# An Alternative Method to Estimate the Effective Energy During Pile Driving Based on Set and Elastic Rebound Records

A. Querelli, F. Massad

**Abstract.** Although the dynamic load test technique has been developed for a long time, it became very popular in Brazil over the last 20 years (especially after the publication of Brazilian standard NBR6122, in 2010). However, the site quality control of driven piles in respect to the resistant capacities remains with the practical application of the so-called "dynamic formulas". Most of the established formulations use a key parameter of the driving event: the driving system efficiency, whose calibration is performed with dynamic monitoring. However, assuming a single value for the efficiency in order to extrapolate it to non-tested piles of the site requires a much more careful analysis of the test than that which has been carried out in practice. The aim of this paper is to discuss the issues involving the efficiency assessment based on the dynamic test and to propose an alternative method of estimating the effective energy during driving through measurements of set and elastic rebound. The method application was verified in comparison with 692 dynamic test records (526 in concrete and 166 in steel piles) of twelve sites spread across seven different locations in Brazil.

**Keywords:** driven piles, dynamic testing, PDA, effective energy, efficiency, elastic rebound, set.

## 1. Introduction

In order to control the resistant capacity of driven piles, the Brazilian practice is based on two different approaches: by sampling or the universal one. The sampling approach is limited to a small number of piles. It consists in performing pile load tests, sometimes static, but mostly dynamic. The universal controls are the ones extended to all piles of the site. There are two of them to highlight: the set and the elastic rebound. About them, the Brazilian standard NBR6122 states: "the set and the elastic rebound must be measured in all piles" (ABNT, 2010).

The measure of set ($s$) refers to the permanent (plastic) displacement of the pile after a single hammer blow (ABNT, 2010). However, even the Brazilian standard itself recognizes that the magnitude of such a measure is extremely small, making it difficult to be precisely read. Aiming to make it easier, the technical community agreed to denominate as "set" the average permanent displacement of ten sequential blows of the hammer (Alves *et al.*, 2004). The set is the main tool that a site engineer has for demanding the end of driving: in Brazilian practice, a certain value of set is specified to ensure the desired resistance after driving.

The elastic rebound ($K$) is a dynamic measure which indicates the elastic displacements recovered in the blow. Alonso (1991) defines it as "the elastic portion of the maximum displacement of the pile". It is composed by both the elastic shortening of the pile element itself ($C2$) and the elastic displacement of the soil under the pile tip ($C3$), also called "toe *quake*", as displayed by Eq. 1:

$$K = C2 + C3 \quad (1)$$

Both set and rebound can be obtained simultaneously and in a very simple and inexpensive way: a pencil is supported by a stationary reference with its tip pressed against a piece of paper attached to the pile. Then, the pile is hammered (usually ten times), resulting in a graph such as presented in Fig. 1.

In Fig. 1, the record was performed for ten strokes: the measured set ($s$) is 1.3 mm (average penetration) and the elastic rebound ($K$) is 20 mm.

The main utility of these two measurements is to estimate the pile resistant load by means of dynamic formulas. Those equations are very familiar to the technical community and still widely used as tools for design and quality control of driven piles. These formulas are intended to obtain the mobilized resistance by means of: a) the elastic rebound ($K$), b) the set ($s$) and c) the effective applied energy, considering all sources of loss.

There are several criticisms regarding dynamic formulas. Fellenius (2018), for example, stated: "basing a pile design today on a dynamic formula shows unacceptable ignorance and demonstrates incompetence". It is a tough, but reasonable assertion: because of the inherent uncertainties and scatter, the dynamic formulas should not be the primary basis of a foundation design. Instead, some authors highlight that they could be used as auxiliary tools in the quality

André Querelli, M.Sc., Civil Engineer, Escola Politécnica, Universidade de São Paulo, São Paulo, SP, Brazil, e-mail: andre.querelli@gmail.com.
Faiçal Massad, Ph.D., Full Professor, Escola Politécnica, Universidade de São Paulo, São Paulo, SP, Brazil, e-mail: faical.massad@usp.br.
Submitted on March 12, 2019; Final Acceptance on June 2, 2019; Discussion open until December 31, 2019.
DOI: 10.28927/SR.422179

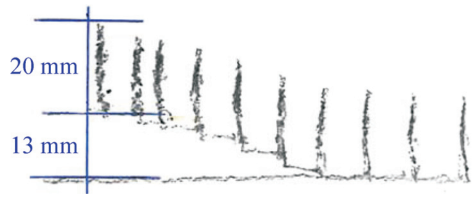


**Figure 1** - Examples of set ($s$) and elastic rebound ($K$) records (not to scale).

and performance control of driven piles. Alves *et al.* (2004) mention that "the use of these formulas should be restricted to the control of the piling uniformity" and also "as an end-of-driving criterion". As Chellis (1951) says: that calculated resistance "is only one item of design information that must be considered with other conditions in order to design the pile foundations intelligently, safely and economically".

## 2. The Danish Formula

At this point, it is important to highlight the well-known "Danish Formula" in order to support the objective methodology of this paper.

It was proposed by Sorensen & Hansen (1957) based on the Engineering News Formula, published by Arthur Wellington in the 20th issue of the journal (1888) and one of the most widely used dynamic equations in the last century (Likins *et al.*, 2012).

The Danish Formula is written as:

$$R = \frac{\eta \cdot W \cdot h}{s + \frac{1}{2}\sqrt{\frac{2 \cdot \eta \cdot W \cdot h \cdot L}{A \cdot E}}} \quad (2)$$

where $R$ is the pile static resistance, $\eta$ the hammer efficiency, $W$ the ram weight, $h$ the drop height, $s$ the pile penetration per blow (set), $L$ the pile length, $A$ the area of the pile cross section and $E$ its dynamic modulus of elasticity (as shown in List of symbols).

Caputo (1983) said that the Danish Formula is "generally applied for steel piles", although its developers Sorensen & Hansen did not make that kind of statement.

Something to emphasize in that formulation is the square root term in the denominator of Eq. 2. Rosa (2000) identified it as the "energy loss due to the elastic deformation of the pile". Sorensen & Hansen (1957) defined it as the "dynamic compression of a pile with a fixed point", $S_o$:

$$S_o = \sqrt{\frac{2 \cdot \eta \cdot W \cdot h \cdot L}{A \cdot E}} \quad (3)$$

In this way, it is possible to observe in Eq. 2 the "tripod" of closely related values of the driving event, which are present (implicitly or explicitly) in almost all dynamic formulas: resistance ($R$), permanent and elastic displacements ($s$, $S_o$) and the effectively transferred energy ($\eta.W.h$).

There are many other equations that are analogous to the Danish Formula, but using the elastic rebound ($K$) or any other form of total elastic displacement in place of $S_o$, as done by the Engineering News Formula (1888), the Hiley's formula (1925) and its variation. One recent and well-known formula that matches this similarity is the complete Energy Approach Equation (Paikowsky & Chernauskas, 1992), which can be written as:

$$R = 2 \cdot K_{sp} \cdot \frac{\eta \cdot W \cdot h}{s + D} \quad (4)$$

where $K_{sp}$ is a coefficient of energy loss by viscous damping effects and $D$ is the maximum pile displacement after blow (sum of $s + K$).

## 3. Dynamic Load Test and The Brazilian Practice

Concerning the dynamic load test, this paper does not aim to discuss the one dimensional wave propagation theory nor the Case and CAPWAP analysis methods, since all of those subjects are widely explored in the technical literature. Therefore, a brief description of the test will be given, highlighting its use and functionalities.

The dynamic loading test (DLT) is one of the most used field tests for pile driving quality control and pile long term performance evaluation. It is based on the one dimensional wave propagation theory and it is standardized in Brazil by NBR13208 (ABNT, 2007).

Basically, it consists in monitoring a pile with both deformation transducers and accelerometers in response to hammer blows. The mostly used microcomputer to perform that test is called "Pile Driving Analyzer", PDA (Beim, 2009).

After the blow, measurements of strain and acceleration are recorded by PDA and converted into two curves over time, related to pile top: one of force ($F$) and another of particle velocity ($v$) multiplied by pile impedance ($Z$). The force is calculated through strain (Hooke's Law) and velocity is calculated by integrating the accelerometer measurements over time. The impedance $Z$ is given by Eq. 5.

$$Z = \frac{E \cdot A}{c} \quad (5)$$

All of these data is readily analyzed by the Case Method and later on by the CAPWAP (Case Pile Wave Analysis Program).

Based on the Case Method analysis, assessments can be made for hammer performance, transferred energy, pile stresses, pile integrity and capacity at the time of testing. These applications have been described by many authors, including Goble *et al.* (1980), Rosa (2000), Alves & Lopes (2004), Vieira (2006), Saldívar (2008), Gonçalves *et al.* (2007), Beim (2009) and many others, which makes it challenging to try to quote them all. Some others, such as Niyama *et al.* (1984), Hussein *et al.* (1993), Rausche *et al.*

(2004), Likins & Rausche (2008), Paraíso & Costa (2010) and Tokhi (2012), for example, additionally emphasized the possibility of evaluating the increase (set-up) or decrease (relaxation) of pile resistant capacity over time.

In Brazil, the current practice of the dynamic load test consists in the procedure proposed by Aoki (1989), which is commonly named as "increasing energy load test". It is performed after the end of driving and the applied energy during the test is gradually increased (from one blow to another) by varying the hammer drop height. The main objective of it is to obtain the maximum mobilized resistance of the pile. Gonçalves *et al.* (2007) pointed out that collecting data of increasing hammer drop heights allows finding sufficient pile displacements to fully mobilize the pile resistance.

As the test energy increases, some care should be taken with sensitive clays due to the loss of shaft resistance in soil remoulding. To overcome that problem, Valverde & Massad (2018) suggested using the concept of "maximum lateral resistance envelope", allowing estimating the mobilized resistance along the shaft that was lost in the first blows of the test.

## 4. Maximum Transferred Energy and Efficiency Calibration

Among the information that results from the dynamic load test (DLT), there is one item to highlight in the present paper: the maximum transferred energy to the pile, designated in PDA as *EMX*. It is calculated through integration of the product of measured force by particle velocity in time, as shown by Eq. 6.

During the DLT, one of *EMX*'s main functions is to confirm to the operator that the effectively applied energy is appropriate to the type of test being performed (constant energy, increasing energy, etc).

$$EMX = \max\left[\int F(t) \cdot v(t) \cdot dt\right] \tag{6}$$

In addition, *EMX* also contributes to estimate the efficiency of the driving system ($\eta$), *i.e.*, the ratio between *EMX* and the nominal applied energy (product of hammer weight by drop height), as displayed by Eq. 7.

$$\eta = \frac{EMX}{W \cdot h} \tag{7}$$

The efficiency of the impact has great value, not only for instant evaluation of the driving system at the time of test, but also to support the use of dynamic formulas in the quality control of the non-tested piles of a given site.

Tavenas & Audy (1972) conducted a study on the limitations of the dynamic formulas, comprising 478 records of pile driving in non-cohesive soil and 45 static load tests. One of their conclusions was that the largest source of scatter using dynamic formulas resided in the wrong consideration of the energy (as nominal, *i.e.* '*W.h*'), without correctly accounting for the energy losses (or an efficiency factor). And so, quoting these authors: "The usual energy estimate being proved erroneous, it is possible to conclude [...] that any pile driving formulas in which this estimate will be used will also be erroneous".

Therefore, one of the major problems when extrapolating dynamic formulas (calibrated by sampling) to all piles of a site is the correct evaluation of the efficiency.

As mentioned by Goble & Likins (1996), the specifications of each dynamic test, *i.e.* the way it will be performed, depends on the purpose for which it is intended. Thus, the authors state: a test to estimate hammer performance requires different procedures compared to a test scheduled for static resistance evaluation.

That raises some criticism to the Brazilian methodology of increasing energy dynamic test in the estimation and adoption of a driving efficiency. The first problem is that just a few strokes are applied in this type of test: typically three to seven blows, which makes the statistical sampling for efficiency evaluation very poor.

In addition, the method requires non sequential blows (different from driving situation): a first blow is applied, then the PDA operator evaluates the calculated information, and only after a period of time, the operator requests another hammer blow to the pile driver. There are no sequential blows, which would take advantage, in terms of efficiency, of the driving "pace" and inertia. It causes some problems both for free fall and hydraulic hammers:

- In free fall hammers, the non-sequential blows require the pile driver operator to use the clutch and the hammer brake in a way that causes some random energy losses, messing with the efficiency evaluation. In a case study of increasing energy test, Rausche (1997) reported efficiency variation between 35% and 52% after a 6 blows test. He concluded that scatter was due to "crane operator effects" and hammer "alignment problems".
- In hydraulic hammers, isolated blows are not setup automatically. For that situation, the operator needs to quickly regulate the oil pressure (increase and instantly decrease) by a manual switch. That causes the drop height not to be exactly the desired one. This situation occurs even with an experienced operator, resulting in poor efficiency evaluation.

That kind of scatter shows that the Brazilian practice of increasing energy test may be unsuitable for evaluation of the hammer efficiency ($\eta$). Thus, if an efficiency value is adopted from increasing energy test in order to extrapolate to all piles of a site, much more (unwanted) inaccuracies could be carried to the resistance estimations of dynamic formulas.

Given all those issues, this paper proposes an alternative to the adoption of a single value for the hammer efficiency. It is based on the estimation of the effectively transferred energy through the set, elastic rebound and some pile geometric and material characteristics.

## 5. Alternative Method for Estimating the Effective Energy

The methodology is based on some simplifying hypotheses that will be presented later, during its development, but it is anchored in two main points:

1. the idealized curve of resistance *vs.* displacement due to the hammer blow.
2. the Chellis formula (1951), which derives directly from Hooke's law.

The deduction starts from the equilibrium between the effective energy applied after the impact and the work done by the pile-soil set against penetration (Paikowsky, 2001).

The application of an effective energy ($E_{ef}$) causes two kinds of displacement: one permanent ($s$) and other elastic ($K = C2 + C3$). At the apex of both, the maximum static resistance ($R$) is mobilized by the pile-soil system. After pile rest, the elastic displacement is recovered, remaining only the set. Also, soil resistance returns to zero, resulting in the ideal graph of resistance *vs.* displacement (linear elastic/perfectly plastic) presented in Fig. 2.

The hatched trapezoidal area in Fig. 2 represents the total work performed by the pile-soil system against penetration. It is equal to the effective energy applied ($E_{ef}$) and may be calculated as:

$$E_{ef} = \frac{[s+(s+K)]\cdot R}{2} = \frac{(s+D)\cdot R}{2} \tag{8}$$

Taking into account that $E_{ef} = \eta.W.h$, it follows from Eq. 8 that:

$$R = \frac{2\cdot\eta\cdot W\cdot h}{(s+D)} \tag{9}$$

On the other hand, the Chellis formula (1951) is written as:

$$R = \frac{C2\cdot E\cdot A}{l} \tag{10}$$

Chellis (1951) defined $l$ as the "length of pile measured from head to center of driving resistance". Thus, it can be considered as a percentage of the full pile length, as shown in Eq. 11:

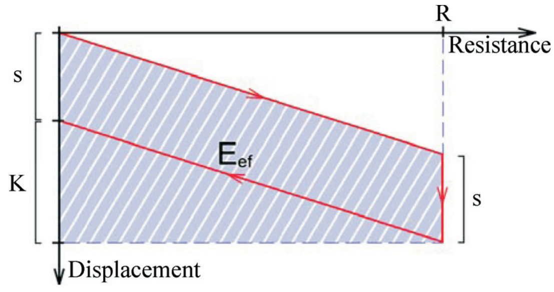


**Figure 2** - Idealized curve of pile resistance *vs.* displacement after impact.

$$R = \frac{C2\cdot E\cdot A}{\alpha\cdot L} \tag{11}$$

That length multiplier ($\alpha$) was proposed by Velloso (1987) and it is calculated considering the distribution of pile shaft and toe resistances. Matching by the resistance ($R$) the previous Eqs. 9 and 11, it follows:

$$(s+D)\cdot C2 = \frac{2\cdot\eta\cdot W\cdot h\cdot\alpha\cdot L}{E\cdot A} \tag{12}$$

The first simplifying hypothesis is that the pile set (plastic displacement) is so small compared to the elastic rebound that it will be disregarded ($s = 0$). Then, it follows that $K = D$, and the Eq. 12 could be written as shown in Eq. 13.

$$D\cdot C2 = \frac{2\cdot\eta\cdot W\cdot h\cdot\alpha\cdot L}{E\cdot A} \tag{13}$$

$C2$ is usually obtained by measuring the elastic rebound ($K$) and subtracting from it the toe quake ($C3$). However, there are two major problems with that practice: a) the quake magnitude is generally unknown and b) establishing a fixed value for the quake assumes that it is invariable with the applied energy. Some authors assume it is constant, equal to 2.5 mm. That is not true: the quake is higher as the energy increases. Thus, Rosa (2000) proposed a very convenient solution: to consider $C2$ as a percentage of the maximum elastic displacement ($K$). Since the set was disregarded ($s = 0$), then $K = D$ and $C2$ is written as:

$$C2 = \kappa\cdot K = \kappa\cdot D \tag{14}$$

Taking that into Eq. 13:

$$D\cdot(\kappa\cdot D) = \frac{2\cdot\eta\cdot W\cdot h\cdot\alpha\cdot L}{E\cdot A} \tag{15}$$

Then, reordering the terms:

$$D = \sqrt{\frac{2\cdot\alpha}{\kappa}}\cdot\sqrt{\frac{\eta\cdot W\cdot h\cdot L}{E\cdot A}} \tag{16}$$

Or, considering the term $\lambda = \sqrt{2\alpha/\kappa}$, Eq. 16 may be rewritten as:

$$D = \lambda\cdot\sqrt{\frac{\eta\cdot W\cdot h\cdot L}{E\cdot A}} \tag{17}$$

Reordering the terms of Eq. 17, the effective transferred energy is given by Eq. 18.

$$E_{ef} = \eta\cdot W\cdot h = \frac{1}{\lambda^2}\cdot\frac{D^2\cdot E\cdot A}{L} \tag{18}$$

That is the base equation of the alternative method, in which the $\lambda$ coefficient is the main parameter and a function of $\alpha$ and $\kappa$.

In the same research that Rosa proposed the solution of considering $C2$ as a percentage of $K$ (Rosa, 2000), the author found mean values for $\kappa$, relating it to the soil at the pile tip, as follows:

- 0.80 for pile tip over sandy soil;
- 0.70 for pile tip over silty soils or clay.

Thus, as $\lambda$ depends on the distribution of pile shaft and toe resistances ($\alpha$) and the soil type at the pile tip ($\kappa$), it can be described as a "site-specific" parameter that might differ from one site (and subsoil profile) to another.

Querelli & Massad (2017) briefly introduced Eq. 18, although not explicitly mentioning the $\lambda$ coefficient. It could be verified that their results showed very similar $\lambda$ values – 1.28 and 1.29 – for two neighboring sites located in Duque de Caxias (state of Rio de Janeiro, Brazil).

More recently, Querelli & Massad (2019) showed another case study involving the deduced methodology. They evaluated the $\lambda$ coefficient for three sites in the Rio de Janeiro metropolitan area, resulting in $\lambda$ values between 1.22 and 1.39.

In the case of the Danish Formula (Sorensen & Hansen, 1957), the adjustment coefficient $\lambda$ implicit in the term $S_o$, given by Eq. 3 and analogous to Eq. 17, is $\lambda = \sqrt{2} = 1.41$ and, therefore, the ratio $1/\lambda^2 = 0.50$. Also, it can be deduced that, as the pile toe is "fixed", $C3 = 0$, $C2 = K$ and $\kappa = 1$. Therefore, it follows that Sorensen & Hansen assumed no shaft resistance, *i.e.* $\alpha = 1$ ($l = L$).

## 6. Case Studies: Characterization and Method Application

Various case studies were selected in order to apply the alternative method, based on Eqs. 17 and 18. They include 12 distinct sites, at seven Brazilian locations with 244 dynamically tested piles: 161 concrete and 83 steel piles. That resulted in 692 records of dynamic load tests of increasing energy: 526 from concrete and 166 from steel piles.

The evaluated subsoils are quite diverse. For example: there are cases of piles length most through marine sedimentary clays (some with organic material), and just a few meters into compact sand (residual), as occurs in sites 2, 3, 9, 10 and 11. Sites 4 and 5 are mostly of residual clays and site 6 of residual sand (of quartz). The piles length in site 12 are mostly through residual sandy silt of gneiss.

The hammer types used in pile driving were only drop hammers for sites 2, 9 and 11 and both drop and hydraulic hammers for sites 3 and 12. There is no information about the hammer types of the other sites.

Table 1 summarizes the site locations, pile materials, amount of tests, dynamic records and cross sections.

In order to evaluate the $\lambda$ values of Eq. 17, linear correlations were made between the two terms, $D$ and $[(\eta.W.h.L)/(E.A)]^{0.5}$. As mentioned above, $D = DMX$ and $E_{ef} = \eta.W.h = EMX$, given by the dynamic load tests. Plotting those linear correlations, it follows that the angular coefficient (slope) is equal to $\lambda$. Therefore, to get the desired effective energy ($E_{ef}$), it is only necessary to use Eq. 18.

### 6.1. Results and discussions

The linear correlations were made: a) grouping the data according to the pile material (concrete or steel); and b) for each site, alone.

#### *6.1.1. $\lambda$ coefficient by material*

The plots in the form $D$ x $[(\eta.W.h.L)/(E.A)]^{0.5}$ for both concrete and steel piles are presented in Fig. 3 and Fig. 4.

The linear correlation through the origin gave the following values for the $\lambda$ coefficient: 1.34 for concrete piles and 1.23 for steel piles. The coefficient of determination ($R^2$) was lower for concrete piles (0.88) than for steel piles (0.92), much because of the scatter caused by the higher $\lambda$ values of sites 1 and 7 (1.71 and 1.56, respectively), which are presented separately in section 6.1.2 (Figs. 5 and 11, respectively).

Therefore, in order to propose an average equation for each material, the effective transferred energy can be calculated by Eqs. 19 and 20:

$$E_{ef} = 0.56 \cdot \frac{D^2 \cdot E \cdot A}{L} \quad \text{(concrete piles)} \tag{19}$$

$$E_{ef} = 0.68 \cdot \frac{D^2 \cdot E \cdot A}{L} \quad \text{(steel piles)} \tag{20}$$

#### *6.1.2. $\lambda$ coefficient by site*

The plots in the form $D$ x $[(\eta.W.h.L)/(E.A)]^{0.5}$ for each site are presented in Figs. 5 to 16 with the linear correlations, where the angular coefficient is $\lambda$, and with the coefficients of determination ($R^2$), that in general are above 90%. Table 2 summarizes these results.

Observing the sites separately, there is greater dispersion in the mean values of $\lambda$ for concrete piles, ranging from 1.23 to 1.71, with an average of 1.40 between sites and Coefficient of Variation ($CV$) of 12.8% (Table 2). For steel piles, the coefficient of variation drops to 7.7%, with $\lambda$ ranging from 1.13 to 1.35 and averaging around 1.25 between sites (Table 2).

#### *6.1.3. Ratio $1/\lambda^2$ by site*

Table 2 also presents mean values of the term $1/\lambda^2$, from Eq. 17, which allows the estimation of the effective energy ($E_{ef} = EMX$). It can be seen that:

a) for concrete piles, the value of the ratio $1/\lambda^2$ ranged from 0.34 to 0.67. The average of the sites was 0.53, with standard deviation of 0.12 and coefficient of variation of 22.6%; and

**Table 1** - Summary of the studied sites.

| Site | City/Location (State) | Piles material | Amount of tested piles | Amount of dynamic load test records | Cross-sections with main dimension (cm) | |
|---|---|---|---|---|---|---|
| 1 | Foz do Iguaçu (PR) | Concrete | 4 | 36 | 36 × | SQ 16.0 / 20.0 / 23.0 |
| 2 | Rio de Janeiro metropolitan area (RJ) | Concrete | 26 | 26 | 10 × | SQ 21.5 / 23.5 / 26.5 / 29.5 |
| | | | | | 16 × | STR 22.9 / 26.9 / 29.8 / 40.6 |
| 3 | Rio de Janeiro metropolitan area (RJ) | Concrete | 23 | 102 | 102 × | SQ 19.5 / 26.5 / 29.5 |
| 4 | São José dos Campos (SP) | Concrete | 35 | 131 | 131 × | SQ 23.0 / 26.5 |
| 5 | São José dos Campos (SP) | Concrete | 12 | 35 | 35 × | SQ 19.5 / 23 5 |
| 6 | São Paulo (SP) | Concrete | 20 | 51 | 51 × | H 40 / 45 |
| 7 | São Paulo (SP) | Concrete | 41 | 145 | 30 × | HS 33 |
| | | | | | 115 × | CC 38 / 42 |
| 8 | Aracaju (SE) | Steel | 4 | 12 | 12 × | HP 25 (73 kg/m) |
| 9 | Rio de Janeiro metropolitan area (RJ) | Steel | 16 | 84 | 84 × | HP 31 (125 kg/m) |
| 10 | Rio de Janeiro metropolitan area (RJ) | Steel | 2 | 9 | 9 × | RL 6.8 |
| 11 | Itajaí (SC) | Steel | 7 | 7 | 7 × | W 25 (25 kg/m) |
| 12 | Belo Horizonte metropolitan area (MG) | Steel | 54 | 54 | 54 × | TB 35.6 / 50.8 / 61.0 |

SQ = Square; STR = Star section; H = Hexagon; HS = Solid Hexagon; CC = Cylinder centrifuged pile; HP and W = H pile; RL = Rail pile; TB = Tubular.

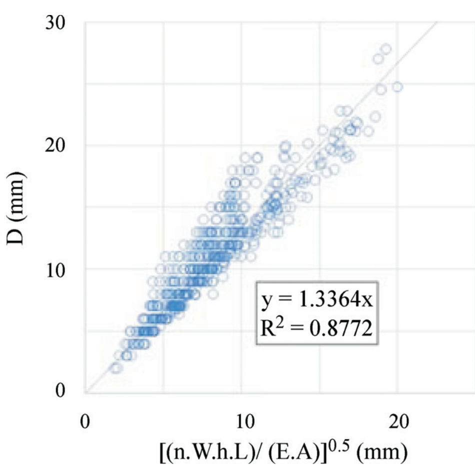


**Figure 3** - All concrete piles.

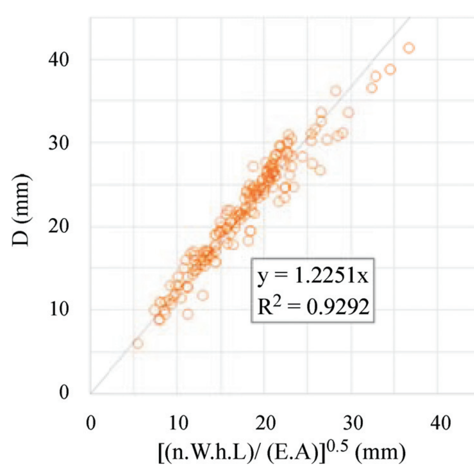


**Figure 4** - All steel piles.

b) for steel piles, the average $1/\lambda^2$ value was 0.65, ranging from 0.55 to 0.78, with a standard deviation of 0.10 and a coefficient of variation of 15.6%.

These results confirm that it is possible to estimate the ratio $1/\lambda^2$ in a consistent way, allowing using the proposed methodology to estimate the effective transferred energy ($E_{ef}$) without needing the hammer efficiency ($\eta$). In order to properly use the method, the analysis shows that it is recommended to perform previous calibrations of $\lambda$ for each site, by means of dynamic load tests (DLT), before the beginning and even during the construction.

## 7. Method Limitations and Cautions

The method stands as an alternative to the assessment of hammer efficiency, since the 12-site case study validated

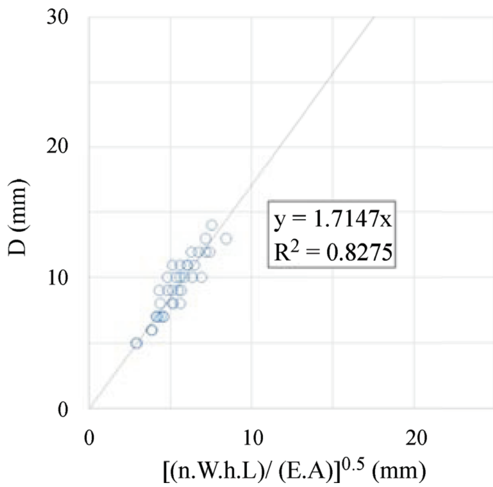


**Figure 5** - Site 1 – concrete.

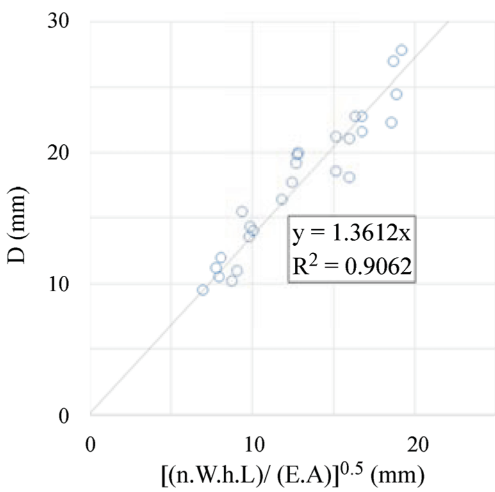


**Figure 6** - Site 2 – concrete.

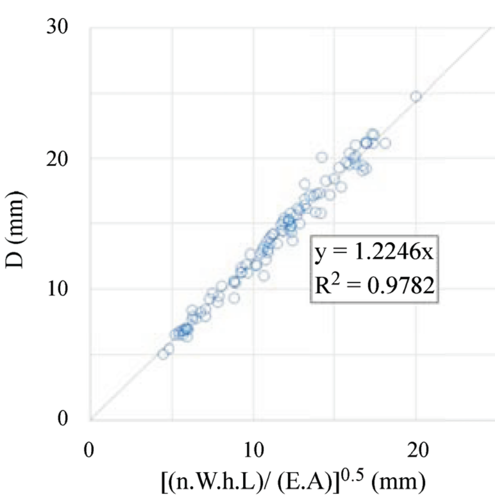


**Figure 7** - Site 3 – concrete.

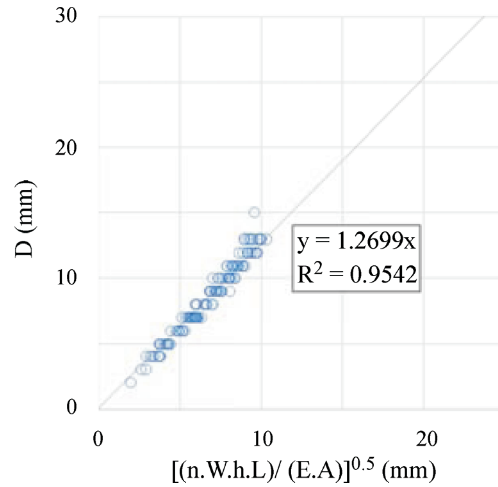


**Figure 8** - Site 4 – concrete.

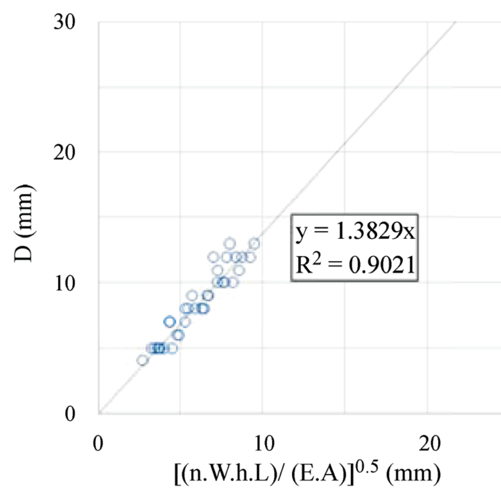


**Figure 9** - Site 5 – concrete.

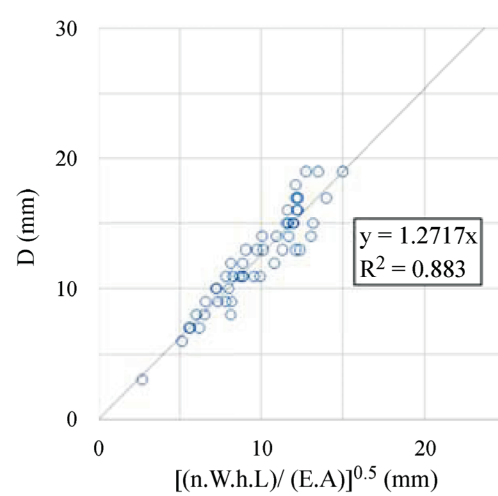


**Figure 10** - Site 6 – concrete.

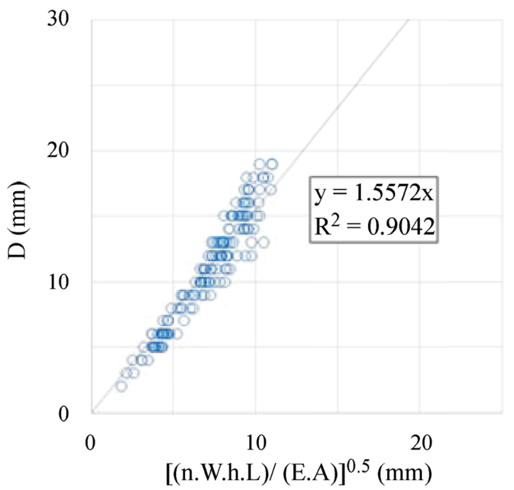


**Figure 11** - Site 7 – concrete.

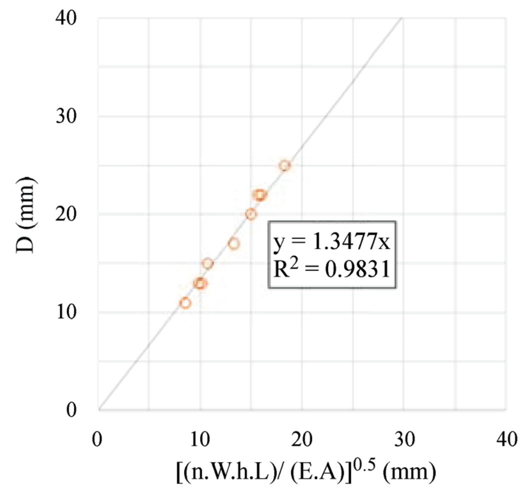


**Figure 14** - Site 10 – steel.

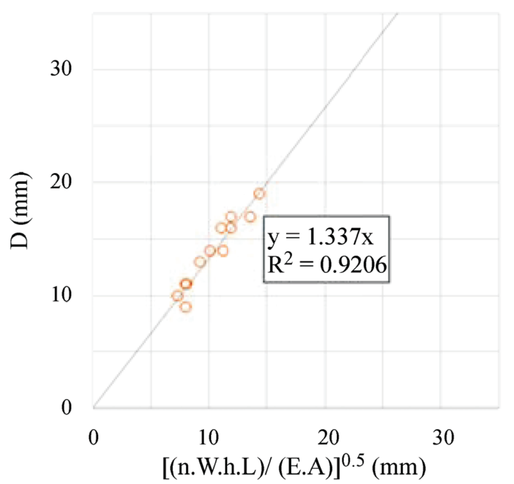


**Figure 12** - Site 8 – steel.

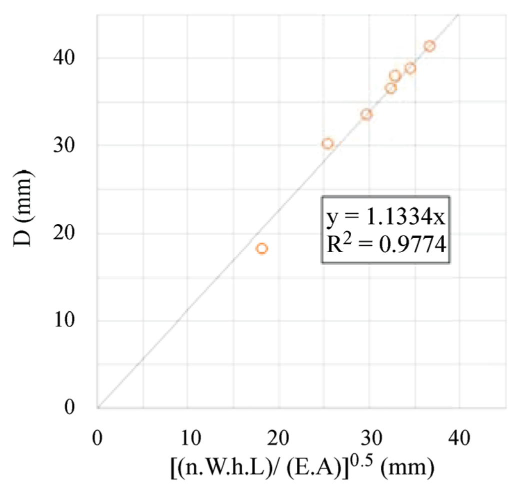


**Figure 15** - Site 11 – steel.

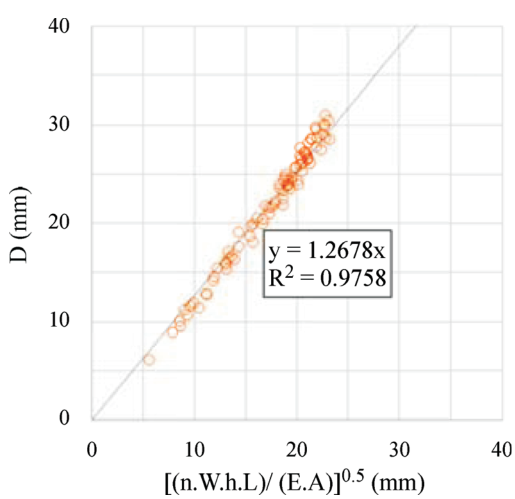


**Figure 13** - Site 9 – steel.

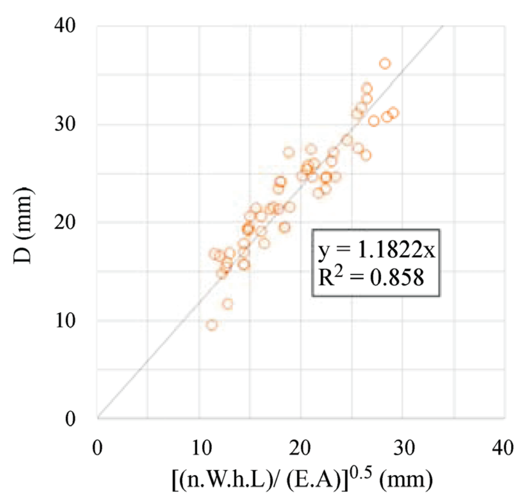


**Figure 16** - Site 12 – steel.

**Table 2** - Values of λ, $1/\lambda^2$ and $R^2$ by site.

| Site | λ coefficient | | ratio $1/\lambda^2$ | | $R^2$ (linear regression) |
|---|---|---|---|---|---|
| | Concrete | Steel | Concrete | Steel | |
| Site 1 | 1.71 | | 0.34 | | 0.83 |
| Site 2 | 1.36 | | 0.54 | | 0.91 |
| Site 3 | 1.22 | | 0.67 | | 0.98 |
| Site 4 | 1.27 | | 0.62 | | 0.95 |
| Site 5 | 1.38 | | 0.52 | | 0.90 |
| Site 6 | 1.27 | | 0.62 | | 0.88 |
| Site 7 | 1.56 | | 0.41 | | 0.90 |
| Site 8 | | 1.34 | | 0.56 | 0.92 |
| Site 9 | | 1.27 | | 0.62 | 0.98 |
| Site 10 | | 1.35 | | 0.55 | 0.98 |
| Site 11 | | 1.13 | | 0.78 | 0.98 |
| Site 12 | | 1.18 | | 0.72 | 0.86 |
| Minimum | 1.22 | 1.13 | 0.34 | 0.55 | 0.83 |
| Maximum | 1.71 | 1.35 | 0.67 | 0.78 | 0.98 |
| Average | 1.40 | 1.25 | 0.53 | 0.65 | 0.92 |
| Std. dev. | 0.18 | 0.10 | 0.12 | 0.10 | 0.05 |
| Coef. of variation | 12.8% | 7.7% | 22.6% | 15.6% | 5.6% |

its application. However, some limitations and cautions are presented below:

- the method was not verified for large quake (high rebound) soils;
- the range of permanent set per blow evaluated was $s \leq 7$ mm; also, in 90% (621) of the dynamic records, the permanent set per blow was $s \leq 3$ mm (hard driving);
- the method is valid when the elastic rebound ($K$) is much higher than the set ($s$), since it is one of the method's simplifying hypothesis;
- when applying it to non-tested pile, the scatter of the elastic modulus ($E$) should be accounted, since it has direct influence on the effective energy estimations.

## 8. Basic Step-By-Step Guide to the Method Application

In order to apply the proposed methodology, the following routine could be applied:

(1) Select the sample of piles to be dynamically tested (DLT);

(2) Perform the dynamic load tests (of increasing energy) in the selected piles, measuring the set ($s$) and elastic rebound ($K$) of each blow;

(3) With that pile geometry and the results from the DLT, plot the graphs in the form $D$ *vs.* $[(E_{ef}.L)/(E.A)]^{0.5}$ for all blows of the tested piles;

(4) The calibrated and site-specific λ value could be obtained from the plot, as it is the slope of the linear correlation through the origin [0,0];

(5) Measuring the set ($s$) and elastic rebound ($K$) of blows in non-tested piles, that calibrated λ value could be applied to Eq. 18 in order to estimate the effective transferred energy.

## 9. Conclusions

The proposed method to estimate the effective transferred energy is a practical and, in the authors' view, relevant contribution to quality control of driven piles, since two very simple records ($s$ and $K$) are needed – besides the knowledge of the λ coefficient, calibrated through dynamic load tests of increasing energy on selected piles of a given site.

After λ calibration, it is possible to use Eq. 18 and apply the calculated effective energy ($E_{ef}$) to dynamic formulas, in order to estimate the resistance of non-tested piles.

There is no need to know the hammer efficiency (η), which presents huge scatter in the same site and difficulties to be calibrated to a single, fixed value (even with dynamic instrumentation). The methodology also does not require input of soil type, since it is considered in the site-calibration of the λ coefficient.

The case studies showed the importance of first calibrating the λ coefficient, since it differs from site to site and the correlations ($R^2$) showed to be good to excellent when considering each site alone; the site-specific calibrated λ ranged from 1.22 to 1.71 in the concrete piles' sites and 1.13 to 1.35 for the steel piles' sites, with $R^2$ from 0.83 to 0.98.

Due to λ variability and the influence of both α and κ parameters (site-specific attributes), it is important to mention that if the site area is extensive, if the local subsoil is very heterogeneous or even if the pile lengths are quite different, it is strongly recommended to divide the site in regions and perform more than one calibration of λ in order to contemplate all of the local particularities. Investigations about other reasons for λ variability were outside the scope of this paper. In this sense, the influence of hammer type or pile crossections in λ was not evaluated.

Although the proposed method dispenses the knowledge of hammer efficiency (η), as a "last but not least" comment, it is reasonable to say that engineers have to be cautious with overly simple methods when representing complex events.

## References


ABNT (2007). Estacas – Ensaio de Carregamento Dinâmico – Método de Ensaio - NBR 13208. Associação Brasileira de Normas Técnicas. Rio de Janeiro, RJ, Brazil, 12 p.

ABNT (2010). Projeto e Execução de Fundações - NBR 6122. Associação Brasileira de Normas Técnicas. Rio de Janeiro, RJ, Brazil, 91 p.

Alonso, U.R. (1991). Previsão e Controle das Fundações. Edgard Blucher Ltda, São Paulo.

Alves, A.M.L. & Lopes, F. de R. (2004). Estacas de deslocamento: Tipos, aplicações e controle de execução. Proc. Seminário sobre Fundações Profundas, Porto Alegre, Brazil.

Alves, A.M. de L.; Lopes, F. de R. & Danziger, B.R. (2004). Métodos dinâmicos para previsão e controle do comportamento de estacas cravadas. Teoria e Prática na Engenharia Civil, 4(4):12-21.

Aoki, N. (1989). A new dynamic load test concept. Proc. XII International Conference on Soil Mechanics and Foundation Engineering, Rio de Janeiro, Brazil, v. 1, pp. 1-3.

Beim, J.W. (2009). Aplicação da equação da onda na análise da cravação e ensaio de fundações: passado, presente e futuro. In: Engenharia de Fundações: Passado Recente e Perspectivas – Homenagem ao Prof. Nelson Aoki. Rio de Janeiro.

Caputo, H.P. (1983). Mecânica dos Solos e Suas Aplicações, v. 2. LTC, Rio de Janeiro, 5ª ed.

Chellis, R.D. (1951). Pile Foundations. McGraw-Hill, New York.

Fellenius, B.H. (2018). Basics of foundation design. January 2018. Vero Beach: Pile Buck International, Inc. Available at https://www.fellenius.net/papers.html. Downloaded in 28 dez. 2018.

Goble, G.G.; Rausche, F. & Likins, G.E. (1980). The analysis of pile driving - A state-of-the-art. Proc. International Seminar on the Application of Stress-Wave Theory on Piles, Stockholm, Sweden, pp. 131-161.

Goble, G.G. & Likins, G.E. (1996). On the application of PDA dynamic pile testing. Proc. Fifth International Conference on the Application of Stress-Wave Theory to Piles, Orlando, United States, pp. 263-273.

Gonçalves, C.; Bernardes, G. de P. & Neves, L.F. de S. (2007). Estacas Pré-Fabricadas de Concreto: Teoria e Prática, 1st ed. Author's edition, São Paulo.

Hiley, A. (1925). A rational pile-driving formula and its application in piling practice explained. Engineering (London), v. 119(3100):657-721.

Hussein, M.H.; Likins, G.E. & Hannigan, P.J. (1993) Pile evaluation by dynamic testing during restrike. Proc. 11th Southeast Asia Geotechnical Conference, Singapore, pp. 535-539.

Likins, G.E.; Fellenius, B.H. & Holtz, R.D. (2012). Pile driving formulas: past and present. Proc. ASCE Geoinstitute Geo-Congress, Full-Scale Testing in Foundation Design, State of the Art and Practice in Geotechnical Engineering, Oakland, United States, pp. 25-29.

Likins, G.E. & Rausche, F. (2008). What constitutes a good PDA test? Proc. Eighth International Conference on the Application of Stress Wave Theory to Piles, Lisbon, Portugal, pp. 403-407.

Niyama, S.; Martins, J.A. de A.; Medeiros Jr., C. & Likins, G.E. (1984). Dynamic pile instrumentation in a calcareous sand close to PCR-2 platform, Brazil. Proc. Second International Conference on the Application of Stress Wave Theory on Piles, Stockholm, Sweden, pp. 306-312.

Paikowsky, S.G. (2001). Load and Resistance Factor Design (LRFD) for dynamic analysis of deep foundations. Proc. 15th International Conference on Soil Mechanics and Foundation Engineering, Istanbul, Turkey, v. 2, pp. 981-984.

Paikowsky, S.G. & Chernauskas, L.R. (1992). Energy Approach for capacity evaluation of driven piles. Proc. Fourth International Conference on the Application of Stress-Wave Theory to Piles, Rotterdam, Netherlands, pp. 21-24.

Paraíso, S.C. & Costa, C.M.C. (2010). A eficácia do ensaio de carregamento dinâmico na avaliação do efeito de "setup" em estacas cravadas. Proc. XV Congresso Brasileiro de Mecânica dos Solos e Engenharia Geotécnica, COBRAMSEG, Gramado, Brazil.

Querelli, A. & Massad, F. (2017). Simplified Hiley's formula calibration for precast concrete piles in Duque de Caxias. Proc. International Conference on Advances in Structural and Geotechnical Engineering, ICASGE17, Hurghada, Egypt.

Querelli, A. & Massad, F. (2019) Proposição de um método alternativo para estimativa da energia efetiva na cravação de estacas. Proc. 9° Seminário de Engenharia de Fundações Especiais e Geotecnia, SEFE 9, São Paulo, Brazil.

Rausche, F. (1997). Hammer Design for Drilled Shaft Testing: Two Case Studies. Notes from PDA Users Day in Cleveland, OH. Pile Dynamics, Inc, Cleveland, pp. 1-13.

Rausche, F.; Robinson, B.R. & Likins, G.E. (2004). On the prediction of long term pile capacity from end-of-driving information. Geotechnical Special Publication, (N. 125):77-95.

Rosa, R.L. (2000). Proposição de Modificação das Fórmulas Dinâmicas de Cravação de Chellis de Uto *et al.* a Partir de Resultados do Método Case. Master's Thesis, Universidade de São Paulo, São Paulo, 196 p.

Saldívar, R.E.R. (2008). Retroanálise Probabilista Aplicada à Análise Dinâmica da Cravação de Estacas. Master's Thesis, Universidade de São Paulo, São Paulo, 147 p.

Sorensen, T. & Hansen, B. (1957). Pile driving formulae – An investigation based on dimensional considerations and a statistical analysis. Proc. 4th International Conference Soil Mechanics and Foundation Engineering, London, England, p. 61-65.

Tavenas, F. & Audy, R. (1972). Limitations of the driving formulas for predicting the bearing capacities of piles in sand. Canadian Geotechnical Journal, 9(1):47-62.

Tokhi, H. (2012). A Practical Method for Improved Implementation of Pile Driving Formula. Master's Thesis, RMIT University, Australia, 207 p.

Valverde, R. & Massad, F. (2018). Maximum envelope of lateral resistance through dynamic increasing energy test in piles. Soils & Rocks, 41(1):75-89.

Velloso, P.P.C. (1987). Fundações: Aspectos Geotécnicos v. 2 e 3. Departamento de Engenharia Civil, PUC, Rio de Janeiro, 5ª ed.

Vieira, S.H.A. (2006). Controle de Cravação de Estacas Pré-Moldadas: Avaliação de Diagramas de Cravação e Fórmulas Dinâmicas. Master's Thesis, COPPE-UFRJ, Rio de Janeiro, 122 p.

## List of Symbols

$s$: Set. Permanent (plastic) displacement per blow
$K$: elastic rebound after the blow
$C2$: pile elastic displacement (structural element)
$C3$: Quake. Elastic displacement of the soil below pile tip
$R$: pile-soil's static resistance
$\eta$: driving system efficiency / impact efficiency
$W$: hammer weight
$h$: hammer drop height
$L$: pile length
$A$: pile crossectional área
$E$: dynamic elastic modulus
$S_o$: dynamic elastic compression of a pile with a fixed point (Sorensen & Hansen, 1957); analogous to $K$
$K_{sp}$: coefficient of energy loss in the soil by the viscous damping effect
$D$: maximum pile displacement after blow (sum $s + K$)
$F$: force calculated in the dynamic load test
$v$: particle velocity calculated in the dynamic load test
$Z$: pile impedance
$c$: wave speed
$EMX$: maximum transferred energy to the pile calculated in the dynamic load test (analogous to '$\eta.W.h$')
$F(t)$: force measured along time;
$v(t)$: particle velocity measured along time
$E_{ef}$: effective transferred energy to the pile (analogous to $EMX$)
$l$: length from pile top to the center of driving resistance
$\alpha$: pile length multiplier in order to consider the center of driving resistance
$\kappa$: elastic rebound multiplier proposed by Rosa (2000)
$\lambda$: site-specific adjustment coefficient
$DMX$: maximum pile displacement after stroke calculated in the dynamic load test (analogous to $D$)
$R^2$: coefficient of determination (linear regression)
$CV$: coefficient of variation (statistics)