CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

Always buried underground and invisible, but foundations are the most important component for structures and buildings. When piles installed, there might be defects along their shafts because of many reasons, including improperly driving, poor quality concrete (or any materials), and mishandling during transportation. During geotechnical design process the common practice is to assume a pile has sound integrity. As earlier mentioned, however, defects may occur. It is therefore essential to have a methodology for inspecting the integrity of installed pile to ensure that their capability is conform to the design calculation.

This chapter reviews both general information and studies related to pile foundation. The general information is required in order to comprehend many aspects of pile foundation, including materials, construction methods, and geotechnical designs. These then provide readers the information related to the main objective of this research namely pile integrity of which is one of the important practices in the field to ensure installed piles have acceptable integrity.

2.2 Pile foundation

According to their usage, foundations can be classified as shallow and deep foundations. Generally the shallow foundation is simply pad foundations; but, it also includes some special foundations such as combined foundations, buoyant foundations, compensated foundations, and raft foundations. The deep foundation, however, comprises several types, including pile foundations, barrettes, and caissons. Nonetheless, the pile foundation can be made from several materials. Thus, it is
necessary to have more classification systems concerning their materials. In addition, quite often the pile foundation is classified according to machines or installation methods. This section provides brief information regarding the pile foundation and its classification.

Generally, piles are used for the following scenarios (as also shown in figure 2.1, Bowles, 1988)

1. To carry and transmit superstructure loads into lower ground that is capable of sustaining such loads.
2. To resist uplift forces and overturning moments.
3. To compact loose sediments.
4. To reduce the settlements in the case a pad foundation in not capable of.
5. To stiffen the soil beneath the foundations support machines.
6. To add extra factor of safety such as under a bridge abutment.

![Diagram](image)

(a) Group and single pile on rock or very firm soil stratum.
(b) Group or single pile "floating" in soil mass.
(c) Offshore pile group.
(d) Tension pile.
(e) Pile penetrating below a soil layer that swells (shown) or consolidates.

Figure 2.1 Different pile configurations for different purposes (Bowles, 1988)
2.2.1 Pile foundation materials

According to Bowles (1988), piles are structural members made from timber, concrete and steel. They are used to transmit superstructure loads to lower ground that is capable of withstand those load without large deformation and settlement. The details for each pile material are described below.

1) Timber piles

Timber piles can be prepared by simply trimming branches off. Because piles are normally buried underground, they are always treated with some preservatives in order to prolong their use. It should be noted that the timber pile if completely installed underground having virtually constant conditions - either completely wet or dry - the deterioration should not be a problem. The drawbacks are that its sizes and shapes are limited according to the tree species. Thus, employing a timber pile requires some regulations. For example, The Chicago Building Cod states that the tip should have a minimum diameter of 150 mm and the butt of 250 mm if the pile is less than 7.6 m and have a 300 mm butt if the pile is more than 7.6 m long. In addition, The New York Building Code limits the capacity of 220 kN for a timber pile having a uniform shaft taper to a tip diameter of 150 mm. If larger load required, a minimum pile tip diameter of 200 mm should be employed. More details can be found in Bowles (1988). During the installation, cares must be taken not to damage the pile head or other parts. Some protections must be carefully prepared during driving. Quite often the timber pile has to be extended by means of splicing. In such case, normally steel sheet and nuts are employed, as illustrated by figure 2.2.

![Diagram of timber pile splicing methods](image)

**Figure 2.2** Example of splices in timber piles (a) using metal sleeve with ends carefully trimmed for fit and bearing (b) using splice plates (Bowles, 1988)
2) Concrete piles

The concrete pile can be further classified as precast concrete piles and cast-in-place piles. For the former, piles are normally cast in a central factory then shipped to a specified site. For that reason, they can be shaped according to the desire of a designer. It is customary to have reinforcement for many reasons. For example, during transportation there will be times when the piles are moved. This of course will generate stress and bending in the pile thereby a reinforcing material required to withstand such extra forces. In general, the minimum pile reinforcement should not less than 1%. (Bowles, 1988). Figure 2.3 displays typically reinforcing steels for concrete piles.

Nowadays, prestressed concrete piles are common in Thailand because of its high resistance to driving stress and applied loads. Generally, they are formed by tensioning high-strength steel having the ultimate tensile stress, \( f_{\text{ult}} \) of around 1705 to 1860 MPa prestressing cables to some value on the order of about 0.5 to 0.7 of \( f_{\text{ult}} \), and casting the concrete pile about the cable. Figure 2.4 shows typical prestressed piles.

![Diagram of concrete pile with reinforcement](image)

Figure 2.3 Typical arrangement of reinforcing steels for concrete piles (PCA, 1951; Bowles, 1988)
Figure 2.4 Typical arrangement of reinforcing steels for prestressed concrete pile (Bowles, 1988)

\[ M_{\text{max}} = \frac{wL^2}{8} \]

(a)

\[ M_{\text{max}} = \frac{wL^2}{2} \]

(b)

\[ M_{\text{max}} = \frac{wL^2}{18} \]

(c)

In all figures \( w \) is the weight per meter (or foot) of pile

\[ M_{\text{max}} = \frac{wL^2}{32} \]

(d)

\[ M_{\text{max}} = 0.021wL^2 \]

(e)

\[ M_{\text{max}} = 0.207L \]

\[ 0.586L \]

\[ 0.207L \]

Figure 2.5 Recommended picking up locations for precast concrete piles and there corresponding bending moment (Bowles, 1988)

As the precast concrete piles have to be transported to a construction site, there will be times when they are lifted and placed. These steps surely create unwanted stresses within the piles thereby extra reinforcing required. To minimise such reinforcement appropriate pick up points have to be predetermined. These points, however, can be obtained by simply evaluating minimum bending moments that correspond to various pick up points, as can be seen in figure 2.5. In general the pickup points should be placed such that the computed bending stress has \( f = M/S \leq f_{pe} \) where \( M \) is from figure 2.5 (Bowles, 1988).
A cast-in-place pile is constructed by first drilling a hole in the ground and then filling it with concrete. Quite often, reinforcement is also installed. Note that the reinforcement by means of reinforcing steels formed by spiral has been called a cage. Figure 2.6 displays examples of cast-in-place concrete pile derived from different companies and configurations.

3) **Steel piles**

The steel piles are normally derived from rolled H shapes or pipe piles. Note that wide-flange beams or I-section beams are also sometimes employed. It should be noted that, however, the H piles can withstand higher stress during driving thereby are
the most popular. However, if the loads to be carried out are low to moderate, other sections could be employed because they are cheaper. The advantage of the H piles is that they have enough rigidity thereby enabling them to be driven in boulders or very stiff layers. One thing should be carefully considered and designed is splicing them. The most common is to weld shorter piles into longer pile required. Sometimes, bolts are used to connect the short sections into a long one. Figure 2.7 shows examples of steel pile splicing for the H- and pipe- piles. In some cases of design, driving points are fabricated such that piles could be driven to the desire depth without any damage, as shown in figures 2.8 and 2.9.

Figure 2.7 Examples of splices for H and pipe steel piles (Bowles, 1988)

Figure 2.8 Shop or on-site fabricated driving points. Labour costs make this generally uneconomical except for small numbers of points. Note that (c) will damage the perimeter soil so that skin friction is reduced in stiff clays (Bowles, 1988)
Figure 2.9 Examples of points for different types of piles. Points are also available in higher-strength steel for very hard driving. (a) (b) and (c) are points for H piles (d) pipe-pile point (f) sheet pile point (Bowles, 1988)

2.2.2 Construction of pile foundations

The construction methods for piles chiefly depend on pile materials. For instance, the installation of steel piles can only be done by driving or pushing (by a machine). In the case of concrete piles, both driving and drilling are common practice. However, other methods have now been developed because of factors such as working areas and limited overhead height. Details with respect to equipment and construction methods for pile foundation can be found in Tomlinson (1994).

2.3 Bearing capacity of pile foundations

If one has decided to use a pile to transfer loads from superstructure into the ground, he/she must have the information with respect to that ground. Then, he/she must carry out calculation to determine the capacity of both the pile and the ground in terms of withstanding the loads. Overall, to accomplish those processes the first step is to obtain insight information with respect to subsurface conditions, including stratum characteristics, water table level, and basic and engineering properties. After that, the calculation to obtain the bearing capacity can be carried out.
Considering the shape of a pile, its bearing capacity is divided into two parts, skin resistance and point resistance, as schematically depicted in figure 2.10. The static calculation of pile bearing capacity is generally obtained from the following equation

\[ Q_p = Q_b + Q_s - W_p \]  

Eq. 2.1

where \( Q_p \) is ultimate pile bearing capacity, \( Q_b \) is ultimate point resistance, \( Q_s \) is ultimate shaft resistance (or skin friction), and \( W_p \) is the weight of a pile. For the \( W_p \), however, it may be omitted if a pile has low to moderate weight compared with the loads to be carried out. The component of \( Q_b \) and \( Q_s \) can be calculated from the equation 2.2 and 2.3, respectively. It should be noted that equation 2.1 yields the ultimate bearing capacity of a pile, i.e., the maximum load the pile can withstand. However, in order to prevent the pile to be overloaded, a factor of safety is required, as shown in equation 2.4. It is customary to employ the factor of safety of 2.5 for the bearing capacity calculation for a common pile foundation. In abnormal cases such as silos, nuclear power plants, and very large structures, the factor of safety should be carefully selected.

\[ Q_{bk} = q_{bk}A_b \]  

Eq. 2.2

\[ Q_{sk} = \sum_{i=1}^{n} q_{sk_i}A_{si} \]  

Eq. 2.3

\[ \text{Allowable load} = \frac{\text{Ultimate pile resistance}}{2.5} \]  

Eq. 2.4
2.4 Stress wave theory

According to Middendorp (2005) the pile integrity test requires some knowledge with respect to the propagation of stress waves through rod-like members such as piles. The stress wave (or sound wave) is applied into a pile by impacting a hammer blow on the pile head, as illustrated by figure 2.11. This stress wave then travels through the pile at the speed of sound \((c)\) to the pile toe and reflects back to the pile head, as shown in figure 2.12.

![Figure 2.11 Impacting a hammer on the pile head (Middendorp, 2005)](image)

![Figure 2.12 Stress wave travelling through a rod (Middendorp, 2005)](image)
The hammer blow results in the response of the pile head as well as the reflection of the pile toe. An accelerometer is attached to the pile head to measure and record those reflection waves. Then, the acceleration is integrated in order to obtain a velocity versus time signal ($v$). Generally, at least 3 hammer blows are required in order to obtain an averaged signal to ensure the consistency of results. For instance, to proof the reliability of results all three signals should have similar patterns. Otherwise, retesting should be carried out. The time ($T$) between the start of a hammer blow and the time of arrival of the reflection from the pile toe is measured during testing. This results in the equation for calculating the pile length ($L$) as

$$L = \frac{ct}{2}$$

Eq. 2. 5

when the stress wave velocity ($c$) is known. Normally, the wave velocity must be an input parameter before conducting a test. If an exact value is not known, a typical wave speed could be entered.

To present the measuring results the time axis ($t$) is scaled to a depth (length, $l$) axis with the equation of $L = ct/2$. Because of shaft resistance the toe reflection may be of small magnitude. Thus, to be able to observe the toe reflection an amplifier is employed. In addition, to remove noises from the measuring signals a filter value may be applied.

The analysis of the pile integrity test is carried out using the theory concerning one-dimension stress wave. Reflections that are generated by impedance changes (or discontinuities) travel to the pile top and are recorded and analysed. The impedance $Z$ is defined as

$$Z = A\sqrt{E\rho}$$

Eq. 2. 6

where $A$ is cross sectional area, $E$ is elastic modulus, and $\rho$ is mass density. Thus, from the equation 2.6, any change in $A$, $E$, and $\rho$ or the combination of them will theoretically generate a reflection from an impedance change (discontinuity). And, the potential causes of the change include:

- Pile toe
- Change in dimension (pile cross sectional area)
- Soil inclusions (especially in the case of cast-in-place piles)
• Cracks
• Joints
• Variations in concrete quality (normally cast-in-place pile)
• Variations in soil strata
• Overlap of reinforcement

As this technique employs the stress wave, there are some limitations that need to be carefully considered such as

• Minor impedance changes may not be detected.
• Gradually increasing and decreasing pile diameter (or impedance) cannot be detected.
• Curved pile shapes cannot be detected.
• Small amount of soil inclusions are not properly detected.
• Local loss of reinforcement cover cannot be detected.
• Thickness of debris layer at pile toe cannot be detected.

2.5 Pile integrity testing methods

Recently, there are two types of pile testing with respect to none-destructive testing methods: high strain- and low strain- tests. Like the names imply, the high strain test requires a pile being undergone the high strain level. While the latter the strain generated during testing is very small, i.e., as low as 1e-6. It should be noted that the low strain testing method comprises several techniques. This report, however, presents the one regarded as surface reflection technique, including the pulse echo method and the transient dynamic response method.

2.5.1 Pulse echo method (PEM)

The pile integrity test by means of the pulse echo method (PEM) is a surface reflection integrity testing technique. It is sometimes called as sonic echo method. However, the sonic echo method seems to have a more popularity in terms of testing name. To conduct testing, a high frequency accelerometer is attached to the pile top by using petroleum gel, wax, or just oily clay. A nylon hammer having around 1 to 3 pounds of weight is employed to strike at the pile top in order to generate small stress waves. The strains generated by the propagating stress waves have a level of around 1με. A typical
test set-up for the PEM test is illustrated by figure 2.7. A plastic material is normally employed as hammer head to reduce the extraneous high frequencies generated by steel. The accelerometer attached to the pile top measures the acceleration at the impact as well as the reflections arriving to the surface of the pile top. The signals are monitored and recorded. Then, they are digitised and integrated using a computer program to obtain a velocity versus time record (Paikowsky and Chemauskas, 2003).

Note that the velocity signal indicates the speed at which the pile material at the point of measurement travels because of the hammer impact as well as the reflected waves generated by the hammer. Figure 2.8 displays a typical velocity versus time signal through filtering. This signal, then, can be further processed using a program that enhances the signal by means of filtering, shifting, pivoting, and magnifying. These processes provide some enhancement of the velocity signal having weak toe reflections thereby reducing the effects of unwanted noises and drifts. Thus, they help in aiding in the interpretation of the signals of pile response.

Eq. 2.7 Typical arrangement for PEM method (Paikowsky and Chemauskas, 2003)
Eq. 2.8 Typical signals obtained from PEM method (a) velocity vs time (b) force vs time
(Paikowsky and Chekauskas, 2003)

The reflected signals are influenced by many factors, including the changes in pile cross-sectional area, concrete quality and density, soil resistance along the pile shaft. These then also alter the pile impedance in the direction of the traveling waves thereby creating the stress wave that propagates backwards the pile top. It should be noted that these reflected stress waves can return in either compression or tension, depending on the type of impedance change. In addition, the pile properties that define impedance $Z$, are the speed at which the stress wave propagates $C$, the elastic modulus $E$, and the cross sectional area $A$, as expressed in the following equation

$$Z = \frac{EA}{C}$$

Eq. 2.9

Figure 2.10 displays the relationship between various changes of pile impedances, travelling wave, and the reflection waves. The reflected tension wave indicates a decrease in impedance. On the contrary, a reflected compression wave indicates an increase in impedance. Combinations of these impedance changes then can create complex reflections at the pile top. One can inspect these changes as well as velocity signals in order to obtain predicted pile integrity. Note that at a time 2L/C, where L is the pile length, the pile toe response can be identified by observing a
reflected tension wave due to softer soil at the pile tip, or a reflected compression wave due to denser soil at the pile tip (Paikowsky and Chemauskas, 2003). Because the PEM is relatively simple and quick, it can often be performed on all types of piles at a site. It should be noted that, however, the technique is generally effective for the cases of the pile length is about 20 to 30 times of pile diameter.

![Diagram of wave travelling, propagation, and reflection vs time and depth](Paikowsky and Chemauskas, 2003)

**2.5.2 Transient dynamic response (TDR) method**

According to Paikowsky and Chemauskas (2003), the transient dynamic response (TDR) method (also known as impulse response method) is also based on the PEM, but an instrumented hammer is used to generate the impact pulse. An accelerometer attached in a hammer, or a force transducer built in an impulse hammer allows one to determine the impact force (due to hammer mass) in addition to the velocity signals normally obtained by the PEM. And, because a force transducer is not attached to the pile, only the impact force is recorded. Then, the force and velocity records can be converted from time domain to the frequency domain plot using a Fast Fourier Transform (FFT). The ratio between the velocity spectrum and the force spectrum provides the mobility spectrum (V/F in the frequency domain, as presented in figure 2.11).
The TDR method provides additional insight compared to the PEM method interpretation technique. For example, certain frequencies could be identified and correlated to pile length and distance to variations either in the pile impedance or in the soil. Furthermore, the low frequency components of around less than 100 Hz can provide an indication of the dynamic pile stiffness.

Eq. 2.11 A mobility spectrum (V/F vs frequency) using records obtained by the TDR machine (Paikowsky and Chemauskas, 2003)