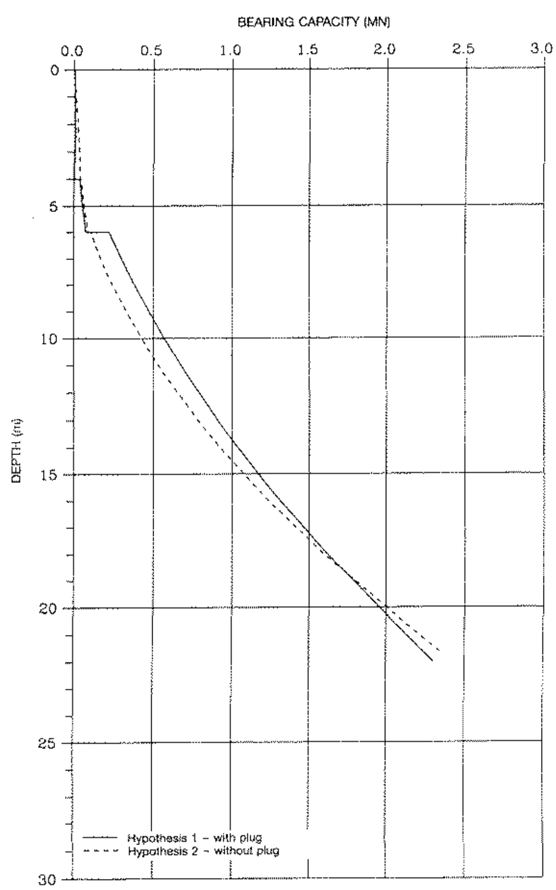
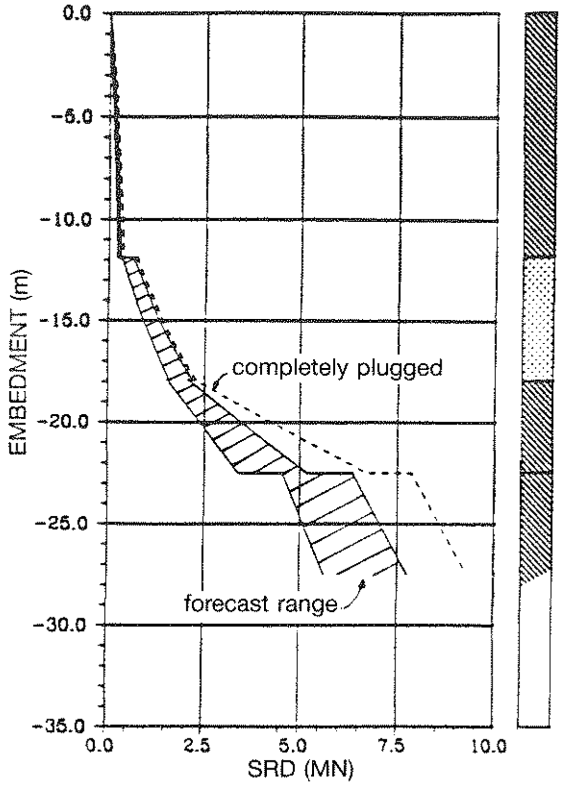
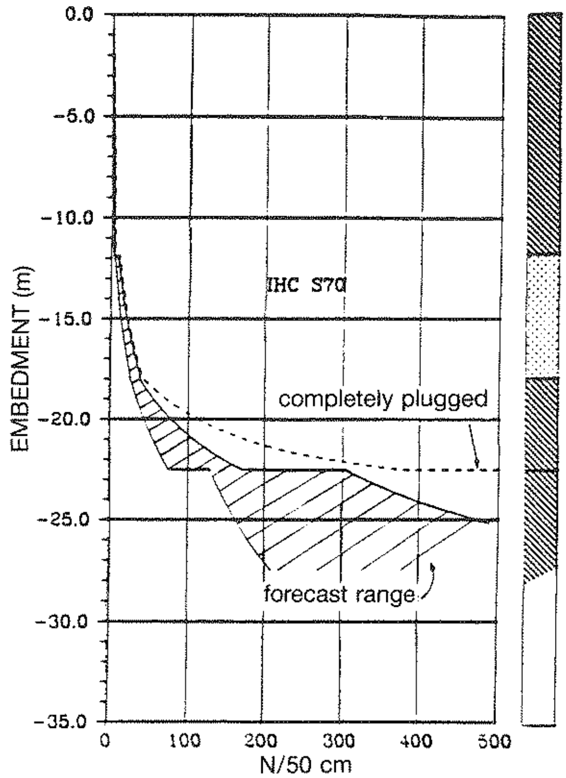


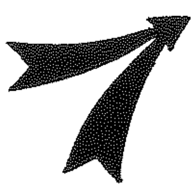
Figure 10: BEARING CAPACITY (HP 360 × 109 PILE) USING API RP2A-92



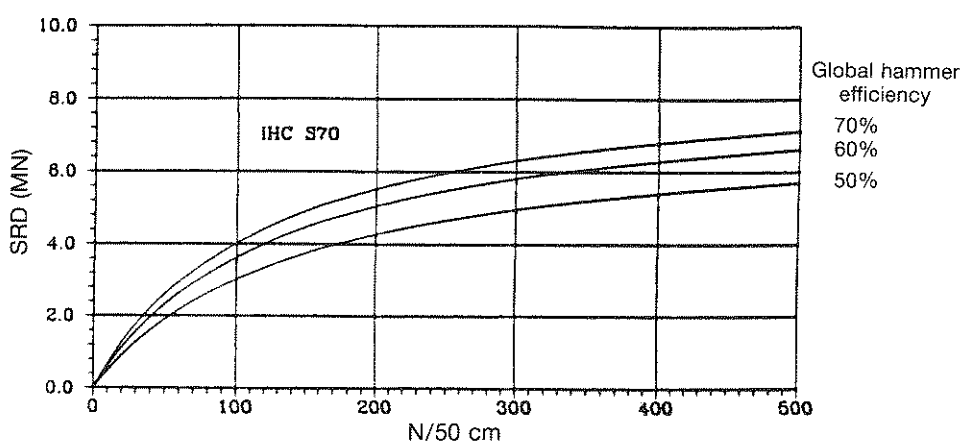
a) Forecast of soil resistance to driving



c) Forecast driving curves

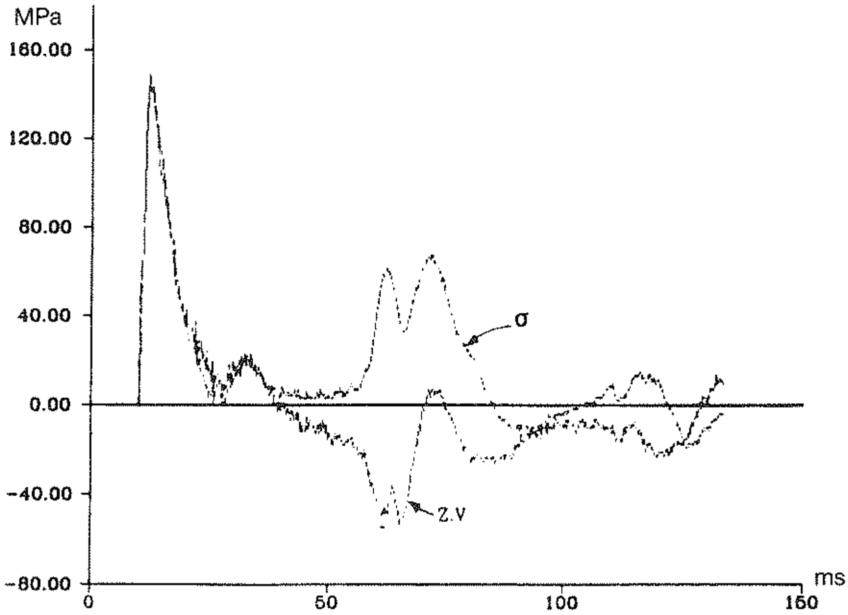


Maximum stress = 215 MPa

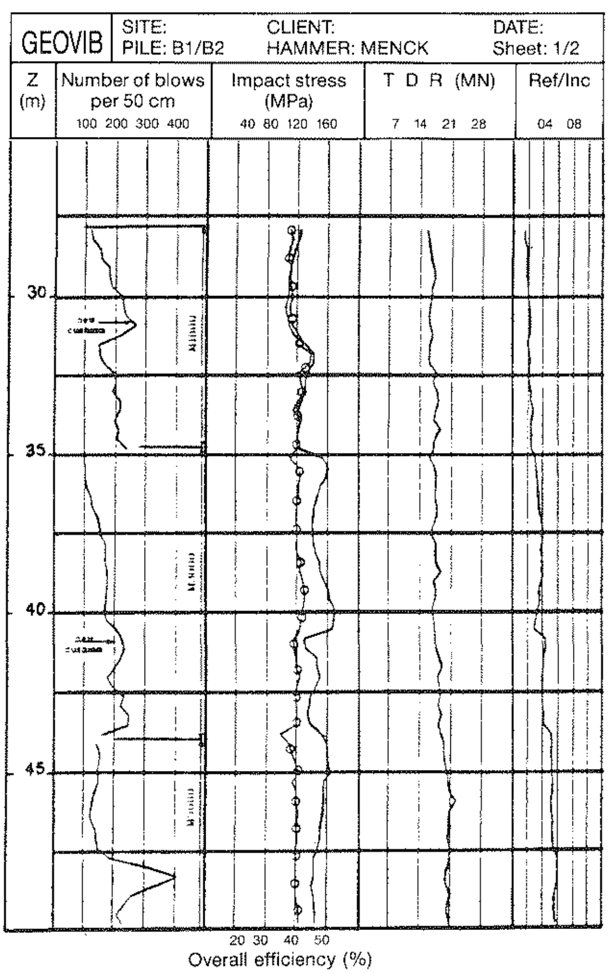


b) Simulation of driving

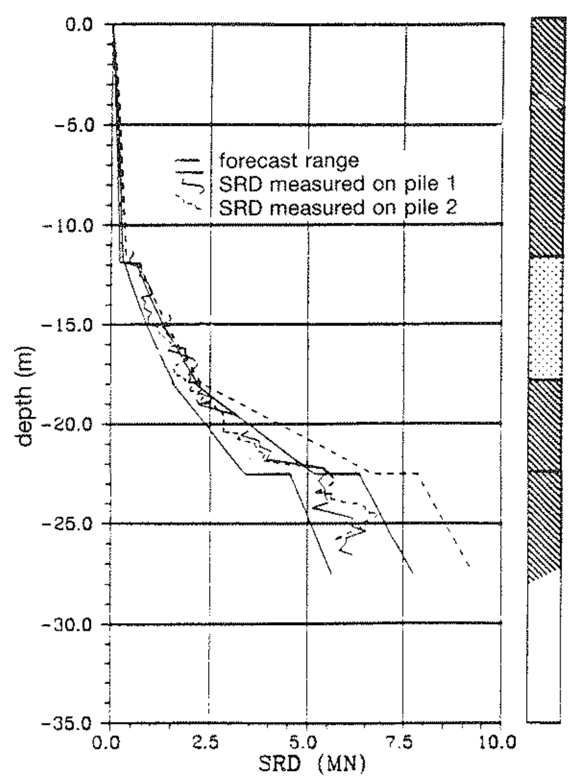
Figure 11: DRIVING FORECAST



a) Curves of stress and velocity

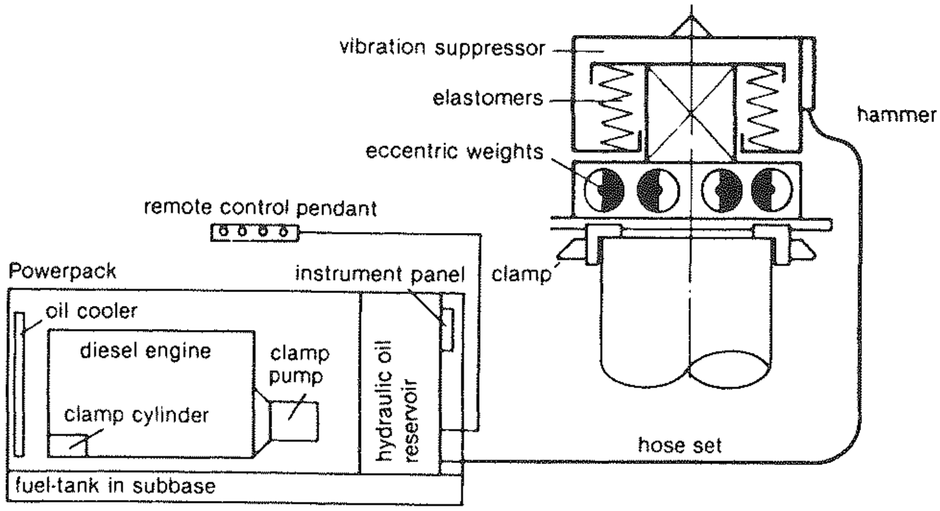


b) Driving parameters obtained on site

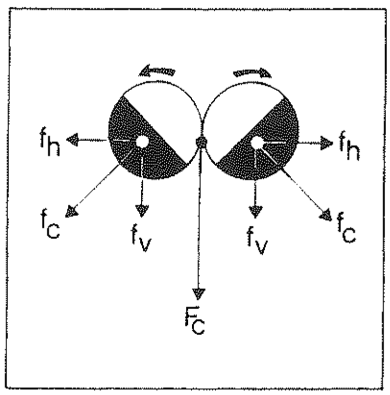


c) Comparison of measured and forecast SRD

Figure 12: DYNAMIC MEASUREMENTS DURING PILE DRIVING



PRINCIPLE OF OPERATION



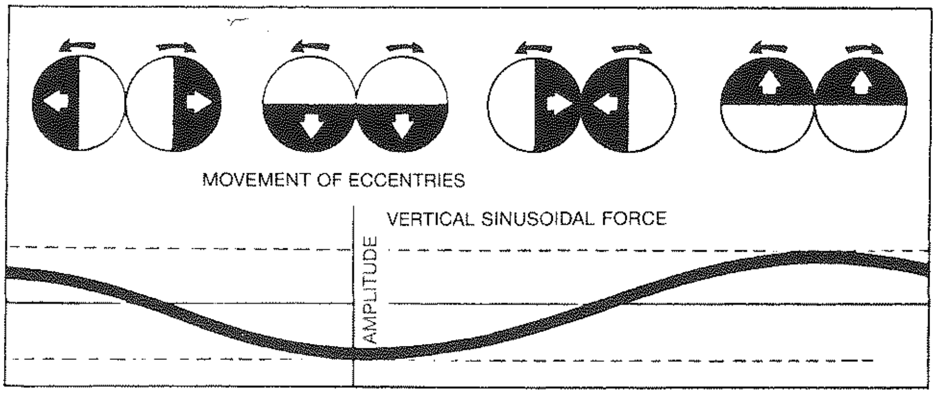


Figure 13: PRINCIPLE OF VIBRATOR OPERATION

Bibliography

[1] Fascicule 62 – Titre V of the CCTG.
Code for the design and calculation of civil engineering foundations, 1993 (Decree 93-446 of 25 March 1993).

[2] American Petroleum Institute (API) (1984, 1986, 1991 and 1992).
Recommended practice for planning, designing and constructing fixed offshore platforms.
API RP2A, 14th edn. 1984, 16th edn. 1986, 19th edn. 1991 and 20th edn. 1992.

[3] The new „Fascicule 62 Titre V“ and its aplications to the design of deep civil engineering foundations on H-piles and sheetpiling under axial load. Recapitulation.
ProfilArbed, ISPC Technical and Marketing Department, 1993.

[4] Operating method for the loading test (1974).
LCPC [Laboratoire Central des Ponts et Chaussées] publication, Paris, May 1974.

[5] ISSMFE 1977.
Recommended procedures for carrying out penetration tests.
International Society for Soil Mechanics and Foundation Engineering.

[6] De Beer E. E., Lousberg E. and Weber L. (1989).
Amélioration de la capacité portante de pieux métalliques HP [Improvement of the carrying capacity of HP steel piles] Annales des Travaux Publics de Belgique, N° 6, 1989.

[7] NF P 94–150 Static test of isolated pile under axial compression, French Standard (AFNOR), October 1991.

[8] US. Department of Commerce, NBS Monograph 58 (1962).
Corrosion of steel pilings in soils.

[9] SNCF – Notice générale EF 4D2 n° 2 (1989).
Protection contre la corrosion des structures métalliques enterrées. – Etude et réalisation des différents procédés.

[10] Meunier J. (1984).
Laws of soil pile interaction in a pile driving simulation program.
2nd Conference on Application of Stress Wave Theory, Stockholm, pp. 326–331.

[11] Smith E. A. L. (1962).
Pile driving analysis by the wave equation.
Transaction ASCE, paper 3306, vol. 127, part 1.

[12] Puech A., Poulet D. and Boisard P. (1990).
A procedure to evaluate pile drivability in the difficult soil conditions of the Southern part of the Gulf of Guinea.
22nd Offschore Technology Conference, Houston, paper 6237, pp. 327–334.

[13] Sevens R. S., Wilstie E. A. and Turton T. H. (1982).
Evaluating pile drivability for hard clay, very dense sand and rock.
14th Offshore Technology Conference, Houston, paper 4205, pp. 465–481.

[14] Toolan F. E. and Fox D. A. (1977).
Geotechnical planning of piled foundations for offshore platforms.
Proc. Institution of Civil Engineers, part 1, pp. 221–224.

[15] De Beer E. and WeberL. (1985).
Belgian Geotechnical Volume published for the 1985 golden jubilee of the International Society for Soil Mechanics and Foundation Engineering (p. 9–25). Leuven: Ceuterick.

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7-12-04-1-E