Attenuation of Ground Vibration Induced by Pile Driving

Atténuation d'Oscillation dans les Sols Entrainee par le Battement de Pieux

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SYNOPSIS Attenuation of ground vibration induced by pile driving is investigated using an equation established for Rayleigh waves. The attenuation equation is modified to include two components, one as a function of the coefficient of attenuation of soil and the distance from source of vibration and the other as a function of rated hammer energy and ground resistance. The former relationship is expressed as influence values while the latter as a family of curves. Parameters in the attenuation equation were evaluated from vibration data monitored during installation of test piles. The agreement between the relationship established and ground vibrations monitored at a number of project sites is found to be satisfactory.

INTRODUCTION

In many areas where soft alluvium and marine deposits prevail, deep foundations may be the only means of foundation support for heavy structures. In comparison with cast-in-place piles and caissons, driven piles have often proven to be the most effective and economical means of deep foundation construction. However, the by-product of pile driving--vibration--could seriously endanger neighboring structures and render driven piles unacceptable as a foundation solution. Prediction and control of ground vibration induced by pile driving are important tasks for engineers.

Due to the complexity of the mechanics of energy dissipation in pile driving, theoretical evaluation of ground vibration induced by pile driving would be extremely cumbersome and may not be practical for engineering applications. A semi-empirical relationship expressing the amplitude of particle velocity as a function of distance from the pile tip r, coefficient of attenuation of the soil α, rated hammer energy E, and ground resistance in terms of blow count B may be the most practical for engineering applications. This relationship would enable the engineers to estimate the magnitude of ground vibration induced by pile driving.

Damage criteria have been investigated by Crandell (1949), Wiss (1968), and Wiss and Nicholls (1974) among others. Crandell suggested an "Energy Ratio" (ER) to define the severity of vibration of the structures while Wiss recommended the "peak particle velocity" as the best parameter to assess damage potential of ground vibration. Wiss concluded that a peak particle velocity of 2 in./sec(5 cm/sec) is a good criterion for residential structures and 4 in./sec (10 cm/sec) for commercial and engineered structures to avoid damage due to vibration. However, a working criterion should be established for individual structures according to its age and conditions.

With the predicted ground vibration induced by pile driving and the damage criteria established for individual structures, engineers would be able to control pile installation so that damage to existing structures can be minimized.

IMPACT PILE DRIVING

When a pile is installed by using an impact hammer, the hammer is dropped on the cushion and the pile head, forcing the pile to penetrate into the ground. As soon as the pile penetrates the ground, the external energy from the dropping hammer is transmitted through the cushion and the pile to the pile tip and dissipates in the surrounding soil mass in the form of strain energy which propagates as elastic waves. Some energy losses are accumulated through the compression of the cushion, elastic deformation of the pile, heat and sound and skin friction of the pile. These losses were discussed in detail by Parola (1970). However, based on observations, the major portion of the energy is transmitted to and released at the pile tip.

Following the release of strain energy at the pile tip, the energy is transmitted in the soil mass in various patterns of waves. The wave arriving at a given point first is called the primary (P) wave or compression wave, which is followed by the secondary (S) wave or shear wave. While these two waves are called "body waves", the P waves can travel in solids, liquids or gases and S waves can travel only in a solid because their existence depends on the ability to resist and transmit shear.

In contrast to body waves in the ground, there are others which travel only on a free surface. The type of surface wave which was first predicted mathematically by Lord Rayleigh is known as the Rayleigh wave or R wave. This surface wave which is similar in many ways to a water wave, is generated and propagate within a wave length of the surface.

A maximum depth of penetration of 300 ft (100 m) has been reported for R waves (Leet, 1960; O'Neill, 1971). Other surface waves such as Love wave, C wave, H wave and other surface waves were described by Leet (1960) and Richter (1958).

The arrival of P and S waves at a point on the ground surface is referred to by Lamb (1904) as the minor tremor and the arrival of the R wave as the major tremor. It has been shown by Ewing, Jardetzky and Press (1957) that the amplitude of particle velocity of the body waves decreases in proportion to the ratio of $1/r$ (where r is the distance from the source of vibration), except along the surface of the half-space, where the amplitude of particle velocity decreases as $1/r^2$. The amplitude of particle velocity of the R wave decreases as $1/\sqrt{r}$.

ATTENUATION OF GROUND VIBRATION

The strain energy released at the pile tip is transmitted spherically into the surrounding soil mass in various wave forms and is eventually dissipated through damping. Damping of strain energy comes in two forms; one is "geometrical damping" which occurs in an elastic system due to spherical spreading of the energy and the other is "material damping", which depends on the material properties of the soil medium. Both geometrical and material damping can be combined in an expression for R wave attenuation (Bornitz, 1931) as:

$$A = A_1\sqrt{\frac{r_1}{r}}\ \mathrm{Exp}\left[-\alpha\ (r-r_1)\right] \tag{1}$$

where A = amplitude of particle velocity at distance r from source

A_1= amplitude of particle velocity at distance r_1 from source

r = distance from source to point in question

r_1= distance from source to point of known amplitude

α = coefficient of attenuation

Similar equations can be established for body waves. However, body wave amplitudes are usually significantly smaller than Rayleigh wave amplitudes. Since the maximum amplitude of particle velocity is of interest to engineers in pile driving, only Equation 1 will be required for piles driven within 300 ft (100 m) of the ground surface where R waves exist.

Equation 1 consists of two distinct components--a dimensionless "influence value" as expressed by $\sqrt{r_1/r}\ \mathrm{Exp}\left[-\alpha\ (r-r_1)\right]$ and A_1, the amplitude of a reference velocity at a given point. The influence value represents the effect of damping and therefore is expressed as a function of distance r and coefficient of attenuation α. The amplitude of the reference velocity A_1 for piles of the same cross-sectional area, depends on input hammer energy E and ground resistance which can be expressed in terms of blow count B:

$$A_1 = f(E,\ B) \tag{2}$$

INFLUENCE VALUE I_1

For convenience, $r_1 = 1$ is selected for this investigation. When $r_1 = 1$, Equation 1 becomes

$$A = A_1\sqrt{\frac{1}{r}}\ \mathrm{Exp}\left[-\alpha(r-1)\right] \tag{3}$$
$$= A_1 \cdot I_1$$

where

$$I_1 = \sqrt{\frac{1}{r}}\ \mathrm{Exp}\left[-\alpha(r-1)\right] \tag{4}$$

and A_1 is the reference velocity at r_1=1.

Equation 4 defines the influence value for attenuation of particle velocity induced by an unit excitation at the pile tip through a single layer soil medium with a coefficient of attenuation α. The influence values for α ranging from 0.01 to 0.05 ft^{-1} (0.03 to 0.15 m^{-1}) are shown in Figure 1 for a maximum r of 140 ft (43 m). Pile driving beyond this limit would not normally generate significant vibration of concern.

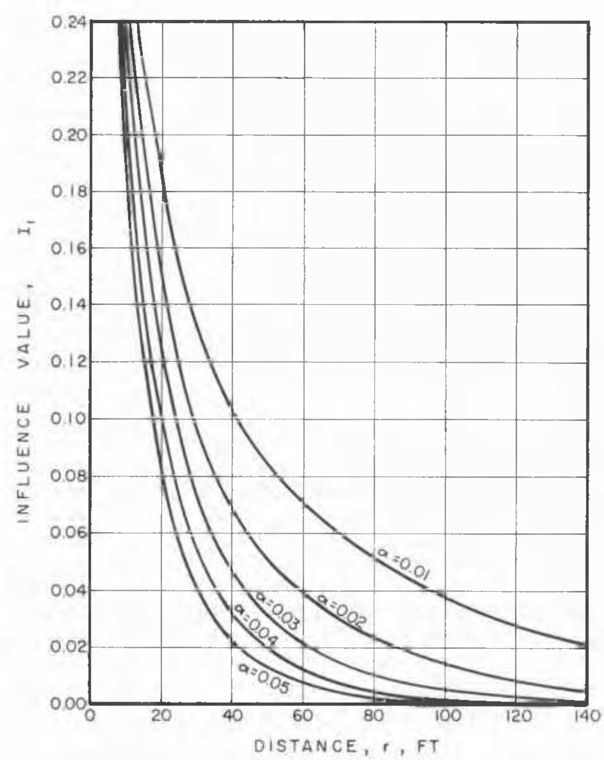

Figure 1. Influence Value

COEFFICIENT OF ATTENUATION α

The coefficient of attenuation α is a property of the soil. This coefficient is normally obtained from an on-site steady-state vibration test. The coefficient of attenuation can also be obtained from a pile driving test, provided that the ground vibration is monitored during test driving. If these two tests are not available, a simple method can be used to evaluate the coefficient by dropping a heavy hammer on the ground while the ground vibration is monitored at various distances.

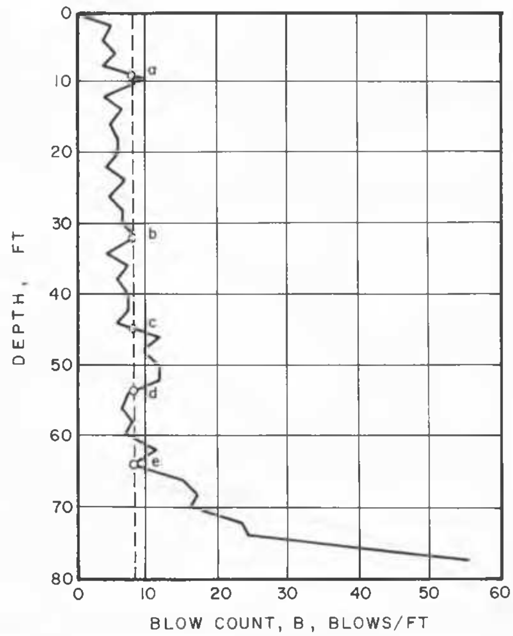

Figure 2. Typical Pile Driving Record

A typical pile driving record is shown on Figure 2. At points a, b, c, d and e, where a constant blow count is received, the energy released at the pile tip will be the same. The particle velocities generated at various distances from the pile tip become a function of the distance r and the coefficient of attenuation α.

When the particle velocity induced by a given blow count is plotted versus distance r, an exponential relationship is found (Figure 3). The value of α can be obtained by substituting data from Figure 3 into Equation 1 using the smallest r value as r_1 and the corresponding A value as A_1.

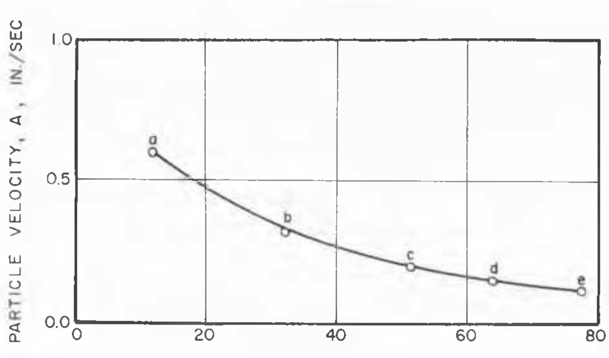

Figure 3. Particle Velocity Versus Distance

REFERENCE VELOCITY A_1

The reference velocity A_1 is a particle velocity which would be generated at r = 1 ft from the pile tip during driving. Due to the complicated mechanics of energy transmissions and losses in the pile driving, the reference velocity A_1 as a function of rated hammer energy and blow count as defined in Equation 2 would be extremely cumbersome. An empirical relationship established from field data would be most practical for engineering applications.

The empirical relationship can be obtained by introducing into Equation 3 the value of α and the particle velocities induced by various blow counts driven with hammers of different sizes. This relationship would be applicable for piles of the same cross-sectional area used in the investigation. Correction will be required for piles of other sizes.

In this investigation, the rated hammer energy is adopted as a parameter so that the procedures developed would be applicable to different type of impact hammers. However, the rated hammer energy, particularly for diesel and double acting hammers, should be corrected whenever possible to reflect the actual energy transmitted to the pile.

FIELD MONITORING

To evaluate α and A_1 as described in the preceeding sections, it is essential to obtain field data in an area where subsurface conditions are least complicated. It is also necessary to have different size of hammers with constant energy for this investigation.

An ideal site was selected for this investigation in Halawa Valley on the island of Oahu, Hawaii. The investigation was conducted during test pile installation for the Aloha Stadium and Interstate H-1 Halawa Interchange construction.

The Halawa Valley floor was formed primarily by a marine fluvial deposit overlain by a recent alluvial deposit. Borings drilled indicated that the subsurface material consists of soft to medium stiff black clayey silt on the order of 100 ft (30 m) in thickness underlain by the bedrock. This deposit is relatively homogeneous and has an average liquid limit of 75 percent and a plasticity index of 25 percent. The ground water table varies from near the surface to a few feet below.

During the installation of test piles, induced particle velocities were monitored using a Sprengnether tri-mode engineering seismograph at a number of ground stations. The test piles were $16\frac{1}{2}$ in. (42 cm) octagonal pre-stressed concrete piles ranging from 52 ft (15 m) to 80 ft (24 m) in length. When the penetration exceeded the pile length, a second section was spliced on top using a 5 ft (1.5 m) long steel splice can. These piles were installed using Vulcan 08, 10 and 14 hammers, with rated energies of 26,000, 32,500 and 42,000 ft-lb (35,300, 44,000, and 57,000 N-m) respectively. These hammers are single acting hammers activated by compressed air. The single acting hammer is ideal for this investigation due to its constant energy output.

A typical particle velocity record in three components is shown on Figure 4. The components of the particle velocity are rather irregular due to convolution of various waves. The maximum amplitude of particle velocity in three dimensional space is obtained using

$$A = A_{max} = \sqrt{A_V^2 + A_L^2 + A_T^2}$$

where A_V, A_L, and A_T, are the amplitudes of particle velocity in vertical, longitudinal, and transverse directions at the instant when the maximum vector sum occurs.

It is important to note that the pile length has very little effect on the amplitude of the induced particle velocity as evidenced by the consistency of the amplitudes before and after splicing a pile.

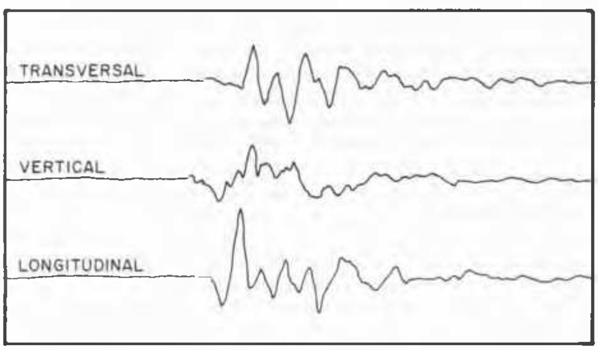

Figure 4. Typical Particle Velocity Record

EVALUATION OF α AND A_1

The coefficients of attenuation α were calculated according to procedures previously described for 12 test piles monitored. The α values are found to range from 0.01 to 0.02 ft^{-1} (0.04 to 0.06 m^{-1}) for the soft to medium stiff saturated clayey silt material. These values agree with those obtained by Barkan (1962). A list of α values obtained by Barkan is shown on Table 1.

Table 1, Coefficient of Attenuation After Barkan (1962)

Soil Type	Coefficient of Attenuation α (ft^{-1})	(m^{-1})
Saturated fine grain sand	0.03	0.1
Saturated fine grain sand in frozen state	0.018	0.06
Saturated sand with laminae of peat and organic silt	0.012	0.04
Clayey sand, clay with some sand and silt above water level	0.012	0.04
Saturated clay with sand and silt	0.012-0.036	0.04-0.12
Marly chalk	0.03	0.1
Loess and Loessial soil	0.03	0.1

A mean value of α = 0.015 ft^{-1} (0.05 m^{-1}) is then used in Equation 3 for the evaluation of A_1, the reference velocity at r_1=1 ft. For 16½ in. octagonal piles, A_1 as a function of rated energy of hammer E and ground resistance in terms of blow count B is shown in Figure 5.

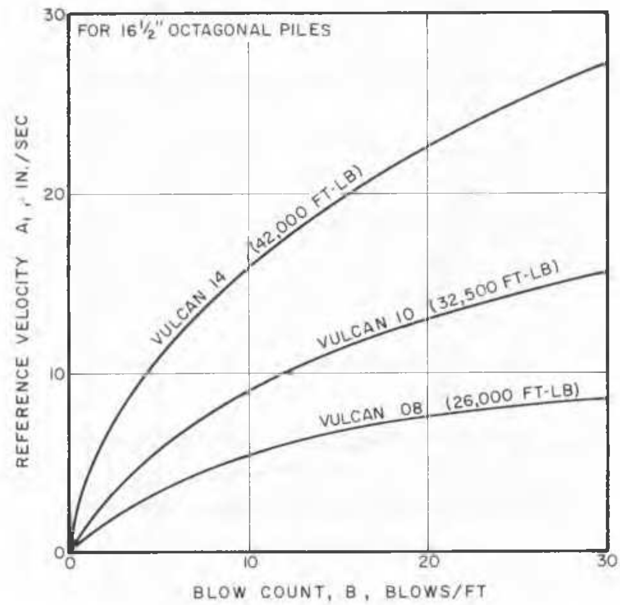

Figure 5. Reference Velocity versus Blow Count

The reference velocity A_1 is extrapolated from Figure 5 for the rated energy of hammer E and blow count B beyond the range of field monitoring. The extrapolated A_1 for 16½ in. piles is shown in Figure 6.

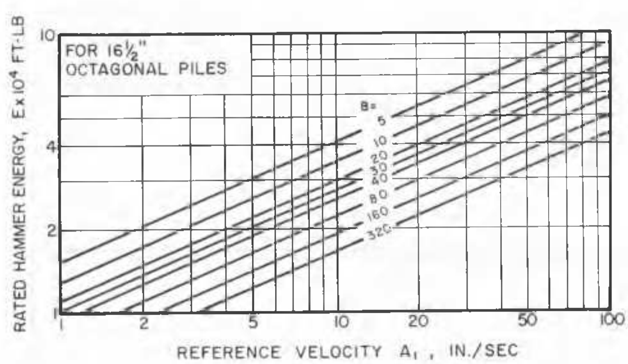

Figure 6. Reference Velocity versus Rated Hammer Energy

EFFECT OF PILE SIZE

Piles with smaller cross-sectional areas would penetrate soil strata with less resistance. Therefore, less ground vibration would be anticipated from driving smaller piles. A correction factor will be required for using Figures 5 and 6 when a pile other than a $16\frac{1}{2}$ in. octagonal (cross-sectional area = 225 in.^{2} (1450 cm^2)) is used. Based on data obtained at other sites, the following relationship may be used for the necessary correction:

$$\frac{(A_1)_a}{(A_1)_b} = \frac{(\text{Cross-Sectional Area})_a}{(\text{Cross-Sectional Area})_b} \qquad (5)$$

CASE STUDIES

The procedures described in this paper were used to estimate the maximum particle velocities which would be induced by pile driving at two project sites where ground vibrations were monitored during pile installation.

At site A, $16\frac{1}{2}$ in. octagonal pre-stressed concrete piles were driven with a Delmag D-22 diesel hammer. The hammer energy was reduced to 25,000 ft-lb (34,000 N-m) to minimize vibration. A coefficient of attenuation of α = 0.04 ft^{-1} (0.13 m^{-1}) is obtained from data monitored in test driving. (If the coefficient of attenuation is not available, the value listed in Table 1 may be used). The influence values for various distances are obtained from Figure 1. A_1 = 16 in./sec (40.6 cm/sec) is selected for an anticipated maximum blow count of 140 blows/ft. The estimated maximum particle velocity for the anticipated maximum blow count is then obtained by multiplication of the reference velocity (A_1 = 16 in./sec) by the influence value. The estimated maximum particle velocity is shown on Figure 7 as an envelope of the monitored particle velocities generated by various blow counts (equal or less than the maximum) in the field.

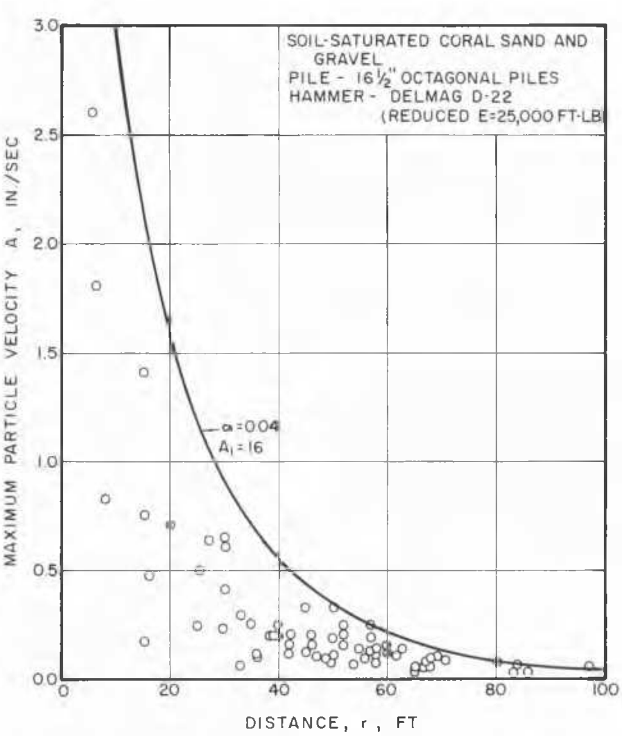

Figure 7. Case Study, Site A

At site B, both $16\frac{1}{2}$ in. octagonal and 12 in. square pre-stressed concrete piles were driven with a Link-Belt 520 diesel hammer. The energy of this hammer, measured with a pressure gauge connected to the bounce chamber was 25,000 ft-lb (34,000 N-m) per blow. A coefficient of attenuation of α = 0.04 ft^{-1} (0.13 m^{-1}) is obtained from test pile data. For $16\frac{1}{2}$ in. octagonal piles, A_1 = 8.5 in./sec (21.6 cm/sec) is selected for an anticipated maximum blow count of 30 blows/ft. The reference velocity is adjusted to A_1 = 5.4 in./sec (13.7 cm/sec) for the 12 in. square piles according to Equation 5. The estimated maximum particle velocity is shown as a solid curve for the $16\frac{1}{2}$ in. piles and as a dashed curve for the 12 in. square piles on Figure 8. These curves in general, form an envelope of the monitored particle velocities from various blow counts.

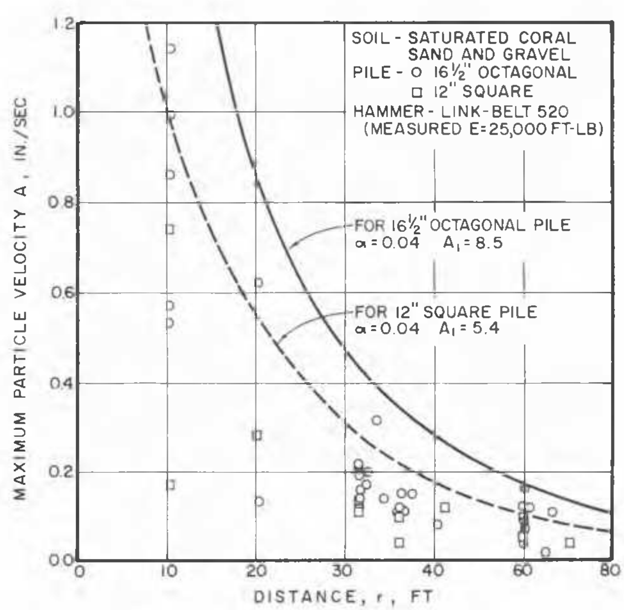

Figure 8. Case Study, Site B

CONCLUSIONS

A procedure is developed for the prediction of maximum particle velocities induced by pile driving in a single layer soil medium. A coefficient of attenuation and a reference velocity are required for the particle velocity estimation. The reference velocity as a function of the rated hammer energy and ground resistance has been developed from data obtained in test pile installations. The coefficient of attenuation is evaluated from test pile data for individual subsurface conditions. Influence values can be obtained for a coefficient of attenuation, and a distance from the pile tip. The particle velocity can then be obtained by multiplication of an influence value by a reference velocity determined by the hammer energy and the ground resistance encountered.

With the agreement of estimated maximum particle velocity and monitored data during pile installation, it is concluded that the procedures presented in this paper would be satisfactory for engineering applications. Furthermore, it is concluded that the major portion of the energy input is released at the pile tip, and that the energy transmitted in the form of R waves is of primary engineering concern.

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