Al-Nahrain Journal for Engineering Sciences (NJES) Vol.20 No.2, 2017 pp.477-485
Special Issue - Proceedings of the 4th Eng. Conf. (21April 2016, Al-Nahrain Univ., Baghdad, IRAQ)

The Dynamic Effect of Pile Installation in Sand on Nearby Piles

Talib K. Qassim AlSheakayree
Department of Civil Engineering
College of Engineering
Al-Muthanna University
alsheakayree1@gmail.com

Qassun S. Mohammed Shafiqu
Department of Civil Engineering
College of Engineering
Al-Nahrain University
qassun@yahoo.com

Asma T. Ibraheem
Department of Civil Engineering
College of Engineering
Al-Nahrain University
drasma2005@yahoo.com

Abstract

Driven piles have often been used in many civil structures to provide structural loading support. However, the unavoidable vibrations induced by pile driving processes may cause varying degrees of damage to adjacent structures. This research presents experimental studies to investigate the transmitted vibrations induced by impact of pile driving on vicinity piles. In the experimental work, a small scale model was tested in a sand box (steel container 1 × 1.5 × 0.8 m) with pile driving hammer device to install the impact pile in sand soil by dropping weights (1, 2, 3, 4 and 5 kg) for different heights of falling (4, 8, 12, 16 and 20 cm). The peak particle velocity was measured at a head of the vicinity piles by vibration meter device. In this study, several piles on different distances away from the vibration source were studied. The experimental results indicate that the peak particle velocity for vibrations emitted with impact pile driving is increased with increasing the energy and the penetration depth of pile driving for all vicinity piles and it can be decreased without change in the driving energy by decreasing the weight of hammer and increasing the height of falling hammer. Vibration intensities are attenuated with increasing surface distance from the pile driving and the peak particle velocity decreased uniformly with surface distance from the pile driving for piles. Also, through laboratory model representation and evaluation of the results obtained in the laboratory, the empirical relations which were determined based on the scaled-distance concept, are appropriate and give results very close and can be relied upon to represent the transmission of vibration resulting from the impact of pile driving to nearby piles.

Keywords: Pile Installation, Dynamic Effect, Nearby Pile, Sand Soil.

1. Introduction

Piles are widely used for transmitting building loads from ground surface through weak soils to more competent soil or rock strata .During pile driving, a falling hammer transfers energy into the pile-head at impact and advances the pile into the ground. The transferred energy then travels down through the pile and some of the energy transmits to the surrounding soil through the pile-toe and some through the pile-shaft. The magnitude of vibration at any point in the ground, arising from any activity, is dependent on the amount of energy transmitted into the ground by the source, the rate of attenuation of the energy as it propagates through the ground and the distance of the observation point from the location at which the energy enters the ground (Dowding, 1996).

The pile driving vibrations in the ground create issues particularly in urban areas such as unwanted noise, environmental disturbance, and potential hazard for the neighboring properties because of the vibrations generated by pile driving. Many case studies have shown that ground vibrations due to pile driving often cause damage to the adjacent structures that are vulnerable to the ground shaking (Dowding, 1999; Kim, et al., 2000 and Woods, et al., 2004). The damage due to pile driving occurs either directly or via settlement of soil beneath foundations in the proximity of pile driving operations.

The potential damage to the adjacent structures can be prevented by conducting pre-construction surveys, monitoring and controlling the vibrations on site, and predicting the anticipated vibrations prior to pile driving (Dowding, 1999). Understanding the conditions under which those vibrations will cause damage is important to avoid excessive vibrations and damage claims. Thus, development of numerical methods that predict ground vibrations prior to pile driving becomes essential.

Previous studies on the analysis of pile driving have mostly focused on assessing the drivability, the bearing capacity, and driving efficiency of piles (Smith, 1960; Mabsout, et al, 1995 and; Liyanapathirana, et al., 2001). Only a limited number of research studies focused on the ground vibrations due to pile driving and their effects on adjacent structures (Ramshaw et al., 1996 and; Masoumi et al., 2007and 2008). Although the numerical models predicted ground vibrations consistent with the experimental data, they have not taken into account the essential soil and source parameters such as the nonlinear constitutive behavior of soil, friction between the pile and the soil, variation of pile penetration depth.

The effects of vibration on structures have also been the subject of extensive research. Much of this work originated in the mining industry, where vibration from blasting is a critical issue. The following is a discussion of damage thresholds that have been developed over the years.

A study by (Chae, 1978) classifies buildings in one of four categories based on age and condition. Table (1) summarizes maximum blast vibration amplitudes based on building type.

The Swiss Association of Standardization has developed a series of vibration damage criteria that differentiate between single-event sources (blasting) and continuous sources (machines and traffic) (Wiss, 1981). The criteria are also differentiated by frequency. Assuming that the frequency range of interest for construction and traffic sources is 10–30 Hz, Table (2) shows criteria for 10–30 Hz.

Table 1: Building vibration criteria (after Chae, 1978)

Category	PPV (Single Blast) mm/s (in/s)	PPV (Repeated Blast) mm/s (in/s)
Buildings of Substantial Construction	100 (4)	50 (2)
Residential, New construction	50 (2)	25 (1)
Residential, Poor Condition	25 (1)	12.5 (0.5)
Residential, Very Poor Condition	12.5 (0.5)	-

Table 2: Swiss Association of Standardization vibration damage criteria (after Wiss, 1981)

Building Class	Continuous Source PPV (in/sec)	Single-Event Source PPV (in/sec)
Class I: buildings in steel or reinforced concrete, such as factories, retaining walls, bridges, steel towers, open channels, underground chambers and tunnels with and without concrete alignment	0.5	1.2
Class II: buildings with foundation walls and floors in concrete, walls in concrete or masonry, stone masonry retaining walls, underground chambers and tunnels with masonry alignments, conduits in loose material	0.3	0.7
Class III: buildings as mentioned above but with wooden ceilings and walls in masonry	0.2	0.5
Class IV: construction very sensitive to vibration; objects of historic interest	0.12	0.3

Konan (1985) reviewed numerous vibration criteria relating to historic and sensitive buildings, and developed a recommended set of vibration criteria for transient (single-event) and steady-state (continuous) sources. Konan recommended that criteria for continuous vibration be about half the amplitude of criteria for transient sources. Table (3) summarizes the recommended criteria.

Table 3: Vibration criteria for historic and sensitive buildings (after Konan, 1985)

Frequency Range (Hz)	Transient Vibration PPV (in/sec)	Steady-State Vibration PPV (in/sec)
1–10	0.25	0.12
10–40	0.25–0.5	0.12–0.25
40–100	0.5	0.25

Dowding (1996) suggests maximum allowable PPV for various structure types and conditions. Table (4) summarizes these values.

Table 4: Building structure vibration criteria (after Dowding, 1996)

Category	Limiting Peak Particle Velocity mm/s (in/s)
Industrial Buildings	50 (2)
Residential	12.5 (0.5)
Residential, New construction	25 (1)
Historic Buildings	12.5 (0.5)
Bridges	50 (2)

The American Association of State Highway and Transportation Officials (AASHTO, 1990) identifies maximum vibration levels for preventing damage to structures from intermittent construction or maintenance activities. Table (5) summarizes the AASHTO maximum levels.

Table 5: Maximum vibration levels for preventing damage (after AASHTO, 1990)

Category	Particle Velocity mm/s (in/s)
Historic sites or other critical locations	2.5 (0.1)
Residential buildings, plastered walls	5.0-7.5 (0.2-0.3)
Residential buildings in good repair with gypsum board walls	10-12.5 (0.4-0.5)
Engineered structures, without plaster	25-37.5(1.0-1.5)

2. Materials and Testing

2.1 Materials

in table (6) sieve analysis carried out its grain size distribution according to ASTM (D422-2007) as shown in Figure (1). The soil was classified as SP type (Poorly graded sand) according to the Unified Soil Classification. ASTM (D453-2007) and ASTM (D2454-2007) were used to determine the maximum minimum dry unit weight. The principle for achieving a relative density was used to fill the box for test. To achieve that, the raining technique using special equipment as shown in Figure (1) is adopted. Direct shear box test was performed in general accordance with ASTM (D 3080-98). The direct shear box test has several particle sizes to box size requirements when preparing specimens for testing. The minimum specimen width should not be less than 10 times the maximum particle size diameter and the minimum initial specimen thickness should not be less than six times the maximum particle diameter.

Table (6): Physical properties of the sand used in present tests.

No.	Index property	value
1	Specific gravity (Gs)	2.68
2	D_{10} (mm)	0.16
3	D_{30} (mm)	0.33
4	D_{60} (mm)	0.43
5	Coefficient of uniformity (C_u)	2.7
6	Coefficient of curvature (C_c)	1.6
7	Maximum dry unit weight γ_{max} (kN/m^3)	18.5
8	Minimum dry unit weight γ_{min} (kN/m^3)	15.89
9	Dry unit weight (kN/m^2) at R.D = 37%	16.71
10	Maximum void ratio e_{max}	0.674
11	Minimum void ratio e_{min}	0.438
12	Relative density (R.D %)	37
13	Angle of internal friction (ϕ) at R.D =37%	31
14	Soil classification (USCS)	SP

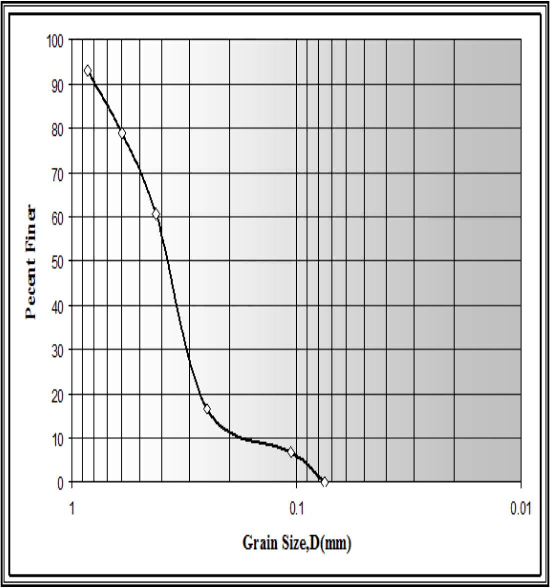

Figure 1: Grain size distribution for sand used in the present study.

2.2 Model Setup Formulation

To simulate the pile load test in the field, a new apparatus was manufactured and consists of the following:

1. Steel container.
2. Raining frame.
3. Pile driving hammer device.
4. Vibration meter device.

A steel container was constructed to contain the dry sand soil on which the tests on the model piles were to be performed. The container is used to prepare the test sample. The internal dimensions are 1500 mm long, 1000 mm wide and 800 mm deep as shown in Figure (2). The internal sides of the container were covered with 25 mm polystyrene sheets as isolation or absorbing boundary in order to reduce reflection of generated waves from pile driving (Al-Omari, 2014).

Raining frame is a Hook-crane with longitudinal beam and hydraulic jack was used to achieve any desired elevation and anywhere with container steel. The longitudinal beam was used to carry the cone that is used to pour the sand. This configuration of raining frame helps get a uniform density by controlling the height of fall. The longitudinal beam and hydraulic jack with the cone ensure that each particle drops in equal height and uniform intensity. To control the height of fall, additional tubular elongations at the cone end were used by using adapted tubes. The raining frame is illustrated in Figure (3).

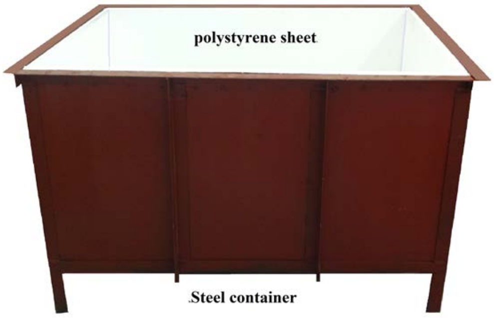

Figure 2: Steel container with polystyrene sheets.

Figure 3: Sand Raining Frame used for controlling density.

Pile driving hammer device is the driving system consists of a steel base with (300mm) in diameter and 150 mm in thickness. Two columns are fixed vertically (20mm) in diameter to support all parts of driven system, these parts are shown in Figure (4). The main part in driving hammer is the steel rod, it contains steel helmet in the rod head and steel cylinder which is used as a base for dropping the hammer weight. The steel helmet was manufactured with pad that is suitable for all model hammers that are used in the tests. These parts are designed to make sure the fixity of piles and as possible to reserve the vertical direction for pile penetration without tilting during the driving process.

Vibration meter device is the device used for this task includes essentially a vibration meter as shown in Figure (5). The vibration meter converts the velocity to be measured into an electrical signal. The voltage signal is sensed by an electrical unit. The signal is then fed to the vibration meter for recording the waveform. The vibration transducers may be either displacement, velocity or acceleration type depending on the electrical voltage signal induced in it. The vibration was measured by fixing vibration pick-up on the top face of the foundation connected to the vibration meter by wire. The vibration meter is connected to the computer and the information of time versus velocity, velocity and frequency were recorded in excel sheets.

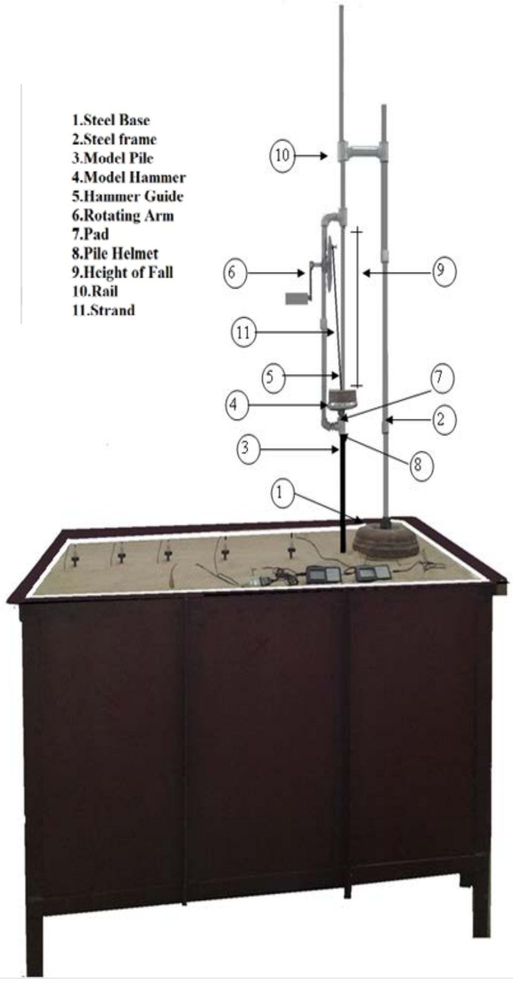

Figure 4: Piles hammer with steel container.

Figure 5: Vibration meter device.

3. Experimental Results and Discussion

To study the transmission of the vibrations induced by impact pile driving to the vicinity soil and piles, many variables that will influence magnitude of vibrations induced by impact pile driving as following were studied:

- Driving energy (Eo): many energies have been used changing the weights (Wh) and heights of falling (Hf).
- Penetration depth of pile: the length of pile driving (L_p) is divided into 10 penetration depths (D_p). Each depth was 30 mm and use (penetration depth/ length of pile driving) ratio D_P/L_P = 0.1, 0.2, 0.3, 0.4, 0.5, 0.6, 0.7, 0.8, 0.9, 1. The vibration meter device was used to measure peak particle velocity (PPV) mm/s at each penetration depth.
- Distance from the impact pile driving: many distances from vibrations source have been used (300, 600, 900 and 1200 mm) or (Distance / L_P= 1, 2, 3 and 4), for points at surface soil and piles.

The extent of structure damage due to pile driving is believed to be related to the magnitude of ground vibration that is often quantified in terms of peak particle velocity (PPV). There are many specifications and provisions that define allowable (or permissible) PPV values, and the vibration meter device recorded only the peak particle velocity (PPV). Furthermore, the dynamic strain induced during the passage of a wave is proportional to the particle velocity; it is strain which causes damage (New, 1986).

4. Vibrations Induced with Impact Pile Driving

Three types of ground waves are induced by impact pile driving:

End pile waves: spherical wave emitted from the pile toe (i.e. spherical waves caused by the dynamic resistance at the pile toe).

Pile shaft waves: cylindrical waves propagating laterally from the pile shaft. Shear stress waves are generated along the skin of the pile due to the friction between the pile and soil particle

Surface waves: which are generated from the pile shaft near the surface either by friction or by whip. Surface waves can also be generated from the refraction of body waves at the ground surface at a critical distance from the pile.

A stress wave will be generated simultaneously in the pile and in the hammer. These three wave types are affected by the velocity-dependent soil resistance at the pile-soil interface as shown in Figure (6).

5. Transmitted Vibrations Induced by Impact Pile Driving to the nearby piles

There are three types of ground waves emitted with impact pile driving in shallow and deep foundations as shown in Figure (6); end pile waves, pile shaft waves and surface waves.

Many energies have been used by taking various weights (W_h) and different heights of falling (H_f) **(E_o (J) =W_h (kg) *g (m.s^{-2}) x H_f (m))** to study the influence of driving energy on peak particle velocity (PPV).

Many piles are located adjacent to the impact pile driving and the peak particle velocity is recorded at head of these piles directly and at distances {(Distance / L_P) = 1, 2, 3 and 4} from the pile driving.

At first pile {(Distance / L_P) = 1} from the impact pile driving, Figures (7) to (12) showed that the peak particle velocity recorded on the head of pile, increase with increasing the energy (E_o) and the penetration depth of pile driving because of the vibrations emitted from the pile shaft and pile toe for impact pile driving interacted with along the shaft of pile nearby, and this increase can be seen at the second half of the pile driving (the percentage of increase = 100-130%).

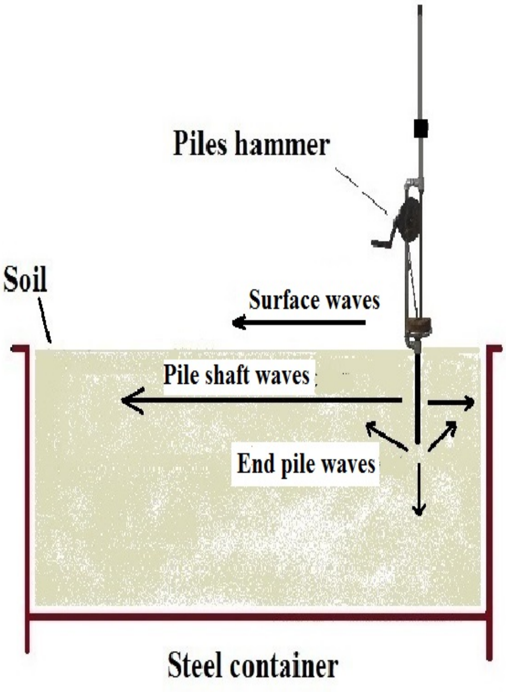

Figure 6: Transfer of vibrations from the hammer, through the pile, into surrounding soil.

The depth of the pile toe increases as driving progresses and the length of the pile shaft also increases. The source therefore changes throughout the drive, whether the source is the toe of the pile, the pile shaft or a combination of the toe and shaft. The nature of the ground into which the pile is driven and the distance from the pile to the measurement location also change continuously during the driving of a pile.

From Figure (13), it can be seen that the peak particle velocity increases without change in the energy but with change in the weight of hammer and height of falling**,** therefore it can be indicated that the influence of driving energy can be decreased by reducing the weight of hammer and increasing the height of falling hammer, because the weight of hammer is more effective than the height of falling hammer.

Vibration intensities are attenuated with increasing distance from the driving pile position and consequently their effect on nearby piles case as shown in Figure (14).Generally; this may be attributed to the fact that there are two types of vibration attenuation: geometric damping and material damping. Geometric damping occurs due to the expansion of the front wave with increasing distance from the wave source and material damping occurs due to the various physical parameters of the soil medium (Wiss, 1981).

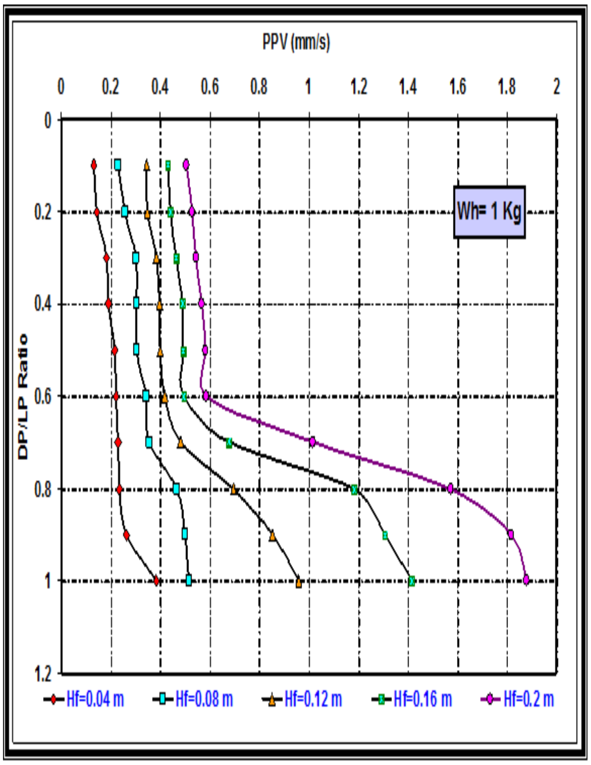

Figure (7): Influence of driving energy and penetration depth of pile on PPV with adjacent piles at (W_h = 1 kg).

6. Empirical Relations for Estimating Vibrations

In this section, a statistical approach is developed, in which a ground vibration monitoring distance corresponding to a frequency-independent threshold peak particle velocity (PPV) is determined based on the scaled-distance concept (Woods, 1997) and statistical results of ground vibration data collected from previous pile driving models in the state of footing, pile and without foundation.

A scaled-distance approach was used to approximate PPV values because it is a simple and well-known assessment of ground vibration attenuation generated by blasting or pile driving (Woods, 1985). The scaled-distance is usually defined as the horizontal distance from the vibration source scaled by the square-root of pile hammer energy, which reads (Wiss, 1981):

$$\mathbf{SD=D/}\sqrt{W} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ (4.1)$$

Where *D* is the horizontal distance from the driven pile to the point of interest, in m; and *W* is the hammer energy transferred to the pile (E_0), in J. Wiss applied the *SD* to approximate the PPV with the following relationship (Wiss, 1981):

$$\mathbf{PPV=k\ (SD)}^{-n} \ldots\ldots\ldots\ldots\ldots\ldots\ (4.2)$$

Where *k* is the velocity at one unit of distance, in mm/s; and *n* is the slope of the amplitude attenuation in the log-log PPV vs. scaled distance chart.

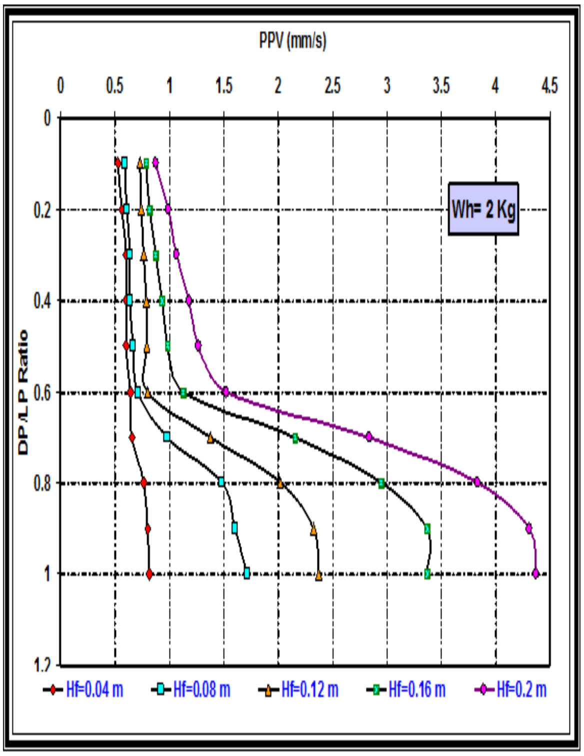

Figure 8: Influence of driving energy and penetration depth of pile on PPV with adjacent piles at (W_h = 2 kg).

The best fit line was first developed using the presently obtained results between the PPV values and the SD based on the regression analysis with Excel as shown in Figure (15) for pile. If a strong correlation exists between the PPV and SD for the data collected from the pile driving models, it can be directly used to determine the peak particle

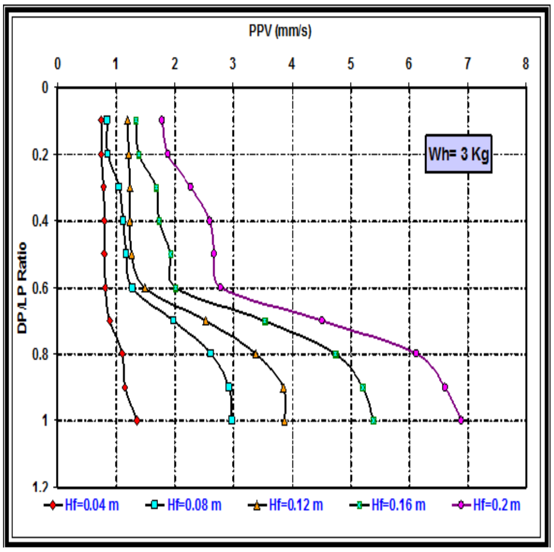

Figure 9: Influence of driving energy and penetration depth of pile on PPV with adjacent

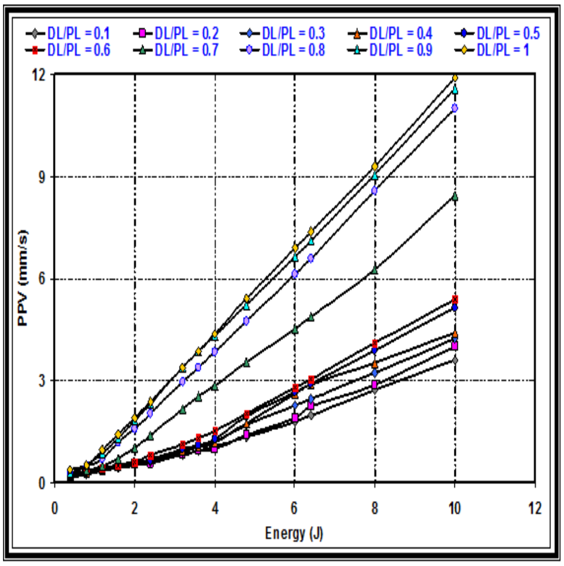

Figure 12: Influence of driving energy and penetration depth of pile on PPV with adjacent

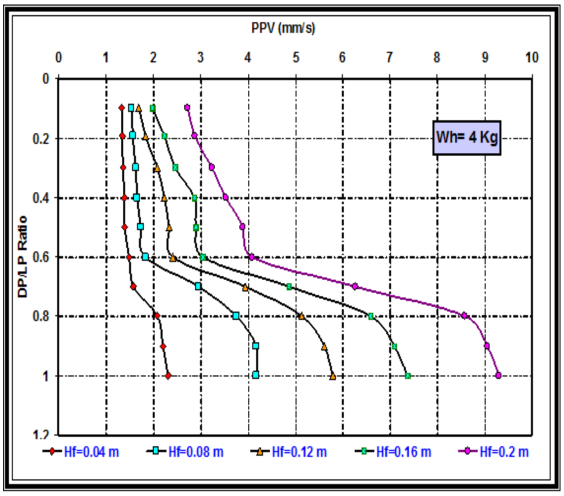

Figure 10: Influence of driving energy and penetration depth of pile on PPV with adjacent piles

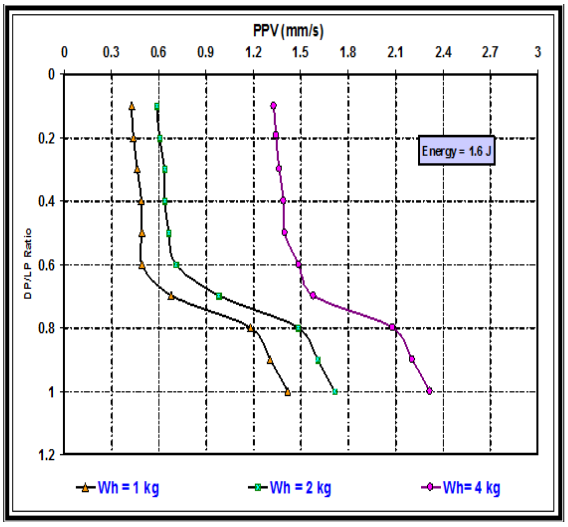

Figure 13: PPV for various weight and falling height of hammer without changing in driving

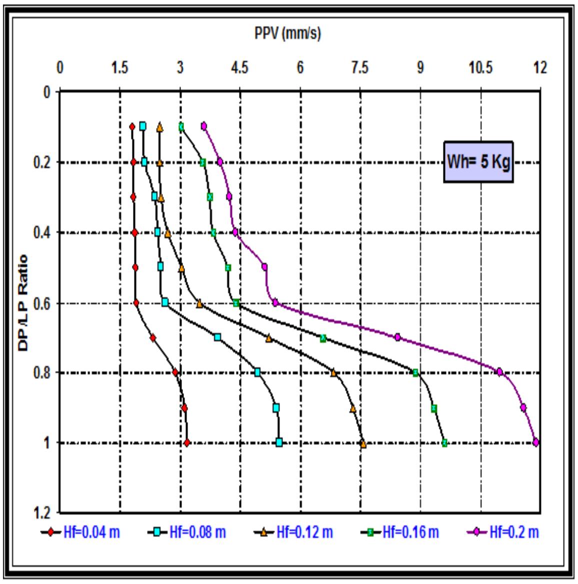

Figure 11: Influence of driving energy and penetration depth of pile on PPV with adjacent

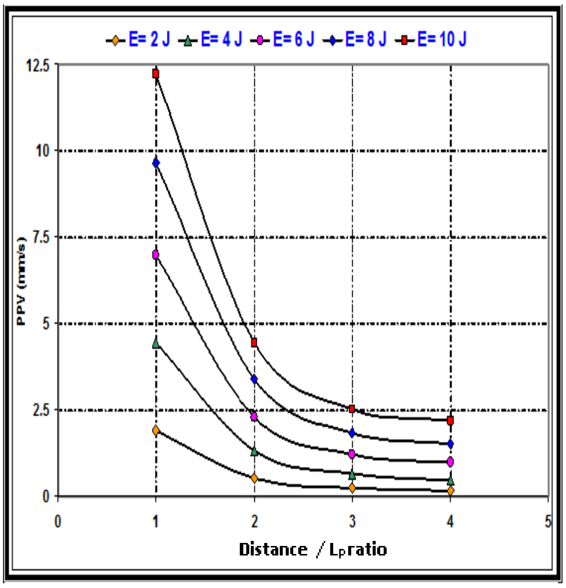

Figure 14: Influence of the distance from the vibration source on attenuation of vibrations on

velocity at any energy and any distance from source vibrations.
The relationship, k, n and regression value R^2 from Figure (15) are given below.

$$\mathbf{PPV= 0.8601\ (SD)^{-2.2953}}$$

k=0.8601, n=-2.2953 and regression value R^2=0.9901

Figure (16) presented the peak particle velocity with distance from the source vibrations obtained from pile driving models and empirical relations.

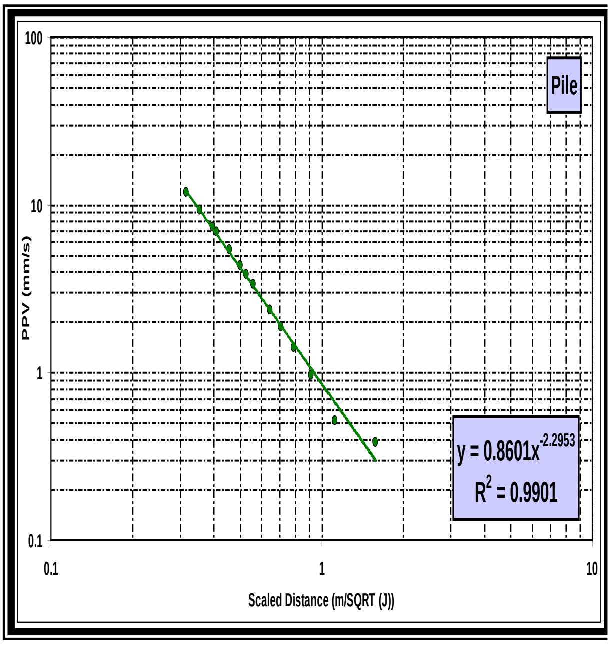

Figure 15: The scaled distance corresponding to the peak particle velocity with pile.

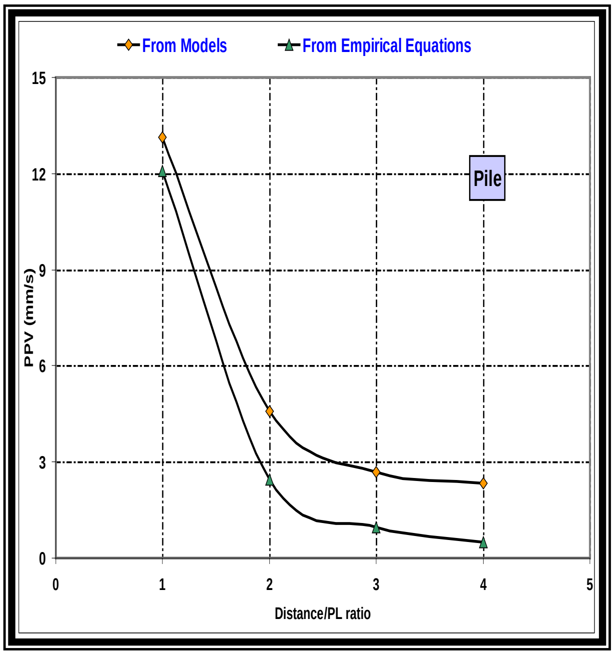

Figure 16: The peak particle velocity with distance from the source vibrations for pile (E= 10 J).

7. Conclusions

1. Three types of ground waves were emitted from impact pile driving to vicinity soil and piles: end pile waves, pile shaft waves and surface waves.

2. The peak particle velocity (PPV) for vibrations, which are emitted with impact pile driving, increase**s** with increasing the energy.

3. The peak particle velocity for vibrations, which are emitted by impact pile driving, increases without change in the energy but with change in the weight of hammer and height of falling**,** therefore the influence of driving energy can be reduced by decreasing the weight of hammer and increasing the height of falling hammer.

4. The peak particle velocity for vibrations, which are emitted with impact pile driving, increase**s** with increasing the penetration depth of pile driving.

5. The peak particle velocity for vibrations, which was emitted by impact pile driving, has effect on deep foundations much more than soil (without piles).

6. Vibration intensities are attenuated with increasing surface distance from the pile driving and the peak particle velocity is reduced uniformly with surface distance from the pile driving for soil and pile.

7. Influence of penetration depth of pile is more than the influence of driving energy (E_o) for piles.

8. The peak particle velocity (PPV) from empirical relations is determined based on the scaled-distance concept (Woods, 1997) and the statistical results of ground vibration data collected in the present work from the pile driving models in the case of footing, pile and that without pile.

References

[1] **AASHTO.** (1990). Interim Guidelines on Foundations, AASHTO LRFD Bridge Design Specifications American Association of State Highway and Transportation Officials, Washington, D.C.

[2] **Al-Omari, R. R., Asour, H. M., Abbass, B. J.** (2014), “Effect of Boundary Isolation on Dynamic Response on Pile Groups”, Second European Conference on Earthquake Engineering and Seismology, Istanbul, August 25-29, 2014, Turkey.

[3] **Chae, Y. S.** (1978). Design of excavation blasts to prevent damage. Civil Engineering—American Soci ety of Civil Engineers 48(4):77–79.

[4] **Dowding, C**. (1999). Construction vibrations. Englewood Cliffs, NJ: Prentice-Hall..

[5] **Dowding, C. H.** (1996). Construction vibrations. Prentice-Hall. Englewood Cliffs, NJ.

[6] **Kim, D., & Lee, J.** (2000). Propagation and attenuation characteristics of various ground vibrations. 19, 115–126.

[7] **Konon, W.** (1985). Vibration criteria for historic buildings. Journal of Construction Engineering and Management 111(3):208–215.

[8] **Liyanapathirana, D., Deeks, A., & Randolph, M.** (2001). Numerical modelling of the driving response of thin-walled open-ended piles. 25, 933–53.

[9] **Mabsout, M., Reese, L., & Tassoulas, L.** (1995). Study of pile driving by finite element method. 121(7), 535-543.

[10] **Ramshaw, C., Selby, A., & Bettess, P.** (1998). Computation of the transmission of waves from pile driving. Ground dynamics and man-made processes. London, UK: Thomas Telford Publishing. Masoumi et al., 2007and 2008

[11] **Smith, E.** (1960). Pile driving analysis by the wave equation. 86 (EM4), 35–61.

[12] **Wiss, JF** (1981) Construction vibrations: state-of-the-art. Journal of the Geotechnical Engineering Division, Proceedings of the American Society of Civil Engineers 107, no. GT2.

[13] **Woods, D. M.** (2004), “Geotechnical Modeling,” Abbots Leigh Version 2.2.

[14] **Attewell, P. B., and I. W. Farmer**. (1973). Attenuation of ground vibrations from pile driving. Journal of Ground Engineering 6(4):26–29.

التأثير الديناميكي لدق ركيزة في الرمل على الركائز المجاورة

طالب كامل قاسم
قسم الهندسة المدنية
جامعة المثنى

قاسيون سعد الدين محمد شفيق
قسم الهندسة المدنية
جامعة النهرين

أسماء ثامر أبراهيم
قسم الهندسة المدنية
جامعة النهرين

الخلاصة

ركائز الدق غالبا ما تستخدم في الكثير من البنى التحتية المدنية وذلك لتقديم الدعم والاسناد لاحمال المنشأت الثقيلة. ومع ذلك، فإن الاهتزازات الحتمية الناجمة أثناء عمليات دق الركائز قد تسبب باحداث الضرر للمنشات المجاورة وبدرجات متفاوتة. في هذه البحث تم إجراء دراسة عملية وذلك لبحث انتقال الاهتزازات المتولدة مع دق الركيزة في التربة الرملية الى الركائز المجاورة. حيث تمت الدراسة العملية في المختبر على نماذج مصغرة تتكون من حاوية من الحديد بابعاد (1× 1.5 × 0.8 م) ومطرقة ركائز لتثبيت الركائز في التربة الرملية من خلال اسقاط اوزان مختلفة (1, 2, 3, 4, 5 كغم) سقوطاً حراً وبارتفاعات مختلفة (4, 8, 12, 16, 20 سم). وقد تم تسجيل أعلى سرعة اهتزاز للجسيمات عند سطح الركائز المجاورة بأستخدام جهاز قياس الاهتزازات. استخدم في هذه الدراسة عدة ركائز تبعد بمسافات مختلفة عن مصدر الاهتزاز. أعلى سرعة اهتزاز للجسيمات تزداد مع زيادة طاقة مطرقة دق الركيزة ومع زيادة عمق الاختراق لركيزة الدق لجميع الركائز المجاورة. ويمكن تقليل اعلى سرعة اهتزاز للجسيمات بدون تغيير لطاقة الدق المطلوبة لدق الركيزة من خلال تقليل وزن المطرقة الساقطة وزيادة ارتفاع سقوطها. كما ان شدة الاهتزاز تضعف مع زيادة المسافة عن ركيزة الدق وان النقصان بشدة الاهتزاز تكون منتظمة مع زيادة المسافة للاسس السطحية بين الركئز. من خلال تمثيل النموذج المختبري وتقييم النتائج التي تم الحصول عليها في المختبر, وجد ان العلاقة الوضعية المستنتجة من البيانات التي تم الحصول عليها من التجارب العملية, تكون ملائمة وتعطي نتائج قريبة جداً ويمكن الاعتماد عليها لتمثيل انتقال الاهتزازات الناتجة من دق الركيزة الى الركائز المجاورة.

الكلمات المفتاحية: تنفيذ الركائز، التأثير الديناميكي، الركائز المجاورة، الترب الرملية.