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Everybody with experience in reinforced concrete construction has encountered columns that, upon dismantling of the forms, exhibit air voids and honeycombing. Although these columns may have been cast with good-quality concrete, in properly assembled forms and with careful vibration, they still exhibit defects. Cast-in-situ piles are also columns, but instead of forms made of wood or metal we have a hole in the ground. This hole may pass through layers of dumped fill, loose sand, organic matter, and ground water, which may be fast flowing or corrosive. Obviously, such conditions are not conducive to a high-quality end product. The fact that on most sites we still manage to get excellent piles is only a tribute to a dedicated team that makes this feat possible: The geotechnical engineer, the structural engineer, the quantity surveyor, the contractor, the site supervisor and the quality control laboratory. This is obviously a chain, the strength of which is determined by the weakest link. This booklet is devoted to the last link: quality control and assurance or in short QC/QA.

A flaw is any deviation from the planned shape and/or material of the pile. A comprehensive list of events, each of which can lead to the formation a flaw in a pile (Either cast-in-situ or driven) is presented by Fleming et al (1992): The use of concrete that is too dry, water penetration into the borehole, collapse in soft strata, falling of boring spoils from the surface, tightly-spaced rebars etc. In small diameter piles, dry chunks of concrete may also get jammed in the reinforcement cage, creating large air-filled voids (Figure 1). This is an interesting illustration of a flaw in a pile that was produced under ideal conditions: It was bored in hard clay above ground water level, and then cast in the dry. If this could happen here, it could happen just anywhere and according to Murphy’s Law, it certainly would.
The use of slurry in pile construction stabilizes the surrounding soil, but adds another risk factor to the pile integrity (Figure 2). Diaphragm wall elements (barrettes), invariably constructed under slurry, may also have flaws (Figure 3).

Flaws may also develop after the piles have been cast. The use of explosives for nearby excavation and careless trimming of the pile heads (Figure 4) are two common causes. Therefore, we have to face the fact that on any given site some piles may exhibit flaws. Of course, not all flaws are detrimental to the performance of the pile. Only a flaw that, because of either size or location, may detract from the pile’s load carrying capacity or durability is defined as a defect. The geotechnical engineer and the structural engineer are jointly responsible to decide which flaw comprises a defect.

Figure 1: An obviously defective pile
Figure 2: A bored pile cast with bentonite

Figure 3: Diaphragm wall constructed with polymer slurry
The existence (Or non-existence) of flaws in piles often triggers a highly emotional debate. Nevertheless, it is of utmost importance to handle this matter rationally and, at the first stage, to gather maximum information as to the location, size and severity of all flaws. This can be achieved with an integrity-testing program to be specified together with the specification for pile construction (ICE 1996). With this information, we can then arrive at a rational decision. This can be one of the following four options:

- Disregard the flaw and accept the pile as-is
- Accept the pile as a partial support and strengthen the superstructure
- Repair the defective pile
- Reject the defective pile altogether, and adopt proper remedial measures.

To be effective, the integrity testing program should take place as soon as this becomes practical. For stress-wave methods, this means at a typical age of one week.
after casting. Piles can be tested sooner under favorable conditions, but sometimes we have to wait longer. One instance is when the concrete contains a larger-than-usual amount of retarding admixture. The result should be reported as soon as the testing agency has had ample opportunity to analyze and evaluate the results. To avoid unpleasant misunderstandings, the specifications of the project should state, in no uncertain manner, that the integrity test results should serve as an exclusive acceptance criterion.

The cost of a quality control program for a given site is very reasonable, and in any case much lower than the potential loss caused by an undetected defect. The choice of a testing firm should not be made on the lowest bidder basis, but shall take into account the qualifications and record of the firm. In any case, the technical director of the firm should be an experienced geotechnical engineer, well versed in the geology of soils, piling techniques and wave propagation theory.
Of Bats, Mice and Waves

The final products of any piling operation (And in many cases the operation itself) are practically invisible. You might even say that they are shrouded in complete darkness. Under such conditions, judging the integrity of a finished pile may seem like a hopeless task.

Not everyone, however, feels hopeless in the dark: Bats, for instance, are quite comfortable! Their secret, as every child knows, is the clever utilization of sound waves. Submarine sonar operators applied this technique decades ago, and piling people (being naturally conservative) followed much later. The two techniques currently dominating pile integrity testing, namely the sonic method and cross-hole ultrasonic method, both utilize sound waves. Such waves travel not only in air and water, but also in solids, where they are also called stress waves or elastic waves. We shall therefore begin with a short introduction to the propagation of stress waves in solids. Later on, when dealing with each of the testing methods, we shall review those principles of wave propagation theory applicable to each specific method.

The last part provides an overview of other testing methods, not based on stress waves.

In this work, we shall use the term “pile” in the general sense. Thus, it will include all deep foundations, whether driven, bored or augered. Specifically, the foundation types called “caissons” “piers” and “drilled shafts” do belong to this category as well. Piles may consist of concrete (cast in situ or precast), steel or wood.

When part of a solid body in a state of equilibrium is dynamically loaded at some point, the effect of this load does not reach immediately all parts of the body. Due to inertial effects, the stress and deformation caused by the load radiate from the point
of application at a finite speed towards all directions. Any given point in the body will be at complete rest both before and after the effect of loading has passed it. This phenomenon, illustrated in Figure 5, is defined as stress-wave propagation.

![Figure 5: Propagation of waves caused by a hammer impact on the upper left-hand corner](image)

Waves are created not only by transient loading, but also by steady periodic loads. Harmonic loading, where the load amplitude is a sinusoidal function of time, is a good example. Since transient loads of any shape may be resolved by Fourier Transform
into a series of elementary harmonic waves, the theoretical treatment will by the same for both.

The basic equation governing the relationship between wave propagation speed \( c \), wavelength \( \lambda \) and frequency \( f \) is:

\[
c = \lambda f
\]

Equation 1

To get a better glimpse into the practical significance of wavelength, let us consider the following example: For mice, a space the size of a typical pile may seem like a palace in which they can roam freely in all directions. A man, on the other hand, will feel the space rather crowded, and at best will be able to move either down or up the pile (Figure 6). The conclusion is that while a pile may seem one-dimensional to a man, a mouse will rightly think that the same pile is three-dimensional. Similarly, the theoretical treatment of waves in solid media depends on the prevalent wavelength: When the wavelength is in the order of pile diameter/width or larger, the problem becomes one-dimensional. This is typical of the stress-waves created during pile driving and by testing methods in which the pile top is hit with a hand-held hammer. When the typical wavelength is small in relation to the geometrical dimensions of the pile, the problem becomes three-dimensional and the classical theory of wave propagation in solid bodies applies. Since piles are relatively slender bodies, the waves have to be rather short, that is of ultrasonic frequencies.
Figure 6: Long vs. short waves in pile testing
The sonic method

Basic Principles

The sonic method for the integrity testing of piles (ASTM 2007) is aimed at routinely testing complete piling sites. To perform this test, a sensor (usually accelerometer) is pressed against the top of the pile while the pile is hit with a small hand-held hammer. Output from the sensor is analyzed and displayed by a suitable computerized instrument, the results providing meaningful information regarding both length and integrity of the pile. The sonic test is fast and inexpensive, with typically less than one minute needed to test a given pile (Amir & Amir 2008).

To have a better understanding of the inner workings of the method, let us first go through the basics of one-dimensional wave propagation in prismatic rods. We shall then discuss the specific instrumentation, go through the interpretation and end this chapter by discussing the capabilities and limitations of the method.

Waves in prismatic rods

The wave equation

Since piles do basically resemble prismatic rods, the analysis of waves in prismatic rods is evidently of much importance to those involved in driving piles and in testing them. The mathematics involved are fairly simple, provided we first make a few reasonable assumptions:

- The wavelength is equal to or larger than the pile diameter.
- The rod is prismatic with a constant cross-section A, elastic with Young’s Modulus E and homogeneous with mass density ρ.
- Cross sections remain plane, parallel and uniformly stressed.
• Lateral inertia effects are negligible.

Let us now examine an element $\Delta x$ along the rod (Figure 7). If we denote the stress above the element by $\sigma$ and below the element by $\sigma+(\partial \sigma/\partial x) \Delta x$, the unbalanced force on the element is $A.(\partial \sigma/\partial x) \Delta x$. If we denote particle displacement by $u$, the particle velocity is given by $\partial u/\partial t$, the particle acceleration by $\partial^2 u/\partial t^2$ and the strain $\epsilon$ by $\partial u/\partial x$. According to Newton’s second law (Force equals mass times acceleration),

$$A \cdot (\partial \sigma/\partial x) \Delta x = A \cdot \Delta x \cdot \rho \cdot \partial^2 u/\partial t^2 \quad \text{Equation 2}$$

Because of elasticity, $\sigma = E \cdot \partial u/\partial x$ (Hooke’s Law). Substituting in and eliminating both $A$ and $\Delta x$ on both sides, we get:

$$E(\partial^2 u / \partial x^2) = \rho(\partial^2 u / \partial t^2) \quad \text{Equation 3}$$

or:
\[
\frac{\partial^2 u}{\partial t^2} = c^2 \cdot \frac{\partial^2 u}{\partial x^2} \quad \text{Equation 4}
\]

where:

\[
c^2 = \frac{E}{\rho} \quad \text{Equation 5}
\]

This is the well-known wave equation in one dimension. The general solution of this differential equation, due to D’Alembert, is the sum of two functions having the following form:

\[
u = f(x-ct) + g(x + ct) \quad \text{Equation 6}
\]

We can easily verify the validity of this solution by substitution. To understand the physical significance of this solution, let us consider the first term in Equation 6, assuming the argument \(x-ct\) is constant. In such a case, the function \(f(x-ct)\) is also constant. If we wish to keep it constant, we have to increase \(x\) by \(c \Delta t\) when \(t\) is increased by \(\Delta t\). This means that \(f\) describes a wave with a constant form, moving in the positive \(x\) direction at a constant speed \(c\). Likewise, \(g\) describes a wave moving at the same speed, but in the opposite direction. Both \(f\) and \(g\) are determined by the initial conditions.
From Equation 6, the particle velocity \( v \) is equal to \( f' (x - ct) + g' (x + ct) \). For the sake of simplicity, we shall in the following consider only the outgoing wave, thus \( v = -f.c \). We may further omit the minus sign if we assume the compressive wave to be positive.

The force acting on a given cross-section is:

\[
P = \sigma.A = \varepsilon.E.A = (EA/c)v = Z.v \quad \text{Equation 7}
\]

This means that the force acting on any section in the rod at a given moment is proportional to the particle velocity. \( Z \) is defined as the impedance of the section with typical dimensions of kg/sec. Other expressions for the impedance are: \( Z = A \cdot (E\rho)^{1/2} \) or \( Z = \rho.c.A \).

As we mentioned before, this relation is true only for the outgoing waves, while for returning waves, \( P = -Z.v \).

A word of caution: particle velocity \( v \) and wave speed \( c \) are two distinct entities:

Particle velocity \( v \) is a function of the initial conditions, such as the blow intensity. The particle velocity caused by tapping with a handheld hammer will be much lower than that caused by a large piledriving hammer. The wave speed \( c \), on the other hand, depends only on the material constants \( E \) and \( \rho \) of the pile, and will therefore be identical in both cases.

**Reflection from the end**

When waves move in a finite rod, they will eventually reach the end \( x = L \). The wave will be reflected from the end, the nature of the reflection depending on the boundary conditions at the end.
When the end is fixed, \( u (L, t) = 0 \). In this case, the wave will be reflected unchanged, by which we mean that a compressive wave will be reflected as compressive, and vice versa.

We can prove this result without any recourse to mathematics. Although infinite rods are rather uncommon in practice, they are quite useful in theory: Let us then try and visualize an infinite rod in which two compressive waves of equal shape are approaching the point \( x = L \) from both sides (Figure 8). When the two waves meet, the displacements associated with them will cancel each other, so that the point \( x = L \) will stay stationary. This obviously satisfies the boundary conditions.

The stresses at this point, however, will add up and reach twice the normal value. An observer on either side will conclude that a compressive wave will be reflected unchanged from the fixed end. The same reasoning, by the way, also holds for tensile waves.

![Figure 8: Two opposite compressive waves meeting at \( x = L \)](image)
If one of the meeting waves is compressive and the other one tensile (Figure 9) the stresses will mutually cancel at $x = L$. This conforms to the boundary condition at a free end $\sigma(L,t) = 0$. Since the particle velocities associated with both waves are directed to the right, they will add up at $x = L$, doubling the normal particle velocity.

The tensile wave in Figure 9 will continue moving to the left, while the compressive wave will travel to the right. Our faithful observer will reach the inevitable conclusion that a wave reflected from a free end will change sign: From compressive to tensile, and vice versa.

If we take a prismatic rod with a given length $L$, and apply a dynamic load to the end $x=0$, the wave created will travel along the rod and return, the duration of the whole trip $T = 2L/c$.

**Discontinuities in rods**

A discontinuity in a rod is defined as an abrupt change in either cross section (From $A_1$ to $A_2$) or material properties $E$ and $\rho$. When a wave traveling in a rod meets such a discontinuity, one part of it will be reflected back while another part will go on beyond the discontinuity (Figure 10). Let us represent the incident wave parameters by the index $i$, while $r$ and $t$ will denote the reflected and transmitted waves, respectively.
From equilibrium:

\[ A_i (\sigma_i + \sigma_r) = A_2 \sigma_i \]  \hspace{1cm} \text{Equation 8}

Moreover, from continuity:

\[ v_i - v_r = v_i \]  \hspace{1cm} \text{Equation 9}

Since \( v = \pm \sigma A / Z \), Equation 9 may be re-written as:

\[ \left( \frac{A_1}{Z_1} \right)(\sigma_i - \sigma_r) = \left( \frac{A_2}{Z_2} \right) \sigma_i \]  \hspace{1cm} \text{Equation 10}

From Equations 10 and 8 we get the following relationships:
\[ \sigma_r = \frac{Z_1 - Z_2}{Z_1 + Z_2} \sigma_i \]  
Equation 11

and:

\[ \sigma_1 = \frac{2Z_2}{Z_1 + Z_2} \frac{A_1}{A_2} \sigma_i \]  
Equation 12

These two equations enable us to calculate the behavior of a wave as it moves along a rod of an arbitrary shape. A convenient way to visualize the process is by using the method of characteristics, due to Riemann, which represents the wave propagation in the x-t plane.

As we have already seen, the solution of the wave equation is in the form of \( f(x - ct) \) and \( g(x + ct) \). If the functions \( f \) and \( g \) are constant, these solutions describe two sets of straight lines in the x-t plane with a slope of \( c \) (downward) and \( -c \) (upwards), respectively. These lines are called characteristics.

Figure 11 shows the characteristics for a rod with a reduced cross-section. The figures besides the lines are the respective stresses, calculated from Equations 11 and 12.

Using the method of characteristics we draw a graph showing the velocity at the top versus time. Such a graph is called a reflectogram.
Figure 11: Characteristics for a rod with a reduced cross-section ($A_2 = A_1/2, A_3 = A_1$)

(After Vyncke & van Nieuwenburg 1987)

Damping

All former analyses of wave phenomena in prismatic rods were based on the assumption of zero skin friction. When the rod is embedded in some solid material, however, the situation is different: Rod particle displacement, which is associated with the wave, will give rise to skin friction forces in the opposite direction. To visualize the effect of these forces, let us consider the case of a compressive wave traveling downwards in a rod (Figure 12):
The equilibrium condition means that:

\[ P_1 - P_2 - F = 0 \]  \hspace{1cm} \text{Equation 13}

Because of continuity, \( v_1 = v_2 \). If there is no change in the impedance of the rod, \( Z_1 = Z_2 \) and therefore \( P_1 = -P_2 \). If we substitute this in the equilibrium equation 13, we get:

\[ P_1 = -P_2 = \frac{F}{2} \]  \hspace{1cm} \text{Equation 14}

This means that the friction force \( F \) gives rise to a pair of waves, each equal in magnitude to \( F/2 \): A reflected compressive wave and a transmitted tensile wave.

The reflected wave is of the same type of the incident wave: A compressive wave will cause a compressive reflection, and vice versa. Thus, the reflection due to skin friction is similar to an increase in the impedance.
The transmitted component $P_2$ is superposed on the incident wave. Since $P_2$ is of opposite sign, the net result is a weakening, or damping, of the wave. The total energy in the rod is decreased, the difference being radiated to the surrounding medium.

Let us assume that the surrounding medium acts as a linear viscous material, that is:

$$F = \eta C \nu$$  \hspace{1cm} \text{Equation 15}$$

where $F$ is the friction developed per unit length, $C$ is the perimeter of the rod and $\nu$ the particle velocity. The incident wave is associated with an internal force $P = Z \nu$, reduced by $\Delta P$ due to friction. The ratio between the transmitted and the incident force is:

$$R = 1 - \Delta P/P = 1 - F/2Z\nu = 1 - \eta C \nu/2A (E \rho)^{1/2} \nu = 1 - [(C \eta / 2A) (E \rho)^{1/2}] \quad \text{Equation 16}$$

For a rod with either a circular or a square cross section, $C/A = 4/D$ and then $R$ becomes

$$1 - [2C \eta/D (E \rho)^{1/2}].$$ If the length of the rod is $L$, a dynamic force $P_0$ applied to one end will arrive at the other end as:

$$P_L = P_0 R^L \equiv P_0 e^k \quad \text{Equation 17}$$

Where $k = (L/D) [2 \eta/(E \rho)^{1/2}]$. Clearly, a small increase in the $L/D$ ratio will lead to a sharp decrease in the force (or velocity) reaching the end. The important contribution of $L/D$ ratio to damping is therefore apparent.
For a linear visco-elastic medium surrounding the rod, the friction is given by \( \eta (\partial u / \partial t) + ku \). For harmonic waves, Novak et al. (1978) suggested the following parameters:

\[ \eta = \pi D \sqrt{G \rho_s} \text{ and } k = 2G, \]
where \( G \) and \( \rho_s \) are the shear modulus and the density of the surrounding medium, respectively.

With known damping parameters, we can correct the characteristics (and the resulting reflectogram) accordingly.

**End rigidity**

So far, we have dealt with two extreme boundary conditions: A free end and a fixed end. In reality, however, we often encounter ends with a finite rigidity. For the sake of simplicity, let us assume that the medium beyond the end exhibits a linearly viscous behavior. In such a case, the stress \( \sigma \) developed in the medium is proportional to the particle velocity \( v \):

\[ \sigma = \eta . v \quad \text{Equation 18} \]

If the area of the end is \( A \), the force developed is \( P = A \eta v \). The impedance of the end is equal to the ratio between the force and velocity, that is:

\[ Z_e = P / v = A \eta \quad \text{Equation 19} \]

The impedance of the rod directly near the end is \( Z_r = \rho c A \). Thus, the impedance ratio is:

\[ Z_e / Z_r = \eta / \rho c \quad \text{Equation 20} \]
If $\eta >> \rho c$ the end will behave as a fixed end and if $\eta << \rho c$ the end will respond as a free end.

In the special case when $\eta = \rho c$ there will be no reflection from the end.

Vyncke & van Nieuwenburg (1987) studied the case of an end with an elastic behavior. They concluded that the reaction of the spring is frequency-dependent: For high frequencies the end is practically free, while for low frequencies it is fixed. A mixed transient pulse will thus be distorted upon reflection.

**Conical rods**

In many cases we may encounter rods where the cross section increases (or decreases) gradually. Such a situation occurs in conical piles, as well as in the lower part of underreamed (belled) piles. The simplest idealization of such a situation is the straight conical rod (Figure 13) in which the cross-sectional area of such a cone is given by:

$$A(x) = A_0 \frac{x^2}{a^2}$$  \hspace{1cm} Equation 21

this problem was solved for a small cone angle $\Omega$, in which the stresses on any spherical surface ($r$ constant) are uniform and parallel. The analysis is similar to that for a straight rod, however the relation between stress and velocity is not linear as it was for the straight rod. In fact, the impedance $Z = P/v$ is now a complex function of, among other things, the coordinate $x$ and the wavelength $\lambda$. 
When a pulse is moving down a prismatic rod that widens into a cone (underream), we can expect some form of reflection, indicating an increase in impedance. The nature of the reflection depends on both the cone angle and the wavelength: A small angle may create no reflection, while a large angle will. Furthermore, small wavelengths will pass unhindered, while longer waves will be almost fully reflected. The reflected pulse may miss some of its higher frequencies, and may thus look quite different from the incident wave.

**Sawtooth-profiled rods**

Certain piling techniques produce a pile that has a sawtooth profile (Figure 14). We may regard this as a special case of the straight rod with a series of enlargements and constrictions, together creating a multitude of reflections and refractions. Unlike the case of the straight rod, the reflectogram here will show almost no reflection at $t_1=2L/c$. Instead, it will show a strong reflection after a longer time $t_2>t_1$. The reason for this is as follows: For a typical situation, the wave that travels straight thorough
the obstacle course will look like a recruit after an obstacle course: very weak indeed.
The stronger reflection we see at $t_2$ is, in fact, a combination of different waves that have already been reflected and transmitted several times on the way. Clearly, this takes some more time, so that naturally $t_2 > t_1$.

![Figure 14: Prismatic rod with a sawtooth profile](image)

The sawtooth rod, like its conical counterpart, acts like an acoustic filter, letting high frequencies pass while stopping low frequencies (Vyncke & van Nieuwenburg 1987).

The PILEWAVE program

The PILEWAVE program (http://www.piletest.com/Show.asp?page=PileWave), simulates wave propagation in rods of different shapes. The pile shape library included in the program consists of many typical shapes, from a straight cylinder to a sawtooth profile. In addition, we can use the mouse (An unavoidable creature when dealing with waves) to draw any rod profile we like. The program then plots the characteristics,
enabling us to follow the waves as they are reflected and transmitted. As a bonus, we also obtain the resulting reflectogram.

Additional features of the program are the option to change both skin friction and amplification. When we have high skin friction, the toe reflection may become very weak. Under such conditions, we must apply exponential amplification to compensate for the damping and obtain a clear, well-balanced reflectogram.

**Wave speed in concrete**

Most piles are probably manufactured of concrete, either cast-in-situ or precast. Therefore, the velocity of wave propagation in concrete should be of much interest to us.

As we have already demonstrated, the wave speed in a rod is given by \( c = \sqrt{\frac{E}{\rho}} \)

The mass density \( \rho \) is determined at the moment of concreting, and thence does not change. Young’s Modulus, on the other hand, tends to increase as the concrete hardens, albeit at a decreasing rate. As a result, \( c \) increases with the age of the concrete.

Figure 15 gives a relationship between \( c \) and the age of the concrete for three common concrete grades (Amir 1988). We can see that from the age of one week onwards, \( c \) lies between the rather narrow limits of 3,600 and 4,400 m/sec, or 4,000 m/sec ± 10%. This means that whenever we have no information regarding the concrete, assuming a speed of 4,000 m/sec is an excellent first guess.

The general relation between concrete compressive strength and wave speed is given by (Amir 1988):
\[ c = Kf_c^{1/6} \]  

Equation 22

Figure 15: Wave speed in a concrete rod as a function of grade and age (From Amir 1988)

When testing piles on a large site, we have to remember that different piles have different ages. Moreover, in many cases the concrete used for piles contains an admixture for retarding its hardening. Small differences in the content of retarder may influence the rate of hardening. As a consequence, we find ourselves in a situation where each pile has its specific wave speed. Statistical analysis of a piling site (Klingmuller 1992) showed that actual wave speed varied between 2,500 and 6,000 meters per second. While the lower limit may be explained by the age of the concrete, only poor data can explain the higher values. Since the real speed for every pile is usually unknown, we have to compromise by assuming a site-average velocity, and accepting an error of ±10% in the reported lengths.
Instrumentation

Steinbach & Vey (1975), who used a makeshift system consisting of an oscilloscope and amplifiers, were probably the first to make use of the sonic method. Although their results look pretty crude by modern standards, they were still able to get a rather convincing reflection from the toe.

The first-generation of commercial pile-testing equipment, which soon followed, still consisted of purely analog components, based on an oscilloscope and a Polaroid camera. A typical result from such a system is shown in Figure 16.

![Figure 16: 3 reflectograms of a 12 m long pile (ca. 1980) on the oscilloscope screen (note second reflections)](image)

The early eighties saw the transition to the second-generation: Digital systems based on purpose-built computers and some proprietary operating system. Third-generation equipment appeared a few years later, once ruggedized laptop computers became commercially available, and used some version of DOS. Today, practically all
testing systems in use are computerized, belonging to either second or third generation.

Fourth-generation testing equipment, which recently became available, makes optimum use of the accelerated progress in both computing power and software capabilities, with a heavy accent on the software aspect. It runs under the world’s most popular operating system - MS-WINDOWS and Android - with all the resulting advantages.

With modern systems, the sonic test can provide us with more information than just the actual pile length: Changes in cross-section, the existence of inclusions, cracks and other discontinuities, poor concrete quality, amount of fixation of the toe, etc.

A modern system for sonic testing of piles (Figure 17) consists of the following components:

A suitable wave generator. In spite of the impressive name, what it means in fact is a good-quality nylon hammer with replaceable tips (They quickly wear out when hitting rough concrete).
A transducer that is pressed against the top of the pile and is sensitive to motion. The transducer serves both to trigger the system when it senses the hammer blow, and subsequently to receive all the reflections. This item should be very sensitive and at the same time very rugged. Commercial accelerometers are particularly suited to this task.

An analog-to-digital (A/D) converter that turns the continuous analog signal from the accelerometer into a discrete series of numbers a computer can analyze. The A/D converter should have at least a 12-bit dynamic range, so it can handle signals as small as 1:4000 of the maximum.

Modern systems combine the accelerometer, the A/D converter and a microcontroller to form a digital sensor. The signals from this sensor are transmitted to the computer by either a USB controller through cable, or totally wireless using the Bluetooth protocol. Digital sensors have a distinct advantage in being practically immune to external noise.

A portable computer. There is a vast selection of notebook computers one can take to a site, but only a handful that will survive a large number of such trips. These “hard-hat” computers should be immune to dust and occasional spray, and have screens that will not fade under direct sunlight.

Dedicated software that handles the input and displays the results in a form that is easy to interpret. The usual presentation of the results is of pile-head velocity vs. time, or time-domain presentation. As a rule, the time axis is multiplied by $c/2$ and thus transformed to length base. This form is also called a reflectogram.

An optional component of the sonic equipment is the instrumented hammer, included in certain systems. The instrumented hammer contains an internal force
transducer, which measures the force created when the pile head is hit. The time-history of the force is then stored in the computer for further analysis.

Procedure

Preparation for testing

The minimum age for testing is often quoted as five days after casting. There are, however, exceptions both ways to this rule: In soft soils, and when the top is smooth, meaningful testing can be performed even on the next day. On the other hand, when the soil is hard and the concrete includes an excessive amount of retarder, testing should be postponed accordingly.

The hammer blow, which forms the input for the test, should be sharp and uninterrupted. This requires first of all free access to the top of the pile. If, for any reason, the pile was not cast to ground level the contractor should excavate a suitable pit around it to allow accessibility. The spiral reinforcement above the concrete should be removed, as should any soil heaped on the pile.

The second condition is that the exposed concrete should be of full strength, clean and dry. In most cases, this is not the situation: For piles cast under slurry, the top will usually contain a low-strength mixture of concrete and slurry. In augered (CFA) pile the insertion of the reinforcement will cause segregation and bleeding reaching the top. Even when the piles are cast in the dry the surface of the concrete is exposed to direct sunlight without proper curing, causing the concrete to dry up. For this reason, the upper part of the concrete should be trimmed off until good quality concrete is reached. The exact amount which is to be removed cannot be determined in advance, and in some cases may exceed one meter. Once this operation is complete, all loose chunks of concrete should be removed. The pneumatic jackhammer is ideally suited
for this purpose: It is powerful enough to be cost effective, but not too much as to damage the pile. In addition, compressed air is useful for flushing the surface after the breaking stage and removing all the debris.

Under difficult testing condition (High friction, high L/D values) preparing a small testing surface with a disc grinder may be very beneficial.

Testing

Testing is carried out by pressing the transducer to the pile head and hitting the concrete with a plastic hammer. For good coupling, a small amount of suitable putty (such as HBM AK-22) should be spread on the bottom of the transducer. The transducer records both the hammer blow and the reflected waves and the results are further processed by the computer. The output for all piles shall clearly identify the project, pile number, date, time, depth scale and wave speed on which the measurements are based, as well as the results of at least three similar hammer blows.

Signal treatment

The analog output from the accelerometer is first turned into a digital signal by a suitable A/D converter. This component is located inside the digital transducer or, if an analog transducer is used, in a special circuit which is attached to the computer.

To turn the raw data into an acceptable reflectogram, the software must go through the following operations, some of which are performed automatically, while others are done interactively by the operator:

Integration: The input must be converted from acceleration to velocity

Filtering: To eliminate high-frequency noise and obtain a smooth curve
Rotation: The curve is turned so that the area enclosed between it and the horizontal axis is minimized.

Amplification: Because of friction damping, the stress wave is weakened as it progresses. To obtain a legible reflectogram, it has to be compensated for damping, usually exponentially. Certain systems also allow for linear amplification, which enhances the upper part of the pile. In a well-balanced reflectogram, the reflection from the toe has the same amplitude as the hammer blow on the top.

Normalization: No two blows are equal in intensity, so that the resulting velocities differ too. In order to assist understanding of the results, the vertical scale is adjusted so that the maxima and minima of the reflectogram occupy most of the available vertical space.

Averaging: A typical reflectogram will include a consistent component (signal) and a random component (noise). As a result, no two blows will yield the same reflectogram. To enhance the resolution of the system, it should include an option for the averaging of successive signals.

Presentation

We already mentioned that the test-results are displayed as a pile head velocity vs. depth. The European practice is to present the initial hammer blow downwards, while Americans prefer to show it upwards (Figure 18). There is some physical justification to the European approach: Since velocity is a vector, it should be plotted in the right sense. The hammer blow imparts the head a downward velocity, so that the reflectogram should preferably be plotted accordingly. Nevertheless, this is mainly a question of convenience.
The second question is where to fix the zero point, from which we start measuring the elapsed time. A good hammer blow lasts anything between one and two milliseconds, so it is quite important where we place our cursor (Figure 19). This choice must be consistent, and applied to all reflections. Certain systems use the point where pile head starts to move (point A), but since the maximum point of the initial blow (point B) has a smaller radius of curvature, it is better defined.
Interpretation

Having gone through all this, we hopefully have a clear reflectogram, and wonder what it can teach us about the pile. In fact, this is the inverse problem of taking a given pile shape in a given soil profile and drawing the characteristics and the reflectogram. Like all inverse problems, it has no unique solution and we have to look for additional information: The piling method, the soil profile and the supervisors’ field notes. While actual testing can be learned fairly quickly by qualified personnel, interpretation should be left to geotechnical engineers with thorough knowledge of wave propagation theory, soil mechanics and piling techniques (ICE 1988). Of course, experts may make a few learned mistakes from time to time, but the ignorant make a lot of stupid mistakes all the time. Expert interpretation is hence the key to successful sonic testing.

Qualitative interpretation

The first step in analyzing the reflectogram is qualitative, and is performed immediately following each test. This is done by mentally comparing the graph to a
catalogue of various pile shapes and their respective reflectograms (Rausche et. al 1988). Some typical cases are presented in Table 1.

If our graph falls into one of these categories (except for the last one, of course), it means that we can explain the significance of what we have in hand. If the opposite is true, we have to try another spot on the pile. Although theoretically the exact location we hit is of no significance (Fukuhara et al. 1992), in practice different spots react differently. If all our attempts fail, either the pile top was not sufficiently prepared for testing, or the specific pile is just not amenable to sonic testing.

On the basis of qualitative interpretation Klingmuller (1992) divided all the piles on a site into six categories: Piles belonging to classes 1 to 4 were accepted without reservation, class 5 piles were considered as candidates for dynamic load testing prior to acceptance and those belonging to class 6 were rejected altogether.
Table 1: Typical piles with respective reflectograms

<table>
<thead>
<tr>
<th>PILE PROFILE</th>
<th>DESCRIPTION</th>
<th>REFLECTOGRAM</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image" alt="Straight pile, free end, length as expected" /></td>
<td>Straight pile, free end, length as expected</td>
<td><img src="image" alt="Reflectogram" /></td>
</tr>
<tr>
<td><img src="image" alt="Straight pile, fixed end, length as expected" /></td>
<td>Straight pile, fixed end, length as expected</td>
<td><img src="image" alt="Reflectogram" /></td>
</tr>
<tr>
<td><img src="image" alt="Straight pile, free end, shorter than expected" /></td>
<td>Straight pile, free end, shorter than expected</td>
<td><img src="image" alt="Reflectogram" /></td>
</tr>
<tr>
<td><img src="image" alt="Increased impedance" /></td>
<td>Increased impedance</td>
<td><img src="image" alt="Reflectogram" /></td>
</tr>
<tr>
<td><img src="image" alt="Decreased impedance" /></td>
<td>Decreased impedance</td>
<td><img src="image" alt="Reflectogram" /></td>
</tr>
<tr>
<td><img src="image" alt="Locally increased impedance" /></td>
<td>Locally increased impedance</td>
<td><img src="image" alt="Reflectogram" /></td>
</tr>
<tr>
<td><img src="image" alt="Locally decreased impedance" /></td>
<td>Locally decreased impedance</td>
<td><img src="image" alt="Reflectogram" /></td>
</tr>
</tbody>
</table>
Table 2 presents Four typical examples of reflectograms obtained using the PET system on 600 mm diameter bored piles. All piles were bored to an approximate depth of 11 m. The examples show, in that order:

- A normal pile
- A pile with an enlarged cross section (collapse during boring)
- A pile with necking (collapse during casting)
- A pile not amenable to interpretation due to an ill-prepared head.
<table>
<thead>
<tr>
<th>PILE</th>
<th>Length (m)</th>
<th>Details</th>
<th>Reflectogram</th>
<th>Remarks</th>
</tr>
</thead>
</table>
| C31  | 11.6       | Date:2002-08-08  
c =4000 
Amp =26 | ![Reflectogram](image) | No anomalies |
| X212 | 11.1       | Date:2002-05-12  
c =4000 
Amp =21 | ![Reflectogram](image) | Increased cross section @ 5m |
| C36  | ?          | Date:2002-08-08  
c =4000 
Amp =55 | ![Reflectogram](image) | Necking @ 4.2m – no toe reflection |
<table>
<thead>
<tr>
<th>PILE</th>
<th>Length (m)</th>
<th>Details</th>
<th>Reflectogram</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>C23</td>
<td>?</td>
<td>Date:2002-07-18 c =4000 Amp =10</td>
<td><img src="image" alt="Reflectogram" /></td>
<td>Head not clean – no toe reflection</td>
</tr>
</tbody>
</table>
Quantitative Interpretation

When both the form of the pile and the skin friction distribution are known, a synthetic reflectogram may be drawn using Equations 11, 12 and 14. The inverse problem, however, has no unique solution even for the zero-friction case (Vyncke & van Nieuwenburg 1987, p. 15, Danziger et al. 1976). Approximate solutions may be obtained by signal-matching techniques (Middendorp & Reiding 1988 p. 33-43) that work as follows:

First, a reference reflectogram for a cylindrical pile is chosen. In certain cases, such a reference reflectogram may be obtained by averaging a large number of reflectograms over a given site.

The soil friction function is then varied until the synthetic reflectogram calculated for a cylindrical pile is identical to the reference reflectogram.

Using the friction distribution thus obtained, the pile profile (or rather impedance profile) is varied until a good match is reached for the pile under consideration. Under ideal conditions, this profile may be close enough to the real one. The signal matching described above can be performed either interactively or automatically (Courage & Bielefeld 1992 pp. 241-246) but both methods are subject to the same limitations.

Figure 20 is an example of signal matching performed by the PET software. The PET signal matching tool uses drag and drop method to draw both the shape of the pile and the skin friction distribution. Obviously, the solution presented is only one of many possible ones.
Figure 20: Quantitative interpretation using PET signal matching
**Instrumented hammer analysis**

If we used an instrumented hammer for testing, we can utilize the additional data in two different approaches:

**Time domain:** From Equation 7 we know that the velocity created by the blow is proportional to its force. Thus, if we normalize the force time-history during the blow, we can plot it so that it will coincide with the initial part of the reflectogram. If, however, there is an impedance change close to the top, the reflected wave will separate the plot of the velocity from that of the force (Reiding 1992). This enables us to observe impedance changes that are close to the top and would otherwise go unmarked.

**Frequency domain:** Another approach is based on presenting the test result not as a reflectogram but in the frequency domain: The measured velocity is resolved into its component frequencies by a process known as Fast Fourier Transform (FFT). When a rod with a finite length $L$ is hit at one end, it will resonate at a frequency of $c/2L$ and at whole products of this frequency. To get a consistent picture, however, we must normalize the velocity spectrum and divide it by the respective force spectrum. The variable $v/F$ thus obtained, which is the inverse of the impedance, is termed **admittance** or mobility. An idealized mobility plot is presented in Figure 21 and shows a periodic curve corresponding to the pile length. With suitable training, we can use the mobility plot to draw certain conclusions regarding the profile of the pile.
The mean value of the wavy-shaped part of the plot is called the characteristic mobility \( M_0 \) of the pile, equal to \( 1/\rho c A \). A comparison of \( M_0 \) with the expected admittance may provide a clue as to the integrity of the pile.

The initial, low-frequency part of the mobility plot is in many cases quite straight. Because of the low frequency we may neglect both viscous and inertial effects, and assume that the maximum force \( F_{\text{max}} = k \cdot u_{\text{max}} \), where \( k \) is the stiffness of the pile head and \( u_{\text{max}} \) is equal to the maximum displacement. The maximum velocity is obtained by:

\[
\nu_{\text{max}} = \omega \cdot u_{\text{max}} = 2\pi f_{\text{max}} u_{\text{max}} \quad \text{Equation 23}
\]

Therefore, the dynamic stiffness of the pile head is:

\[
k = F_{\text{max}}/u_{\text{max}} = 2\pi [f_{\text{max}}/(\nu_{\text{max}}/F_{\text{max}})] = 2\pi/s \quad \text{Equation 24}
\]
Where $s$ is the slope of the straight line. The dynamic stiffness $k$ is of course applicable only at very low strains, and has little to do with the behavior of the pile under working loads.

**Evaluation of the sonic method**

The sonic test is a powerful quality-assurance tool, but we must never forget that it is not omnipotent. The following paragraphs contain a short discussion of both the capabilities and limitations of the sonic method. We will then discuss the problems created by the unavoidable presence of noise.

**Capabilities**

Since the sonic method is based on the use of stress-waves, it can identify only those pile attributes that influence wave propagation. The following items may often be detected (England 1995):

1. Pile length.
2. Inclusions of foreign material with different acoustic properties.
3. Cracking perpendicular to the axis
4. Joints and staged concreting.
5. Abrupt changes in cross section.
6. Distinct changes in soil layers.

**Limitations**

All physical measurements have limitations, and the sonic test probably has more limitations than any other test. For instance, the sonic test will normally not detect the following items:
1. A toe reflection when the L/D ratio roughly exceeds 20 (in hard soils) to 60 (in very soft soils)
2. Gradual changes in cross-section
3. Minor inclusions and changes in cross-section smaller than ±25%
4. Impedance changes of small axial dimension
5. Small variations in length
6. Features located below either a fully-cracked cross section or a major (1:2) change in impedance
7. Debris at the toe
8. Deviations from the straight line and from the vertical
9. Load-carrying capacity

**Noise**

Noise is the enemy of all physical measurements, and its magnitude in relation to the measured signal is of utmost importance. We usually distinguish between internal noise and external noise. In modern instruments with digital sensors and miniaturized electronics internal noise is practically non-existent. As to external noise, it may be due to numerous sources (Amir & Fellenius 1996), namely:

1. Surface (Rayleigh) waves created by the hammer blow are reflected from the boundaries, causing high-frequency noise.
2. Often a short piece of casing is used at the top of a pile during concreting and later pulled out, resulting in a sharp decrease of the cross section at the bottom of the casing. This decrease creates regularly repetitive reflections that (except for the first one) appear as medium-frequency noise.
3. Trimming of the pile tops usually leaves a rough surface. When a concrete protrusion is hit with the hammer, it may break and create random noise.
4. High-frequency noise may also be produced by careless hammer blows that may hit reinforcement bars.

5. When the top of the pile is not trimmed enough, or not at all, it may produce pure noise (usually of a wavy form). This may lead to severe misinterpretation by inexperienced persons, where faulty piles may be declared sound, and vice versa.

Noise should always be reduced to the minimum level. Regular noise (Items 1 and 2) may sometimes be treated by mathematical filtering. Random noise (Items 3 and 4) is reduced by averaging a larger number of blows. Testing a poorly-prepared pile (Item 5) is a waste of time, so whenever one is identified the test should be repeated after proper trimming.

Needless to say that a noisy signal is practically worthless, as it masks important features that could otherwise be detected.

For high L/D ratios and high skin friction, we have to apply high amplification to the signal. As a negative side effect this will also magnify the noise. It seems, therefore that while some degree of noise may be tolerated in short piles in soft soils, it becomes unacceptable when piles become longer and soils harder.

To reduce external noise, it is sometimes recommended to smoothen a small patch on the top of the pile. This can be done by troweling the fresh concrete immediately after casting or by grinding the hardened concrete with a disc saw.

Even under good conditions, there will be piles in which it will be difficulty to pinpoint the toe reflection. In such piles the error in the reported length may exceed the ±10% value usually quoted.

**Accuracy**

Any given pile has at least three so-called “lengths”:

1. The design length as shown in the drawings
2. The as-made length, for instance the depth to which the pile was drilled or augered, less the amount trimmed off.

3. The effective length, which is the length between the top and the uppermost total discontinuity.

The main parameter that the sonic test is supposed to deliver is the effective length of the pile. This subject is often loaded with emotion, and anybody involved with pile testing should approach it with due care. The accuracy of the reported effective length depends on two main factors.

The first one is how good our reflectogram is: To give an accurate value for the effective length, the reflectogram should have a high signal-to-noise ration, with a clearly defined toe reflection. A poor reflectogram will either show no toe reflection or, what is worse, will show multiple features, each of which may be mistaken for the toe reflection. In addition, when the bottom part of the pile was poorly constructed, it will “smear” the toe reflection over a considerable length. In such a case it may be rather difficult to pinpoint the toe reflection.

The second item is how good our guess for the wave speed is. From experience, wave speed in concrete piles can vary in the range between 3,600 and 4,400 m/s. This means that an initial guess of 4,000 m/s ensures that the error due to this cause will not exceed 10%. We can greatly improve on this assumption if we take into account the concrete grade and age (Amir 1988). Even then, we can never expect all the piles on a site to have an exactly uniform wave speed, especially when testing is done shortly after construction.

To improve accuracy, we shall need at least a few reliable as-made depth measurements made in the open holes before concreting. Bilancia et al. (1996) compared length measurements made during construction to those obtained by the sonic method, and found differences up to 6 percent. Thus, if we consider all relevant factors, it is safe to assume that the sonic method is accurate within an order of 10%.
Frequency domain vs. Time domain

Although the presentation of the sonic test results in the frequency domain offers more options for advanced analysis, it is still subject to the same limitations as the time domain analysis. Like the reflectogram, the mobility plot is sensitive to both skin friction and noise: With high values of skin friction it becomes practically flat, and provides very little information. The high frequency noise, which we mentioned above, may, if not treated properly, give rise to erroneous interpretation.

A typical hammer blow lasts around 1 millisecond. This means that it is hardly possible to detect frequencies below 1 kHz. The calculation of the pile head stiffness, therefore, is necessarily based on scant information. Indeed, related research work (Rausche et al. 1992 p. 617) indicated that the dynamic stiffness values obtained by this method are at best questionable.

An extensive research project carried out in the Technical University of Braunschweig (Plassman 2002) reported as follows: “In comparison to the analysis of the time domain, there is no further improvement in the interpretations of the pile geometry”.

To summarize, choice between the two methods is a question of personal preference. The frequency domain method is a different way to look at the results, and can be a useful tool in the hands of trained people. The method may provide some information as to the properties of the pile but these are also obtainable from time domain analysis if we use an instrumented hammer. The interpretation in the frequency domain is more elaborate and less intuitive than in the time domain. Moreover, because it allows more sophisticated analytical techniques, frequency domain analysis can easily lead to over-confidence in the results.

International competitions

Nowhere were the limitations of the sonic method better demonstrated than in a number of testing competitions held in recent years. Due to the heated debate that followed each competition, it is worthwhile to discuss a few of them. More details are presented in Appendix A.
Sample specifications

1. **Introduction**: Sonic testing is intended to give information regarding pile lengths, continuity and concrete quality. It is able to locate flaws in piles with regard to depth, character and severity, but does not address the question of pile capacity.

2. **Testing agency**: The sonic test shall be carried out by a firm experienced in this kind of work and approved by the Engineer. A Geotechnical engineer with proven experience shall supervise site work and carry out the interpretation of the results.

3. **Equipment**: The sonic test shall be performed using a computerized system of reputable origin with digital sensors. All components shall be recently validated and in good working order. All software shall be of the latest released version.

4. **Piles to be tested**: All piles shall be tested at a minimum age of five days after casting, unless instructions to the contrary shall be given by The Engineer.

5. **Preparations**: Before commencing the test, the Contractor shall ensure that there is adequate access to the pile. The Contractor shall remove, where applicable, the spiral reinforcement above the pile head. The concrete at cutoff level shall be of full strength, clean and dry and free of laitance, free chunks, etc., to the satisfaction of The Engineer. Where ordered, The Contractor shall prepare smooth testing surfaces using a disc grinder.

6. **Testing method**: Testing shall be carried out by pressing the transducer to the head, hitting it with a plastic hammer, recording the reflected waves and processing the results by the computer. The output for all piles shall clearly identify the project, pile number, date, time, depth scale and wave speed on which the measurements are based, as well as the results of at least three hammer blows.
7. Reporting: A final report for each testing stage shall be presented not later than three working days after completion of that stage. The report shall consist of a printout of the original output, as well as a summary table including, for every pile tested, the depth and the engineers' interpretation regarding its integrity.

Table 3: Limitations of sonic testing

<table>
<thead>
<tr>
<th>PILE PROFILE</th>
<th>DESCRIPTION</th>
<th>REFLECTOGRAM</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>High L/D ratio and/or high skin friction - no toe reflection</td>
<td><img src="image" alt="Reflectogram" /></td>
</tr>
<tr>
<td></td>
<td>Progressive changes in cross-section</td>
<td><img src="image" alt="Reflectogram" /></td>
</tr>
<tr>
<td></td>
<td>Minor inclusions</td>
<td><img src="image" alt="Reflectogram" /></td>
</tr>
<tr>
<td></td>
<td>Impedance change of small axial dimension</td>
<td><img src="image" alt="Reflectogram" /></td>
</tr>
</tbody>
</table>
Parallel Seismic Testing

The main drawback of the sonic method is that the pile top should be accessible. If the pile is part of a capped pile group sonic testing is highly impractical. If the cap supports a sea-wall or bridge pier (Figure 22) sonic testing is impossible. Yet, in numerous cases we are asked to establish the depth of the existing piles when the structure is due for upgrade. A similar situation occurs when deep excavations are planned in urban areas, adjacent to old and sensitive buildings. Under such circumstances, the Parallel Seismic method (Afnor 1993) is the accepted means for measuring the depth of foundations supporting structures.

To perform the test, a plastic tube with 40-50 mm inner diameter is installed in the ground and filled with water. The tube should be vertical, at a distance of 0.5 to 1.5 m from the measured element, and extend at least 5 m below
and extend at least 5 m below the expected bottom of the tested foundation (Figure 24). In dry soil the annular space around the tube should be grouted, while in saturated soil this is unnecessary. A suitable spot on the superstructure above the pile is then repetitively hit with a heavy hammer equipped with a triggering switch. Simultaneously, a hydrophone connected to the instrument is lowered in the tube in stages of about 500 mm each and the arriving waves recorded.

A typical graph of FAT vs. depth (Figure 23) consists of two straight branches. The slope of the upper one V1 is equal to the wave speed in the pile and the lower one V2 is equal to the wave speed in the ground. The depth of the break point indicates the depth of the pile. Figure 24 shows the PSI (Parallel Seismic Instrument) by Piletest.com.
Figure 23: Principle of the Parallel Seismic method

Figure 24: The PSI (Parallel Seismic Instrument) by Piletest.com
Ultrasonic Testing

Basic Principles

The sonic method belongs to the external test-methods, as it accesses only the top of the pile. Ultrasonic logging, on the other hand, is intrusive and necessitates the prior installation of access tubes (Usually two or more) in the pile. Before the test they have to be filled with water (to obtain good coupling) and two probes are lowered inside two of the tubes. One of these probes is an emitter of ultrasonic pulses and the other a receiver. Having been lowered to the bottom, the probes are then pulled simultaneously upwards to produce an ultrasonic logging profile. The transmitter produces a series of acoustic waves in all directions. Some of these waves do eventually reach the receiver. The testing instrument then plots the travel time between the tubes versus the depth. As long as this time is fairly constant, it shows that there is no change in concrete quality. An anomaly, showing a sudden increase of the travel time at any depth, may indicate a flaw. The cross hole method has been described in a number of recognized standards (Amir & Amir 2008a). As we shall see later, the cross-hole method may be further refined to produce high-quality results.

A variation of ultrasonic logging is the single-hole method (Brettmann & Frank 1996, Amir 2002): In this method both emitter and receiver are lowered, with a fixed vertical spacing, in the same tube (or hole). In this case the waves do not travel across, but rather along, the pile axis.

While the access tubes introduce an extra expense item, the cross-hole compensates for this by allowing the testing equipment to approach potential flaws. An additional advantage of this test is the enhanced resolution: while the sonic test uses a wavelength of at least two meters, the cross-hole method utilizes ultrasonic frequencies, with typical wave lengths of 50 to 100 mm. Since resolution is strongly dependent on the wave length, the cross-hole method enables us to detect much smaller flaws.
Because of these short wave lengths, the one-dimensional theory, on which we based the sonic method, is no more applicable. Thus, we have to look into the theory describing the behavior of waves in space.

Three-dimensional wave propagation

In our review of wave propagation in prismatic rods, we discussed only one type of waves - longitudinal waves. These could be either compressive or tensile.

Body waves moving in a three-dimensional elastic space, however, include two main types: P-Waves and S-Waves. When a P-Wave travels close to a free surface it changes into a third type, namely Rayleigh waves (Figure 25).

When the displacements associated with the wave occur on the axis of wave propagation (as in the rod), the waves are called longitudinal or P-Waves. The particles in this case may move either in the same direction (compressive wave) or in the opposite direction (tensile wave). P-Waves are dilatational, which means they are accompanied by volume expansion or decrease.

![Figure 25: Main types of body waves](image)

When the wave causes particles to move perpendicular to its direction of motion, the waves are known as S-Waves or shear waves. S-Waves are distortional and as such do not cause any volume change in the medium. The character of S-Waves is further determined by the direction of
displacement: When particle displacements are horizontal, the waves are called SH-Waves. SV-Waves, on the other hand, are characterized by vertical motion. A general S-Wave may be decomposed into its SV and SH components.

As mentioned before, waves radiate from the point of application of any dynamic load. As the distance from this point increases, the waves may be assumed to travel as a plane front. All material particles then move in this plane (S-Waves) or perpendicular to it (P-Waves). This assumption leads to substantial simplification in the mathematical treatment: If we denote the particle displacement by $\psi$, then the propagation of plane waves is governed by the following differential equation:

$$\frac{\partial^2 \psi}{\partial t^2} = c^2 \frac{\partial^2 \psi}{\partial x^2}$$

Equation 25

Here $c^2 = \frac{E}{2\rho(1+\nu)}$ for S-Waves and $c^2 = \frac{E}{\rho} \frac{(1-\nu)}{(1+\nu)(1-2\nu)}$ for P-Waves. $c$, as usual, is the wave propagation velocity. For Poisson’s ratio $\nu=0.25$, the velocity ratio $c_P/c_S$ is equal to $\sqrt{3}$, which means that P-Waves propagate much faster than S-Waves. For good-quality concrete, $c_S=2,580 \text{ m/sec}$ and $c_P = 4,470 \text{ m/sec}$. In comparison, $c$ for a rod of the same material is equal to 4,080 m/sec.

To visualize the behavior of plane wave, we often prefer to trace a single normal to the wave front, and follow its’ path. As an analogy to optics, the trajectory of such a normal is termed a ray path.

Attenuation

When a wave front moves away from the emitter, it expands with distance. If we assume that no energy is lost during the process, the energy per unit area of the front (or per ray) declines with distance. The rate of decrease depends on the shape of the front: If it is spherical, the wave energy per unit area will change as the inverse of the square of the distance from the source:
\( \frac{E_2}{E_1} = \left( \frac{r_1}{r_2} \right)^2 \) \hspace{1cm} \text{Equation 26}

Where:

\( E_1 \) and \( E_2 \) denote the respective wave energies at points 1 and 2, respectively, while \( r_1 \) and \( r_2 \) are the respective distances of points 1 and 2 from the source. Since the energy \( E \) is proportional to the square of the amplitude \( A \), Equation 25 can be re-written as:

\( \frac{A_2}{A_1} = \frac{r_1}{r_2} \) \hspace{1cm} \text{Equation 27}

Equation 26 describes what we define as geometric attenuation, which occurs in all types of media and is the only source of energy loss in perfectly elastic materials. In real materials, which also exhibit viscous and frictional behavior, part of the mechanical energy of the waves is constantly converted into heat. This phenomenon, defined as material loss or intrinsic attenuation, is represented by the following equation (Santamarina et al. 2002):

\( A_2 = A_1 e^{-kf(r_2 - r_1)} \) \hspace{1cm} \text{Equation 28}

Where \( k \) is a medium-dependent constant and \( f \) is the wave frequency. Equation 28 shows clearly that material attenuation causes the wave amplitude to decrease with increasing distance from the source, at a rate that is dependent on the frequency. Higher frequency waves attenuate much faster than those with lower frequencies and therefore have a smaller range.

Since wave amplitudes may vary over a few orders of magnitude, it is convenient to express amplitude ratios on the decibel scale, defined as \( \text{dB} = 20 \log(A_2/A_1) \). Since \( \log 2 \approx 0.3 \), every 6 dB roughly represent an amplitude ratio of 2 or a 50% decrease.
Figure 26: Attenuation of Ultrasonic Waves in Concrete

Figure 26 is an example obtained by passing ultrasonic waves in a concrete pile while varying the distance between the emitter and the receiver. The exponential character of the attenuation is immediately apparent, as is the linear dependence of FAT on distance traveled.

\[ y = 0.22x \]

\[ y = 0.1x + 2 \]

Waves in an elastic half-space

When a dynamic load is applied at the free surface of a boundary, both P- and S-waves radiate from the point of application into the body. In addition, this load will produce surface waves that are not dissimilar to the ripples created by throwing a stone into calm water. These surface waves, named
after Lord Rayleigh, include longitudinal and transverse components. They are confined to a thin layer close to the surface, and propagate at a velocity that is slightly smaller than $c_S$.

Inside an infinite uniform body, plane body waves will propagate in straight lines. In a half-space, some waves will eventually hit the boundary $x = 0$ and will be reflected back into the body having undergone several changes:

When the waves hit the boundary at a right angle, P-Waves will change sign: Compressive waves are reflected as tensile, and vice versa. This follows directly from the requirement that the boundary be stress-free.

When a wave meets the boundary at an angle $q_1 < 90^\circ$, it undergoes mode conversion and is reflected back as two distinct waves: A P-wave is reflected at the angle of incidence $q_1$, and an SV-wave is reflected at a different angle $q_2 < q_1$ (Figure 27). See also Kolsky (1963) p. 24.

When the incident wave is of SV type, it will create two distinct reflected waves: An SV-wave will be reflected at the angle of incidence $q_2$, and a P-wave at a larger angle $q_1$.

![Figure 27: Reflection of P-Waves from the boundary $x = 0$](image)
The amplitudes of both reflected waves depend on the amplitude of the incident wave, the angle of incidence and Poisson’s ratio (Kolsky 1963 pp. 27-28).

Waves in Non-homogeneous Media

Up to this point we have dealt with homogenous media. For our purposes, a medium is defined as non-homogenous when it fulfils the following two conditions:

- The medium includes zones with different characteristic impedance \( Z = \rho c \).
- The typical size of these inclusions \( d \) is much larger than the wavelength \( \lambda \).

In such media, whenever a wave hits an inclusion, its propagation pattern will change at the interface. Here we shall distinguish between two cases:

**Normal incidence**

When a wave with an amplitude \( A_i \) travels from a region with the impedance \( Z \) to a region with another impedance \( Z' \) through a normal interface, it will be partly reflected back with an amplitude \( A_r \) while another part will pass through the interface with an amplitude \( A_t \).

Therefore, when a wave meets a weaker region \( (Z'<Z) \) it will change sign: A compressive wave will be reflected as a tensile wave, and vice versa. If, on the other hand, \( Z'>Z \) the reflected wave will preserve the sign of the incident wave.

This ratio \( A_t/A_i \) is always positive, which means that the transmitted part of the wave will preserve the sign of the incident wave.
**Oblique Incidence**

When a wave meets the interface at an angle $\theta_1$ to the normal, it will create four new waves: Two of them reflected and two - refracted. (Figure 28).

![Figure 28: Reflection and refraction of P- and SV-waves](image)

**Media with inclusions**

A generalization of the case, which we discussed above, is that of a given medium with an inclusion of an arbitrary shape having different properties. This situation, by the way, is of most importance to those who are in the business of locating flaws in cast-in-situ concrete piles. From our previous discussion we may now realize that in a typical case, an incident wave will create four distinct waves when hitting the inclusion. Of these, two will enter the inclusion at different angles, and each of those may create four more waves upon leaving the inclusion. Thus, for each ray path hitting a typical inclusion, two waves will be reflected back from its boundary and a number of waves will leave it at various locations and angles. Clearly, a close-form treatment of the general case (An inclusion with an arbitrary form) is rather hopeless. Idealized cases, such as a plane wave in an infinite
medium with a spherical or cylindrical inclusion, may be treated by analytical methods using infinite series (Graff p. 394 ff.).

A typical case of a plane P-wave hitting a cylindrical inclusion is demonstrated (For a single ray path) in Figure 29. In this example, a single wave creates three pairs of waves exiting the cylinder. We note immediately that none of these follows the original trajectory! The amplitudes of each wave and its sign (either compressive or tensile) are shown as a fraction of the amplitude of the incident wave.

From this exercise we can also conclude that in a non-homogeneous medium, consisting of zones with different acoustic properties, waves will propagate in broken or curved lines.
Instrumentation

A modern system for ultrasonic testing of piles (Figure 30) consists of the following components (Figure 31):

![Figure 30: The CHUM system for pile integrity testing](image1)

![Figure 31: Ultrasonic Testing Instrumentation](image2)
A pair of probes (emitter and receiver). Each of these is equipped with a ceramic piezo-electric ring and the applicable electronics. The frequency of the system is determined by the properties of the rings, with the usual range being 40 to 100 kHz. This conforms to wavelengths in the order of 40 to 100 mm.

A depth encoder(s) that produces a predetermined number of pulses per revolution.

Pulse generator and signal processor.

A computer that handles the following functions:

- Identification of the project, the pile and the profile
- Input of the control parameter for the specific measurement
- Monitoring the depth from the encoder output
- Controlling the pulse rate of the emitter
- Saving the output, processing the data and displaying the results.

Pile preparation

The access tubes specified for testing purposes are in most cases made of either PVC or “black” ungalvanized steel. The diameter of the tubes depends, of course, on the diameter of the probes: The normal size is between 38 and 50 mm (1.5” to 2”), with sufficient wall thickness to ensure stability. If Splicing is done by butt-welding, due care should be taken to prevent welding material from penetrating into the tubes and cause jamming of the probes. The use of an external bushing is also an option. Both ends of the tube should be sealed with suitable screwed caps or with welded plates. This is mandatory in order to eliminate penetration of debris or drilling mud into the tubes. Never should bending and hammering be used to close the bottom of the tube, as this may cause jamming of the probes (Figure 32).
The number of access tubes cast in the pile concrete is a function of the pile diameter, the importance of the pile and, of course, economic consideration. A good rule of thumb (ASTM 2008) is to specify one tube per each 30 cm of pile diameter. Thus for a pile with a diameter of 1.2 m, four tubes will normally do. For best effect, the tubes should be equally spaced inside the spiral reinforcement and rigidly attached to it by wire or spot welding. Where tubes are extended below the reinforcement cage, they have to be stabilized by suitable steel hoops.

When rigid PVC tubing is used, we have to face the risk of physical damage to the tube due to rough handling. In addition, separation ("debonding") of the tube from the surrounding concrete has occasionally been reported. On the other hand, the acoustic impedance of PVC is much lower than that of steel, so higher energy is transmitted through a plastic tube, something which can be advantageous for long range testing. Typical tube arrangements are presented in Table 4.

In piles the tubes are usually named according to their orientation: The northernmost tube is designated N and the others accordingly. A profile is then named after the tubes between which it passes, such as N-S or S-W.
In barrettes, the tubes should be located so that the resulting profiles will be diagonal. Profiles that are close and parallel to the wall may often produce false alarms. As distinct from piles, the tubes are usually denoted by numbers. The maximum distance between tubes depends on the kind of equipment used, but should usually not exceed two meters.

Table 4: Access tube arrangements

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<table>
<thead>
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<tbody>
<tr>
<td>Single tube</td>
<td>2 tubes - 1 profile</td>
<td>3 tubes - 3 profiles</td>
</tr>
<tr>
<td>4 tubes - 6 profiles</td>
<td>Straight barrette</td>
<td>T-shaped barrette</td>
</tr>
</tbody>
</table>

Procedure
Before we begin testing a pile, the access tubes have to be checked for free access and filled with water (to obtain good acoustic coupling). Two pulleys are then inserted into the tubes, at least one of which is equipped with a depth meter. The probes (emitter and receiver) are then inserted over the pulleys and lowered into the tubes. After reaching the bottom of the tubes, the operator must first ascertain that they are at the same level. Probes may be found at different levels because of poor tube workmanship or because one or both tubes is blocked by debris. The probes are then pulled simultaneously upwards with smooth motion until they reach the pulleys. During this time the emitter produces a continuous series of pulses, sending waves in all directions. The vertical distance between successive pulses is determined by the operator, with 50 mm. being a good typical figure.

Some of these wave paths do eventually reach the receiver. The pulses arriving at the receiver are recorded and processed by the testing instrument. The operator may then see the results on the screen.

Presentation of the results

According to the French Standard (AFNOR 2000), the test report should, as a minimum, present for each pile full identification details: the project description, job number, date, pile number and tube arrangement. For each profile in the pile the first arrival time (FAT) of each pulse versus depth should be plotted. Later standards (ASTM 2008) demand, in addition, a plot of the relative energy versus depth. For this purpose, the energy carried by each pulse is defined as that area enclosed between the pulse envelope and the horizontal axis. The energy units are typically mVμsec. If the maximum possible energy is taken as reference, the fraction of this value calculated for each pulse is defined as the relative energy. Since energy may vary over several orders of magnitude, the scale for relative energy is usually logarithmic. ASTM (2008) also requires the presentation of a “waterfall” representation. This form is obtained by transforming each pulse into a narrow band. The highest peaks of the pulse are shown in black and the deepest troughs by white. Intermediate values are shown as gray scale. All the bands are then stacked together, giving an overview of the whole profile.
In addition to the above plots modern instruments, such as the CHUM, are capable of showing the following (Table 5: Presentation of the results):

Apparent velocity versus depth - obtained by dividing the tube spacing by the FAT.

Attenuation (in decibels) versus depth shows the inverse of relative energy, and has the advantage of making quantitative assessment possible.

The CHUM can combine any number of the above plots, with the exception of relative energy and attenuation.

Table 5: Presentation of the results

[Graphs showing First Arrival Time (FAT) and Apparent velocity]
c. Relative energy (no scale)

d. Attenuation (dB)
Interpretation

While any good technician can successfully carry out a cross hole test, the interpretation of the results should be left to experienced geotechnical engineers. The two main parameters that have to be considered when looking at the test results are the FAT and the measured energy.

First arrival time (FAT)

The first item is the shape of the FAT vs. depth plot. As long as FAT is more or less constant, it indicates that there is no change in concrete quality between the tubes along the pile. A local increase of the arrival time is considered as an anomaly that may be the result of an obstacle (or flaw) on the
straight path between the probes at the corresponding depth. The ultrasonic waves will either travel through this flaw at a reduced velocity or bypass it, with a respective increase in travel time.

Experimental work (Stain & Williams 1991) indicated that lower-strength concrete has little effect on either transmission time or signal attenuation. On the other hand, segregated (honeycombed) concrete, bentonite-polluted concrete and pockets of pure bentonite slurry do strongly affect the signal.

FAT picking by sight for every pulse is fairly straightforward. But when dealing with thousands of pulses, as is the case in a typical pile, it becomes an impractical proposition. We have therefore to use some automatic process, or suitable algorithm, to perform this task for us. We have at our disposal several methods designed exactly for this purpose:

**Fixed threshold**

In this method, we tacitly assume that the first arrival time occurs when the signal strength first exceeds a predetermined signal level, or threshold. Since this can give erroneous results in noisy pulses, the refined version of this method allows the operator to dictate a lower-bound value for the FAT, depending on the probe separation. The fixed threshold method is very simple, but may give us totally wrong answers in weak signals (Figure 33).

![Figure 33: FAT picking by fixed threshold in strong (left – 10V scale) and weak signal (right – 1V scale)](image-url)
**Dynamic threshold**

The dynamic threshold overcomes the main drawback of the fixed threshold method, by assigning to each pulse a different threshold, which is a fraction of the maximum amplitude. Thus for a strong signal we have a high threshold, and for a weak signal a correspondingly low threshold. A good choice of the fraction is crucial to the success of this method.

**Moving windows (STA/LTA)**

This method uses a long moving window in which a short window is included in the right hand side (Figure 34). As both windows are shifted from the origin to the right, pixel by pixel, the ratio between their respective average amplitudes (STA/LTA) is constantly calculated. The FAT is set at the point where the ratio exceed a predetermined value,

![Figure 34: The moving windows FAT picking method](image)
Automatic FAT picking

This iterative algorithm assumes that the FAT is located between the origin and the maximum pulse amplitude $V_0$. To start with, the program plots the pulse envelope, and the time in which the maximum point occurs is defined as $T_0$. The algorithm then performs the following tasks (Figure 35):

- A diagonal is drawn from $(0,V_0)$ to $(T_0,0)$.
- The point where this diagonal first crosses the envelope is $(T_1,V_1)$.
- A diagonal is drawn from $(0,V_1)$ to $(T_1,0)$.
- Steps 2 and 3 are repeated until $T_{n+1} = T_n$, which is then defined as the FAT.

The major advantage of this method is that, unlike all others, it requires no arbitrary parameters.

Figure 35: Automatic FAT picking
Accuracy

England (1995) itemized the factors that influence the accuracy of the test in terms of arrival time. Among these are:

- Free movement of the probes within tubes of larger diameter
- Probes not being at the same level
- Tubes not exactly parallel
- Air gap or different material around the tube
- Measurement resolution

According to England, the accumulated error due to all these factors may amount to 20 percent. He therefore suggests that only larger variations should justify further investigation.

To the above factors, Amir et al. (2004) also added the normal scatter of concrete strength and the FAT picking error. As a result of all this, they estimated that the total error may reach 10% of the FAT value plus 27 \( \mu \)sec. The effect of this error is very marked when the tubes are closely-spaced.

From the above, it is obvious that for a given pulse, FAT is not a unique value, as it depends strongly on the algorithm used and, within that algorithm, on the parameters selected. Figure 36 amply proves this point. Moreover, experience has shown that no single algorithm will give satisfactory results for all types of pulses. FAT picking is, therefore, a process which calls for careful judgment. Deciding which method is preferable is sometimes difficult, and at present the best standard against which a particular method can be judged is the visual method.
Energy is the other useful parameter which we obtain from the ultrasonic test. It is true that in uniform concrete the both energy and FAT depend on the distance traveled (Figure 26), but still it pays to study them separately. The main reason for this is that, unlike FAT, which can assume many values, energy is practically invariant.

If the waves pass through an inclusion, both parameters should be affected. A local increase in FAT without a corresponding decrease in energy may mean that we are using the wrong FAT picking method. If, on the other hand we notice a decrease of the measured energy without an increase in FAT, it usually is a result of a constriction of the travel path and the defect is located out of the straight line connecting the probes (Figure 37).
Special techniques

Tomography

When a flaw is located in a certain region of the pile the test may be repeated, but this time with the probes at different levels (Figure 38). This technique, borrowed from medicine, is called *tomography* and it enables us to define both the location and size of the inclusion.

![Figure 38: Ultrasonic tomography](image)

To perform pile tomography, we start the test normally, pulling both probes simultaneously upward. When an anomaly is encountered, we proceed as follows:

- Pulling is stopped about one meter above the anomaly.
- One of the probes (let’s call it A) is lowered until the signal becomes very weak.
- The second probe (let’s call it B) is lowered a little (say 100 mm) and probe A raised until the signal is almost lost.
- Probe B is further lowered in stages, and probe A alternately lowered and raised until the signal is lost.
- The procedure is repeated until the anomaly is fully formed on the screen.
The probes are brought to the same level, and the test continued in the normal fashion all the way to the top.

The above technique produces horizontal readings, as well as reading inclined both ways. To analyze the problem, the zone tested is arbitrarily divided into a grid of small elements called *pixels* and the FAT is then calculated for all rays passing through the zone. At this point, we can choose among three analytical approaches:

**Simultaneous equations**

If we define the inverse of wave speed as *slowness* $s = 1/c$, the arrival time for a ray $i$ is:

$$t_i = \sum L_{ij} s_j$$  \hspace{1cm} \text{Equation 29}

where $L_{ij}$ is the distance traveled by ray $i$ in pixel $j$, and $s_j$ is the slowness of pixel $j$. If the pixels are small enough, all $L_{ij}$ values can be replaced by 1 with little effect on accuracy. Since the number of measurements usually exceeds the number of pixels, the resulting system of equations is overdetermined, and can be solved with suitable mathematical techniques (Bowen & Dianguang 1992). The resulting slowness values can then be converted into wave speeds and plotted in contour form. As we have already seen, the wave speeds can correlated with concrete quality.

The same procedure, which is carried out in two dimensions on a single profile, can be used in three dimensions for the whole pile. In this case, the pile is divided into elementary voxels, or volume pixels. The mathematical treatment is the same. An important point with 3-dimensional tomography (3DT) is that distances between the access tubes must be measured very accurately.

The 3DT output can be presented in various forms:

Using special viewing software enables dynamic presentation with which the user can rotated the pile, look at it from different angles and zoom in on areas of special interest (Figure 39).
Horizontal and/or vertical cross sections where anomalies are found (Figure 40).

In all forms, the user can choose a threshold wave speed that will emphasize the inclusions on the blank background which denotes sound concrete.
Fuzzy logic tomography

The analysis can be simplified, at some sacrifice to accuracy, by using fuzzy logic techniques (Santamarina 1994). In this approach, every pixel is assigned a quality index on a scale of 1 to 10. To obtain the quality index, all the rays passing through the region are scrutinized, and the FAT values of the inclined rays corrected for inclination, according to the following expression:

\[
FAT_{corr} = FAT - \frac{S}{\sqrt{S^2 + (\Delta y)^2}}
\]

Equation 30

Where \(FAT_{corr}\) is the corrected arrival time, \(S\) is the tube spacing and \(\Delta y\) is the difference in probe level. The rays with the minimal \(FAT_{corr}\) in the region are assigned a fuzzy value of 10 and the ray with the maximum \(FAT_{corr}\) – a fuzzy value of 1. The rays between these boundaries are given intermediate values by interpolation. Since a “good” ray cannot pass through a “poor” pixel, the quality index of a pixel is equal to the highest fuzzy value of all the rays passing through it. The procedure is dynamic, and the inclusion forms in real time on the screen as more readings are added (Error! Reference source not found.).

![Figure 41: Stages in fuzzy-logic tomography (note “shadow” areas)](image)

1. The real inclusion. 2. Horizontal measurements determinates the vertical coordinates \(y_1\) and \(y_2\). 3. Adding inclined (/) measurements reveals the location of the defect. 4. Reverse inclined (\(\backslash\)) measurements refine the defect shape (note unavoidable “shadow” areas).
A real-life example demonstrating the capabilities of the method is presented in Figure 42

![Image](image)

**Figure 42**: Result of fuzzy logic tomography (left) and the exposed defect (right)

**Parametric tomography**

In the parametric approach, we start with two assumptions:

- At any given level there is only one inclusion.
- The inclusion is in the shape of an upright rectangle.

From the horizontal rays, we know $y_1$ and $y_2$, that is the lower and the upper coordinates of the rectangle, respectively. To define the rectangle, all we need just two more parameters – the left hand coordinate $x_1$ and the right hand coordinate $x_2$ (Figure 43). The procedure works as follows:
First we define some rectangle by assuming the values of $x_1$ and $x_2$.

We take all rays that we logged through the region. For rays passing through the assumed rectangle, we assign the highest measured value of FAT. Those rays that skip the rectangle get the normal FAT value of the region.

For every ray, we find the difference $\Delta t$ between the measured and the assumed FAT. The sum of the squares of the differences $\Sigma(\Delta t)^2$ is defined as the residual of the assumed rectangle.

The procedure is repeated for a large number of rectangles, with $x_1$ and $x_2$ varied between 0 and $S$. That rectangle with the lowest residual is considered the best representation of the inclusion.
**Attenuation-based tomography**

Up to this point, we presented the classical approach, that of FAT-based tomography. Since a given inclusion may have a stronger influence on the energy than on the FAT, attenuation-based tomography may sometimes produce superior results (Amir & Amir 2009). One of the advantages of this approach is that energy, unlike FAT, is practically invariant. Incidentally, attenuation, like FAT, is linearly-dependent on distance. The exact function can be derived from Figure 26 or from actual measurements made on a defect-free zone of the pile.

Tomography can also be based on either/and combination of FAT and energy, and experience has shown that this is a very promising approach.

**Evaluation**

Tomography provides us with a two-dimensional picture of the profile we are interested in, and thus can be a powerful technique. Nevertheless, it has a few inherent shortcomings:

- The method fails close to either the upper or the lower ends of the pile (Figure 44).
- Since we have only a limited range of viewing angles (none close to vertical), we shall always get shadows that will extend the defects horizontally (Figure 43).
- The solution is not necessarily unique, and in certain cases may yield erroneous results (Santamarina 1994).
Single-hole testing

Single hole ultrasonic testing (Figure 45) is a variation on the conventional cross hole method. It is extremely useful under the following conditions:

- If cross hole testing was planned, but some tubes were accidentally damaged, and only one is available for testing
- In piles without tubes where core drilling was performed
- In hollow driven piles
- In small diameter piles where there it is impractical to install multiple piles and the sonic test is inapplicable
In this method, both probes are lowered into the same tube, one above the other. As long as the probes are closely spaced, the first arrival will belong to the waves passing in the water filling the tube. At a larger spacing, the wave traveling through the surrounding concrete will arrive earlier, due to the higher wave speed. Thus, both FAT and energy provide information about the quality of the concrete around the hole (Figure 45).
The single hole ultrasonic method can find defects as thin as 3 cm if they surround the tube, or defects 10 cm thick at a distance of 7 cm from the tube (Amir 2002). A word of caution: it is impossible to apply the method in steel tubes, since they intend to refract the waves towards the horizontal (Figure 47). In fact, the FAT recorded in steel tubes will represent the propagation in the water filling the tube.
Sample Specifications

Introduction: For ultrasonic testing, at least two access tubes are cast in the pile. An ultrasonic transmitter is lowered in one of the tubes and a receiver - in the other. Both transducers are connected to a computer that plots the time of first arrival FAT, as well as the energy, versus depth. As long as FAT and energy are approximately constant, the pile is considered acceptable. A local anomaly, in the form of late arrival or decreased energy, may mean that the concrete between the tubes is defective at this level. Installing a larger number of tubes along the perimeter of the piles will provide almost complete coverage of the cross-section.
Testing agency: The ultrasonic test shall be carried out by a firm experienced in this kind of work and approved by The Engineer. Both site work and interpretation of the results shall be carried out by a Geotechnical engineer with proven experience.

Equipment: The test shall be performed using a computerized system of reputable origin. All components shall be recently calibrated and in good working order. All software shall be of the latest released version.

Piles to be tested: All piles shall be tested at a minimum age of Seven days after casting, unless instructions to the contrary shall be given by The Engineer.

Access tubes: All tubes shall be new, straight and clean both inside and outside, with a minimum internal diameter of 50 mm. All connections shall be welded or glued externally, taking care that no welding material penetrates inside. Both ends of the tube shall be tightly plugged. The Contractor shall take all due care to avoid any damage to the tubes during manufacturing, installation, concreting and the trimming of the head.

Preparation for testing: Before commencing the test, the Contractor shall ensure that there is adequate access to the pile. The Contractor shall use a suitable saw to cut the top of the access tubes cleanly at a level of 200 to 300 mm above the concrete, unless ordered otherwise. The tubes shall then be tested for access throughout their length using a 1” steel bar as a dummy. Having passed this test, the tubes shall be completely filled with water.

Reporting: A final report for each testing stage shall be presented not later than three working days after completion of that stage. The report shall consist of a printout of the original output, as well a summary table including, for every pile tested, the depth and The Engineers' interpretation regarding its integrity.
Radioactive integrity testing

Radioactive testing of piles (Preiss 1971) was the first commercially available method for the integrity testing of piles. In this method, the probe hung by a cable is lowered into a steel tube prepared in advance. The probe (Figure 48) consists of the following components:

A weak radioactive source (Usually Cesium 137)
A lead shield
A photon detector (Geiger counter or similar device)
Electronic circuitry and computer

![Radioactive Probe (Schematic)](image)

Figure 48: Radioactive Probe (Schematic)

The source emits gamma rays (photons) to all directions, and these are backscattered by the surrounding material. When the rays hit an inclusion with a lower density, a larger proportion of them is backscattered and trapped by the counter. Thus, the photon count serves as a measure of the average density of the surrounding concrete: A higher count means lower density, and vice versa.
The instrument can be calibrated to give direct density readings, but here a word of caution is appropriate: The photon count includes the effect of all the elements surrounding the counter, including the tube and the water in the tube (if such exists). Therefore, the instruments should be calibrated separately for each tube diameter and wall thickness combination and for both wet and dry situations.

Theoretically, there is no limit to the range of gamma ray penetration. The radiation energy, however, decreases quickly with penetration. For Cesium 137, for instance, it is halved for every 50 mm penetration into concrete. Therefore, available radioactive pile testers have a typical range of only 50 to 75 mm around each tube (Davis & Hertlein 1994).

The main drawback of the radioactive method, in addition to the limited range, is the utilization of dangerous material. The use of radioactive material is regulated in all civilized countries, and operating such an instrument for pile testing normally requires a special license.

The main advantages of the radioactive method over stress-wave methods are as follows:

It is possible to test the pile immediately after concreting

The method is suitable for acceptance of pile grouting, where stress-wave methods fail.

In spite of these advantages, the radioactive method is rapidly losing ground due to the great progress made in cross-hole ultrasonic testing.
References

Amir, J.M. & Fellenius, B.H (1996): Discussion on “Small strain sonic pile tests - the need for caution” by K.Y. Wong, Ground Engineering Vol. 29 No.1


Komornik, A. & Amir, J.M.: Quality Control at Pier NB-2, Proc. 5th Intl. Conf. DFI, Bruges,


ASTM (2008): Standard test method for integrity testing of concrete deep foundations by ultrasonic crosshole testing, Designation D6760-08, Philadelphia PA


Ellway (1993): Objectives of competition are unclear, Ground Engineering, June p. 8


Stain, R (1993): Test’s integrity is questionable, Ground Engineering January/February, p. 7
Stain, R (1993a): Competition was not applicable, Ground Engineering, April p. 15