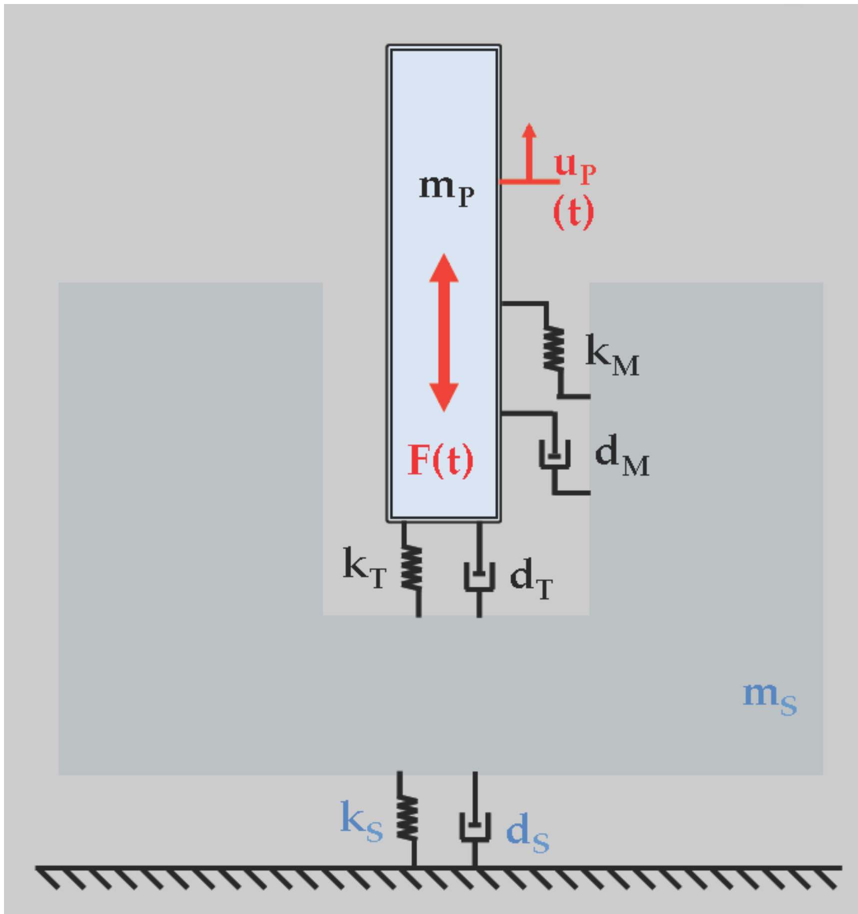

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Engineering Assessment of Ground Vibrations Caused by Impact Pile Driving

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ABSTRACT: Ground vibrations are an important design consideration for piles driven by impact hammers. The first task is to determine allowable vibration levels; the second task is to predict the intensity of ground vibrations during driving and the attenuation of ground vibrations with increasing distance. The paper describes the Swedish vibration standard which is applicable for pile driving. Commonly used vibration parameters, associated with the evaluation of vibration measurements, are discussed. The importance of pile impedance for ground vibrations is highlighted. A simplified calculation method is proposed which can be used to estimate vertical ground vibration velocity as a function of distance from the driven pile. Two case histories have been evaluated and compared with theoretical predictions, using the proposed method of analysis.

KEYWORDS: Ground vibrations, Pile driving, Vibration limits, Vibration parameters, Prediction, Frequency, Damage

1. INTRODUCTION

Piling is the most widely used foundation method for heavy structures. The geotechnical engineer can choose among a variety of piling methods, but in most cases, driving piles by impact hammer is the most cost-effective alternative. Under unfavourable conditions, driving piles or sheet piles can cause environmental problems, such as noise, ground movements and vibrations with risk of damage to adjacent buildings or other structures. Therefore, it is important for the designer of foundation projects to be able to predict maximum vibrations, which can be generated due to pile driving. In spite of extensive efforts devoted to improve the understanding of the pile driving process, surprisingly few attempts have been made to develop practical design methods, which can be used to predict ground vibrations as aid in the design and construction process.

Massarsch and Fellenius (2008) presented a comprehensive concept that encompasses the entire pile driving process, addressing the impact of the pile hammer, propagation of stress waves along the pile, and transfer of vibrations from the pile along the shaft and at the toe to the surrounding soil. However, geotechnical engineers need practical methods to assess ground vibrations, especially during the preliminary design phase of a project. To meet this need, a simplified and straight-forward method is presented which does not require extensive theoretical knowledge.

In order to evaluate the impact of ground vibrations on the surroundings it is necessary to understand the different parameters which can be used to define ground vibrations. Ambiguity when using definitions of vibration parameters is not uncommon even in the scientific literature. Therefore, definitions of the most important vibration parameters are presented. Another important task for the design engineer is to compare the predicted level of ground vibrations with vibration limits stated in codes or guidelines. Environmental concerns with respect to noise and ground vibrations can restrict or even prohibit driving of piles. Vibration standards were primarily developed for blasting applications, but are also often used to regulate vibrations from construction activities such as pile driving or ground compaction. This paper describes the application of the Swedish standard which regulates permissible ground vibrations caused by driving of piles, sheet piles, or ground compaction. An overview of existing international vibration standards applicable to pile driving induced vibrations has been compiled by Massarsch and Fellenius (2014a).

2. DEFINITION OF VIBRATION PARAMETERS

The understanding of which parameters can be used to describe vibrations is an important requirement when assessing building damage. The following sections describe the most important parameters required for evaluating the effect of ground vibrations on buildings and building foundations. A comprehensive discussion of these parameters and their interpretation is given by Chameau et al. (1998).

2.1 Vibration Amplitude

Vibration amplitude can be defined as the departure of a point on a vibrating body from its equilibrium position. It is equal to one-half the length of the vibration path. A typical vibration record from pile driving is shown in Figure 1. The following relationship exists between different expressions of vibration amplitude.

$$a = 2\pi f v = (2\pi f)^2 d \qquad (1)$$

where a = acceleration, v = particle velocity, d = displacement, and f = vibration frequency. It is thus possible to derive for sinusoidal vibrations the corresponding amplitude values, when one amplitude value (displacement, velocity, or acceleration) and the vibration frequency are known. The maximum value of vibration velocity (peak value) occurring during the measuring period is in many standards defined as peak particle velocity (PPV). If particle motions are measured in three orthogonal directions (x, y, and z) simultaneously, it is possible to calculate the vector sum, $|v_i|$ of the three components.

$$|v_i| = \sqrt{v_{x1}^2 + v_{y1}^2 + v_{z1}^2} \qquad (2)$$

In the case of sinusoidal vibrations, the average vibration amplitude, x_{rms} (displacement, velocity, or acceleration) can be expressed by the ratio of root-mean-square (RMS).

$$x_{rms} = \sqrt{\frac{1}{n}\left(x_1^2 + x_2^2 + x_3^2 + x_4^2 + ... + x_n^2\right)} \qquad (3)$$

where x_n etc. are the set of n vibration values. The RMS value, which is frequently used to describe the average vibration intensity, corresponds to the area under the half wavelength. In case of sinusoidal vibrations—and only then—it is related to the peak amplitude, v_{peak}.

$$v_{rms} = 0.7\, v_{peak} \qquad (4)$$

The relevance of the RMS value depends strongly on the duration of the signal. The so-called CREST factor (peak to average) is the ratio between the peak value and the RMS value. In the case of transient vibrations, which are typically generated by impact pile driving, the duration of the largest motions is small compared to the total length of the signal. For such vibrations, it is possible to choose the minimum amplitude of interest (i.e. minimum value which is of relevance) and calculate the RMS amplitude from the time that the minimum amplitude is exceeded for the first time to the time when the minimum amplitude is exceeded for the last time in the record. The peak value of the wave is the highest value the wave reaches above a reference, normally zero. This definition is used in the above equations. Note that in engineering applications, frequently the peak-to-trough value (vertical distance between the top and bottom of the amplitude) is used to express vibration intensity, which ambiguity has caused numerous interpretation errors.

2.2 Strain

Vibrations passing through material impose strain, which can be calculated from the particle velocity and the wave speed. Strain, ε, caused by propagation of a compression wave (P-wave) can be determined from Eq. (5), if the particle velocity, v_P measured in the direction of wave propagation, and the wave speed, c_P are known.

$$\varepsilon = \frac{v_p}{c_p} \tag{5}$$

Shear strain, γ can be calculated from the particle velocity measured perpendicular to the direction of wave propagation, v_s and the shear wave speed, c_S (Eq. 6).

$$\gamma = \frac{v_s}{c_s} \tag{6}$$

Shear strain is an important parameter when assessing settlement in granular soils or disturbance of cohesive soils.

2.3 Vibration Frequency

The time history of the vibration record shown in Figure 1 can be transferred into the frequency domain, Figure 2. The frequency content of a signal is important when assessing the effect of vibrations on structures. The simplest method of estimating the dominant frequency is by examining "zero crossings" of the time history. This method works reasonably well for simple, periodic signals, but is less reliable for complex, multiple-frequency signals.

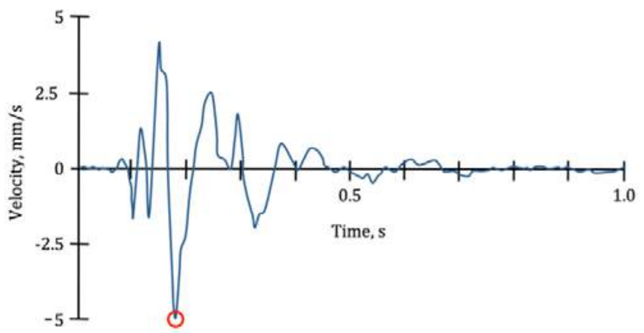

Figure 1 Vertical vibration velocity as function of time. Recording station was on the ground 10 m away from where a precast concrete pile was driven into sandy soil. The pile toe was located 3 m below the ground surface. The value of the peak particle velocity (PPV) is indicated (Massarsch and Fellenius 2014)

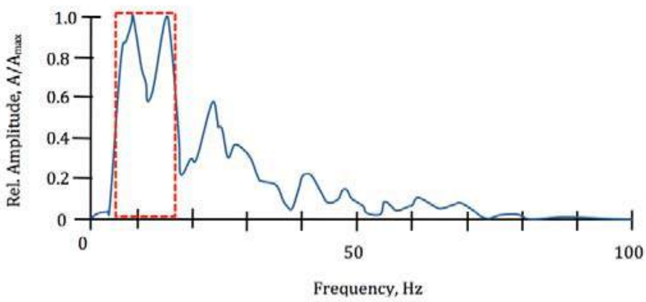

Figure 2 Frequency spectrum of the time history shown in Figure 1. The dominant frequency range is indicated

A common method of estimating the frequency content (spectrum) of a signal is to perform a Fast Fourier Transformation (FFT). The resulting values are usually presented as amplitude and phase, both plotted versus frequency. A related quantity, which is also widely used to estimate the power of a signal, is the power spectrum, which in the frequency domain is the square of FFT´s magnitude. Despite its widespread use, there are several limitations associated with the use of Fourier methods for estimating spectra. The Fourier method implicitly assumes that the signal is stationary. For transient signals such as from impact pile driving and blasting, as well as for many discontinuous signals, this assumption is not strictly valid.

The relationship between vibration frequency and vibration amplitudes according to Eq. 1 is shown in Figure 3. The black line marks as an example a vibration velocity of 30 mm/s. At a frequency of 10 Hz, the corresponding displacement amplitude is 0.48 mm and the acceleration is 0.19 g. If the vibration frequency decreases at constant vibration velocity, the displacement amplitude will increase. Correspondingly, if the vibration frequency increases at constant vibration velocity, the acceleration amplitude increases.

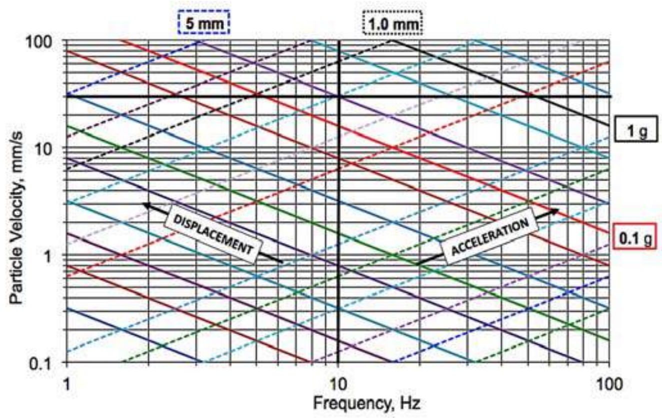

Figure 3 Relationship for sinusoidal vibrations between frequency and particle velocity, acceleration (full lines), and deformation (dotted lines), cf. Eq. (1)

2.4 Wave Length

The wave length is an important parameter when assessing the risk of damage due to propagation of waves in the ground. The wave length, λ can be determined from the following relationship.

$$\lambda = 2\pi f \tag{7}$$

where: f is the vibration frequency. The largest risk of damage to structures from ground vibrations exists when the wave length corresponds to approximately the building length, (Massarsch 2000).

3. DAMAGE POTENTIAL OF VIBRATIONS

It is difficult to give general recommendations regarding the damage effects of displacement, velocity, and acceleration as the sensitivity of structures can depend on many factors. However, the following general observations can be made, which are not generally appreciated:

- Displacement: at low frequencies, displacement is the most relevant measure of structural damage as the failure mode is generally due to "static" stress caused by displacement (Hooke's law), i.e., damage caused by exceeding material strength.
- Velocity: indicates how often the displacement is applied in a given time period and is thus related to the fatigue mode of failure (material degradation). As can be seen from Eqs. (5) and (6), strain, which causes material distortion (settlement), depends on vibration velocity and is therefore particularly important when assessing settlement in loose granular soils.
- Acceleration: at high frequencies, the failure mode is normally related to the applied dynamic force caused by inertial forces (Newton's law).

It should be pointed out that the above failure modes (stress—fatigue—dynamic force) can overlap and the proper selection of vibration parameter must reflect the type of problem. A detailed discussion of damage caused by ground vibrations has been presented by Massarsch and Fellenius (2014b).

4. SWEDISH VIBRATION STANDARD

Authorities in many countries are increasingly aware of the importance of environmental problems and apply standards more rigorously than in the past. Vibrations from construction activities are normally not likely to cause damage to buildings or building elements. However, buildings with poor foundation conditions may be very sensitive to vibrations, and, damage may then be instigated or existing cracks and fissures be aggravated. In the case of vibration-sensitive foundation conditions, such as mixed foundations or foundations on loose, granular soils, damage can be caused by vibration-induced increase of pre-existing total and/or differential settlements. This aspect is not included in most vibration standards. Due to the difficult ground conditions in Sweden, pile driving is used frequently also in vibration-sensitive areas and extensive experience regarding the effects of driving preformed piles has been accumulated.

For these reasons, the Swedish Standard SS 02 52 11, "Vibration and shock—Guidance levels and measuring of vibrations in buildings originating from piling, sheet piling, excavating, and packing [sic] to estimate permitted vibration levels" was established in 1999. The standard—which is not widely known outside Scandinavia—was particularly developed to regulate construction activities and is probably the most elaborate standard currently available. It deals with vibrations caused by piling, sheet piling, excavation and soil compaction. Guidance levels of vibrations acceptable with respect to potential building damage have been chosen based on more than 30 years of practical experience in a wide range of soils. Under the Swedish standard, a risk analysis must be carried out for construction projects, involving the prediction of maximum ground vibration levels and statement of permissible vibration levels for different types of structures. The proposed vibration values do not take into account psychological effects (noise or discomfort) on occupants of buildings. Neither do they consider the effects of vibrations on sensitive machinery or equipment in buildings.

The vibration levels in the standard are based on experience from measured ground vibrations (vertical component of particle velocity) and observed damage to buildings, with comparable foundation conditions in Sweden. The vibration level, v, is expressed as the peak value of the vertical vibration velocity. It is measured on bearing elements of the building foundation closest to the vibration source and is determined from the following relationship.

$$v = v_0 \; F_b \; F_m \; F_g \tag{8}$$

where: v_0 = vertical component of the uncorrected vibration velocity in mm/s, F_b = building factor, F_m = material factor and F_g = foundation factor. Values for v_0 are given in Table 1 for different ground conditions and construction activities, and are maximum allowable values at the base of the building. It should be noted that in the Swedish standard, the limiting vibration values are independent of vibration frequency. The main reason is that within the frequency range of vibrations generated by pile driving and soil compaction, the dominant frequency usually varies within a narrow range (typically 5 to 30 Hz).

Buildings are divided into five classes with respect to their vibration sensitivity cf. Table 2. Classes 1 – 4 apply to structures in good condition. If they are in a poor state, a lower building factor should be used.

Table 1 Uncorrected vibration velocity, v_0 (mm/s)

Foundation Condition	Piling, Sheet piling or Excavation	Soil Compaction
Clay, silt, sand or gravel	9 mm/s	6 mm/s
Moraine (till)	12 mm/s	9 mm/s
Rock	15 mm/s	12 mm/s

Table 2 Building Factor, F_b

Class	Type of Structure	Building Factor, F_b
1	Heavy structures such as bridges, quay walls, defence structures etc.	1.70
2	Industrial or office buildings	1.20
3	Normal residential buildings	1.00
4	Especially sensitive buildings and buildings with high value or structural elements with wide spans, e.g., churches or museums	0.65
5	Historic buildings in a sensitive state as well as certain sensitive historic buildings (ruins)	0.50

The structural material is divided into four classes with respect to their vibration sensitivity, cf. Table 3. The most sensitive material component of the structure determines the class to be applied.

Table 3 Material Factor, F_m

Class	Type of Building Material	Material Factor, F_m
1	Reinforced concrete, steel or timber	1.20
2	Unreinforced concrete, bricks, concrete blocks with voids, light-weight concrete elements	1.00
3	Light concrete blocks and plaster	0.75
4	Limestone, lime-sandstone	0.65

Table 4 defines a foundation factor. Lower factors are applied to buildings on shallow foundations, whereas buildings on piled foundations are accorded higher factors due to their reduced sensitivity to ground vibrations.

Table 4 Foundation Factor, F_g

Class	Type of Building Material	Foundation Factor, F_g
1	Slab, raft foundation	0.60
2	Buildings founded on friction piles	0.80
3	Buildings founded on end-bearing piles	1.00

The following example illustrates the practical application of the standard: Piles are to be installed in the vicinity of a residential building with brick walls, which are supported by toe-bearing piles in clay. If the following factors are chosen according to Tables 1 to 4: v_0 = 9 mm/s, F_b = 1.00, F_m = 1.00, F_g = 1.00, the maximum allowable vertical vibration velocity, v, measured at the base of the foundation is 9 mm/s.

5. VIBRATIONS CAUSED BY IMPACT PILE DRIVING

When a pile is jacked into the soil, the process is usually slow and without vibrations. However, a more effective installation method is pile driving by an impact hammer. The impact of the pile hammer on the pile helmet generates a stress wave that propagates through the pile. Dynamic forces develop along the interface between the pile and the surrounding soil, which can give rise to vibrations. The vibrations propagate in the form of different wave types depending on whether the waves are emitted along the pile shaft and/or from the pile toe. Vibrations attenuate with increasing distance from the pile although in some soil layers and buildings, they may actually become amplified due to resonance effects.

Vibration propagation caused by pile driving is complex, as illustrated in Figure 4. The pile hammer (1) transfers the kinetic energy via the pile cushion (2) to the pile (3). The stress wave in the pile generates a dynamic shaft resistance, R_M along the pile shaft (4) and dynamic toe resistances, R_T at the pile toe (5).

Vibrations are generated along the pile shaft and at the pile toe and the distribution of dynamic forces along the pile-soil interface depends on the variation of the dynamic soil resistance. The vibrations propagate in the form of waves to the surrounding soil layers. At the toe, compression waves (P-waves) and shear waves (S-waves) occur, which both extend as spherical waves in all directions. When the waves reach the surface, they are reflected and refracted. The refracted waves are spread as surface waves (R-waves), which propagate with lower attenuation than body waves along the ground surface. A detailed description of the dynamic aspects of impact pile driving and arising ground vibrations has been presented by Massarsch and Fellenius (2008). In the following, a simplified method of assessing the dynamic pile driving process is presented.

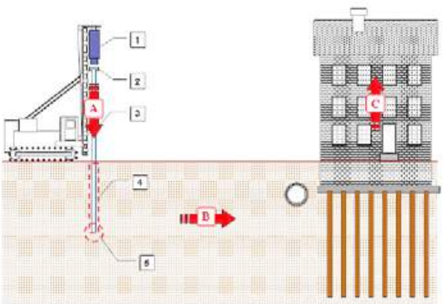

Figure 4 Propagation of stress wave from impact hammer along the pile and into the surrounding soil and building, (Massarsch 2004).

Vibrations are the result of the energy and force imparted by the hammer impact, which both are governed by the hammer impact velocity. The impact velocity can be calculated according to Eq. (8).

$$v_0 = \sqrt{2\,g\,h} \tag{8}$$

where: v_0 = hammer impact velocity (m/s), g = gravity constant (m/s^2), h = hammer height-of-fall (m). It is important to realize that the impact velocity—which is an important parameter for the generation of ground vibrations—is independent of the hammer mass. The maximum particle velocity in the pile that can be generated by the impact can be determined from the hammer and pile impedances according to Eq. (9).

$$v^P = \frac{v_0}{1+\dfrac{Z^P}{Z^H}} \tag{9}$$

where: v_P = maximum particle velocity in pile (m/s), v_0 = hammer impact velocity (m/s), Z^P = pile impedance (Ns/m), Z^H = hammer impedance (Ns/m).

The pile impedance can be determined from Eq. (10).

$$Z^P = A^P c^P \rho^P = \frac{E^P A^P}{c^P} \tag{10}$$

where: Z^P = pile impedance (Ns/m), A^P = pile cross sectional area (m^2), c^P = stress-wave speed in the pile (m/s), ρ^P = density of the pile material (kg/m^3), E^P = modulus of the pile (Pa). Equation (9) shows that the particle velocity depends on the ratio of the impedances of the pile and the hammer. The hammer impedance is determined similarily. If the hammer and pile impedances are equal, Eq. (9) becomes Eq. (11), indicating that the pile particle velocity is half the hammer impact velocity.

$$v^P = 0.5\,v_0 \tag{11}$$

where: v^P is maximum particle velocity in the pile (m/s), and v_0 is hammer impact velocity (m/s). The dynamic force of the stress wave in the pile, which decides the intensity of the dynamic soil resistance, can be calculated according to Eq. (12).

$$F_i = Z^P\,v^P \tag{12}$$

where: F_i = dynamic force in the pile (N), Z^P = pile impedance (Ns/m), v^P = maximum particle velocity (m/s).

5.1 Ground Vibrations due to Pile Driving

Ground vibrations are often caused when the pile encounters significant toe resistance. The dynamic toe resistance can be calculated according to Eqs. (13) and (14) (Massarsch and Fellenius 2008).

$$R_T = 2\,v^P\,Z_P \tag{13}$$

$$Z_p = A^P c_p \rho_s \tag{14}$$

where: R_T = toe resistance (N), v^P = maximum particle velocity (m/s), Z_P = impedance of the soil (with respect to P-waves) below the pile toe (Ns/m), A^P = pile cross sectional area (m^2), c_P= = compression wave speed in the soil (m/s), ρ_s = the soil bulk density (kg/m^3).

The compression wave speed in loose, water-saturated soil is about equal to the wave speed in water, but it may be larger in dense or very dense soil. The soil density in most coarse-grained soils ranges from about 1,700 through about 2,200 kg/m^3. Notice that the soil impedance is different to the pile impedance. Combining Eqs. (13)

and (14) results in Eq. (15), showing a relation for the dynamic toe resistance as a function of the pile particle velocity (the other parameters are non variables in a specific case).

$$R_T = 2\, v^p\, A^p\, c_p\, \rho_s \qquad (15)$$

where: R_T = toe resistance (N), v^P = maximum particle velocity (m/s), A^P = pile cross sectional area (m^2), c_P = compression wave speed in the soil (m/s), ρ_s = the soil bulk density (kg/m^3).

5.2 Empirical Method for Estimating Ground Vibration

The engineering practice has developed empirical methods for assessing the potential of ground vibrations from pile driving. Equation (16) shows a relation often used.

$$v = k\frac{\sqrt{W}}{r} = k\frac{\sqrt{M\,g\,h}}{r} \qquad (16)$$

where: v = vertical component of the ground vibration (m/s), k = empirical factor (m^2/s/√Nm), W = impact energy transferred from the hammer to the pile (Nm), r = distance from vibration source to observation point on the ground surface (m), M = hammer mass (kg), g = gravity constant (m/s^2), h = hammer height-of-fall (m).

The empirical factor, k in the equation is not dimensionless, an aspect which is important when applying the relationship. Brenner and Viranuvut (1977) reported results from vibration measurements from pile driving in different soil types, Figure 5. Note that Figure 5 shows vibration velocity in mm/s while the distance is given in meters. Often in the literature the average, k = 0.75 is used to estimate ground vibrations.

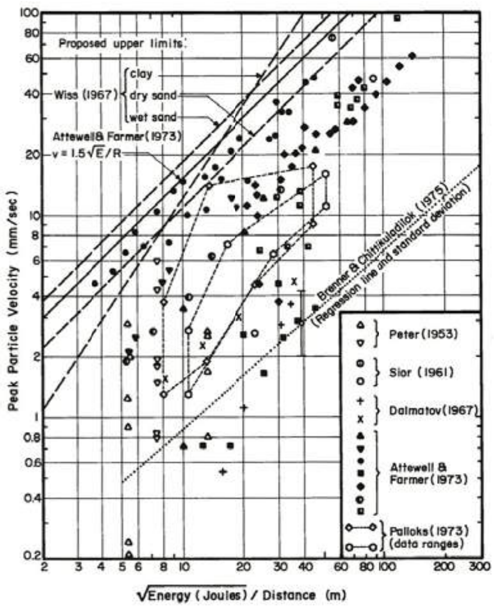

Figure 5 Results of vibration measurements at pile driving with indication of the spread of the k-factor (Brenner and Viranuvut 1977). Note that different units of length are used for particle velocity and distance.

Figure 5 shows that the spread of the measured values is large, in spite of plotting data in a double-logarithmic diagram. Note that the depth of the vibration source (pile penetration depth) is not specified in Eq. (16) nor in Figure 5. Another aspect which limits the application of this semi-empirical relationship is that neither pile characteristics nor the geotechnical conditions are included in the relationship.

When considering vibrations with respect to the distance to the pile, it is important to take into account the depth of the vibration source, which is usually located at some depth below the ground surface. Accordingly, Eq. (16) modifies to Eq. (17).

$$v = k\sqrt{\frac{M\,g\,h}{z^2 + x^2}} \qquad (17)$$

where v = vertical component of the ground vibration (m/s), k = empirical factor (m^2/s/√Nm), W = impact energy transferred from the hammer to the pile (Nm), M = hammer mass (kg), g = gravity constant (m/s^2), h = hammer height-of-fall (m), z = pile penetration depth, (m), x = horizontal distance from pile to observation point at ground surface, (m). Figure 6 defines the parameters used in Eq. 17.

A common misunderstanding is to consider the distance from the vibration source to the observation point to be the horizontal distance, x. Instead, the actual distance, r, from the vibration source to the observation point should be used. Under any circumstance, it is recommended to apply the depth of the vibration source (often the pile toe), r rather than to assume the horizontal distance, x at the ground surface.

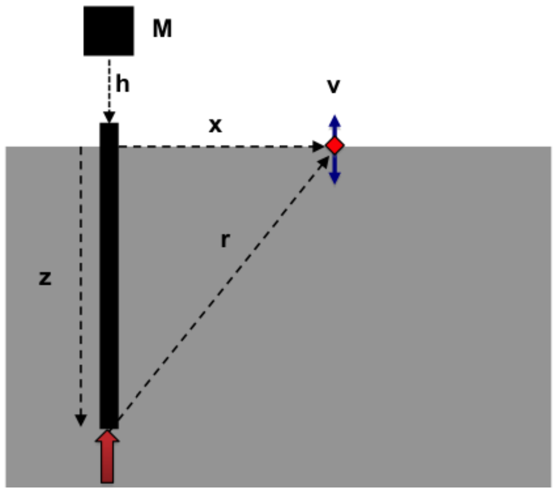

Figure 6 Definition of parameters used in Eqs. (16) and (17)

Note that Eqs. (16) and (17) are based on the energy imparted to the pile. They do not consider loss of energy between the pile hammer and the pile, nor the dynamic soil resistance along the pile shaft and at the pile toe. It can be assumed that the driving energy (hammer mass and height of fall) is well-matched to ensure penetration of the pile. However, if the dynamic soil resistance is small and the applied driving energy is larger than required (easy driving), vibration levels calculated according to Eq. (17) will be too high. Thus, Eq. (17) gives an upper limit of ground vibrations generated at the pile toe at medium to hard driving conditions. On the other hand, if substantial dynamic soil resistance ("hard" driving) is generated during pile penetration along the pile shaft, cylindrical wave can be generated, which are not considered in Eq. (17). The effect of vibrations emitted along the pile shaft can be estimated according to the procedure outlined by Massarsch and Fellenius (2008)

Heckman and Hagerty (1978) demonstrated that the k-factor depends on the pile impedance. Massarsch and Fellenius (2008) analyzed their data and found that the k-factor is a linear function of the inverse of the pile impedance, Z^P, per Eq. (18), as shown in Figure 7.

$$k = \frac{436}{Z^P} \qquad (18)$$

where: k = empirical factor (m^2/s/√Nm), Z^P = pile impedance (Ns/m).

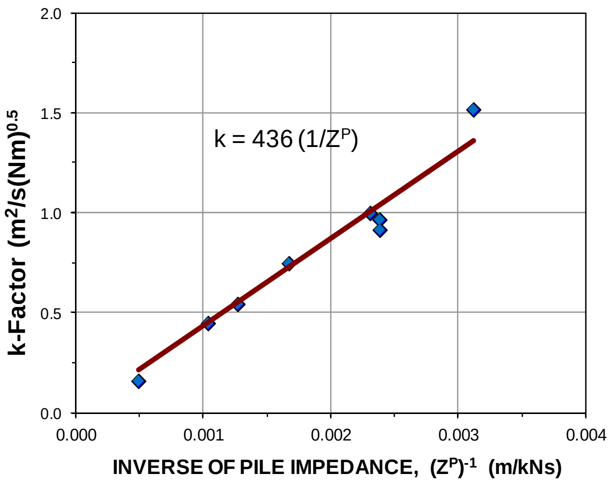

Figure 7 Relationship between the pile impedance Z^P and the empirical coefficient k in Eq. (16), re-analyzed data from Heckman and Hagerty (1978).

Note that the constant in Eq. (18) is not dimensionless. Combining Eqs. (16) and (18) results in Eq. (19), which can be used for estimation of ground vibration from pile driving.

$$v = \frac{436}{Z^P} \frac{\sqrt{F^H W_0}}{r} \tag{19}$$

where: v = vertical component of the vibration velocity (m/s), k = 436 ($m^2/s/\sqrt{Nm}$), Z^P = pile impedance (Ns/m), F^H = hammer efficacy factor (--), W_0 = nominal energy of pile driving hammer (Nm), and r = distance (m) from vibration source (the pile toe) to an observation point on the ground surface.

5.3 Sample Calculations

The theoretical relations described in the foregoing section can be used in practice. As an example, assume a concrete pile with a cross sectional area A^P = 0.09 m^2 (0.3 m x 0.3 m), wave speed, c_p = 3,900 m/s, concrete density, ρ^P = 2,630 kg/m^3, which combine to an elastic modulus, E^P = 40 GPa. The pile will be driven through a medium dense soil. Eq. (10) gives the pile impedance Z^P = 923 kNs/m. Figure 8 shows vertical ground vibration velocity vs. distance from source for five hammers used with the same 0.5-m height-of-fall, but having different mass—ranging from 3,000 through 7,000 kg. F^H = 1.0 was applied to the calculations.

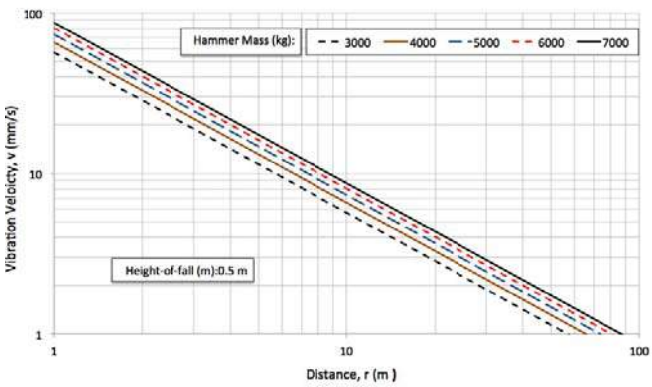

Figure 8 Variation of vertical ground vibration velocity as function of hammer mass for increasing distance from vibration source (assumed at pile toe) to observation point at ground surface, according to Eq. (19).

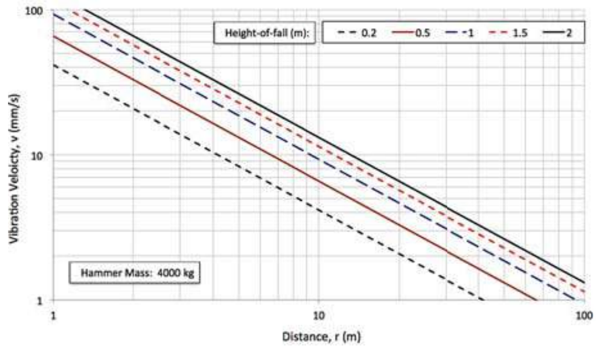

Figure 9 Variation of vertical ground vibration velocity of hammer with mass of 4,000 kg, as function of height-of-fall for increasing distance from vibration source (assumed at pile toe) to observation point at ground surface, according to Eq. (19).

In the next example, the square concrete pile is to be driven to a dense soil layer at 10 m depth. The hammer mass is 4,000 kg and the hammer height-of-fall is assumed to range between 0.2 and 2.0 m. The calculation results according to Eq. (19) are shown in Figure 10. Note that in this case, the horizontal distance at the ground surface is shown in linear scale.

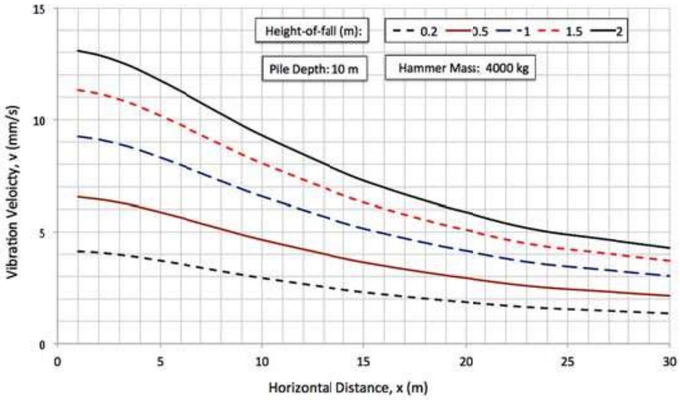

Figure 10 Variation of vertical ground vibration velocity caused by a hammer with mass of 4,000 kg used at a range of height-of-fall, assuming the vibration source (pile toe) is at 10 m depth.

6. CASE HISTORIES

Most case histories only provide information regarding pile type and vibration velocity as function of horizontal distance from the pile. Only few case histories are available that report ground vibration velocity with information on pile imped ance and measured pile penetration depth. The measured ground vibration for the following case histories are compared with the semi-empirical method presented above.

6.1 Case History - Sweden

Nilsson (1989) performed comprehensive vibration measurements during the driving of a series of test piles in southern Sweden, near the city of Skövde. A detailed presentation and interpretation of the tests was given by Massarsch and Fellenius (2008). The main objective of the vibration measurements was to establish site-specific driving methods and to select the optimal pile type which would minimize ground vibrations. Ground vibrations were of major concern due to the fact that several buildings and installations in the vicinity were susceptible to vibrations.

The soil profile in the test area consisted of about 2 m to 4 m of surface fill, well-compacted, alternating layers of furnace slag sand size particles and sand and gravel. Below followed a relatively homogeneous, 12 m thick layer of medium stiff clay with average undrained shear strength of 30 kPa deposited on a layer of sand with a thickness of 7 m on glacial till. Bedrock was encountered at a depth of about 25 m below the ground surface. The groundwater table was located about 3 to 4 m below the ground surface at the top of the clay layer. Unfortunately, data from more detailed geotechnical investigations (such as penetration tests or soil sampling) are not available.

The existence of a stiff surface layer on top of the clay suggested that vibration problems would likely occur during the early stage of the driving. Vibration problems could also be expected during seating of the piles into the bearing layer at 24 to 25 m depth. The permissible vibration values with respect to damage to the existing structures and installations were estimated according to Swedish standard SS 02 52 11, which has been described above. As the piles were driven into sandy, clayey soils, according to the Swedish standard, a vibration velocity, v_0, equal to 9 mm/s was chosen (Table 1). The buildings were of normal type (F_b = 1), constructed of reinforced concrete (F_m = 1.2), and with foundations on toe-bearing piles (F_g = 1.0). Therefore, according to Eq. (8), the maximum allowable vibration velocity (the vertical component) was v_{max} = 10.8 mm/s. In order to assess whether pile driving would exceed the maximum allowable vertical vibration velocity of 11 mm/s, the driving of two pile types—reinforced concrete and steel pipe piles—was monitored by extensive ground vibration measurements (Nilsson 1989).

6.1.1 Concrete Piles

Three precast concrete piles, called Piles 7, 8, and 10, were driven using an impact hammer. Two of these had the upper pile section covered with an asphalt layer to reduce the potential effect of negative skin friction. Each concrete pile was built up of three segments of lengths (13.3 + 10 + 6 = 29.3 m), which were connected in the field with a mechanical type splice. The lower pile segment (starting segment) had a 270 mm x 270 mm square cross section and the upper two pile segments a 235 mm x 235 mm square cross section. The concrete pile had a wave speed of 4,000 m/s, a bulk density of 2,400 kg/m^3 resulting in an elastic modulus of 39 GPa. Thus, the impedances, Z^P, of the starting and upper pile elements were 711 and 552 kNs/m, respectively.. The piles were driven by a hydraulic hammer type Banut with a mass of 4,000 kg. During the driving through the overburden soils, the hammer height-of-fall was kept to 0.40 m. It was increased to 0.50 m during the termination driving in the stiff glacial till at a final depth of approximately 25 m. Figure 11 shows pile penetration resistance (blows/500 mm penetration) as a function of the accumulated (total) number of blows during the driving to 25 m depth to seating the pile in the stiff glacial till. The figure shows the four depths where detailed vibration analyses was carried out.

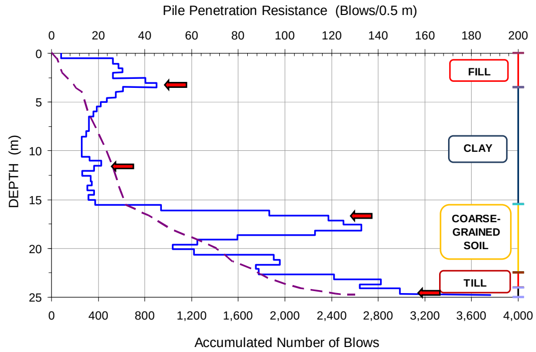

Figure 11 Pile penetration resistance during driving of the concrete pile with hydraulic hammer to 25 m depth. Also indicated are main soil layers and, by arrows, the depths at which detailed vibration analyses were carried out. (Data from Nilsson 1989).

6.1.2 Steel Pipe Piles

Three steel piles of type Gustavsberg, made of ductile steel, called Piles 11, 19, and 20, were also installed. Piles 19 and 20 consisted of six segments of 5 m length (total length of 30 m) and Pile 11 had three segments of 5 m length joined by welding in the field. Piles 19 and 20 had diameter 170 mm and wall thickness 13 mm, while Pile 11 had diameter 118 mm and wall thickness 10 mm. Thus, the larger piles had an impedance of 245 kNs/m and the smaller pile an impedance of 130 kNs/m.

The steel pipe piles were driven to a total depth of 9.8 m (Pile 19), 19 m (Pile 20) and 25.5 m (Pile 11), using an impact hammer with mass of 1,500 kg and 300-mm height-of-fall. During a brief testing phase, the 4,000 kg Banut hammer was also used at the height-of-fall ranging from 100 to 300 mm. It was observed that during the easy driving with the heavy pile hammer, ground vibrations were lower, as opposed to when it was driven with the lighter hammer.

6.1.3 Vibration Measurements – Concrete Pile.

Vibration measurements were performed using five geophones placed at 10, 20, and 40 m distance from the respective test pile. A data logger recorded the peak value of vibration velocity at each hammer blow as well as the depth of the pile at each measurement. Figure 12 shows the vertical vibration velocities measured at 10 (V1), 20 (V2), and 40 (V3) m horizontal distances from Pile 10 as function of selected pile toe depths.

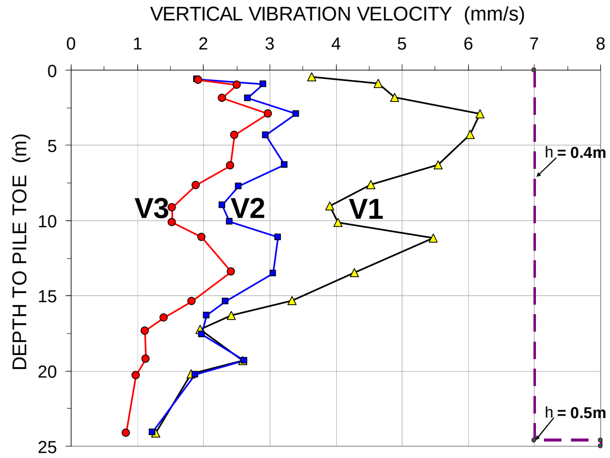

Figure 12 Vertical vibration velocities measured when driving concrete Pile 10. The measurements were taken at 10 (V1), 20 (V2), and 40 (V3) m horizontal distance as function of pile penetration depth. The hammer height-of-fall is indicated as "*h*". (Data from Nilsson 1989).

When the pile was driven through the surface fill, the magnitude of the vibration amplitudes at 10 and 20 m distance are relatively equal, compared to that at 40 m distance. The vertical vibration velocity decreases markedly with increasing horizontal distance from the pile. At a pile depth range of 17 to 25 m, the direct distances from the pile toe to the measurement points V1 and V2 were 26 m and 32 m, respectively. The distance difference is small in terms of vibration propagation, which explains why the measured vibration amplitudes are almost the same.

Figure 13 shows the results of vibration measurements for the three composite concrete piles, driven with impact hammer (4,000 kg) and 0.4 m height-of-fall (raised to 0.5 m during the termination driving). The vibration velocity curves (one dashed and one dotted), calculated by means of Eq. (19) for the two pile impedances, are shown as well as the measured values of vertical vibration velocity. F^H = 1.0 was applied to the calculations.

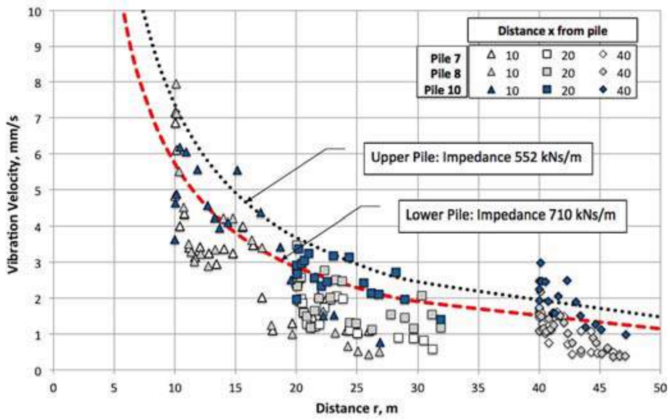

Figure 13 Vertical vibration velocities measured when driving the composite concrete piles plotted together with two values of pile impedance calculated from Eq. (19). (Data from Nilsson 1989).

The calculated lines lie above the measured values with the exception of measuring point V3 at 40 m distance, where vibrations during driving of Pile 10 exceeded the calculated values in a few instances. However, the Eq. (19) calculations agree surprisingly well with the measured ground vibrations, considering the complexity of the problem. Note that the diagrams in Figure 13 is in linear scale, which gives a more realistic comparison between measured and calculated values as opposed to a diagram showing values in log-log scale, otherwise commonly used for reporting vibration measurements.

6.1.4 Vibration Measurements – Steel Pipe Pile

The results of vibration measurements during driving of the steel pipe piles using a Banut hydraulic hammer are shown in Figure 14. The three steel pipe piles were generally driven with a 1,500 kg hammer and 0.3 m height-of-fall. $F^H = 1.0$ was applied to the calculations.

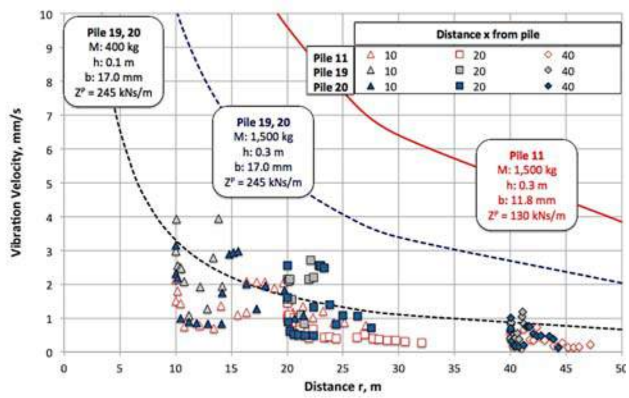

Figure 14 Vertical vibration velocity measured when driving the steel pipe piles plotted together with values calculated from Eq. (19). Also shown are vibration velocities calculated for driving with reduced energy: hammer mass of 400 kg and height-of-fall 0.1 m. (Data from Nilsson 1989).

It is apparent that in the case of steel pipe piles, the vibration velocities calculated by Eq. (19) are larger than the measured ground vibrations and especially so for the pile with smaller diameter (Pile 11). In the opinion of the authors, the main reason for the measured vibration velocities being smaller than those calculated, is the easy penetration of the small steel pipe piles when being driven by a heavy hydraulic hammer.

That is, the driving energy was larger than needed, that is, the ratio of actual energy in the wave down the pile was smaller than usual. The dashed lines in the figure show vibration velocities calculated for a lower driving energy (hammer mass 400 kg) with reduced height-of-fall (0.1 and 0.3 m) used during easy driving. The calculated values for the low energy driving lie closer to the measured values. Another explanation of the discrepancy between calculated and measured vibration amplitudes could be that the relationship shown in Figure 7 does not apply correctly to piles with very low impedance. Note that in Figure 7, only one data point was in the range of low pile impedance. Additional data for piles with low impedance are needed to substantiate the validity of Eq. (19) for piles with very low impedance.

6.2 Case History - Thailand

Brenner and Chittikuladilok (1975) measured vibrations due to driving precast concrete piles in Bangkok clay at two sites called Lak Si, located north of Bangkok, and EGAT, located south of Bangkok. The piles were precast concrete piles driven to depths ranging between 18 and 28 m. A large number of measurements were carried out at the ground surface at different distances from the pile during pile penetration. The paper also includes information on pile cross-section and pile material which is required information for the calculation of ground vibrations according to the above concept.

6.2.1 Site Conditions

At both sites, the ground surface was raised by an approximately 2 m thick fill consisting of sand (Lak Si) and sand and gravel (EGAT). Below the fill, the profile consisted of very soft to soft Bangkok clay followed at a depth of 14 to 15 m by stiff Bangkok clay. At the EGAT site, an approximately 3 m thick layer of loose, fine clayey sand was found, interbedded in the stiff clay between depths about 6.5 through 9.5 m depths. Cone penetration tests were performed at both sites and Figure 15 shows the distribution of cone stress, q_c. Note the high cone penetration resistance at the EGAT site at about 8 m depth.

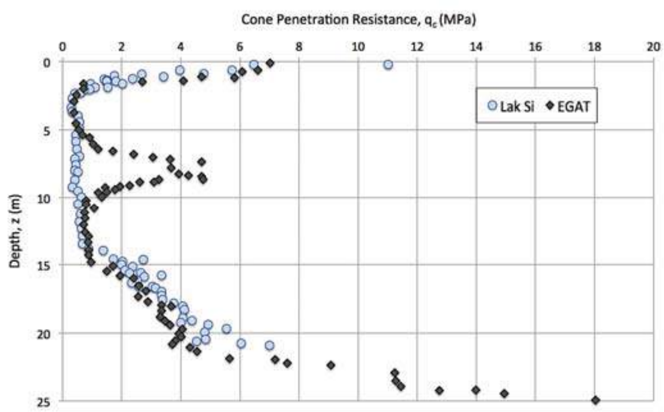

Figure 15 Distribution of cone stress, qc, at two Bangkok test sites. (Data from Brenner and Chittikuladilok 1975).

At Lak Si, concrete piles with a pile area of 0.068 m^2 were installed. The density of the concrete piles was assumed to 2,440 kg/m^3. The elastic modulus of the concrete piles was 39 GPa. The wave speed was 4,000 m/s. The pile impedance of the concrete piles was thus 658 kNs/m. At the EGAT site, concrete piles with a 0.180 m^2 pile area were installed. The density of the concrete piles was assumed to 2,440 kg/m^3. A 39 GPa elastic modulus and a 4,000 m/s wave speed were assumed for the concrete piles. Table 5 provides a summary of the pile and hammer data for the two sites.

Table 5 Summary of Pile Driving Information

Site	Lak Si	EGAT
Pile cross section (m^2)	0.0675	0.180
Pile length (m)	18	25
Pile impedance (kNs/m)	658	1,755
Mass of hammer (kg)	4,700	7,000
Hammer height of fall (m)	0.5	1.0

Driving the concrete pile through the hard sand fill layer close to the ground surface would have risked breaking the piles. Therefore, a steel probe was used to pre-bore through the fill. No information is available about the steel probe and no vibration measurements were reported from the driving through the fill layer. Despite the pre-boring, the concrete piles had to be driving through the sand fill. During pile driving, vertical vibration velocity was measured at the ground surface at several distances from the respective pile. The pile penetration depth was recorded for each measurement record. The following general observations were reported by Brenner and Chittikuladilok (1975):

- A sudden decrease in vibration occurred when the pile toe penetrated from the surface sand fill into the soft clay layer.
- At the EGAT site, a significant increase in vibration took place when the pile toe encountered the layer of fine sand at depth between 6.5 and 9.5 m, producing there a relative maximum in ground vibration velocity.
- When the pile toe reached the stiff clay layer, a pronounced increase in vibration level occurred due to the greater penetration resistance.
- A distinct increase in vibration intensity from driving through the stiff clay layer, however, appeared to occur only at surface points away from the pile.

The results of ground vibration measurements and calculated vibration levels according to Eq. (19) are shown in Figure 16 for the Lak Si site. Vertical ground vibrations were measured at four distances at the ground surface (10, 18, 25, and 30 m).

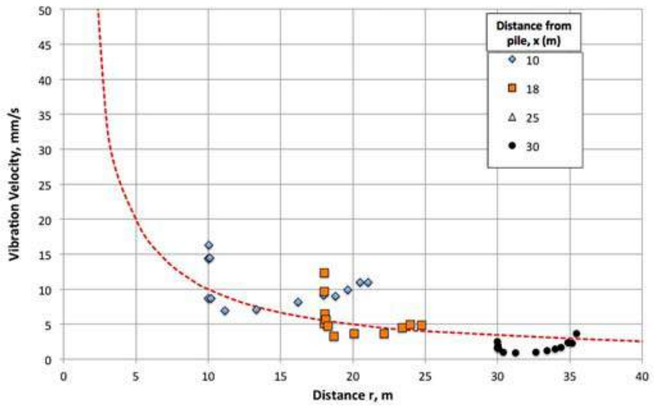

Figure 16 Lak Si Site: vertical vibration velocity measured during driving of concrete piles, driven with impact hammer (mass 4,700 kg) and height-of-fall of 0.5 m. Also shown are the calculated vibration velocities according to Eq. (19). (Data from Brenner and Chittikuladilok 1975).

The measured vibration velocities are in general agreement with those calculated according to Eq. (18), although at shorter distance, measured ground vibrations are slightly higher than those calculated. One likely reason for this difference between measured and calculated values could be that a significant part of the vibration energy was generated by the dynamic soil resistance acting along the pile shaft at approximately. In Eq. (18), the source of vibrations has been assumed to be located at the pile toe. Figure 17 shows the measured vertical vibration velocity at lateral distances ranging from 1 to 20 m from the pile.

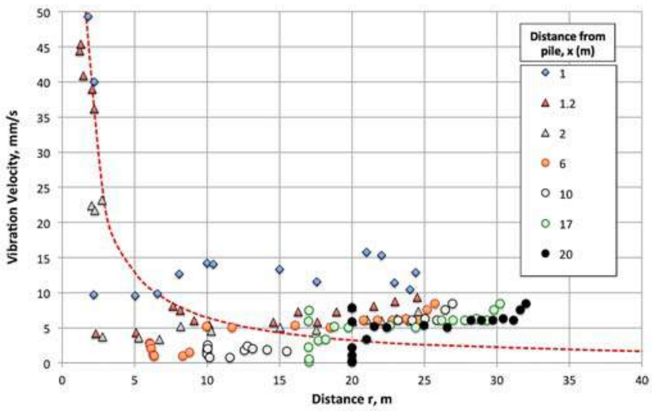

Figure 17 EGAT Site: vertical vibration velocity measured during driving of concrete piles, driven with impact hammer (mass 7,000 kg) and 1.0-m height-of-fall. Also shown are the calculated vibration velocities according to Eq. 18. (Data from Brenner and Chittikuladilok 1975).

At the EGAT site, the general trend of measured ground vibrations as function of distance to the pile toe is good. Close to the pile, measured ground vibrations are in very good agreement with calculated values according to Eq. (18). At increasing distance, measured values are somewhat higher than those predicted. This effect can be explained by the contribution of vibrations generated along the pile shaft at about 7 m depth, adding to the vibrations generated at the pile toe. Also, surface waves can become more important at larger distance from the vibration source, an effect which is not included in the analysis according to Eq. (18). However, in general, measured vibration attenuation is in good agreement with the proposed method of analysis.

7. SUMMARY AND CONCLUSIONS

When planning and designing a piling project, the geotechnical engineer must answer two questions: how strong will the maximum ground vibrations be during pile driving and which vibration levels are acceptable for the specific case.

The definition of vibration parameters is an important aspect when evaluating vibration measurements. Therefore, relevant parameters are described and defined.

A method of defining permissible ground vibrations from impact pile driving according to the Swedish vibration standard is presented. The standard takes into consideration the ground conditions, building standard and foundation conditions.

The standard is used in the Nordic countries of Europe with good results. It should be noted that the geological conditions and housing standards in Sweden—which can have importance for the dynamic response of buildings—may not be applicable in countries with significantly different foundation conditions and construction methods. This aspect needs to be taken into consideration when applying the Swedish vibration standard.

The concept is presented which describes the factors, which are of significance for the generation of ground vibrations due to impact pile driving. A simple approach is proposed for the prediction of vertical ground vibrations, which takes into account pile driving energy (hammer mass and height-of-fall) and pile impedance. Based on limited data, an almost linear relationship was found between the inverse of pile impedance and the *k*-factor. Figure 7 shows that the *k*-factor depends on the inverse of the pile impedance. This aspect of analysing ground vibrations due to impact pile driving has previously not been considered in a quantitative way.

Prediction of ground vibrations caused by impact pile driving is a complex task. The objective of the paper is not to predict the ground vibrations during all phases of pile driving. Rather, emphasis has been placed on predicting upper limits of vertical ground vibrations in the near-field, i.e., to a distance equal to about two pile lengths.

A method is presented which makes it possible to estimate the vertical component of ground vibrations during hard driving, taking into consideration the importance of pile impedance. However, the proposed method is based on simplified assumptions and needs to be verified by field measurements. Equation (19) defines the parameters on which the prediction of vertical ground vibrations in the near-field is based. Vibration attenuation is predicted, assuming that the distance from the source of dynamic driving resistance below the ground surface to a point at the ground surface is known. Thus, the actual distance from the energy source to a point on the ground surface should be used as the distance (r), and not the horizontal distance at the ground surface.

It should be noted that vibrations can be emitted at the pile toe as well as along the pile shaft. Geotechnical investigations are needed to determine where along the pile vibrations are primarily emitted. In the present method, it is assumed that the primary source of vibration is at the pile toe.

Sample calculations have been presented which illustrate the effect of hammer mass and hammer height-of-fall on ground vibrations as function of distance from the vibration source.

Two case histories with very different ground conditions, pile types, and driving methods were analyzed. The test objective was to determine an upper boundary of ground vibration velocity, which can occur during the driving process. The agreement between calculated and measured vertical ground vibrations is reasonable, considering the complexity of the problem. Vibration attenuation has been shown in linear scale, which give a better understanding of the accuracy of vibration attenuation than logarithmic diagrams.

The results from the evaluation of case histories confirm that the general trend of vibration attenuation is captured by the simple relationship given by Eq. (18). The pile impedance is an important parameter.

The main conclusion of the pile driving tests is that Eq. (18) surprisingly well predicts ground vibrations generated by driving concrete piles, and moderately well in the case of steel pipe piles with significantly lower impedance. The values calculated according to Eq. (18) are generally considered to give conservative results.

It is important to point out that the method does not consider the variation of dynamic soil resistance along the pile toe and pile shaft, which is possible when applying the more complex concepts proposed by Massarsch and Fellenius (2008). Therefore, when special care is needed, the assessments should be verified and adjusted by results of field measurements. Especially in the case of easy pile driving (low pile penetration resistance), the proposed method will overestimate actual ground vibrations.

8. ACKNOWLEDGEMENT

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