

CHAPTER 2

Loading Rate Effect and Literature

1. Introduction

Soils like many other materials, exhibit strong time-dependent behaviour, which can be translated in term of creep, relaxation or strain-rate effect. The degree of this rheological behaviour varies with the type of soil (sand and the opposite, clay), the type of structure, the soil stress history (Mitchell, 1976).

It is widely recognized that cohesive soils behaviour is very sensitive to the rate of loading, i.e. there is a general agreement that an increase in rate of deformation results in an increase of the undrained shear strength (Richardson and Whitman, 1963; Berre and Bjerrum, 1973). Generally, an increase of 10 ... 20% of strength is measured per logarithm cycle of strain rate (10 times). Loading rate effects can be considered sometimes as minor for sands, sometimes not. And for problems involving dynamic loading, the influence can be significant.

Loading rates have also affected the axial capacity of piles. Kraft et al. (1981) reported that the ultimate bearing capacity of piles embedded in clay increases by about 40% to 75% when the loading rate is increased by about three orders of magnitude. The development of the Constant Rate of Penetration Test (Whitaker, 1963) has shown that the rate of penetration enhances pile shaft resistance in clay soils (Whitaker and Cooke (1966), Burland et al (1988), etc...). Indeed, a gradual loading rate increase for

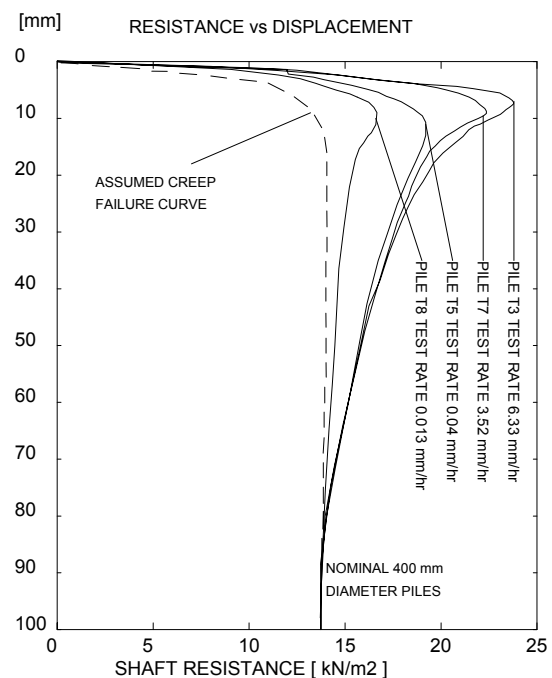


Figure II-1 : Shaft friction deduced from different CRP tests (after Lyndon et al., 1994)

many CRP tests in clay exhibits a variation of peak resistance as illustrated in [Figure II-1](#). It is therefore concluded that the peak resistance is induced by the testing regime employed, and is therefore a function of the rate of load application.

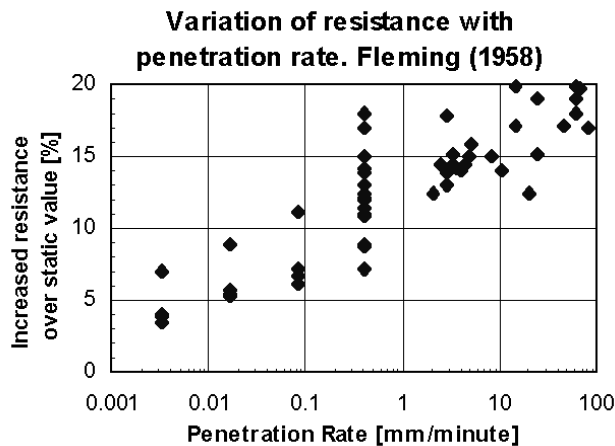


Figure II-2 : Model pile resistance vs penetration rate (after Fleming, 1958)

For cohesionless materials, the increased skin friction due to rate is also apparent: a series of experimental results from model piles in sands are illustrated in [Figure II-2](#) (Fleming, 1958).

Pile driving and dynamic pile testing are situations where the effect of the loading rate are exacerbated because of the high impact velocity of the hammer on the pile head. Smith (1960), has observed numerous driving processes on many piles in many soil configurations. He developed the first practical model of hammer-pile-soil system used for numerical analysis of pile driving to foresee the drivability and the bearing capacity of driven piles. He introduced a

simple parameter, called the Smith damping factor, linearly dependent upon pile velocity to describe the additional resistance of the soil due to the dynamic solicitation.

This first attempt of modelling of the loading rate effect led the way followed by scientists and contractors to differentiate the static term from the measured resistance. This parameter has been evaluated after correlations with Static Load Tests. More elaborated models have been built based on the simplicity of the Smith approach or including a sophisticated description of the loading rate effect.

This chapter presents these developments including the tests carried out to highlight the phenomenon, the general approach based on a global parameter and the analytical models developed in terms of fundamental soil parameters with the aim to describe more precisely the loading rate effect.

2. Definitions

The Smith's empirical parameter includes many aspects of pile-soil behaviour occurring during the loading, either along the pile shaft or at the base, or either in the direct interface or into the close vicinity in the surrounding soil. These parameters can be summarized as:

- The inertia of the soil and the level of coupling between the soil and the foundation;
- The excess pore pressure generated in the soil due to driving;
- The undrained behaviour of the pile/soil interaction during dynamic loading;
- The failure mechanisms different during static and dynamic loading events;
- The losses of energy due to the viscous intrinsic behaviour of soils;

- The hysteretic losses of energy of the loading-unloading soil behaviour.

All these effects are commonly taken into account by a simple viscous parameter because of the analogy with the mechanical models where a loss of energy can be modelled by a dashpot. It is also due to the ease and the simplification in the solving process to use only one parameter. This section is focused on the succinct definition of the underlying behaviours introduced by some of the authors of the analytical models.

2.1. The hysteretic term

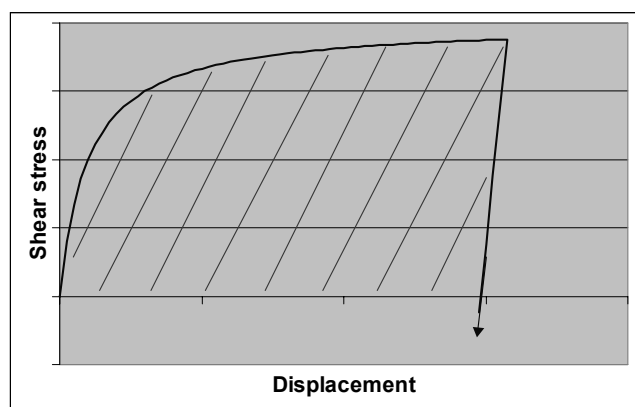


Figure II-3 : Representation of the hysteretic damping

The pile/soil interaction is governed by a stress-strain or stress-displacement relationship. The form of this function portrays not only the intrinsic behaviour of the soil during the loading (elastic, elasto-plastic, hyperbolic, rigid-plastic,...) but also during the unloading. Often, the soil does not follow the same stress path for loading and unloading processes, generating a permanent settlement and a loss of energy used to settle permanently (Figure II-3). This behaviour is sometimes governed by the initial tangent modulus. It is due to the nature of the intrinsic behaviour of the soil. The hysteretic

behaviour leads to damping of a free system governed by this law, and is called hysteretic damping. It is not related to the rate of loading and is solely defined by the pile/soil interaction law. It is not a velocity dependent term even if Svinkin et Abe (1992), describe a correlation between the hysteretic damping and the CASE damping factor.

2.2. The viscous term

The viscous term is related to the intrinsic behaviour of soil material. It is similar to the notion of damping introduced by Smith and follows the next formulation for a normal stress:

$$\sigma = \varepsilon \cdot E + E' \cdot \frac{\partial \varepsilon}{\partial t}$$

Equ. II-1

- | | |
|---|---|
| - σ is the stress generated [Pa]; | - E is the elastic modulus [Pa]; |
| - ε is the strain generating this stress [-]; | - E' is the modulus of viscosity [Pa.s ⁻¹]. |

(after Holeyman, 1988)

This visco-elastic behaviour is dependent of the loading and unloading rates. It is also relevant to the soil behaviour and is related to the dissipative behaviour of the soil. The typical viscous term is often neglected due to its small contribution to the total loss of energy. Some authors do not neglect it, such as Nguyen et al. (1988) or El Naggar et al. (1994). The viscous term is also often combined with the

increase of ultimate resistance observed when the loading rate is stepped up. This observation is made after the soil sample or the soil reaches failure, so the observation concerns the ultimate resistance and can be interpreted as the sum of all above mentioned behaviours. But the term used for the present definition is not the same and does not have the same amplitude than the global term (even if both parameters have the same denomination in literature). The effect of internal viscous damping on the strength of concrete is shown on [Figure II-4](#) in a similar behaviour than expected for soil.

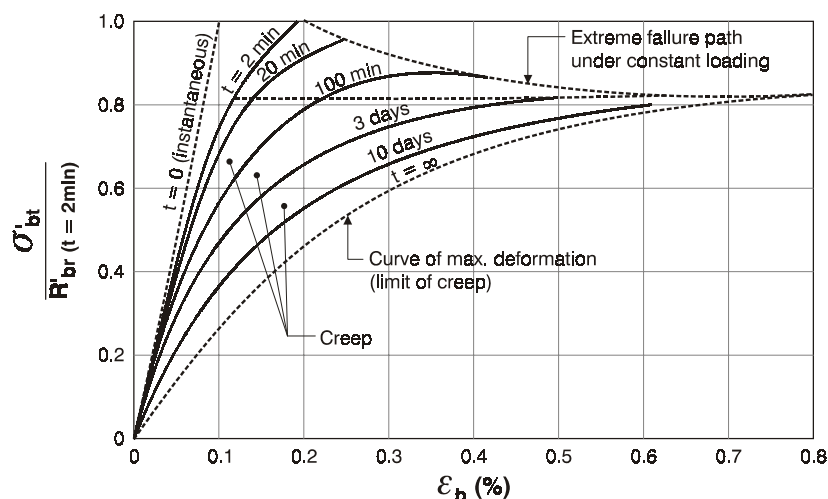


Figure II-4 : Effect of loading rate on concrete strength

2.3. The radiation term

The radiation term is one of the most important factor responsible of the loss of energy during the dynamic loading, more for the pile shaft than the pile base behaviour (Holeyman, 1992). This term can lead the pile to failure by itself (Heritier and Paquet, 1984).

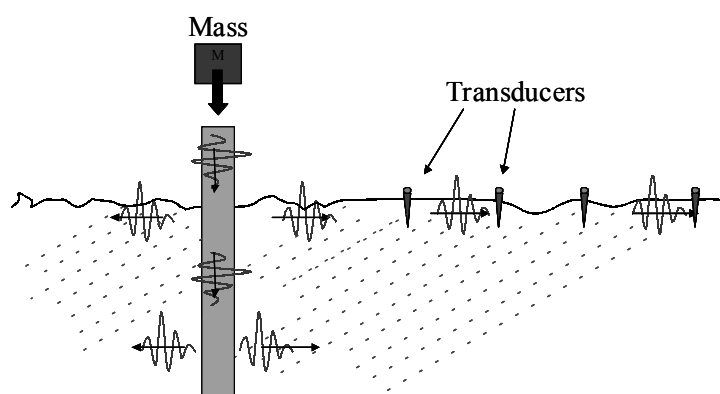


Figure II-5 : Energy radiation in the surrounding soil

The pile forces due to impact generate downward pile motion and soil forces react against the pile penetration. The energy dissipation associated with the soil resistance occurs in the interface between the pile and the soil but also in the surrounding soil by creation of the soil motion (linked to its inertia). This behaviour can be highlighted by transducers placed on the soil surface in the vicinity of the pile ([Figure II-5](#)). Several factors such as soil type,

pile shape and pile volume, pile shaft smoothness or soil stiffness and volumetric mass modify this motion. Moreover, the level of soil mobilization is inversely related to the transmitted wave. The closer the soil is to failure, the less radiation energy is transmitted. The slip layer created with the failure between the pile and the soil acts like a barrier that no wave can pass through.

Several authors tried to take this term into account out of the general viscous damping: Novak et al., 1978; Holeyman, 1985 & 1988; Randolph and Simon, 1986; Corte and Lepert, 1986; Mitwally and Novak, 1988; Paquet, 1988; Randolph and Deeks, 1992, Likins et al., 1992, and many others.

2.4. The excess pore pressure generated by the dynamic loading

Generally the pore pressure is getting higher during a dynamic pile loading test or driving. This behaviour causes an instant decrease of the effective stresses and an underestimation of the bearing capacity. It is the main reason explaining the set-up effect, and motivating restrike tests a few hours, days or weeks after driving (depending on the soil type) in order to check the bearing capacity. With this delay, the excess pore pressure has sufficient time to be gradually dissipated. Moreover, the analysis must be performed on the first blows of the restrike to avoid new excess pore pressure to be generated. A lot a authors have written on this subject in the last 30 years (Randolph et al., 1979; Skov & Denver, 1988; Svinkin et al, 1995; Svinkin, 1996, 1997 & 2000; Paikowsky et al. 1996; Axelsson, 1999, Chow et al. 2000, Seidel & Kalinowsky, 2000) but it is not specifically a loading rate effect. Nevertheless, some authors, working on the driving of pile models in clay and sand in calibration chambers (Brown et al. 2002; Eiksund & Nordal, 1996) observe that the pore pressure response in the region below the pile is a function of the stress distribution. Some negative pore pressure can occur increasing the dynamic resistance due to the instant blow.

2.5. The static and rheologic denominations

The term generally used to describe the loading rate effect is called the “viscous” damping factor determining the so-called “dynamic” resistance. In order to be clear with each term used or not within each reviewed theory, the following denominations are used to describe each parameters:

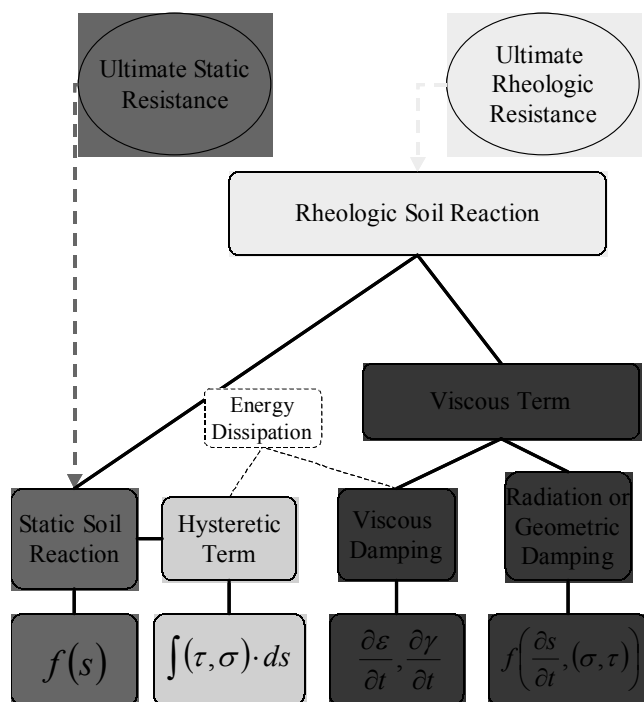


Figure II-6 : Overview of loading rate effects

The **radiation damping** or geometric damping reflects the transfer of energy into the soil surrounding the pile by moving it. This term is directly linked to the loading rate and to the level of mobilization;

The **hysteretic and viscous dampings** are intrinsic parts of the soil behaviour. Both are energy dissipating, it is the reasons why they are both called “damping”. One is velocity dependent, the other is function of the strain path and the soil motion. Both are linked to the soil reaction and they are associated in a term called the dissipative term;

The sum of the viscous and the radiation terms is introduced as a **viscous term** added to the static soil reaction.

It is defined as directly linked to the

velocity of the moving element (pile, soil or interface) and is dependent to the loading rate;

The sum of the static soil reaction and the viscous term defined here above is called the **rheologic soil reaction** by analogy with the rheology (science of the deformation and flow of materials);

The ultimate and maximum reaction of the soil is called the **static ultimate resistance** and the sum of the static and viscous ultimate resistances is called the **rheologic ultimate resistance**.

Figure II-6 summarizes each terms and their associations. Both circles also express the ultimate resistances available.

3. The global damping parameter J

This section is focused on the mainly used velocity dependent relationship in the pile driving and dynamic loading test domains. First based on the approach of Smith (1960), followed by Coyle & Gibson (1968) and the theoretical approaches of Goble et al. (1975) and Rausche et al.(1985), the first attempt to describe the added soil resistance during driving was a velocity dependent approach. This section aims to describe these attempts in the framework of the driving and dynamic load testing with a particular attention made on correlations with static pile tests and typical parameters (soil type, transmitted energy, ...).

3.1. The Smith approach

Smith (1960) developed a method to obtain the ultimate load versus set by using a finite difference method in which the wave equation theory is implemented. The pile/soil system is idealized by appropriate springs, dashpots and masses (with respect to the mechanical properties of the pile material) (Figure II-7). The model used to describe the pile/soil interaction is the elasto-plastic relationship allowing for loading and unloading (oabcdef on Figure II-8) for the shaft friction and only the compression mode (oabcf on Figure II-8) for the base reaction.

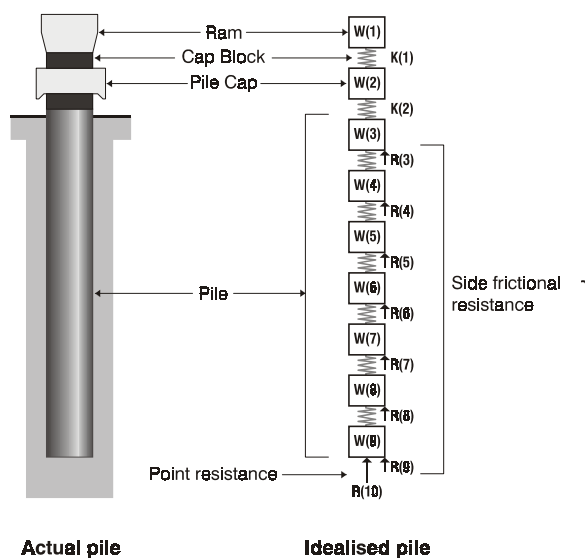


Figure II-7 : Idealization of pile Smith (1960)

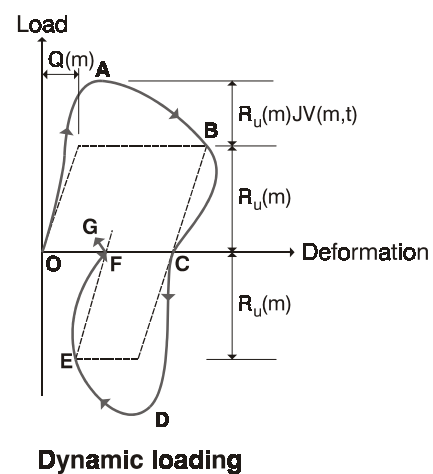


Figure II-8 : Static and dynamic load-deformation curves for the Smith model

Smith decomposed the observed soil reaction in a static term (dashed line) and in a so-called dynamic term (called “viscous” in this research). The static behaviour is pile displacement dependent and defined by the quantities Q and R_u where Q is the quake or the maximum soil deformation that may occur elastically and R_u is the ultimate static soil resistance (Equ. II-2). The static behaviour can be replaced by a spring in an equivalent rheological model where the spring stiffness can be deduced from Q and R_u :

$$K = \frac{R_u}{Q} \quad \text{Equ. II-2}$$

The viscous term is added as a linearly pile velocity dependent term of which the main factor is J_s , called the Smith damping factor [s/m]. The viscous reaction is also proportional to the static soil reaction. Smith considers that the fast driving of the pile causes the flow of the soil around the base. According to that, he considers that the soil reaction must be proportional to the rate of the mobilization.

$$R_{rheol} = R_s + |R_s| \cdot J_s \cdot v \quad \text{Equ. II-3}$$

R_{rheol} is the rheologic soil reaction [N];

R_s is the mobilized static soil reaction [N];

v is the pile velocity, soil is motionless [m/s];

J_s is the Smith standard damping [s/m].

The value of R_s in absolute value is related to the velocity sign.

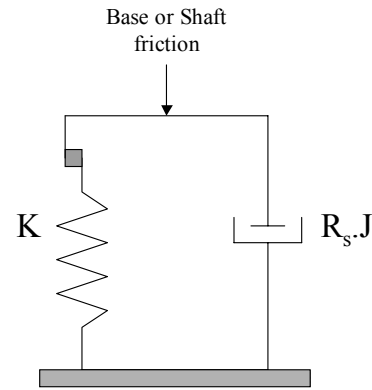


Figure II-9 : Equivalent rheological soil model

The addition of both terms models the rheologic soil reaction and is described by the load-deformation curve of [Figure II-8](#) and the equivalent rheological model on [Figure II-9](#). This model is used either for the base or the shaft reaction. The soil beyond the slip layer is considered fixed. So all the energy lost in transmission in the surrounding soil is included in the rheologic parameters of the equivalent soil model. The observations made by Smith concerned the ultimate resistances but this form is used for the intermediate steps during the numerical processing in many programs.

3.2. The J values and the evolution of the relationship

Initially, the quake Q is shown equal for base and shaft, (0.1 inch ~ 2.5 mm) and J_s is found to be less for the shaft than the base :

$$Q = 0.1 \text{ inch} \sim 2.5 \text{ mm (base and shaft)} \quad \text{Equ. II-4}$$

$$J_s(\text{shaft}) = \frac{1}{3} J_s(\text{base}) = 0.05 \text{ s/ft} = 0.16 \text{ s/m}$$

Forehand and Reese (1964) suggested empirical values for Q and $J_s(\text{base})$ as seen on [Table II-1](#).

Soil	Q [in.]	J _s (base) [s/feet]
Coarse Sand	0.1	0.15
Sand gravel mixed	0.1	0.15
Fine sand	0.15	0.15
Sand and clay or loam (50% of pile in sand)	0.2	0.2
Silt and fine sand underlain by hard strata	0.2	0.2
Sand and gravel underlain by hard strata	0.15	0.15

Table II-1 : Empirical values of Q, J_s - Smith approach (after Forehand and Reese, 1964)

Q values can be derived from pile-settlement theory in the elastic theory or the soil-resistance curves. According to this theory, the value of Q must be greater with depth and larger at the pile base than at the pile shaft. Coyle and Gibson (1970) relate the value of J_s(base) to the internal friction angle ϕ' for sands and to the liquidity index (LI) for clays.

GRLWEAP recommends using the following values for damping and quake:

	Soil Type	Pile Type	Q [inch/mm]
Shaft quake	All soil types	All types	0.1/2.5
Toe quake	All soil types, Soft Rock	Open ended pipes	0.1/2.5
	Dry soils or very dense or hard soils	Displacement piles of ϕ D	D/120
	Submerged soils or loose or soft soils	Displacement piles of ϕ D	D/60
	Hard rock	All types	0.4/10
			Damping Factor J_s [s/ft]/[s/m]
Shaft damping	Non cohesive soils		0.05/0.16
	Cohesive soils		0.2/0.65
Toe damping	In all soil types		0.15/0.5

Table II-2 : Recommended values for quake and damping factor (after GRLWEAP manual, 2000)

Coyle and Gibson (1970) and Wu et al (1989) found that Smith's original damping parameters vary with pile velocity (decreasing as velocity increase). Consequently, the basic formulation evolved. A non-linear relationship between the viscous resistance and the loading velocity was highlighted and the simple model of Smith was modified according to the works of Coyle and Gibson (1968, 1970), Litkouhi and Poskitt (1980) and Heerema (1979).

$$R_{rheol} = R_s \cdot (1 + J_N \cdot v^N) \quad \text{Equ. II-5}$$

studies give generally the value of N and J_N for some kind of soils: N = 0.18 (clay); 0.2 (sand) and J_N different from J_s. (Coyle and Gibson, 1970) - N = 0.2 for sand and clay (Heerema, 1979) - N = 0.2 for clays (Litkouhi and Poskitt, 1980). Coyle et al. (1972) determined for various soil from model pile tests in a range of 0.3-1.8 m/s that the Smith's approach is acceptable for the base resistance (N=1) and they modified only the shaft damping parameters: N = 0.35 and J_N = 1.98 s/m^{-N} (Coyle et al. 1972). Finally, data of published damping constant were compiled in "The Interim Report" (GRL, Inc. 1992):

Soil Type	Damping constant N=1		Soil Type	Damping constant N≠1			
	Shaft [s/m]	Base [s/m]		Shaft		Base	
				(s/m) ^N	N	(s/m) ^N	N
Clay	0 – 2.4	0.009 – 0.87					
Sand	0.17 – 0.5	0.33 – 0.5	Clay	1.339 – 4.106	0.17 – 0.57	0.683 – 1.99	0.17 – 0.37
Silt	0.33 – 0.34	0.3 – 0.5	Sand	1.015 – 1.649	0.2	1.015 – 2.536	0.2

Table II-3 : Range of damping factors J (after Interim Report, GRL and Associates, Inc, 1992)

Rausche et al. (1992) suggest the following equation to bypass the difficulties encountered by the numerical instabilities around 0 of Equ. II-5:

$$R_{rheol} = R_s \cdot \left(1 + J_R \cdot v_x^N \cdot \frac{v}{v_x} \cdot \frac{R_a}{R_s} \right) \quad \text{Equ. II-6}$$

- J_R is the Rausche damping factor [s/m^N];
- v_x is the maximum velocity achieved up prior the time under consideration [m/s];
- v is the classical variable, the velocity of the moving body [m/s];
- R_a is the maximum static resistance mobilized prior the time under consideration [N];
- R_s is the static soil mobilization at the time of consideration [N];
- R_{rheol} is the rheologic soil reaction equal to the sum of the static and viscous term [N].

This modification respects the non linearity of the velocity dependent relationship and makes the result free of numerical disturbances but the coefficient J_R is dependent of the level of maximum mobilization of the blow (determined by the couple (v_x , R_a)).

Finally the main results remain simple: the ultimate viscous soil resistance is velocity dependent and its dependency is not linear for shaft and possibly for the base. This non-linearity is expressed by an exponent = 0.2 either for sand and clay.

The CASE method (Goble et al. (1975); Rausche et al. (1985), introduced in chapter 1 (Equ I-26), uses also a velocity dependent relationship to take into account of the load rate effect but with no links to the static soil reaction in the viscous term. The dimensionless parameter J_c can be associated to the Smith and the Coyle and Gibson ones.

3.3. Attempt of classification of J

Many researchers have tried to correlate the results of dynamic loading tests with static loading tests performed on same or similar pile in the same conditions in order to differentiate the base and shaft friction behaviours and the parameters of the static and viscous terms. One of the purposes of these researchers was to find a relationship between the soil damping factors and a type of soil.

Paikowsky et al. (1994) presented a large study of dynamic load tests analysis (372 cases) and summarized the values of Smith damping coefficients needed to obtain the best match between measured and calculated force or velocity signals (by using CAPWAP analysis). The damping factor J_s were sorted according to soil type for base and shaft. The authors conclude from this large statistical study that there is no specific correlation between the type of soil and the damping parameters used. The spreading of J_s is too wide. Other authors tried to correlate data from pile driving to damping parameters using the wave equation analysis (CAPWAP).

Likins et al. (1996) performed a systematic correlation study on 82 piles loaded dynamically and statically (from the GRL Database). The authors estimate that no clear trend can be established even if the average Smith damping constants slightly decreases for both skin and base when the soil grain size increases. The shaft quake seems to be constant (0.1 inch), as expected. The same type of correlations was also carried out on data coming from the GRL Database (**Thendean et al. 1996**) and one more time, the Smith damping factors vary strongly for each soil type and no correlation can be extracted from these analysis.

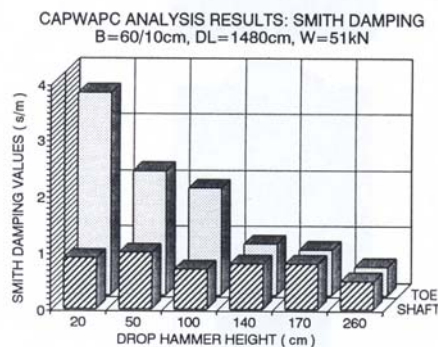


Figure II-11 : Estimated damping parameters (after Aoki et al. 1992)

Besides the classification of the damping factors with soil types, **Svinkin (1995, 1996)** proposes to consider the rheologic resistance with the damping factor as a function of time in order to take account for the set-up effect either for the static pile bearing capacity or the viscous soil reaction. Svinkin shows the evolution of the pile –soil response with elapsed time after end of driving, the pile-soil stiffness increases with time. Svinkin proposes to get the damping factor variable with respect to time or to another parameter characterizing the soil consolidation around the pile (like the shear modulus or the frequency of the fundamental mode of the pile system).

Aoki et al. (1992) describes the results of a systematic study performed on dynamic data coming from a restrike test on a concrete tube pile driven in dense sand. The blows (given by a free falling mass) were characterised by successive increasingly heights (from 0.2 to 2.6m). It can be observed that:

- The resistance mobilized at the base and along the pile shaft evolves with the transmitted energy and are not synchronised (transfer of resistance);
- The quake (for base and shaft friction) is not a constant value but evolves with the energy transmitted. Moreover, the ratio of base resistance on the quake value gives an almost constant value, similar to a spring stiffness.
- The associated values of the damping factor exhibit an evolution of constant reduction of the base damping factor with the increasingly height of drop. Figure II-11.

Many authors tried to use the Standard Penetration Test to estimate the Smith rheological parameters (the damping factor and the quake). **Goble et al. (1992)** and **Abou-Matar et al.(1996)** determines them from SPT measurements (acceleration and force on SPT-rod head). The method uses an unconstrained minimization routine to match the base soil reaction.

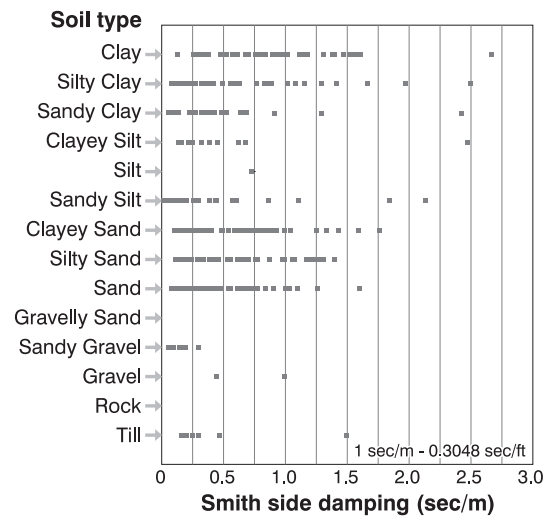


Figure II-10: classification of Smith damping factor vs. soil type (after Paikowsky et al. 1994)

Liang (2000) proposes a method to correlate the Smith damping coefficient with SPT blow count. The method uses GRLWEAP to simulate the SPT results. Assumptions are made for the quakes (shaft and base) and the evaluation of the damping factor is achieved through an iterative process and a parametric optimisation where the matched unknown is the N value and the variable are the damping factors (comparisons are made with the real SPT-N values).

Concerning the CASE damping parameter, the same approach to correlate the value of J_c with soil type was also investigated and resulted in such a poor conclusions similarly to the previous analysis for the Smith viscous parameters. The authors of the method (**Goble et al. 1975**) estimate the value of J_c based on the comparison and analysis of driving data with respect to a corresponding static load test (the static resistance being evaluated with the Davisson criterion). **Figure II-12** shows a plot of the J_c range within 20% of the statically observed capacity. The values of J_c have been sorted by soil types (as for the Smith damping factor) and the results are also scattered for each soil type. The negative values of J_c have been removed because of their “non-logical “ meaning.

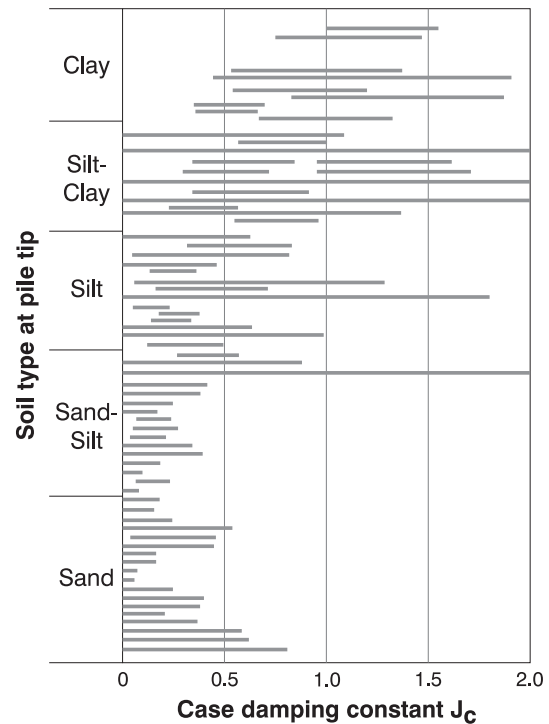


Figure II-12 : Dimensionless damping factor J_c vs. soil type (after Goble et al. 1975)

Other researchers have tried to link this parameter to the soil type. **Paikowsky et al. (1994)** have performed a large study of dynamic load tests analysis by using CAPWAP analysis in order to back-calculate the value of J_c needed to obtain the best match between measured and calculated force or velocity signals. The scattering is too wide to conclude something relevant. Some typical values for the CASE damping factor J_c are given by:

Soil Type	Suggested range of J_c	Best value of J_c
Sand	0.05-0.2	0.05
Silty sand or sandy silt	0.15-0.3	0.15
Silt	0.2-0.45	0.3
Silty clay or clayed silt	0.4-0.7	0.55
Clay	0.6-1.1	1.1

Table II-4 : suggested value for the Case damping factor J_c (after Rausche et al. 1985)

3.4. Conclusions

After a wide review of the empirical methods used to determine some influence of the Smith based damping factor with soil or driving method related parameters, it is not possible to conclude to some particular trend. It is rather the opposite.

All the authors try to find a correlation between a determined parameter and a given soil oriented parameter. The results tend to highlight the impossibility to classify the global damping parameter.

The reasons are simple as following: The damping parameter described by these relationships includes some behaviours that are not correlated with the same soil parameters, all these behaviours identified in Section 2 do not have the same magnitude at the same instant because of their origin and particular links to stress or strain evolution; second, the levels of mobilization for both dynamic and static tests are not of the same order; finally, each author concentrates his works on one parameter whereas a global study should be more appropriate.

The model described by Smith was a first attempt requiring more development with further scientifically improvements. Moreover, the Smith (or Coyle-Gibson) law is representative of the ultimate behaviour of the soil. It is only a viscous soil response corresponding to the failure state produced at a certain velocity. The trend observed, even if it is well established, can therefore only describe this situation. There is a need to study the low strain field with large range of applied velocity in order to develop the knowledge of the intermediate velocity dependent relationships. It seems to be possible to correlate the value of J with more and more parameters from the pile, soil or driving system description. But its global and artificial role prevents any general trend to be found. All the contradictions observed in the literature show the inadequacy of the use of this all-inclusive parameter J and the necessity of a more developed model including other viscous or so-called “dynamic” influences like the radiation and the dissipative damping. This is the subject of the next section.

4. Analytically based models

In parallel with the empirical refinements of the Smith model, analytically-based alternative models have been proposed by many authors. This aim is to differentiate the terms included in the global and empirical damping factor J and to derive their parameters from standard geotechnical soil properties. The models are generally separated in shaft and base responses. The shaft main theoretical studies are primarily based on the work of Novak et al. (1978) and the base main approaches are generally based on the work of Lysmer and Richart (1966). For these reasons, a short presentation of these theories is firstly covered, followed by a brief description of some models separated in shaft friction and base terms. The models related to Novak are firstly described, followed by others models chronologically. The model related to the base are described according to the same sequence than the friction ones.

All these models are generally characterised by a main term, displacement dependent, often defined as the static term and modelled by a spring (linear or not). Another important parameter of these models is the velocity dependent term (pile, soil or interface velocity) representative of the radiation damping in the soil what is a major difference compared to the Smith based models already presented. Besides these two constants, each model describes the dynamic pile/soil interaction with some differences that are explained hereafter.

The models are firstly described according to the shaft modelling. The firsts models describe the

4.1. Shaft friction models

4.1.1. Novak et al. (1978)

Novak et al. (1978) developed the plane strain soil reactions under dynamic loading, based on the work of Baranov (1968): for harmonically vertical vibrations¹, the plane strain solution of soil displacement for an infinitely long massless and rigid cylinder vibrating in an infinite elastic, homogeneous and isotropic medium is given by a complex formulation:

$$w(a_0, r) = f(a_0, r, G, \beta, P_v, i) \quad \text{Equ. II-7}$$

Where:

- $w(a_0, r)$ is the vertical displacement amplitude corresponding to a distance r from the centre of the cylinder and to the dimensionless frequency a_0 ;
- $a_0 = \frac{\omega r_0}{V_s}$ is the dimensionless frequency of the cyclic motion;
- r_0 is the pile radius;
- ω is the circular frequency;
- V_s is the shear wave velocity of the soil $= \sqrt{\frac{G}{\rho_s}}$;
- P_v is the vertical exciting force;
- G is the initial shear modulus of the soil;
- β is the soil material damping ratio ($= 0.05$ or 0 in function of the soil).

Equ. II-7 can be developed into a complex stiffness:

$$K = G(S_{v1} + iS_{v2}) \quad \text{Equ. II-8}$$

Figure II-12 shows the variation of the real and imaginary parts S_{v1} and S_{v2} . The real part S_{v1} is almost constant for $a_0 > 0.3$ and S_{v2} increases monotonously, almost linearly, with the frequency. Within that frequency domain, the soil reaction can be modelled by a spring and a dashpot whose constants are frequency independent and defined by:

¹ The main assumptions of this model are : -1- soil medium infinite, homogeneous, isotropic, visco-elastic with an internal damping frequency independent. -2- The cylinder is rigid, infinite and massless -3- no separation between soil and pile, -4- small displacements. -5- harmonic vibrations.

The spring relates to the elastic and static displacement dependent behaviour of soil and the dashpot relates to the energy lost in radiation in the soil (inertia of soil, vertical motion initiated by the imposed cylinder motion). The units of K and c are related to the unit of length of the pile.

$$K = G \cdot S_{v1} \text{ [N/m/m]} \quad \text{Equ. II-9}$$

$$c = \frac{G_s r_0}{V_s} S_{v2} (a_0=1) \text{ [N.s/m/m]} \quad \text{Equ. II-10}$$

This approach is followed by many researchers to describe the pile shaft soil resistance under dynamic loading. Generally, they consider a horizontally homogeneous soil medium but this leads to over-predictions of the radiation damping because the Novak's theory has been established for small strain where the shear modulus can be considered as non degraded. However pile dynamic testing generates large displacements, large stresses and plastic deformation. Consequently, the shear modulus undergoes a reduction that must be accounted for the radiation damping evaluation. Some of the authors developed models allowing a radial non-homogeneity in the soil accounting for the soil shear modulus degradation.

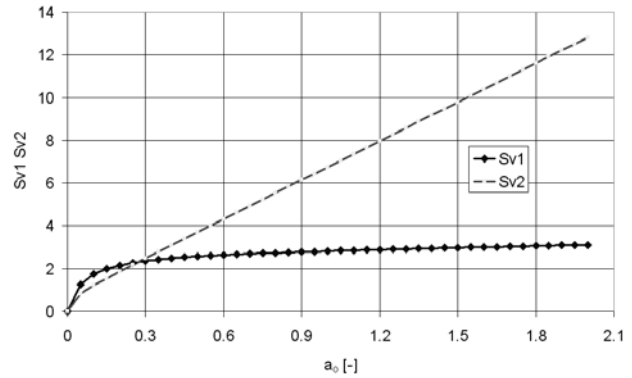


Figure II-13: Parameters S_{v1} and S_{v2} for homogeneous elastic soil medium under plane strain conditions

4.1.2. Other models

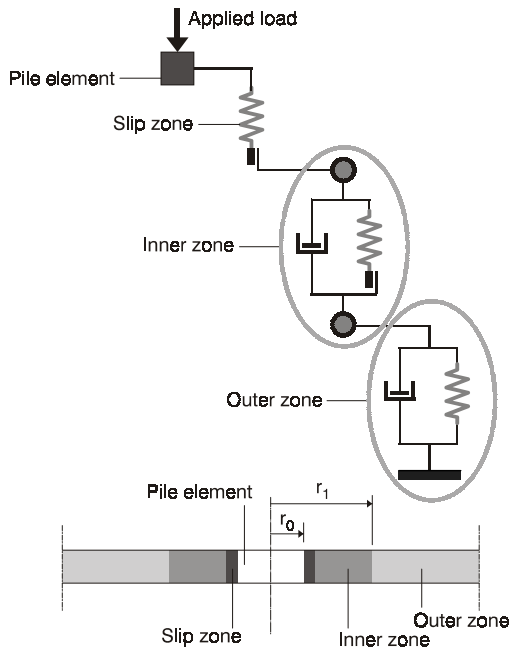


Figure II-14 : Mitwally & Novak and El Naggar & Novak soil model (after El Naggar et al, 1994)

Mitwally and Novak (1988) propose a reduced shear modulus G_m in a massless inner zone (Figure II-14). The radial and non-homogeneous zone is characterized by an internal boundary of radius r_1 . This zone limits the modelled place where the high stress and deformations are presents: “the inner zone”. This zone is surrounded by two particular media: the slip zone where the slippage occurs when the pile displacement yields an exceeding stress in the adjacent soil; and the outer zone extending to infinity where the stress and strain are assumed to remain in the elastic domain already described by Novak et al. (1978). In the inner zone, the soil stiffness is modified and becomes lower at low frequencies and larger at high frequencies. The damping factor becomes lower for all frequencies and the reduction is much greater for higher frequencies.

This model follows the classical elasto-plastic

formulation respecting the value of the stiffness and limited by the ultimate soil resistance determined by the quake value (set to 0.1 inch). The soil and pile motions are independent but both motions are equal until the slippage occurs and the soil stress exceeds the ultimate value. The radiation damping continues to occur during plastic deformation and the static soil resistance is constant during slip.

El Naggar and Novak (1992, 1994) propose the same construction model with substantial modifications: extension to rate dependent parameters - addition of inner masses into the main algorithm - modification of the inner zone particularity: the reduction of the shear modulus is obtained by a stress-strain relationship. The inner field is still representing the nonlinear response of the soil when the pile is strongly excited. But the difference in the modelling comes from the fundamental law used to describe this behaviour. Instead of the reduced shear modulus incorporated in an elasto-plastic relationship, a hyperbolic stress-strain relationship (Kondner, 1963) is chosen. This relationship smoothly indicates the transition between linear and nonlinear behaviours and respects the elastic behaviour in case of small strains and the plastic deformation in case of large strains. Moreover, the instant shear modulus is degraded according to the mobilization level.

For the plane strain assumptions, the displacement at the pile surface is obtained directly after integration of the angular distortion from the radius of the inner field r_1 to the pile radius r_0 :

$$w_0 = \int_{r_0}^{r_1} \gamma \cdot dr = \frac{\tau_0 r_0}{G_i} \ln \left(\frac{\frac{r_1}{r_0} - \eta_0}{1 - \eta_0} \right) \quad \text{Equ. II-11}$$

- τ_0, w_0 is the pile shaft stress-displacement state;
- η_0 is the mobilization ratio at pile shaft $= \frac{\tau_0}{\tau_{ult}}$.

Similarly to the shear modulus, the stiffness associated to the inner field is function of the stress state. A hysteretic behaviour is also assumed by the introduction of a completely elastic unloading phase. There is a possible material damping in the inner field: a dashpot is set in parallel of the non-linear spring described above of which the constant c_m must be suitably chosen.

To account for the loading rate effects, the ultimate soil resistance in the slippage layer is modified to be similar to the velocity dependent Coyle-Gibson approach (Equ. II-5):

$$R_{ult,rheol} = R_{ult,stat} (1 + J \cdot V^N) \quad \text{Equ. II-12}$$

- $R_{ult,rheol}$ is the ultimate rheologic soil resistance accounting for the rate effects [N];
- $R_{ult,stat}$ is the ultimate static soil resistance [N];
- v is the interface velocity [m/s];
- J, N are the rheological parameters [s/m^N ; -].

This variable ultimate soil resistance is managed in the plastic slider representing the slip layer where the slippage between the pile and the adjacent soil is modelled. Either the soil resistance is under the limit and the pile and soil move together, or the limit is exceeded and the soil reaction is kept constant in function of the velocity. The inner field masses are introduced in the model in order to write properly the equations of motion (needed for resolution).

Holeyman (1984, 1985) proposes to describe the shaft friction stress-displacement relationship as a result of an integration of the angular distortion of the soil from a radius R_m to the pile. This distance is fixed according to Randolph and Wroth (1978) who determined that the influence of the pile motion at this distance on the soil vertical stress is insignificant. The reference τ - γ relationship (Equ. II-13) to be integrated on this cylinder is the hyperbolic law proposed by Kondner (1963). This relation allows to progressively introduce plasticity smoothly, indicates the transition between linear to nonlinear behaviours and respects the elastic behaviour in case of small strains and the plastic deformation in case of large strains. Equ. II-14 gives the integration between both limits.

$$\gamma = \gamma_r \frac{\eta}{1-\eta} \quad \text{Equ. II-13}$$

γ is the angular distortion or shear strain [%];

γ_r is the reference shear strain: $\frac{\tau_{ult}}{G_i}$ [%];

η is the mobilization ratio: $\frac{\tau}{\tau_{ult}}$ [-];

G_i is the initial tangent shear modulus [Pa];

τ_{ult} is the ultimate shear stress [Pa].

$$u_s = \int_R^{R_m} \gamma \cdot dr = \frac{\tau_s R}{G_i} \ln \left(\frac{\frac{R_m}{R} - \eta}{1 - \eta} \right) \quad \text{Equ. II-14}$$

τ_s , u_s is the pile shaft stress-displacement state;

η is the mobilization ratio at pile shaft;

R is the pile radius [m];

R_m is the radius of influence.

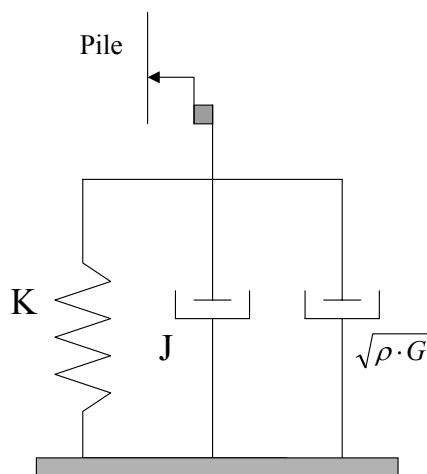


Figure II-15 : Holeyman shaft 1D-model

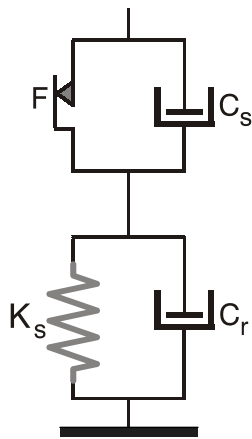
The associated stiffness is function of the stress state that can be illustrated by the degradation of the instant tangent shear modulus who decreases when η reaches 1. This approach allows the direct introduction of the hysteretic damping by the introduction of a completely elastic unloading phase.

This term is considered as the static term, displacement dependent, schematised by a non-linear spring of which the stiffness (K) is function of the stress state. The Holeyman's model is defined by only one degree of freedom, the pile motion, and consequently, the soil is considered as motionless. Hence, the soil reaction describes the soil directly adjacent to the pile.

The dynamic case is described by two terms according to the theory of Novak et al. (1978) already described and to account for the load rate effects by the introduction of a viscous term, non linearly function of the pile velocity, similarly to the results of

Coyle and Gibson (1968) (Equ. II-5). The radiation term is characterised by a damping parameter which is a simplification of the damping term of Novak. Since the model have only one degree of freedom, the radiation term must take the large strains and the plastic behaviour into account. However, this is in contrast to the Novak's assessments. In order to bypass this difficulty, Holeyman substitutes the shear modulus by the instant tangent shear modulus of the static hyperbolic relationship. When the soil is highly in plasticity, the value of G is become very small and the radiation damping is negligible, in agreement with the physics of the problem. Finally, a slider defined as the rheologic ultimate resistance limits the sum of all terms and the soil resistance. this term is also velocity dependent and function of the static ultimate resistance (Equ. II-12).

Randolph & Simons (1985,1986) propose a model also influenced by the works of Novak et al. (1978) for the description of the wave radiation generated in the soil around the pile.



The authors separates the soil in the vicinity of the pile in two zones, namely two degree of freedom. The closest is the perturbed zone where slippage, plasticity and large deformations occur. This zone is very narrow and the soil beyond behaves elastically. Consequently, Randolph & Simons describe the first and thin layer with a slider F (Equ. II-17) representing the ultimate shaft resistance under dynamic conditions according to the non linear velocity dependent relationship proposed by Coyle and Gibson (1968), and a dashpot C_s representing the internal viscous behaviour of the soil material, often set to 0. The outer zone is defined by a static (K_s) and radiation (C_r) terms similarly to the theory of Novak et al. The authors simplified the formulations of Equ. II-9 and Equ. II-10 to obtain Equ. II-15 and Equ. II-16.

Figure II-16 : Shaft model
(after Randolph & Simons, 1986)

$$K_s = \frac{G}{\phi_p} \quad \text{Equ. II-15}$$

$$C_r = \sqrt{G\rho} \quad \text{Equ. II-16}$$

$$F \rightarrow \tau_{rheol} = \tau_{stat} \cdot \left(k + \alpha \cdot \left(\frac{\Delta v}{v_0} \right)^\beta \right) \quad \text{Equ. II-17}$$

Where Δv is the relative velocity between the pile and the soil, v_0 is the reference velocity (of the static test used as reference and α is the rheologic parameter). The soil stiffness is modified under static conditions in order to account for the Randolph and Wroth approach (1978).

Corte and Lepert (1986) and **Warrington (1988)** model the shaft friction with the SDOF system according to the works of Novak et al. (1978) following the simplification of Randolph and Simons (1985, 1986): Equ. II-15 and Equ. II-16. Corte and Lepert introduce a static stiffness equal to the 2/3 of the value of K_s and a limit value equal to the ultimate static resistance.

Chow et al. (1988) and **Lee et al. (1988)** based their works on a rigorous 3D finite element wave equation model for pile driving analysis (Smith & Chow (1982)). The soil resistance of this model is also represented by a spring and a dashpot in parallel. The spring is assumed to behave following an elasto-plastic relationship (Equ. II-15) and the dashpot represents the radiation damping effects (Equ. II-16). The parameters are still derived and simplified from the works of Novak et al. (1978).

The total soil resistance is represented by the sum of both terms. The rheologic ultimate soil resistance is approximated by the Coyle and Gibson approach (Equ. II-12). When the spring force exceeds this limit, the soil dashpot disconnects from the process and radiation damping ceases. It starts again when the spring force becomes inferior to the ultimate resistance. The significant difference with other models is the fact that the failure is only limited by the spring evolution and the damping effects vanishes at this instant. That means also that the rheologic soil resistance undergoes a sudden reduction and a sudden increase when the failure load is exceeded up or down respectively.

Nguyen et al. (1988) developed a model schematically described by [Figure II-17](#).

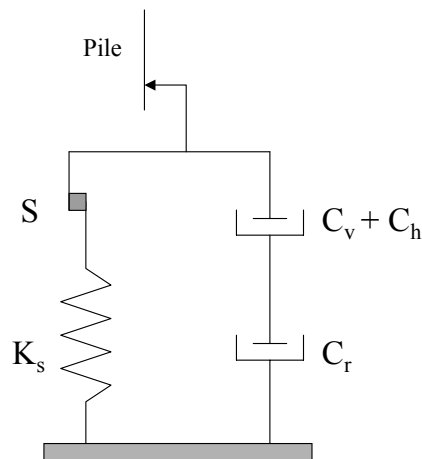


Figure II-17 : Nguyen et al. Model

Dashpots represent the effects of the radiation (C_r) and the material damping. The material damping regroups the intrinsic viscous damping (C_v) and the hysteretic damping (C_h). The spring represents the static soil stiffness (K_s); and the plastic slider (S) represents the ultimate static resistance of the soil. The model is only determined by the pile motion without taking the soil motion into account.

The static and the radiation terms are based on the simplified forms of the Novak's theory (similarly to Equ. II-15 and Equ. II-16). However, the value of G is function of the strain amplitude, what seems to be similar to the Holeyman's approach. The radiation term vanishes when the static term exceeds its ultimate resistance.

Paquet (1988) proposes a shaft model where the pile and the soil have distinct motions depending of the level of stress generated at their interface. This model is described in chapter 1 (SIMBAT model). The model could be described as the model proposed by Randolph and Simons with two degrees of freedom. The slippage limit is determined by the ultimate rheologic resistance of the soil, function of the interface velocity. The reaction of the soil is represented by a spring and a dashpot representing the soil stiffness and the radiation damping. Originally based on the expressions given by Briard (1970) and Novak (1974), the underlying theory is similar to other proposed models. Moreover, the limit value (R_l) of the interface resistance is function of the penetration velocity (Dali & Heritier, 1992).

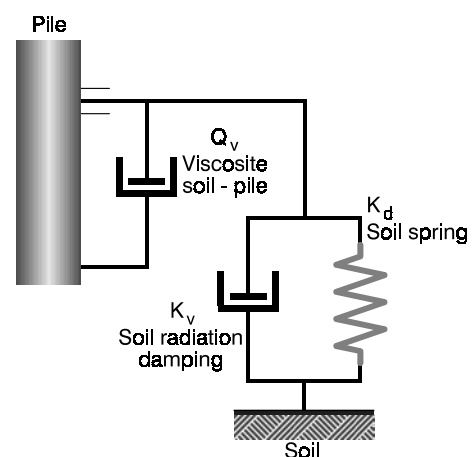
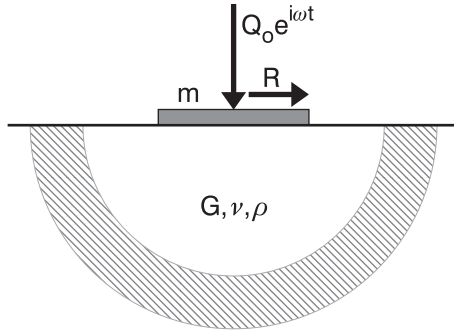


Figure II-18 : SIMBAT shaft model (after Dali & Heritier, 1992)

4.2. The base models

4.2.1. The Lysmer analogue

Similarly to the approach of Novak et al (1978), the Lysmer and Richart approach (1965) is used in most of the models describing the dynamic soil reaction at the pile base. Lysmer detailed the response² of an elastic half-space medium under vertical harmonic solicitations ($Q_0 \cdot e^{i\omega t}$) applied on the centre of a circular and rigid footing resting on a plane soil surface (Figure II-19).



The footing has a radius R , a mass m and the medium has an initial shear modulus G , a soil density ρ and a Poisson's ratio ν . The analogy with the pile base response is considered in the range of frequency undergone during a pile dynamic event and in the range of elastic stresses because of the assessments made by Lysmer and Richart. The Lysmer analogue is based on the works of Reissner (1936) who determined that the displacement in the centre of a flexible circular area under the same loading was function of a complex notation:

Figure II-19 : Lysmer & Richart model

$$\bar{u} = \frac{Q_0 \cdot e^{i\omega t}}{GR} (f_1 + if_2) = \frac{Q_0 \cdot e^{i\omega t}}{GR} \cdot f \quad \text{Equ. II-18}$$

Where f_1 and f_2 are functions of the dimensionless frequency $a_0 = \frac{\omega R_0}{V_s}$ and Poisson's ratio. If the frequency a_0 increases, the relative weight of the complex term in Equ. II-18 increases as well and for high frequencies, the half-space medium response looks like an impedance, and can be modelled with a dashpot velocity dependent.

$$C_z = \frac{R^2}{a_0} \sqrt{G\rho} \cdot \left(\frac{-f_2}{f_1^2 + f_2^2} \right) \quad \text{Equ. II-19}$$

And the associated reaction coefficient (displacement dependent term) is given by:

$$K_z = GR \cdot \left(\frac{f_1}{f_1^2 + f_2^2} \right) \quad \text{Equ. II-20}$$

However, f_1 and f_2 are still functions of the dimensionless frequency $a_0 = \frac{\omega R_0}{V_s}$ and of Poisson's ratio ν , and are difficult to be used in practice. Lysmer and Richart (1965) introduced new developments reducing the influence of the Poisson's ratio and observed that the new formulations of C_z and K_z were almost constant in the usual range of frequencies.

² The subsoil is perfectly elastic, isotropic, homogeneous where only normal stresses are transferred at the interface between footing and soil.

The corresponding spring stiffness and the damping factor are given by the well known Lysmer analogue values (Equ. II-21, Equ. II-22 and [Figure II-20](#)):

$$K_z = \frac{4G_i R}{(1-\nu)}$$

$$C_z = \frac{3.4R\sqrt{G_i \rho}}{(1-\nu)}$$

Equ. II-21

Equ. II-22

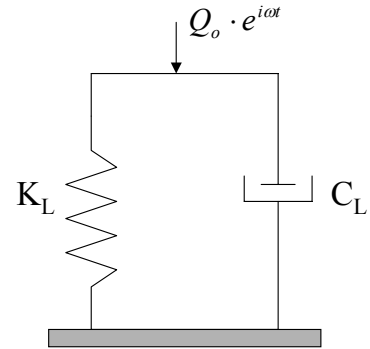


Figure II-20 : Lysmer analogue

The spring corresponds to the static response of the soil and the second term defines the geometrical or radiation damping of the soil assuming the energy losses in the soil by wave propagation. This model is only valid for homogeneous and elastic media.

4.2.2. Other models

For **Mitwally and Novak (1988)**, the base resistance is modelled by the reaction of an elastic half-space to a rigid massless circular disc undergoing harmonic vibration and can be written analogously to Equ. II-8 by:

$$R = G_b \cdot r_b (C_1 + iC_2)$$

Equ. II-23

Where:

- C_i are the dimensionless parameters representing the soil stiffness and radiation damping and function of the Poisson's ratio and the dimensionless frequency a_0 ;
- G_b is the soil shear modulus beneath the pile base [MPa];
- r_b is the pile base radius [m].

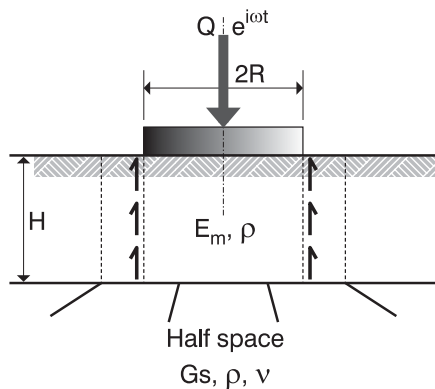


Figure II-21: El Naggat et al. base model

El Naggat and Novak (1992, 1994) improve the model of Mitwally and Novak: the homogeneous elastic half-space is replaced by the addition of two regions: a first layer accounts for heterogeneity and nonlinearity where a hyperbolic law is used. This media is built to allow for radial wave propagation and lateral confinement. This soil layer is underlain by a half-space where plane strain soil reactions are modelled by using the Lysmer and Richart analogue with linear dashpot and a spring in parallel (Equ. II-21 & Equ. II-22). The geometrical properties of the soil layer are evaluated such the static deflection and the dynamic behaviour at the interface with half space were satisfied ([Figure II-21](#)).

In the same way, **Holeyman (1984 & 1988)** modifies the half-space governing the approach of Lysmer and Richart by a two-part medium to account for non-homogeneous half-space and non-linear behaviour of the soil response. [Figure II-22](#) represents an

equivalent axisymmetric solid resting below the footing. It has finite lateral extent determined by the agreement of the response model under static and dynamic conditions (Holeyman, 1984). The model rests on the half-space medium governed by the analogue of Lysmer. The evolution of the cone radius r is function of the depth Z , the pile radius r_0 and the Poisson's ratio. Only the vertical mode of deformation is considered and the cone mass density is equal to the soil medium one.

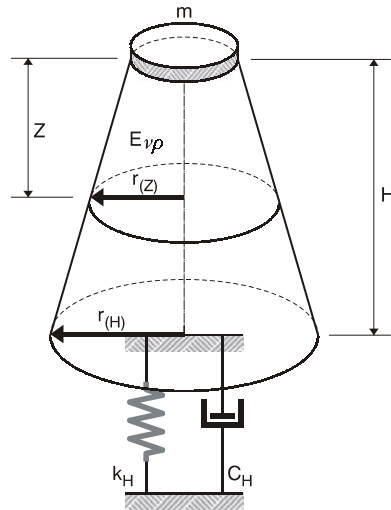


Figure II-22 : Concept of the equivalent solid
(after Holeyman, 1988)

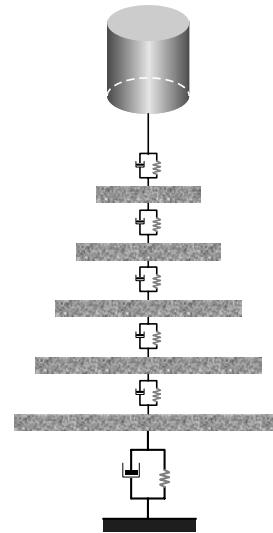


Figure II-23 : Holeyman base model

The model has the advantage to be easily discretized in lumped parameter accounting for the local properties (non-homogeneity) of each layers (Figure II-23). The non linear response of the soil, typical of large deformations, can be also integrated. As the shaft model, the hyperbolic law (Kondner, 1963) can reflect the smooth evolution from elasticity (small strains) to plasticity for the base soil reaction. This model is also characterized by an elastic unloading phase (related to the initial tangent modulus) defining the hysteretic damping of the pile movement. The viscosity effects can be incorporated related to the strain path and strain rate and the axial stress is limited by a value taking account of the loading rate effects (according to the Coyle and Gibson approach - Equ. II-12).

The radiation damping is accounted for by the geometry of the cone and the masses of the lumped elements. The behaviour at the base of the cone is assumed as low strain as sufficient in order to use the Lysmer analogue defined for an elastic behaviour.

The proposition of **Randolph & Deeks (1992)** is also based on a double degree of freedom system like the shaft model developed by Randolph and Simons. Contrary to many base models, this one is not based on the Lysmer analogue but based on the Wolf model (1988) who study the vibration of a circular rigid footing on the surface of a half-space (derived from the analogue of the dynamical spherical cavity expansion (Randolph & Pennington, 1988).

The model includes the slider Q_b that represents the ultimate base resistance under dynamic conditions; the dashpots C_b representing the radiation damping in the soil beneath the pile; the spring K_b representing the static deformation of the soil and the lumped mass M_b that takes into account the inertia of the soil moving under the base.

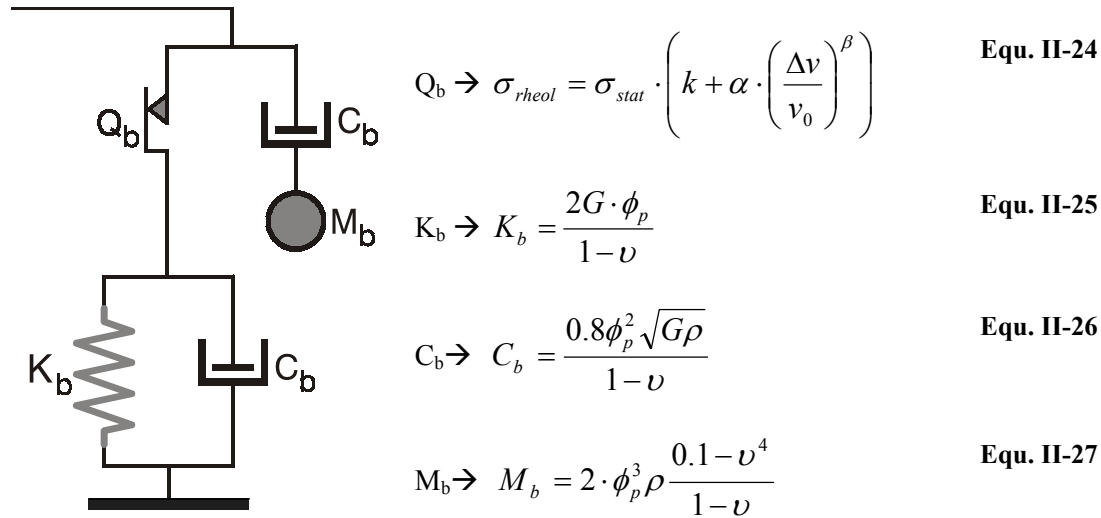


Figure II-24 : Base model
(after Randolph & Deeks, 1992)

The ultimate limit of the based resistance is the general form of the non linear velocity dependent relationship proposed by Coyle and Gibson (1968). The authors specify it is possible to add a viscous dashpot in parallel of the plastic slider in case of a particularly dissipating soil (same effect than C_s for the shaft model). Generally this effect is neglected or integrated in the hysteretic energy losses.

Nguyen et al. (1988) present for the base a model similar to the model described for the shaft friction (Figure II-17). The static and radiation terms are based from the works of Lysmer and Richart (Equ. II-21 & Equ. II-22) and specific formulations account for the viscous and the hysteretic dampings.

The models of **Chow et al. (1988)** and **Warrington (1988)** also use strictly the analogue of Lysmer to describe the base soil reaction.

4.3. Conclusions

A major part of the models described herein uses either the Novak approach and the Lysmer analogue for the shaft and base modelling, respectively. However, both approaches are described for particular conditions including the elasticity and homogeneity of the soil and the elastic range of the strain induced. The high strains generated during a dynamic test or a pile driving cause pile/soil interactions not described by the aforementioned theories. To account for this important feature, some authors introduce elements more or less consequent to the initial model. Three types of upgrade can be observed for the shaft model:

- The modification of the stress-displacement relationships from a linear and elastic formulation to the hyperbolic one. The hyperbolic law (Kondner, 1963) can reflect the smooth evolution from elasticity (small strains) to plasticity and represents the non-linearity of the soil;
- To allow some parameters to evolve with the level of plasticity. The radiation damping is degraded or completely suppressed when the shear stress exceeds the ultimate shear resistance;
- To account for the large strains in a portion of soil surrounding the pile and differentiated from the soil located in the far field.

Generally the authors use one, two or the three upgrades (Holeyman, El Naggar, Simons & Randolph, N'guyen, Paquet, Chow). The authors using the simple Novak approach to model the whole pile/soil interaction model an over-predicted radiation damping.

Concerning the base, a few number of authors modify the Lysmer analogue to account for the large strains and non linearity of the pile soil system. It is probably due to the fact that there is no failure occurrence similarly to the slippage along the pile shaft at the pile base and due to the fact that the radiation damping keeps acting also at large strain (authors like N'Guyen who makes the radiation term vanishing when the pile reaches plastic deformations at the pile base probably over-predicts the static term of the ultimate soil resistance). However the Lysmer analogue hypothesis does not agree with the applied range of loads and the soil response does not correspond. Construction of a typical model used as interface between the high level of solicitation in the vicinity of the pile base and the far field where the soil behaves elastically is a good way of bypassing this difficulty in keeping relevant with the theory and the Lysmer approach (equivalent solid of Holeyman, El Naggar).

On the other hand, for most of the authors, the only velocity dependent term is the radiation damping. The ultimate resistance takes also the loading rate into account but it is only the representation of the failure mode. A few authors introduce a viscous term, also described as velocity dependent by often neglected. This is a major difference with the observations of Smith where the “dynamic” resistance is directly linked to the static soil reaction (contrary to the radiation damping). However, the description of Smith only concerns the failure mode and not the complete description of the soil reaction. The lack of data in the low ranges of velocities and in the intermediate states of soil mobilization brings out this difference of interpretation between the modelling and the observation.

The loading rate effect is taken into account by the Coyle and Gibson formulation of the ultimate rheologic soil resistance: a power law relationship of the velocity multiplying the ultimate static resistance. Many authors use it as plastic slider in order to limit the evolution of the rheologic soil reaction and sometimes to get rid of the radiation term.

In conclusion, all these models are similar by the underlying theory used and by the necessity to describe the radiation behaviour for both base and shaft models. The models differentiate in the way of describing the soil non-linearity (especially in large strain) and the pile/soil motion system. It seems to be required to describe the radiation behaviour on a large range of strain and velocities. The close vicinity of the pile, either at the base or along the shaft must be investigated with a special attention put on the attenuation of the stresses and strains generated by the pile's motion.

5. The loading rate study through the literature

Besides the interest brought by Smith and its first model describing the static and viscous soil reactions, many researchers have performed specific studies in order to isolate the trend and the origin of the effects of an increase of loading rate on the bearing capacity of a pile. These studies have been carried out by observations during pile loading events or laboratory testing (use of models or standard lab procedures).

Generally, the analysis concerned the comparison of the static and the so-called “dynamic” soil reaction when the soil sample, the pile or the model reaches failure under a rapid load application. This empirical approach resulted in the development of many formulations describing the observations

measured. Each of them respects the trend of the respective experiments and was somehow linked to this experiment (assessments, testing procedures, ...). However, it seems possible to classify these relationships modelling the loading rate effects in 4 categories following the mathematical approach or the reference quantity. All of them relate to the penetration velocity (with different units and orders of magnitude) or the time to reach failure (that can be considered as the inverse of the loading rate).

- Linear relationship with proportionality to the static soil reaction (mobilized);
- Linear relationship with relation to the pile properties;
- Power relationship with proportionality to the static soil reaction (mobilized);
- Logarithmic relationship with proportionality to the static soil reaction (mobilized).

This section is dedicated to a rapid overview of these categories with a description of the tests underlain and a list of the testing programs found in literature.

5.1. Linear velocity relationships

5.1.1. Proportionality to the static soil reaction & proportionality to the pile properties

Those two categories refer to basic formulations already introduced. The Smith's approach (section 3.1) was the first to attempt to quantify the loading rate effect based on the dynamic pile loading test results. The relationship is based on the observations of piles during their driving. The "dynamic" contribution to the soil resistance is linearly dependent upon pile velocity and also proportional to the static soil reaction. Equ II-3 resumes this formulation.

The second category refers to the CASE method introduced in chapter 1 and in section 3.3. The "dynamic" contribution to the pile capacity is pile velocity dependent but with no direct link to the static soil reaction (Equ I-26). Moreover, the pile properties (geometry, mechanical properties) influence the load rate effect with to the nominal impedance. This formulation is a mathematical result based on the Wave Equation Analysis applied to the pile driving resolution but looks like the empirical and global relationships described here with the use of the CASE damping parameter J_c and the velocity dependency.

5.2. Complex velocity relationships

5.2.1. Power relationship with proportionality to the static soil reaction

One of the major studies performed on the velocity dependent characteristic of soil resistance within the framework of the pile driving was carried out by **Coyle and Gibson (1968)**. They use a triaxial cell submitted to the falling of a dropping mass from varying heights generating varying impact velocities. Sands and clays of different types have been tested in a velocity range of 0.2 to 3.5 m/s.

A general trend of $\frac{Q_{rheol}}{Q_{stat}}$ is found to be similar for both types of soil at failure (Figure II-25).

The authors obtained a general formulation commonly used:

$$Q_{rheol} = Q_{stat} \cdot (1 + J_N \cdot V^N) \quad \text{Equ. II-28}$$

Q_{rheol} is the soil resistance with all influences [N]; J_N is the damping factor [s/m^{-N}];

Q_{stat} is the static soil resistance [N]; N is the rheologic exponent [-].

According to the authors, $N = 0.2$ for sands and $N = 0.18$ for clays; J_N is correlated with the internal friction angle and the void ratio for sands, and with the liquidity index and the water content for clays.

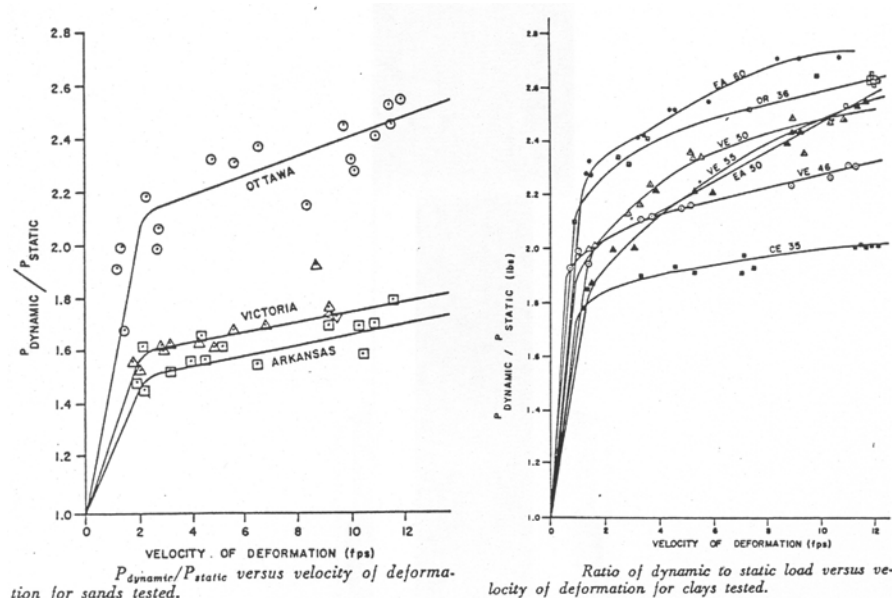


Figure II-25 : Loading rate effect (by mass falling) in a triaxial cell (after Gibson and Coyle, 1968)

Heerema (1979) tested a range of clays and sands by shearing a steel plate under constant normal stress at different velocities. This test can be classified in a middle position between classical tests (simple shearing) and models tests. The velocities varied from $7 \cdot 10^{-4}$ m/s to 0.6 m/s for sand and from $7.81 \cdot 10^{-4}$ m/s to 1.04 m/s for clay. The tests performed on dry sands do not show any probative results on the failure friction but the tests performed on clays (stiff clay – Kontisch, Heather; soft clays – Claymore) exhibit a pronounced trend. The general form of the results is similar to the Coyle and Gibson modelling (Equ. II-28). Heerema suggests the value of 0.2 for N and gives a correlation for the J_N values with the undrained soil cohesion c_u .

Litkouhi & Poskitt (1980) carried out pile loading tests on models of diameter 10 mm driven with a constant rate of penetration in undisturbed and consolidated remoulded clay samples (London, Forties and Magnus clays). The rate of penetration varied from 0.0003 m/s (considered as static velocity) to 1.65 m/s. The authors performed tests with two different models to separate shaft and base behaviours. The velocity dependent trends observed in laboratory can be also modelled with a power law (Equ. II-28). N is fixed to 0.2 for the remoulded samples for base and shaft behaviours. N is found = 0.4 (base) and 0.6 (shaft) for the only undisturbed sample. J is in the range of $[0.2; 0.3]$ s/m^{-N} for remoulded samples and 0.08 s/m^{-N} (base) and 0.06 s/m^{-N} (shaft) for the undisturbed samples. That confirms that both base and shaft frictions capacities have non-linear viscous behaviour.

Briaud and Garland (1985) use the results of lab tests and full-scale static and dynamic pile tests in order to predict the pile behaviour in cohesive soils subjected to axial load. Their study is focused on the analysis of the evolution of the undrained shear strength (S_u) with time to failure t_f .

After analysis of laboratory results focused on undrained shear strength, it was seen that a power law can link the values of S_u obtained for two different failure durations.

$$\frac{S_{u1}}{S_{u2}} = \left(\frac{t_2}{t_1} \right)^n \quad \text{Equ. II-29}$$

- S_{u1} and S_{u2} are undrained shear strengths measured after time to failure t_1 and t_2 respectively.
- n is the viscous exponent.

Statistically, the range of n values is limited to 0.02 for stiff and plastic clays and to 0.16 for soft clays. And the average value is 0.061. Correlations have been found with fundamental parameters of clays as the water content w , the plasticity index PI , the liquidity index LI or the overconsolidation ratio OCR . Afterwards, the authors applied the same equation (Equ. II-29) to 31 couples of load tests performed with two different loading rates on same or on similar piles. The scattering is larger but the range and the average value of n remains similar (average=0,068). Equ. II-29 becomes:

$$\frac{Q_{u1}}{Q_{u2}} = \left(\frac{t_2}{t_1} \right)^n \quad \text{Equ. II-30}$$

Q_{ui} are the ultimate pile capacities reached during test at the time t_i .

The spreading of n is due to the scale and averaging effects between the sample in laboratory tests (10 cm high) and the pile shaft dimensions (several meters length). It is also due to the undrained behaviour maybe not encountered in the slow pile load tests. Site specific values of n can be obtained by performing pressuremeter or cone penetrometer tests at significantly rates of penetration (Briaud et al. 2000). Tests performed in 2000 in sand show that this approach is still valid in non-cohesive soil.

Brown et al. (2002) & Anderson et al. (2003) propose a similar approach to study the load rate effect in clay with a model pile in a calibration chamber. Clay (mix of Kaolin with sand and silt) is tested in an instrumented calibration chamber (pore pressure transducers and accelerometers in the soil). The partially embedded model pile has a diameter of 70 mm and more than 1 m length. It is instrumented in order to measure the pile tip and skin friction loads (load cells) to differentiate both rate of testing effects. The testing program includes Constant Rate of Penetration tests carried out at 0.01, 1, 10, 100 and 500 mm/s (Figure II-26) and load pulse similar to the Statnamic testing (pulses of 150 ms with target loads of increased value and corresponding increased settlement rate - Figure II-27).

It is shown that the magnitude of pore pressure increased with increasing rate of pile penetration, that the stiffness measured during the Statnamic pulses increases with the target loads.

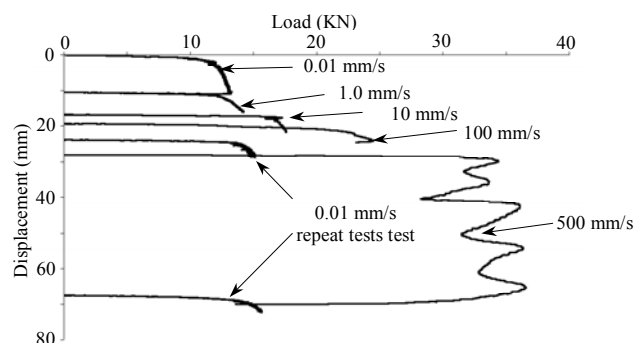


Figure II-26 : Pile load deflection response to CRP tests (after Brown et al., 2002)

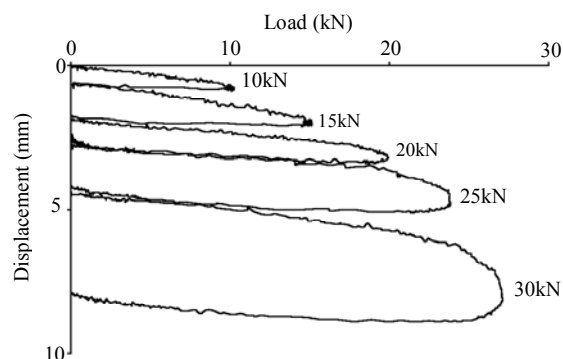


Figure II-27 : Pile load deflection response to static load pulses (after Brown et al., 2002)

The load capacity of the pile is found to be particularly sensitive to the pile deformation rate with significant increase for the rates between 10 to 500 mm/s but this effect is more pronounced for the shaft than the base component of the pile capacity. The static reference pile capacity is chosen as the result of the CRP test performed at 0.01 mm/s. Figure II-28 shows the loading rate effects expressed through the ratio of maximum soil reaction during the CRP tests. The result from Figure II-28 can be modelled by a power law of the Coyle and Gibson type (Equ. II-35). The authors specify that more tests must be performed to differentiate correctly both trends corresponding to the base and shaft friction behaviours (rates between 10 to 100 mm/s and 100 to 500 mm/s).

$$\frac{R_r}{R_s} = 1 + \alpha \cdot \left(\frac{v}{v_0} \right)^\beta$$

Equ. II-31

- v is the pile penetration velocity [m/s];
- v_0 is a normalizing velocity [m/s];
- α and β are the rheologic parameters.

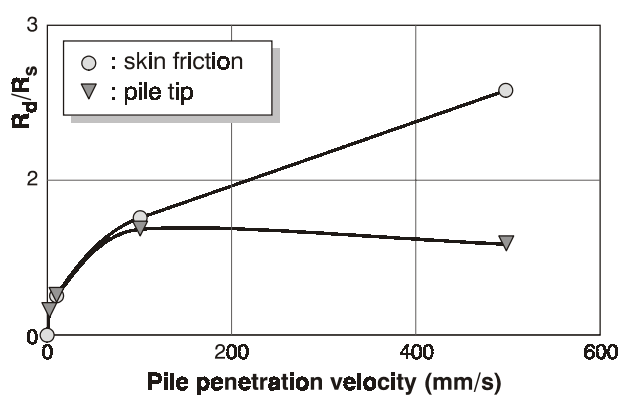


Figure II-28: Rate effects for the pile and base components (after Brown et al., 2002)

The normalizing velocity is taken as the velocity of the static reference test. Here, it is the velocity of the CRP test performed at 0.01 mm/s or 10^{-5} m/s. The values α and β based on the best fit curve possible are 1.22 and 0.32 respectively. A detailed study of the strength behaviour of the consolidated clay has been carried out by **Balderas-Meca (2003)**. He carried out consolidated undrained (CU) triaxial tests at different rates of strain on 100 mm diameter specimens simulating the stress history followed in the calibration chamber. The deviator stress at 3% axial strain was taken as the failure stress. Secant

elastic modulus values, E_{50} , were calculated using the strain at which 50% of the failure deviator stress

was reached. Values of the undrained strengths and moduli obtained at different strain rates increase significantly as the rate of testing increases. The values of the associated rheologic parameters α and β for those two quantities are 0.53 and 0.19 for the undrained shear strength and 1.85 and 0.15 for E_{50} . Brown (2003) suggests to modify the classical analysis method of the Statnamic tests to incorporate the non linear loading rate effect (power law).

5.2.2. Logarithmic relationship with proportionality to the static soil reaction

In parallel to the power relationship of the velocity, a second type of law has been developed by many authors especially based on laboratory testing (except for Bea): the logarithmic relationship of the velocity with the use of a reference velocity (corresponding to the reference static test).

Dayal & Allen (1974, 1975) work on cone penetration tests performed at high velocity. All the transducers of the classical device are available (cone resistance, unit shaft friction). CPT-E is used to penetrate the soil with a constant velocity. The velocities range from 0.0044 ft/s (0.0013 m/s) to 2.662 ft/s (0.812 m/s). The tested soils are dry sand and clayed silt. The cone resistance results show a variation of the dynamic resistance with the loading rate. The authors use the Peck law (1962) in order to analyse the results:

$$\frac{c_u^d - c_u}{c_u} = 0.1 \cdot \log\left(\frac{\dot{\epsilon}^d}{\dot{\epsilon}}\right) \quad \text{Equ. II-32}$$

c_u^d and c_u are the undrained cohesion corresponding to the strain rates $\dot{\epsilon}^d$ and $\dot{\epsilon}$, respectively.

This formula is standardised for the cone approach:

$$\frac{q_c^d - q_c}{q_c} = K_l \cdot \log\left(\frac{v}{v_s}\right) \quad \text{Equ. II-33}$$

Where:

- q_c is the cone resistance at the velocity v ;
- q_c^d is the cone resistance at the reference velocity v_s ;
- K_l is the coefficient of soil viscosity (dependent of the soil nature, the velocity range).

Equ. II-33 is similar to Equ. II-36:

$$\begin{aligned} \frac{q_c^d - q_c}{q_c} &= K_l \cdot \log\left(\frac{v}{v_s}\right) = \frac{q_c^d}{q_c} - 1 \\ \Rightarrow \frac{q_c^d}{q_c} &= 1 + K_l \cdot \log\left(\frac{v}{v_s}\right) \end{aligned} \quad \text{Equ. II-34}$$

Bea (1982) analysed the results of pile load testing (0.355 m diameter tubular pile in plastic clay of the Gulf Coast) based on the works of Cox et al. (1979) (Figure II-29) and proposes the following law:

$$\beta = \frac{Q_{rheol}}{Q_{stat}} = F_1 + F_2 \log\left(\frac{\lambda}{\lambda_s}\right)$$

Equ. II-35

Q_{rheol} is the rheologic soil reaction [kips];

λ_s is the reference loading rate [% Q_{ult}/h];

Q_{stat} is the static soil reaction [kips];

$F_1 = 0.75$ to 1 [-];

λ is the loading rate [% Q_{ult}/h];

$F_2 = 0.10$ to 0.20 [-].

An attempt is made to link F_2 to fundamental characteristics of clays: OCR, liquidity index LI .

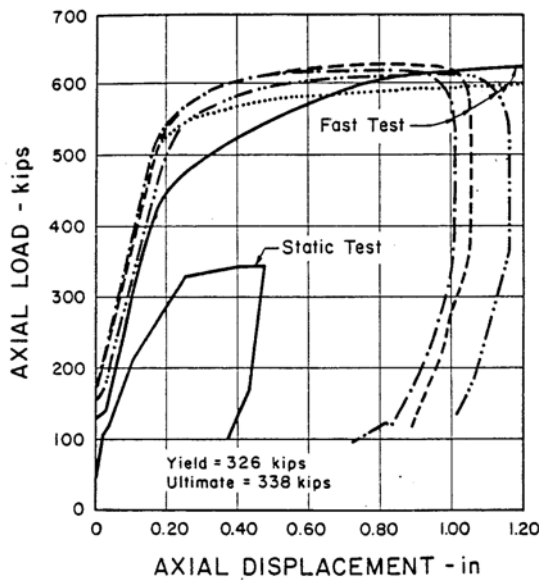


Figure II-29 : Loading rate effects from pile load tests (after Cox et al., 1979)

The majority of the research undertaken in laboratories was focused on the influence of an increase of strain rate on the undrained shear strength of clays S_u . The main results for tests in clay demonstrate that an increase of a factor 100 of the strain rate causes an increase of 25% of S_u (Vail et al., 1979); Graham et al (1983) show that triaxial compression tests increase by about 10...20% the value of S_u for a 10 times increase of the strain rate. Kulhawy & Mayne (1990) work on 26 different clays (normally and overconsolidated) and found an increase of about 10% per cycle of logarithm of strain rate. A tenuous influence of loading rate can be felt on fine sands (Seed and Lundgren, 1954; Whitman and Healy, 1962).

All these results can be modelled by using a dashpot in parallel with a soil spring (responsible of the static term of the soil reaction). The damping parameter is found by multiplying the relative pile velocity with respect to soil by an equivalent damping coefficient C_{eq} :

$$C_{eq} = \frac{\beta}{V} \log\left(\frac{V}{V_{ref}}\right) \rightarrow Q_{rheol} = Q_{stat} \cdot \left(1 + \beta \cdot \log\left(\frac{V}{V_{ref}}\right)\right) \quad \text{Equ. II-36}$$

- Q_{rheol} is the rheologic soil reaction [N];
- Q_{stat} is the static soil reaction [N];
- β is the rate of increase in shearing resistance per log cycle of increase in strain;
- V is the relative velocity of the pile with respect to the soil [m/s];
- V_{ref} is the reference (or static) velocity: 10^{-3} inch/s or $0.0254 \cdot 10^{-3}$ m/s.

All the results can be used in numerical analysis by replacing the value of β by the found percentage of increase of shear resistance per log cycle of strain rate (after Chacko et al., 2002).

Matsumoto & Nishimura (1996) present the results of CU triaxial tests performed on diatomaceous mudstone (soft rock). Tests were performed with two different axial strain rates ($\dot{\epsilon} = 0.1$ and 0.01 %/min - [Figure II-30](#)). Both mudstone samples show a peak shear resistance and thereafter undergo a gradual reduction in shear resistance with increasing strain. The increase of undrained shear strength is similar to the previous results: 12% for a log cycle of strain rate. This deformation rate dependency of the shear resistance of the mudstone is also reported in Sekiguchi et al. (1985) and Maekawa et al. (1991).

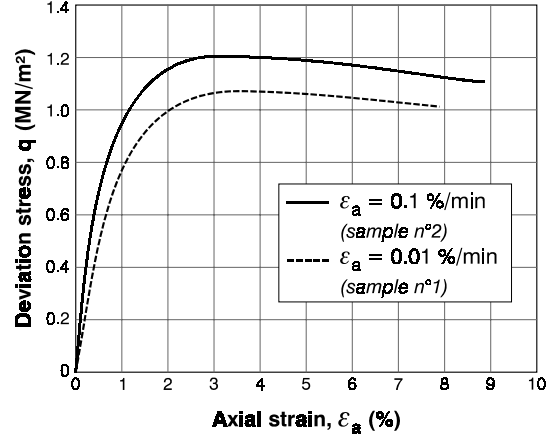


Figure II-30: Stress-strain curves under two shearing rates (after Matsumoto et al., 1996)

Eiksund and Nordal (1996) report on a model pile testing program in Ottawa sand where the dynamic soil resistance and the pore pressure response near the pile base is studied during driving. The model pile is a closed end steel tube of 107 cm long and 63.5 mm in diameter. The studied penetration velocities range from 0.8 to 1800 mm/s. The pile is instrumented with strain gages, accelerometers and LVDT and pore pressure transducers are installed in the soil. The observed increase in resistance is less than 10% for an increase of more than 1000 times in penetration velocity. The effect of the loading rate in sand is quasi non-existent.

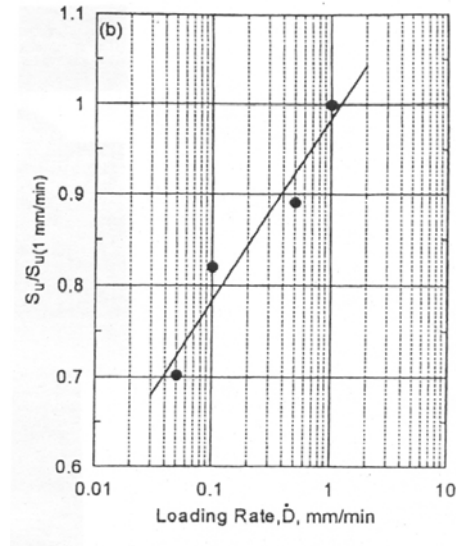


Figure II-31 : Undrained shear strength versus log loading rate (after Al-Shamrani, 2001)

Al-Shamrani (2001) carries out CU triaxial tests on clay samples by increasing the axial strain with a constant rate (0.05, 0.1, 0.5 and 1 mm/min) in order to determine the influence of loading rate on the undrained shear strength. [Figure II-31](#) shows a clear linear relationship in the semi-log plot for the S_u evolution which is similar to Equ. II-36. This linear relationship can be expressed by:

$$S_u(\dot{D}) = S_u(\dot{D}_{ref}) \left[1 + \beta \cdot \log \left(\frac{\dot{D}}{\dot{D}_{ref}} \right) \right] \quad \text{Equ. II-37}$$

- $\dot{D}_{ref}$ and $\dot{D}$ are the reference and the analysed loading rate, respectively;
- $S_u(\dot{D}_{ref})$ and $S_u(\dot{D})$ are the undrained shear strength for $\dot{D}_{ref}$ and $\dot{D}$ respectively;
- β is the rate parameter defined as the slope of the straight line of the undrained shear strength vs the logarithm of the normalized loading rate.

The loading rate of reference ($\dot{D}_{ref}$) is set to 1 mm/min (0.0167 mm/s). The increase of S_u per log cycle of loading rate is found equal to 21.8 %, i.e. in general accordance with the above-mentioned results found in literature. Consequently, the value of β is fixed to be around 0.2. The author points out that the value of β and the shape of the modelled curve on the results are dependent of the range of studied loading range. Indeed, there is some authors that suggested a certain threshold limits the influence of the strain rate on the undrained shear strength for very slow loading rates. The author notes a stiffening effect and an increase of the initial tangent modulus with the strain rate (Al-Shamrani, 2001).

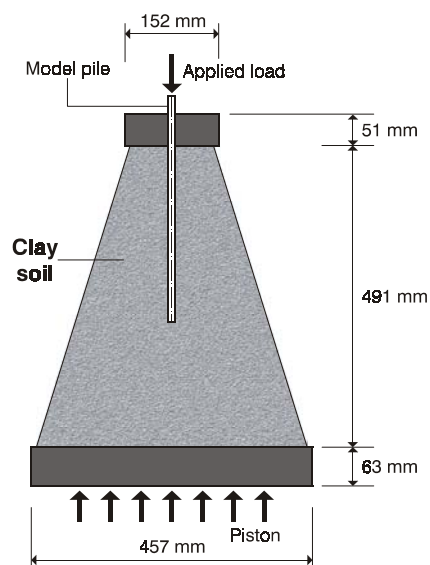


Figure II-32 : Cone-shaped soil chamber (after Horvath, 1995)

In order to find this straight line, the author interprets the results with the equilibrium point method (Midendorp et al., 1992; Horvath et al, 1993). This method is used to account for the rate effects in the statnamic tests interpretation. Horvath applies the method to the QCL and QML adjusted raw results (open square on Figure II-33) to find the adjusted failure loads (black triangles on Figure II-33). He concludes that the straight line is the real influence of the loading rate on the pile capacity without taking the soil damping (according to the author) into account. He concludes also that the soil damping is significant for QCL tests only and is negligible for the QML tests. (Q_{CRP-f} is the CRP failure load.

Horvath (1995) carried out a set of loading tests (CRP-QML-QCL)³ on a model pile embedded in clay. The CRP results are used as reference. The duration of the QML tests is about 0.1 s and of the QCL tests 1.5 to 17 min. The CRP test is performed with a rate of 2 mm/min and the test duration is about 15 s. The maximum load applied on the pile is considered as the normative failure load (Q_{CRP-f}). The model pile is a closed-end steel pipe with a diameter of 12.7 mm and a length of 508 mm. The used soil is a low plasticity clay in an inverted steel cone confining chamber allowing a linearly increasing confining stress on the sample (0 at the top, maximum at the bottom) (see Figure II-32). The pile tests were loaded until failure occurs (when the pile is plunging). The failure load is taken as ultimate applied load. Figure II-33 indicates the influence of rate of loading on the capacity of the pile. The increase of failure load due to the loading rate effect can be plotted with a straight line in a semilog plot.

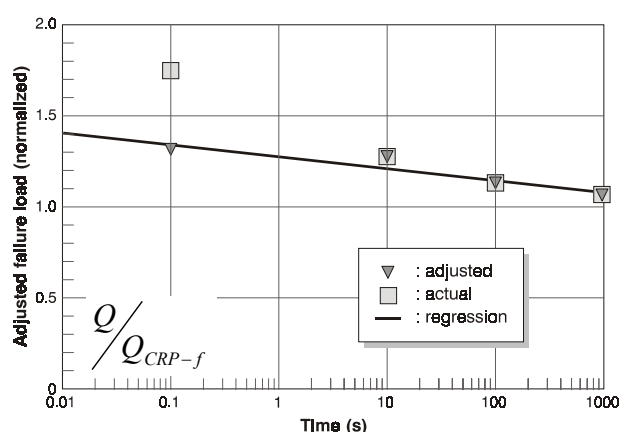


Figure II-33 : Adjusted failure load versus time (logarithm scale) (after Horvath, 1995)

³ CRP means Constant Rate of Penetration Test, QML means Quick Maintained Loading test and QCL means Quick Continuous Loading test.

However, using the Briaud approach (Equ. II-30), and using the time to reach failure instead of the nominal loading durations in a log-log plot, Figure II-33 becomes Figure II-34. The raw points of the Horvath's paper follow a straight line that may be modelled by a power or a logarithmic law. The value of the viscous exponent related to the Briaud and Garland approach equal 0.058 which correspond to the expected range of value for a clay of type "low plasticity". Afterwards, with the adjustment made with respect to Q_{CRP-f} , Equ. II-38 can be written.

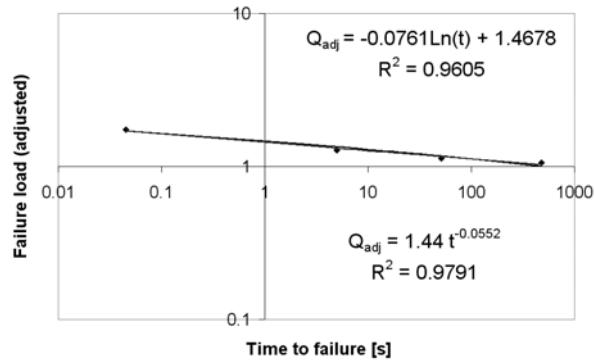


Figure II-34 : Application of the Briaud approach to the results of Horvath, 1995

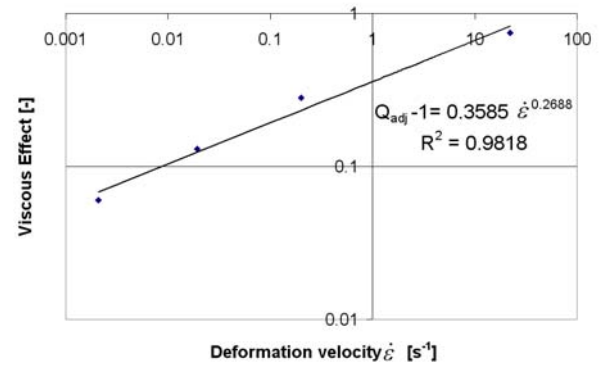


Figure II-35 : Viscous effect versus velocity of deformation

$$Q_{adj} = \frac{Q_{meas,dyn}}{Q_{ref}} = f(Q_{stat}, viscous\ effect)$$

Equ. II-38

If the general Smith approach is followed:

$$Q_{rheol} = Q_{stat} \cdot (1 + viscous\ effect)$$

Equ. II-39

$$Q_{meas,dyn} \equiv Q_{rheol} \quad \& \quad Q_{ref} \equiv Q_{stat}$$

$$Q_{adj} - 1 = Viscous\ effect$$

The velocity of deformation is defined as the inverted time to failure (Holeyman, 1984): $\dot{\epsilon} = \frac{1}{t_f}$ and

Figure II-34 becomes Figure II-35.

The exponent (0.2688) is close to the expected value of 0.2 from the Coyle and Gibson approach and

$$Q_{adj} = \frac{Q_{meas,dyn}}{Q_{ref,stat}} = 1 + J_- \cdot \dot{\epsilon}^{N_-} = c \cdot \dot{\epsilon}^n$$

Equ. II-40

- J. is the damping parameter in a dimensionless formulation of the Coyle-Gibson model;
- N. is the exponent in a dimensionless formulation of the Coyle-Gibson model;
- n is the viscous exponent of the Briaud model.

There is a relationship between the rheologic parameters of both models.

These developments show that all theories mentioned above are connected by mathematical approaches but finally describe the same behaviour with a similar trend, which is strongly non-linear.

5.3. Loading Rate Conclusions

Except for the Briaud and Garland form, all other power relationships can be written in a generic form:

$$\frac{Q_{rheol}}{Q_{stat}} = (1 + J_{LRE} \cdot V^N) \quad \text{Equ. II-41}$$

J_{LRE} and N are the rheologic parameters accounting for the loading rate effects on the bearing capacity of the test performed at a velocity V compared to a test performed at a quasi null velocity.

All logarithmic relationships can be transformed in a generic formulation:

$$\frac{Q_{rheol}}{Q_{stat}} = \left(1 + Y_{LRE} \cdot \log\left(\frac{V}{V_{ref}}\right) \right) \quad \text{Equ. II-42}$$

Y_{LRE} is the rheologic parameter accounting for the loading rate effect on the bearing capacity between a test performed at a velocity V_{ref} (the static test for example giving the capacity Q_{stat}) and the rapid test performed at the velocity V giving the capacity Q_{rheol} .

Except for the proposed law of Briaud and Garland, all proposed power laws work with the absolute velocity of the body considered (the pile, the soil or the in between interface). The reference test is considered with a null velocity. On the other hand, all logarithmic relationships consider a reference velocity, used in the denominator of the analysed velocity. This chosen reference has a great influence on the rheologic parameters (Holeyman, 1984; Al-Shamrani, 2001) like the units of the absolute velocity of the power law category.

The power laws family can be seen as a model type describing an absolute behaviour. The power law assumes that the effect of a increasing loading rate is always an addition of soil resistance. The logarithmic laws family describes a relative behaviour between two tests performed at different velocities. The evolution of the corresponding capacities is evaluated with the logarithmic relationship in function of the corresponding loading rates. Consequently, it is possible to compare two rapid tests, or a test performed with a slower loading rate than the reference one (with a ratio $\frac{Q_{rheol}}{Q_{stat}} < 1$).

All the tests performed concern tests conducted to failure. The tests results used for the models calibration are supposed to be the failure loads, corresponding to the ultimate resistances of soil under the submitted solicitations⁴. The conclusions can only concern a sort of envelope of the loading rate effect on the bearing capacity. It supposes the high strain state of the materials. Nevertheless, the

⁴ Under the assumption that the failure under static and dynamic solicitation is comparable.

results of the tests performed in Sheffield (Anderson et al. 2003) have shown that the results ($S_{u,3\% \epsilon} - E_{50}$) of undrained triaxial tests on clay-silt-sand samples were also influenced by the loading rate.

6. Conclusions

The testing programs and the evolution of the pile/soil models give the keys to delimit the knowledge existing about the loading rate effects. This section tries to resume the above mentioned results in terms of small and large strains, low and large velocities and types of soil.

6.1. Approach low and large strains

6.1.1. Low strains

The main effect due to the loading rate variation in the range of low strains is the radiation damping described by Novak et al. (1978) and used by many authors to totally or partially take the velocity dependent term into account. Unfortunately, too few tests have been performed focused on the loading rate effects on small levels of strain allowing the measurements of energy radiation in a large media. Most of the representative tests performed on a small level of deformation are laboratory tests which do not include the geometrical effect because of their characteristic size. However, there is a need to describe the evolution of the radiation damping because of the possible evolution of its parameter with the strain or stress level (the shear modulus) and because of its significant importance in the rheologic soil reaction especially for low strains.

The particular need to study the small strain range is motivated by the global understanding of the phenomena and also the need to correctly model the complete load history and its effect on the pile/soil interaction including the loading and unloading phases. Indeed, currently, the only thing sure is the validity of the general Coyle-Gibson relationship for ultimate loads and large strains. But these formulations are often used unconditionally on the all load-time history to quantify the loading rate effect.

Rausche et al. (1992) tried to take into account of a specific behaviour in function of the strain level and the loading path. Others researchers involved in the analytically based models tried to do the same approach (by introducing the viscous component) but without any real verifications of the behaviours involved.

6.1.2. Large strains

In the range of large strains, the main loading rate effects are based on dissipative and viscous dampings. Indeed, these two terms are theoretically significant because of the reduction of the radiation damping effect due to the degradation of the shear modulus and the entrance in plasticity. Since all the tests performed to investigate the loading rate effects are generally carried out until failure occurs, the relationship obtained and established as a power law function of the velocity could be interpreted as the sum of these terms. This is based on the fact the radiation damping theoretically vanishes when the soil enters in plasticity. But is this last assumption correct? Or is there still a radiative term for large strains despite the degradation of the shear modulus in the vicinity of the pile? For the base model, the assumptions generally express that the radiation continues to be generated when the plasticity increases. For the shaft, the assumptions assesses the opposite. The difference is

explained with the occurrence of slippage or not. There is a need to understand the radiation occurring at large strains in the close vicinity of the pile shaft.

Second, the high strain results are well known and usable for the characterization of the ultimate rheologic soil resistance depending on the velocity. There is a need to describe accurately the all different terms involved.

6.2. Approach low and large velocities

6.2.1. Low velocities

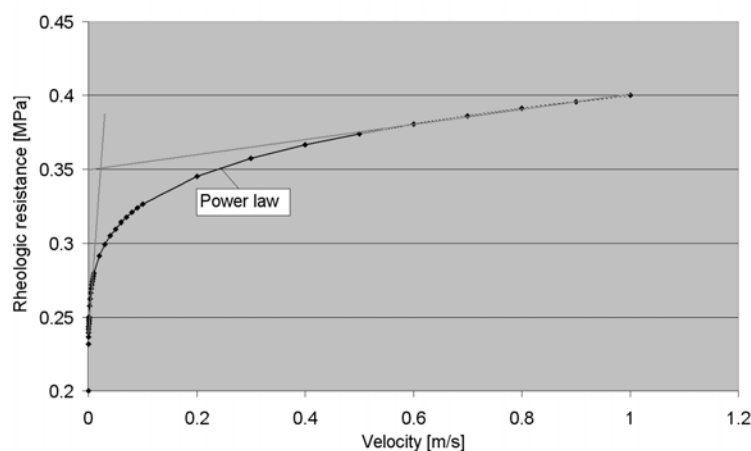


Figure II- 36 : Power law or juxtaposition of two different behaviours for the loading rate influence on the rheologic stress

The tests performed in laboratory and on footing models allow to assess a logarithmic relationship in the range of small velocities that seems to be stiffer than the power-law relationship. Others testing programs investigate the large range of velocities correctly described by the power law, but the trend found is not wide enough to cover a large range of velocities. The problem resides in the connection between both velocity ranges (Figure II- 36): the static and the dynamic ranges of velocities or the stiffest part of the

power-law. Is the power-law relationship representative of one sole behaviour or a juxtaposition of two behaviours of which the predominance is complementary? The domain of low velocities remains unexplored in terms of load rate effects and specific devices must be set-up in order to apply loading with small velocity.

6.2.2. Large velocities

A major part of the tests performed on model piles or on real piles in soil with the purpose to highlight the loading rate effect is carried out with a large velocity. The authors generally tried to vary the range of maximum velocities but the size and the devices used often impose to have a substantial velocity. Consequently, the tested velocities are still too high to analyse the first part of the theoretical power law curve that plunges suddenly to the value 1^5 . The consequences on the constitution of the models and on the further analysis are important because of the loading history where the velocity is not constant and maximum all the time, but evolves from zero to a maximum value and return back to zero. All the velocity and all the strain ranges must be investigated to have a general overview of the effect of the loading rate on the bearing capacity and on the load settlement curve. Currently, the major parts of the conclusions available concern large velocities.

⁵ Where the rheologic resistance is equal to the static one for $v=0$

6.3. Clay and sand particularities

It is widely recognized that cohesive soils have a significant dependency to rate of loading. It comes from observations, experience and particular researches performed on the soil materials. On the other hand, even if this behaviour is assessed as similar for the sands, the amount of information available is much less larger. In the particular study of the loading rate effect, a major part of the tests performed concerns exclusively clay or soil containing a major part of clay. The results respect generally the same trend: a large, pronounced and non linear relationship between the rheologic soil resistance and the loading rate. Attempts were made to classify the rheologic parameters in function of the pile type, it was seen a large scattering of the results forbidding to have the smaller conclusion. The trend is there but not correlated to the soil type.

Sand has the poorest range of investigation and displays contradictory results (Svinkin, Eiksund, Paikowsky, Coyle & Gibson, Litkouhi & Poskitt). Sometimes a velocity trend exist, sometimes not. The explanation might come from the mix of the sand with one or several more “viscous” other materials. The sand can be saturated, mixed with a fine materials or a portion of clay or completely dry and pure. This last classification is almost insensitive to the loading rate where a mix of sand and clay shows a behaviour similar to the clayed one. These observations can support the assumption of a need of more tests, preferably in laboratory in order to evaluate the different components of the loading rate effects.

The following table summarizes the data available for each classification and highlights the necessity to increase the knowledge and to perform tests for sandy material and especially for small strains and velocities. The darker zone is the most characteristic and common tested range.

	Low strains		Large strains	
	<i>Clay</i>	<i>Sand</i>	<i>Clay</i>	<i>Sand</i>
Low Velocity	Cyclic lab tests	Cyclic lab tests	Model piles Lab tests	Models piles Lab tests
Large Velocity	Difficult to perform	Difficult to perform	Piles and models Lab tests	Piles and models Lab tests

Table II-5 : Summary of the data available in function of the strain, velocity level and of the sandy of clayey character of the soil