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Keynote lecture: Technology of pile dynamic testing

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ABSTRACT: Because of the recent larger availability and higher performance of pile testing and monitoring equipment, pile dynamic testing has become part of many present day civil engineering projects. This report covers the past and state-of-the-art technological aspects of pile dynamic testing: testing methods, loading equipment, and measurements, including their acquisition and interpretation. Three major pile dynamic testing methods are distinguished based on means and objectives: high-strain testing performed primarily for bearing capacity, low-strain testing performed primarily for integrity, and high-strain kinetic testing performed for bearing capacity. Historical and recent references are provided on each topic listed.

1 INTRODUCTION

Technological aspects of pile dynamic testing have been rapidly evolving over the past 12 years, although not quite so fast as the interpretation and modelling aspects. This reflects the higher challenge of creating practical rather than theoretical breakthroughs in the area of pile dynamic testing. It has been observed that theoretical developments result from innovative testing and measuring approaches. At this juncture, it is anticipated that higher quality measurements will be one of the key aspects of the development of testing practice over the next decade.

This paper attempts to summarize the present, state-of-the-art technological aspects of pile dynamic testing. After discussing some fundamentals of pile dynamic testing, the following are reviewed: testing methods, loading equipment, and measurements, including their acquisition and interpretation.

Pile dynamic testing may be defined as the testing of piles using dynamic effects (i.e., generating a force or stress within, outside, or at the boundary of a pile through the intervention of mass and acceleration). The most common dynamic interaction between a pile and an accelerated (or decelerated) mass occurs during pile driving, which has prompted the application of stress wave theory to piles. According to the one-dimensional formulation of the phenomena occurring during such an impact, waves travelling downward (↓) and upward (↑) can represent the behavior of the pile (De St. Venant, 1867; Isaac, 1931).

These waves travel at speed c, given by the expression $c = \sqrt{E/\rho}$, where E and ρ are Young's modulus and the specific mass, respectively, of the pile material. The number and complexity of the waves depend on the changes of the cross section of the pile and on the interaction or the pile with the surrounding medium at the pile boundaries (i.e., at the pile head, along the shaft, and at the toe). Essential aspects of these interactions described in detail elsewhere (Goble, et al., 1975) are summarized in Figures 1 and 2 for ease of reference.

The interaction of a mass M hitting the pile head with an impact velocity v_i can be described by the equivalent system shown in Figure 1a. The pile can be represented by a dashpot with a damping factor equal to the pile impedance: $I = \rho cA$, where A is the cross-sectional area of the pile. The cushion protecting the pile head is assumed to be elastic, with a spring constant k. Although the equivalent model of Figure 1a is strictly applicable to a half-infinitely long free pile, it provides reasonably accurate force levels and approximate downward waveforms for most practical cases. Non-dimensional sample results derived from the analysis of the equivalent mechanical system are presented in Figure 1b.

More extensive solutions and charts relating to the mechanical equivalent system shown in Figure 1a have been published (Parola, 1970; Van Koten, 1977; Holeyman, 1984) and can be used to quickly pre-engineer M, v_i, and k with respect to pile impedance, in order to generate the compressive pulse of the desired amplitude and duration, as discussed in more detail in Section 3.2. These solutions establish that the non-dimensional peak amplitude F_{max}/Iv_i decreases essentially as $2I/\sqrt{kM}$ increases, whereas the duration of the impact decreases proportionally with $\sqrt{k/M}$, as

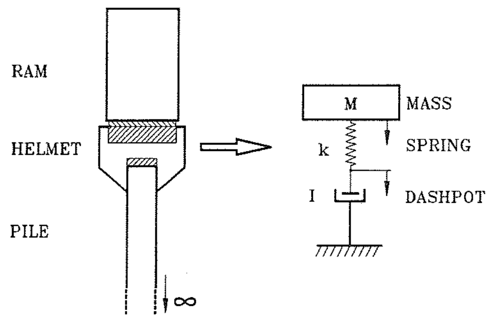


Figure 1a Mechanical Equivalence

indicated in Figure 1b. For waves of very long durations, the pile mechanical behavior should be completed with a spring parallel to the dashpot, representing the equivalent pile static behavior.

Figure 2 illustrates the interaction of a short (relative to the pile length L) downward compressive wave with a component of the shaft resistance at depth z* and with the toe resistance at depth L. For the sake of this illustration, the mobilized shaft resistance Q_F and mobilized base resistance Q_B have been assumed to be constant over the duration of the wave. It can be noted that, upon the passing of the initial downward compressive wave through level z*, two waves are being generated: an upward compressive wave of amplitude $Q_F/2$, which is a newly created reflected wave, and a downward tension wave of amplitude - $Q_F/2$, which combines with the initial

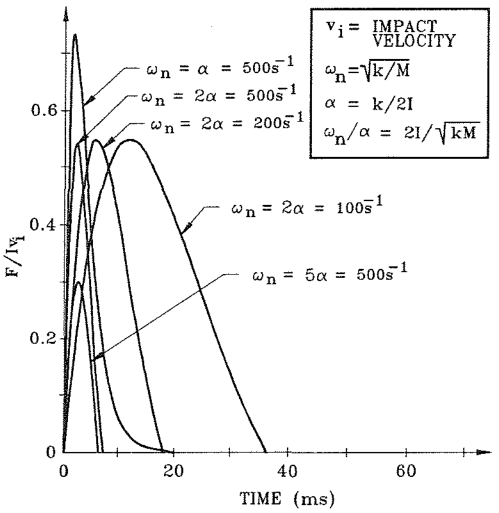


Figure 1b Non-dimensional Force Pulses

downward wave. Upon reaching the pile toe, the resulting downward wave is reflected upward and reversed (i.e., compression becomes tension), with a compressive offset corresponding to the mobilized toe resistance Q_B. On its way back up toward the pile head, the wave again interacts with the shaft friction at depth z* and reaches the pile head after time t = 2L/c.

From this simplified conceptual representation, which can be generalized to the case of a continuous shaft resistance along the whole shaft, it can be

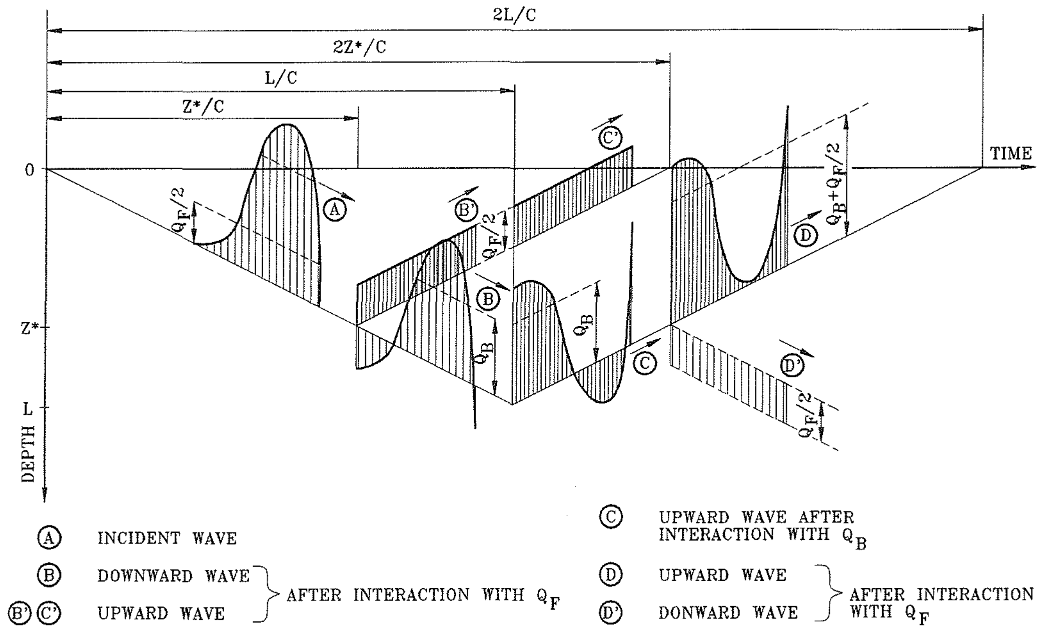


Figure 2 Sets of Waves in Dynamically Loaded Pile

Table 1. Typical Key Attributes of Different Types of Pile Tests

	Integrity Testing	High-Strain Dynamic Testing	Kinetic Testing	Static Testing
Mass of Hammer	0.5 - 5 kg	2,000 - 10,000 kg	2,000 - 5,000 kg	N/A
Pile Peak Strain	2 - 10 μstr	500 - 1,000 μstr	1,000 μstr	1,000 μstr
Pile Peak Velocity	10 - 40 mm/s	2,000 - 4,000 mm/s	500 mm/s	10^{-3} mm/s
Peak Force	2 - 20 kN	2,000 - 10,000 kN	2,000 - 10,000 kN	2,000 - 10,000 kN
Force Duration	0.5 - 2 ms	5 - 20 ms	50 - 200 ms	10^{7} ms
Pile Acceleration	50 g	500 g	0.5 - 1 g	10^{-14} g
Pile Displacement	0.01 mm	10 - 30 mm	50 mm	> 20 mm
Relative Wave Length	0.1	1.0	10	10^{8}

observed that shaft resistance effects can be perceived firsthand at the pile head through upward waves of amplitudes $Q_{Fi}/2$. The distribution of the shaft resistance along the pile depth can be readily interpreted from the time development of the upward reflected waves up to time 2L/c. On the other hand, waves received after 2L/c result from several interactions: shaft resistance on the way down, reversal at toe, toe resistance, and shaft resistance on the way up. These waves are generally more difficult to interpret because they integrate the effects of several depth- and time-dependent variables. A major advantage of the dynamic nature of the loading is that depth-dependent information can be obtained from time-dependent, single-point measurements at the pile head. This is clearly not the case for static testing, where evaluation of the shaft resistance distribution requires measurements at several depths.

Resolution of the shaft resistance terms versus depth (depth resolution) is afforded by the sharp increase of the force at the wave front and by the short length or duration of the original waveform. The sharpness of the wave relative to the pile characteristics can be used as a criterion to separate different types of "dynamic" pile tests. Table 1 provides a summary of key attributes of several known pile test types. Of particular significance to this discussion is the relative wave length Λ, which represents the length of the force pulse in terms of the double length (2L) of the pile. It can be noted from Table 1 that integrity testing is typically characterized by a relative wave length of 0.1, which provides for maximum depth resolution. The dynamic bearing capacity test is typically characterized by a relative wave length of 1, which still allows for depth resolution while providing high-strain testing.

Longer-duration impacts, such as generated by the Dynatest (Gonin et al., 1984) or the Statnamic Test (Bermingham and Janes, 1989), are characterized by a relative wave length Λ of 10 or higher and, therefore, do not allow for depth resolution. It is suggested that, although those tests resort to inertial actions on masses to generate their extended force pulse, they be referred to as "kinetic tests" mainly because the inertial forces within the pile are small compared to the current force being applied and because the interpretation of these tests does not and cannot make use of the wave equation framework.

Figure 3 provides a representation of the pile tests available in terms of relative wave length Λ and of strain level. Figure 3 also presents typical relative wave lengths required to reach 90% consolidation around a pile in sand, silt and clay. This diagram allows, in the writer's opinion, the separation between dynamic, kinetic, and static testing. Compared to static tests, one is faced with the difficulty in kinetic tests of sorting out the velocity dependency on the soil resistance, and in dynamic tests of resolving dynamic

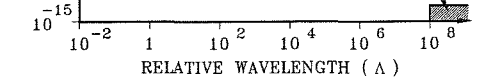


Figure 3 Sharpness and Duration of Force Pulse for Different Pile Tests

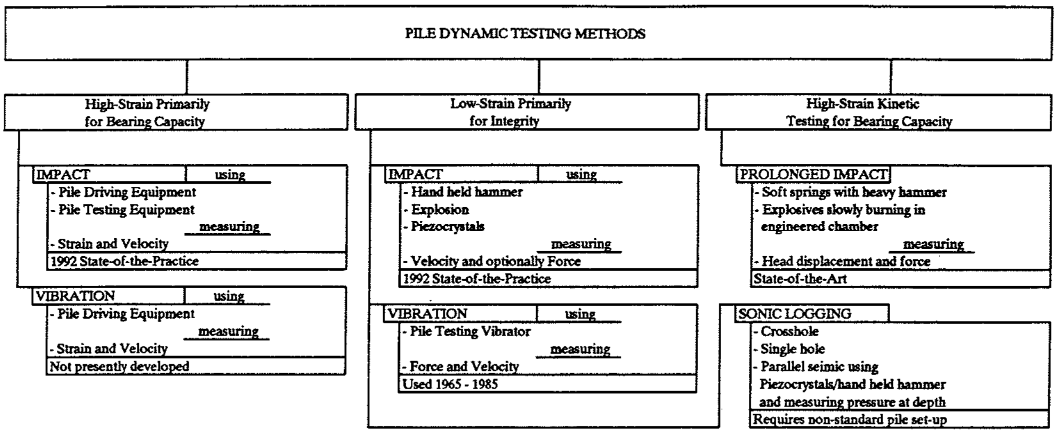


Figure 4 Summary of Pile Dynamic Testing Methods

effects with, however, the advantage of depth resolution.

2 TESTING METHODS

The ideal force wave should be sharp, short, and of high intensity, while ideal measurements should be accurate over a large frequency range. Idealized interpretations and models are discussed in the keynote lecture by Randolph (1992) in these proceedings. Practical constraints limiting the attainment of these ideal features are material strength limitations, energy, mass, and cost limitations; safety; and ease and rapidity of interpretation. As a result of differently biased compromises, several dynamic testing methods have been developed. These methods, which are summarized in Figure 4, are reviewed in the following paragraphs.

2.1 *High-Strain Dynamic Testing*

The most common high-strain dynamic testing involves dropping a mass on the head of a pile that has been cushioned for that purpose. The pile head is monitored during the impact to obtain force and velocity as functions of time (see Figure 5). This test is well documented in the relevant literature: it was one of the first testing applications of driven pile dynamic monitoring as presented in the early days by Goble and Rausche (1970) and is addressed by ASTM Standard D-4945-89 (1989). The primary objective of this type of test is to evaluate the load-bearing behavior of the pile under axial static loading. The load-bearing behavior may be summarized by an allowable load or described by a complete load-movement curve generally derived from distributed shaft and toe resistance terms. The load-bearing behavior requires the exploration of high strains; pile

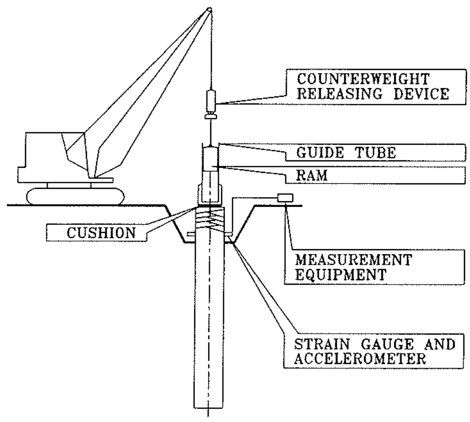


Figure 5 Typical High-Strain Dynamic Test Setup

material strain during that test has a typical maximum value of 500 to 1,000 microstrains (μstr).

Primary difficulties and limitations associated with high-strain testing are the decoding of dynamically mobilized resistance measured during the test into static resistance and the limited transient displacement enforced by the impact. Conversion of dynamic resistance into static resistance is rendered difficult in part because of the following effects:

- Inertial and radiation-damping effects, which are frequency-dependent,
- Differences in the deformation pattern along the shaft and at the base between dynamic and static loading,
- Effect of pore-pressure generation and dissipation, and
- Dependence of the soil's modulus and shear strength on velocity.

For driven piles monitored during driving, one must also contend with the effects of cyclic pore pressure generation and soil setup (or relaxation). Also, and less often mentioned, reliability problems of measurements, especially of the force for cast-in-place piles, and velocity and displacement in general must be contented with. Finally, the development, commercial success, and persistence of early simplistic models, which still represent the bulk of the practice, have deterred most end users from addressing the complexity of the phenomena at hand.

High-strain vibration, although easily implementable in practice, has not seen many applications. Vibrators are regularly used to install sheet piles; however, in that case, axial capacity is not usually a primary concern. Also, vibrations imply cyclic loading, which generates an additional difficulty in the interpretation because of pore-pressure generation and fatigue effects.

Very rare examples of non axial loading have been reported. They relate essentially to lateral loading, rocking, and twisting. These tests are generally not interpreted to evaluate the elementary soil behavior, but rather to verify specific pile dynamic performance criteria.

2.2 *Low-Strain Dynamic Testing*

The most common low-strain dynamic testing involves hitting the pile head using a hand-held hammer and monitoring the pile head to obtain its transient velocity, and optionally the impact force. This test is well documented, but is not, to the writer's knowledge, the object of an official standard. The primary objective of the low-strain dynamic test is to assess the integrity of the pile as a structural member. Anomalies that impair the integrity of a pile and that are expected to be identified by integrity tests include the presence of material of poorer quality than expected (locally and overall) and variations in the cross section of the shaft (e.g., crack, necking, and bulb). Additionally, some idea of the pile and soil behavior at low-strain may be inferred. Because the primary information offered by the test is the manner in which waves travel and are reflected within the pile material, pile material strain during those integrity tests has a typical maximum of only 2 to 10 μstr.

Primary difficulties associated with low-strain integrity testing are:

- Test repeatability (improved to some degree by signal averaging),
- Elimination of spurious vibrations (in hammer and Rayleigh wave effects),
- Discrimination between soil resistance and shaft impedance effects,
- Difficulty in identifying gradual changes in shaft section,
- Masking of potential necking below bulb,
- Overall historical distrust of engineering community towards results, and
- Absence of simple, quantitative and rational interpretation method.

Methods of imparting an impact at the head of a pile are further discussed in Section 3.5 and 3.6.

Of historical interest is the application of maintained vertical vibration at the head of a pile, with a view to obtaining the cyclic mobility (inverse of mechanical impedance) of the pile's head at various frequencies (Davis and Guillermain, 1979). Although experiments using a vibrator are still reported, the "impedance test" is now generally administered using a short impact generated by a hand-held hammer, and converting the collected transient time signals into the frequency domain, generally using a Fast Fourier Transform (FFT).

Other low-strain methods are used to investigate the integrity of piles, although not exclusively relying on the transmission of longitudinal waves. These are the Parallel Seismic Testing, Crosshole Seismic Logging, and Single Hole Seismic Logging (Stain, 1982). These three methods require the provision of casings outside or within the pile shaft.

Parallel Seismic Testing (see Figure 6a) is typically used when the pile head is not accessible. A bore-hole is drilled immediately adjacent and parallel to the pile, and a slotted tube is installed. The boring is usually drilled to within 1 meter (m) of the shaft and at least 3 to 5 m deeper than the presumed pile depth. The cased hole is filled with water, and a hydrophone is lowered down the hole to monitor, at regular depth intervals (typically 0.5 m), the water pressure wave resulting from the impacts imparted on a structural element directly connected to the pile head. Wave arrival time delays are plotted versus depth in order to identify the deep foundation bottom.

Crosshole and single-hole seismic logging are typically used to evaluate the concrete condition of drilled shafts and slurry walls. Casing within the pile generally consists of water-filled tubes attached to the rebar cage before the casting of concrete. Ultrasonic pulses are generated by a piezoelectric motion generator (source), and the resulting water pressure waves are recorded by a hydrophone (receiver). Pulses have a typical duration of 50 microseconds (μs) and result in a concrete strain on the order of 0.1 μstr. As shown in Figure 6b, crosshole logging is performed by simultaneously lowering source and receiver into separate tubes; single hole logging is performed by lowering a source/receiver assembly, separated by a fixed depth interval, into a single hole. Wave arrival time delays and amplitudes are interpreted with a view to identifying zones with poor quality concrete, voids, intrusions, and breaks.

Difficulties and present limitations associated with seismic logging are:

- Planning requirement and interference with construction process,
- Control of casing positions,
- Quality of mechanical contact between tube and concrete,

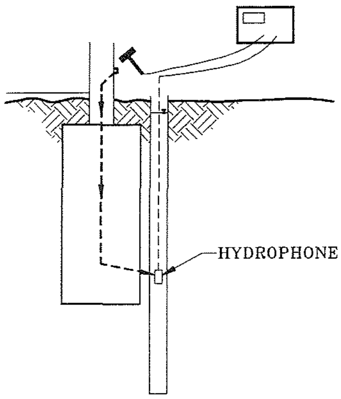


Figure 6a Parallel Seismic Testing (Stain, 1982)

- Defect must fully separate receiver from source (i.e., defect boundary must ideally intercept casing to be detected), and
- Qualitative more than quantitative interpretation.

2.3 *High-Strain Kinetic Testing*

High-strain kinetic testing involves the upward decelerating and/or accelerating of a mass and using its inertial force to generate an axial fast push (or a prolonged pulse) at the head of a pile. Loading equipment as described in the reviewed literature is discussed in Section 3.2 Pile-head force and movement are typically monitored during the event. The duration of the force pulse is on the order of 100 to 200 milliseconds (ms) and thus long enough for the waves to travel back and forth several times (typically 10 to 20) within the pile. As a result of the progressivity and duration of the force pulse, inertial effects on the pile and surrounding soil are regarded as minimal by promotors of this type of testing (Gonin et al., 1984, Bermingham and Janes, 1989). Under typical kinetic testing conditions, resistance distribution cannot be resolved versus depth. Pile load test results are directly based on recorded force and movement at the pile head. Present difficulties and limitations associated with kinetic testing of piles are:

- Differences in deformation pattern along the shaft and at the base between kinetic and static loading,
- Effects of pore-pressure generation and dissipation,
- Dependence of soil resistance on velocity, and
- Inability to resolve depth effects and, therefore, reliance on a single global measurement.

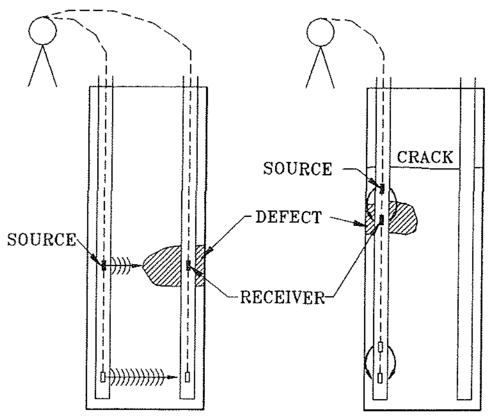


Figure 6b Crosshole and Single Hole Seismic Logging

3 LOADING EQUIPMENT

3.1 *Pile Driving Equipment*

Virtually any type of pile driving equipment can be used in conjunction with dynamic monitoring at the end of driving (EOD). If the energy delivered by the driving equipment is sufficient to drive the pile satisfactorily to the design depth and bearing capacity, it can be presumed that the peak force is at least twice the soil resistance at the end of driving.

When testing a driven pile upon re-strike, its bearing capacity may have increased and a heavier hammer may be appropriate to enforce a sufficient pile displacement. When testing a cast-in-place pile, a number of conditions must be satisfied, as discussed in Section 3.2, that render the use of standard driving equipment generally impractical for testing that type of pile.

The pile driving equipment most suited to high-strain dynamic testing is a system that allows for a maximum of flexibility and control. Ideally, the system should be able to deliver single blows of predetermined energy. Also, easy modification of the mechanical characteristics of the helmet may be desirable. The ability to deliver single blows is required in order to avoid multiple blows that would otherwise cyclically alter the soil resistance. The ability to deliver blows of predetermined energy is desirable when conducting a sequence of blows with increasing energy. These factors make the air, steam, and diesel hammer least suited for dynamic testing. Winch-operated drop hammers and hydraulic hammers, on the other hand, possess the desirable features mentioned above.

3.2 *High-Strain Pile Testing Equipment*

As discussed in Section 1 and illustrated in Figures 1a and 1b, the force pulse resulting from the impact of a

Table 2. Energy Levels for High-Strain Testing

Pile Considered	Timber Ø 300 mm	Steel H-beam 14" x 73	Percast Concrete ◫ 300 mm	Closed-end pipe 406x 375 mm	Bored Pile Ø 1m
Typical working load (MN)	0.4	1.5	1.0	2.0	4.0
Impedance (MN/ms^{-1})	0.3	0.56	0.9	0.8	7.8
2.5% Ø Displacement (mm)	7.5	4 (*)	8.5	10.2	25
Net Momentum Required to Achieve 2.5% Ø Displacement (kg x ms^{-1})	2,250	2,250	7,600	8,200	195,000
Example of Mass x Drop Height (kg x m)	900	900	3,000	3,240	77,000

(*) assumes no soil plug at pile toe

hammer on the head of a pile depends on a number of key parameters:

- M = mass of impacting hammer,
- v_i = velocity of hammer at the beginning of the impact,
- k = spring constant of the helmet, or compression characteristics of the helmet if non-linear, and
- I = impedance of the pile.

The desirable maximum level of the force pulse should be lower than the allowable dynamic structural capacity of the shaft in compression but higher than half the soil resistance to be mobilized. A longer duration of the blow is preferred up to once or twice the return period of the waves in the pile. Within the above-mentioned constraints, a blow with the highest energy should be sought because it would impose a larger transient penetration of the pile.

The charts presented in Figure 7, resulting from straightforward integration of the wave equation for a free half-infinitely long prismatic elastic pile, can be used to address both aspects of maximum force and energy transfer to a pile during impact. The theoretical maximum transient displacement u_{max} can be obtained using the energy transfer function η:

$$u_{max} = \eta \, Mv_i/I \qquad (1)$$

Equation (1) would imply that for a given set of M, k, and I, the maximum transient penetration would be proportional to the impact velocity.

Table 2 provides, for a number of typical cases, the energy levels required for given displacements and in particular for a pile head displacement of 2.5% of the pile diameter, which, in the writer's opinion, represents a reasonable objective. As has been observed from pile-driving experience, larger penetrations for a given energy are obtained through the use of heavier hammers, while softer cushions allow the increase of allowable energy to a given pile without breaking it.

Several pile dynamic loading systems have been developed with the objective of operating with minimum field equipment (at most, a 30-ton crane) and with a quick setup procedure that typically allows 4 to 10 tests to be performed in a single day. The main concern when dropping heavy masses on piles that have a nominal capability to resist overturning

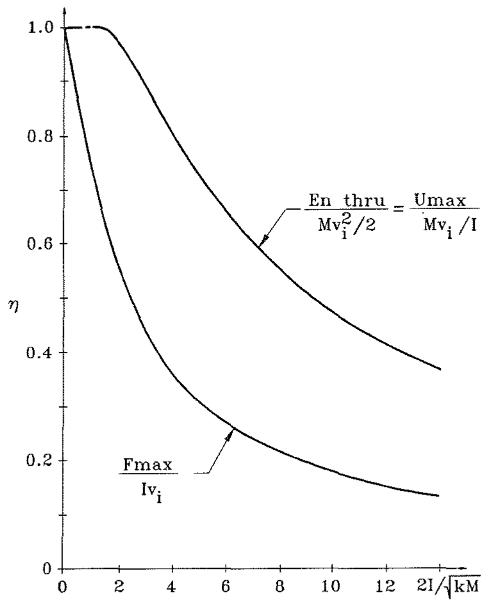


Figure 7 Peak Force and Energy Transfer (Holeyman, 1984)

forces is the safe control of the mass during fall, impact, and rebound. It is, therefore, required that the mass travel be guided by some mechanically restrained system.

The guiding system is provided either by independent crane leads, as used for pile driving, or by a structural element generally connected to the pile. Structural guides connected to the pile load may be either internal (for donut hammers) or external (for solid hammers). Precautions must be taken to prepare and design the pile head so as to generate a vertical and centered drop, and to minimize lateral and overturning efforts. For cast-in-place piles, this generally requires that a well finished concrete surface be provided by casting an additional custom-made concrete pile head.

Van Koten and Middendorp (1980) reported on a 1,000-kilogram (kg) donut-shaped hammer allowed to free fall from a drop height of approximately 1.5 m (see Figure 8). They also reported on a 100-kg hammer that could be accelerated downward by an air-pressure chamber resting on the pile head; the downward movement of the pressurized hammer was triggered by the breakage of a calibrated mechanical fuse (machined bolt) when the design pressure was achieved. Nianci (1980) reported on a truck-mounted hammer weighing 1,500 kg (see Figure 9) with a release mechanism allowing free fall up to 1.5 m. These early systems were limited to a testing capacity of 1.5 Meganewtons (MN).

Higher capacity systems are required to perform high-strain dynamic testing of large-diameter drilled shafts. Rausche and Seidel (1984) reported on a loading system capable of dropping a 20-ton mass from up to 2.5 m onto drilled shafts of 1.1 to 1.5 m in diameter and approximately 40 m long. Peak forces in excess of 30 MN were generated by that system; these forces corresponded to a transient displacement of approximately 20 millimeters (mm). A unique dynamic load test, performed at the bottom of the bored pile prior to casting concrete, was presented by Magnusson, et al. (1984). In this procedure, the soil at the pile toe was tested while being improved by the dropping of a 4-ton hammer from a drop height of up to 4 m. Peak forces on the order of 3.5 MN were generated after 12 ms of impact and under a displacement of approximately 20 mm. A diagram produced by Berggren (1981) allows, according to the author, the conversion of the dynamic mobilized resistance to the equivalent static resistance based on the time required to reach maximum load and the soil shear wave speed.

The loading system used during the prediction tests organized for the Belgian Symposium on Pile Dynamic Testing (Holeyman, 1987a) was a 4,000-kg externally guided hammer, which could fall from 2 m (see Figure 10). Maximum forces generated by this system onto a variety of 1 MN allowable load piles, were on the order of 3 MN after approximately 4 ms of impact. Piles with an approximate impedance of 1 MN/ms^{-1} incurred a typical transient displacement in

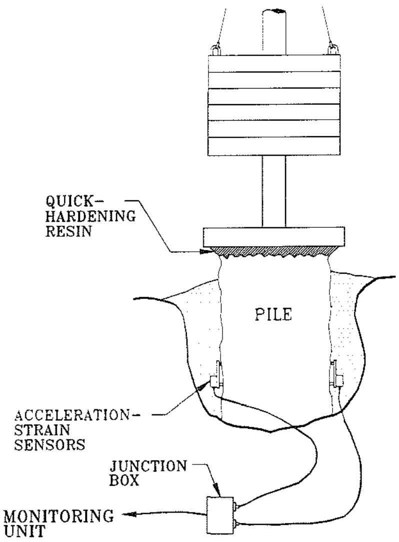


Figure 8 Donut Hammer (Van Koten and Middendorp, 1980)

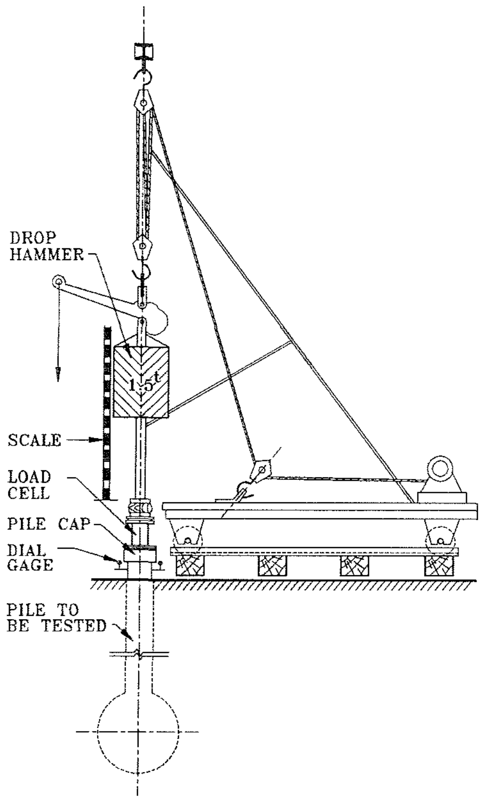


Figure 9 Chinese Loading System (Nianci, 1980)

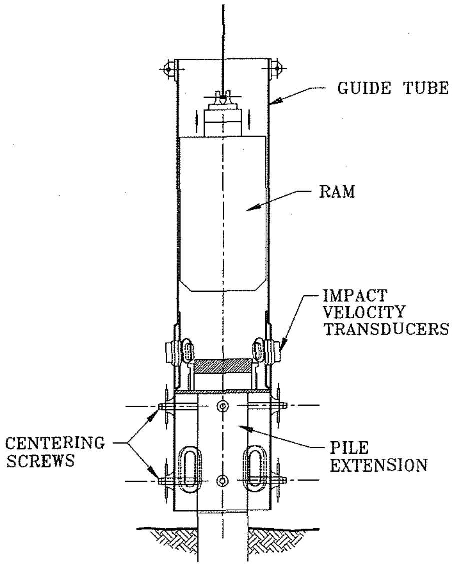


Figure 10 Belgian Loading System (Holeyman, 1987a)

the order of 10 mm.

Cushions used for pile dynamic testing consist of the materials currently utilized for pile driving: wood, plywood, and various types of plastic. Engineered materials are preferable because they retain their initial elastic properties better. It is the writer's opinion that too little engineering has been devoted so far to testing cushions, especially when considering that fatigue is no longer a stringent requirement. Some merit might be found in using impact force modulators such as prestressed spring caps (Iwanoski and Berglars, 1984), nitrogen loaded caps (Jansz et al., 1974), or hammers with engineered deformation properties (Fischer, 1961).

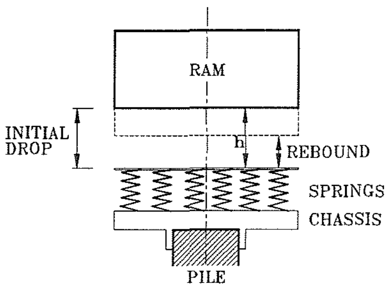
3.3 *Pile Kinetic Testing Equipment*

The drastic reduction in the cushion stiffness of pile dynamic testing equipment discussed above is a feasible way to lengthen the duration of the force pulse. Gonin et al. (1984) have developed a system in France called "Dynatest" that uses soft coil springs for that purpose. The loading equipment is installed on a small tracked vehicle, with a ram weight of 15,000 kg (see Figure 11). After surfacing the top of the pile with quick-setting concrete, the ram weight is raised by two jacks to a height that can vary from 0.1 to 1.4 m. The ram falls freely onto the springs placed on the pile head, rebounds, and is picked up by an automatic system. The Dynatest apparatus is made of two groups of springs to accommodate small and large drop heights. The force pulse is rated at 4 MN maximum and typically lasts 100 to 150 ms (see Figure 11). Reported pile load test results indicate a maximum load of approximately 3 MN for a maximum pile load movement of 20 mm. The Dynatest was developed with a view to quickly check a large proportion of the piles installed at a given site; as such, it entails simple measurements during the test: pile displacement, spring compression, and rebound height.

The controlled combustion of fuel within a pressure chamber is another feasible way to generate a long force pulse. Bermingham and Janes (1989) have developed a system in Canada called "Statnamic", which uses rapidly expanding gases as a soft cushion to transfer forces to the pile head. The Statnamic apparatus consists of a reactive mass placed over a pressure chamber atop a pile to be tested (see Figure 12). Fuel is burned within the pressure chamber, creating large pressures that drive the reaction mass upward at high velocity and push the pile downwards. The total height to which the reaction mass is propelled can reach 2 m. A pressure transducer located in the pressure chamber of known diameter is used to monitor the force pulse while pile head movement is monitored by use of a laser level. The force-time history (shape and length of the pulse) can be modulated by varying the following parameters:

- Reaction mass (typically 5% of test load),
- Amount of fuel, and
- Physical characteristics of pressure chamber (diameter, stroke/or length before gas freely escapes) and venting system.

Test results reported to date indicate a maximum



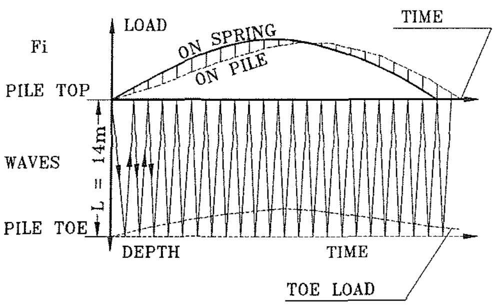


Figure 11 "Dynatest" Apparatus and Load Diagram (Gonin et al., 1984) load of 3.1 MN, using a 15,000 kg-reaction mass, for a maximum pile movement of 50 mm. A typical duration of the force pulse is 100 ms, which is progressive enough to keep acceleration levels within the pile below 1 g. A 30-ton crane is required to set up the equipment atop a pile. Piles with a batter of 1 in 6 have been tested.

3.4 *Hand-Held Hammers*

Hand-held hammers used to hit the pile head for integrity testing typically vary in mass from 0.5 to 5 kg. For a given hammer mass, the intensity and duration of the force pulse will depend essentially on the velocity at impact and the stiffness of the impacting end or tip. Modally tuned impact hammers are available from manufacturers of shock and vibration equipment. Modal tuning eliminates spurious glitches in the force frequency spectrum and provides a "cleaner" blow. Special hammers generally come with tips that allow for soft, medium, and hard hitting, and contain a dynamic force transducer to monitor the impact force.

Electric impact hammers are also available from manufacturers of shock and vibration equipment. These hammers provide a controlled, repeatable, and operator-independent input force typically ranging from 5 to 5,000 Newtons (N). However, they require to be powered by mains and are therefore not convenient for testing on a construction site. Experienced integrity testors also use hardware-store hammers with plastic ends that provide a satisfactory

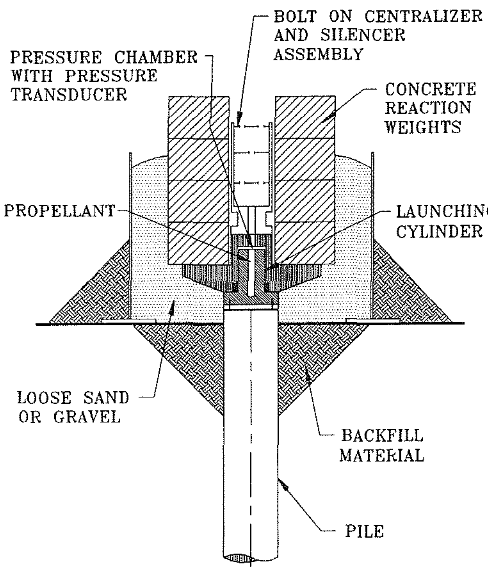


Figure 12 "Statnamic" Apparatus (Bermingham and Janes, 1989)

pulse. In the latter case, the impact force is monitored from an accelerometer mounted within the hammer.

Better records are generally obtained by tapping, rather than violently hitting, the head of the pile with the hammer. Spurious vibrations are usually generated within the hammer if it is manipulated too briskly and if the operator's hand forces the hammer down during contact. Repeatability of records should be ascertained because variations may occur depending on where the blows are given on the pile head and where the motion sensor is placed.

The optimum amount of impact energy, and therefore the choice of hammer, may depend on the type of pile, soil conditions, and main objective of the test. Heavier hammers tend to produce blows that contain lower frequencies, leading to lower dispersive effects within the pile, and are generally better suited to identify the toe of a long pile. Lighter hammers, however, produce sharper pulses that tend to provide more contrast in the waves reflected along the pile shaft (i.e., finer depth resolution).

3.5 *Testing Vibrators*

Vibrators generally used to test piles are of the electromagnetic and piezoelectric types. Electromagnetic shakers operate similarly to a common loud speaker. A coil is driven within a permanent magnet field. The dynamic electromagnetic coil field causes the motion of a heavy mechanical component. In most models, the coil is attached to the structure. The heavy ring-shaped magnets are suspended and oscillate around the coil. The electromagnetic shaker generates force in proportion to input current.

Piezoelectric shakers utilize ceramic disks, which change thickness proportional to an applied voltage. These disks are sandwiched between a heavy mass and a light fixture that attaches to the test pile. Although the displacement is very small, the use of multiple disks and high drive voltages (several kV) can produce large forces.

Electromagnetic vibrators may cover a frequency range of 5 to 2,000 Hertz (Hz), while piezoelectric vibrators cover a frequency range of 200 to 20,000 Hz. Peak force is generally on the order of 100 to 1,000 N. When vibrations are generated by a variable-speed eccentric mass, the only operational parameter is the frequency; the peak force increases with the square of the frequency. For reasons explained in Section 2.2, vibrators are utilized less and less as they become replaced by the simpler hand-held hammer.

3.6 *Explosion*

Zhang Yong-Qian et al. (1980) have presented an

Table 3. Desirable Amplitude and Frequency Ranges for Instrumentation

	High-Strain Testing	Low-Strain Testing	Kinetic Testing
Strain	1,000 μstr @ 0 - 5,000 Hz	NA 10 - 10,000 Hz	NA 0 - 100 Hz
Velocity	5 m/s @ 0 - 5,000 Hz	0.1 m/s @ 10 - 5,000 Hz	0.5 m/s @ 0 - 100 Hz
Acceleration	500 g @ 0 - 10,000 Hz	10 g @ 10 - 10,000 Hz	NA
Displacement	50 mm @ 0 - 2,000 Hz	NA NA	50 mm @ 0 - 50 Hz

interesting method for generating short pulses at the head of a pile. As illustrated in Figure 13, the method involves the release of electrical energy stored in condensers into a liquid contained on top of the pile. The high voltage and current impulse discharge at the immersed spark point generates an exploding gas bubble and thus produces a high-pressure pulse in the water. The fluid pressure pulse is converted to a stress wave at the top of the pile. The exploding pressure of the chamber can be calculated from the electrical input energy, which can be controlled by features of the electrical circuit and, in particular, of the condenser. Although reported results relate to integrity tests, the authors are confident that forces up to 0.8 MN could be generated for high-strain tests, using a condenser of 66 microFarads and an electrical tension of 20 kilovolts (kV).

3.7 *Piezoelectric Cells*

Piezoelectric material changes dimensions when subjected to an electrical field. By assembling a number of piezocrystals in series and subjecting them to an electrically controlled voltage pulse, displacements on the order of 0.1 mm and velocities on the order of 10 mm/s can be generated. The magnitude of the pressure or stress generated depends on the type of medium (fluid or solid) surrounding the crystal, and the boundary conditions of the crystal assembly. Piezoelectric wave generators are used in water for sonic logging as discussed in Section 2.2. Typically, the amplitude of the pressure pulse is a few kiloPascals (kPa), with a duration that can vary between 0.010 and 0.100 ms. Piezoelectric vibrators may also be used for integrity testing of piles. The writer is not aware, however, of the piezoelectric generation of impacts for low-strain pile testing.

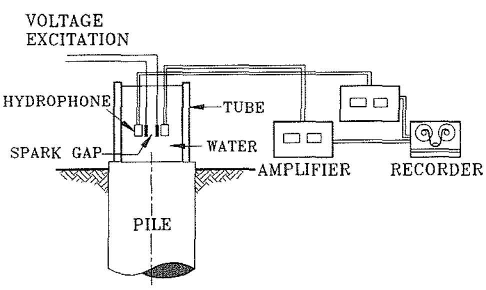


Figure 13 Hydroelectric Method (Zhang, 1980)

4 MEASUREMENTS

As discussed in Section 1.0, quality measurements are at the source of quality interpretation and quality models created to explain observed phenomena. This section deals with the measurement chain, which starts with a sensor sensing the measurand thanks to a physical set-up exploiting a natural law of physics, thereby converting a mechanical change into an electrical voltage difference. The electrical signal can then be read of a voltmeter, magnetically stored, or digitally converted for digital processing.

The measurement chain can be long, and its reliability and accuracy depend on the quality of each of its links. That is why it is, in the writer's opinion, vital that the end users of measuring equipment be fully aware of its working principles, details, and limitations. The first dynamic pile measurements were reported by Glanville et al. (1938); since then, instruments of higher reliability and complexity have been developed. However, the more complex the instruments, the more detail-oriented civil engineers or technicians need to be to keep reliability in line!

Two large classes of measurements can be identified for the purpose of reporting and interpreting pile dynamic tests: (1) measurements involving force, stress, pressure, or strain, which will be referred to as dynamic measurements; and (2) acceleration, velocity, and displacement, which will be referred to as kinetic measurements. As discussed in Section 5.0, a large number of interpretations are based on the force and velocity records collected at the head of the pile. Also discussed are the acquisition of measurements and their storage under a form that allows for their conceptual replay.

Because of the dynamic nature of quantities to be measured, a great deal of attention should be paid to the frequency range within which the measurement is sufficiently accurate (i.e, the "passing band") The passing band is characterized by low- and high-frequency bounds corresponding to a given loss in accuracy. The loss of accuracy can be expressed by a percentage(typically 5 or 10%) and in decibels (dB) (typically 1, 3, or 5 dB, which correspond to approximately 11, 29, or 44%, respectively). While some instruments are better suited to higher frequency ranges, they generally have poorer characteristics at lower frequencies. Perfect instruments do not exist and reasonable compromises that consider the most important frequency range of the phenomena to be measured must be generally accepted. Desirable amplitude and frequency ranges for different types of measurements are given in Table 3.

4.1 *Dynamic Measurements*

Force can be measured by using dynamometers (see load cell in Figure 9), which are "stand-alone" instruments that essentially convert the load acting on them into a measurable displacement or strain that depends on the mechanical structure of the dynamometer. Most commonly, force is derived from the measurement of strain at the pile head, as illustrated in Figures 5 and 8. Force can also be evaluated by measuring a pressure known to be representative of a pressure chamber of known dimensions. Finally, force can also be evaluated by monitoring the acceleration of the hammer.

A few cases of using a dynamometer to monitor force during pile driving, or high-strain testing have been reported (Fellenius and Haagen, 1969, Nianci, 1980). These instruments are based on a strain-gauge assembly bonded to a compressive steel element placed at the head of the pile. These systems have now been generally replaced by the more convenient and much more popular bolt-on transducer (Goble et al., 1975). Bolt-on strain transducers consist of a relatively small and flexible metallic structure that is bolted to the side of the pile section where the force is to be monitored. Several structural shapes of the transducer have been developed (circular, oval, diamond, or polygonal) in order to optimize strain amplification and strain gauge assembly (Beringen et al., 1980; Reiding et al., 1988). Sufficient flexibility of the transducer between the two points of attachment (typically 5 to 10 centimeters [cm] apart) is warranted to avoid modifying the section modulus of the pile, but more importantly, to avoid generating excessive force at the bolts that are attached to a concrete pile using expansive devices.

The bolt-on transducer, in contrast to strain-gauges directly bonded to the pile section, allows the bonding of the strain-gauges under ideal laboratory conditions and their thorough weatherproofing. The frequency range of the transducer is governed by the characteristics of the amplifier required to amplify the microvolts of the wheatstone bridge imbalance into more readily measurable volts. Most setups claim a frequency range covering DC (0 Hz) to 5,000 Hz.

In the case of the Dynatest, the force applied is derived from the compression of the springs. In the case of the Statnamic test, the force applied on the pile is monitored through a pressure gauge located in the pressure chamber.

Although present technology, and in particular the use of radio frequency transmitters, allows the monitoring of heavy hammer acceleration and deceleration (Holeyman, 1987b), the method of obtaining the impact force from acceleration measurements is confined in practice to hand-held hammers. These hammers are obtained by the custom incorporation of an accelerometer into the hitting mass (Likins, 1992). The force is obtained by multiplying the known mass of the hammer by the measured acceleration. The measurement of acceleration is discussed in Section 4.2.

So-called "impedance hammers" available from shock and vibration equipment manufacturers include a piezoelectric impact force transducer. The piezoelectric effect is the displacement of electrical charges within a crystal when strained by an external force. According to the law of electrostatics, the displaced electrical charges that accumulate at major opposing surfaces of the crystal form a voltage signal. The crystal elements in piezoelectric transducers perform a dual function: they act as a precision spring to oppose the applied force and supply an electrical signal proportional to their deflection. An

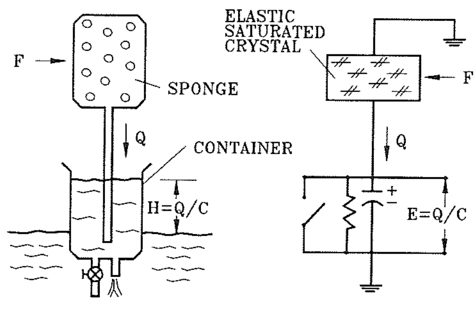


Figure 14 Piezoelectric Analogy

electrical circuit is required to amplify and transform the charge signal into a low impedance voltage signal that can be readily measured and recorded. The operation of this circuit, which is nowadays an integrated circuit lodged in the transducer mount, is based on the accumulation of electrical charges in a capacitor, forming a voltage according to the law of electrostatics. A resistor "slowly" discharges capacitance to eliminate drift by leaking off any static signal components. The electrical circuit and its hydraulic analogy are shown in Figure 14. It is important to observe that the piezoelectric transducers in their usual configuration do not retain a constant voltage and, therefore, are not able to measure a continuous or static stimulus. They work best in the high-frequency range. Some special force transducers are available with a discharge time constant of 2,000 s, which corresponds to a 5% low-pass frequency of 0.0003 Hz.

4.2 *Kinematic Measurements*

Motion of the pile under the impact can be monitored using accelerometers, velocity transducers, and displacement transducers. Motion of the hammer, which may not be used directly in the interpretation of the stress wave measurements, may be monitored through measurement of its position and/or velocity.

Accelerometers are the most commonly used instruments in pile dynamic testing to monitor the pile head movement under the impact. Velocity, which is the most desirable format for interpretation as discussed in Section 5.2, is obtained by direct integration of the accelerogram over time. Because of the transient nature and high-frequency content of the pile impact, piezoelectric transducers are most often used. Implementing Newton's law of motion, accelerometers measure the change of compression of a prestressed piezoelectric crystal resulting from the inertial effects of acceleration onto a seismic mass in contact with the crystal (see Figure 15). As discussed in Section 4.1, piezoelectric sensors work best at high frequencies but are not designed to pick up static components. In the case of piezoelectric accelerometers, this is reflected by a typical pass band of 1 to 5,000 Hz for a 5% accuracy, which corresponds to a discharge time constant of 0.5 s.

Because of instrument imperfections in the low frequency range, problems are typically encountered when integrating the signal. The result of the first integration may require an artificial zeroing at the end of the impact, based on the fact that the pile is supposed to have stopped moving after a certain time. The baseline for the second integration leading to displacement estimates is usually a straight line between the initial and final zero-velocity control points. The results of double integration are generally questionable beyond the maximum transient displacement, and a second artificial correction of the baseline is required to match an independent measurement of displacement. In spite of these shortcomings, piezoelectric accelerometers remain the most commonly used kinematic sensor for pile dynamic tests because of their low cost, ease of operation, and ruggedness.

Accelerometers, which are capable of measuring a static component, are of the piezoresistive type and of the variable capacitance type. Piezoresistive accelerometers have been used by some researchers (Legrand, 1986), but were found to be less rugged than piezoelectric ones. Newly developed variable capacitance accelerometers may, however, improve accuracy and ruggedness. The principle of operation of a variable capacitance microsensor is that varying acceleration causes a minute deflection of a sensing mass that changes the capacitance proportional to the acceleration input. The variable capacitance microsensor is constructed from three silicon elements bonded together to form a sealed assembly (Link, 1992). The middle element is chemically etched to form a rigid central mass suspended by thin membranes. It is fabricated from a single-crystal silicon and measures approximately 2 x 3 x 1 mm. The frequency response for a 5% tolerance is 0 to 1,000 Hz and 0 to 13,000 Hz for a low range (100 g) and mid-range (2,000 g), respectively. It is expected that those accelerometers would provide improved

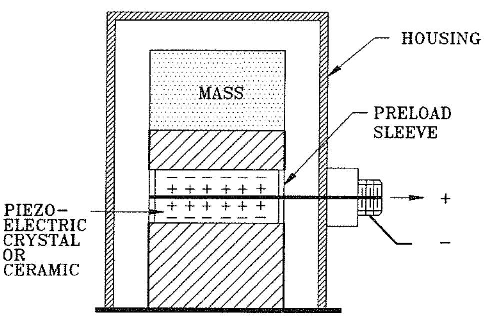


Figure 15 Piezoelectric Accelerometer

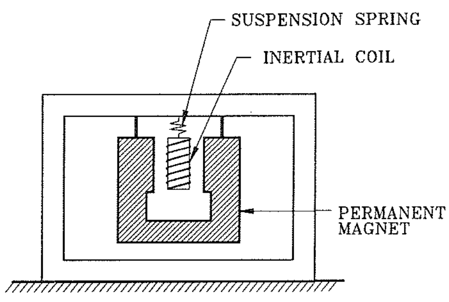


Figure 16 Electromagnetic Geophone

integration tolerance.

Velocity transducers should, in theory, be preferred to accelerometers and displacement transducers because they directly give the measurand utilized for interpretation. Velocity transducers produce their output via a coil moving through a magnetic field. The voltage induced in the coil is directly proportional to the relative velocity between the coil and the magnetic field. Small displacement (2 mm) velocity transducers consist of a coil suspended by springs and a permanent magnet, which is held by the case of the instrument (see Figure 16). This instrument, commonly called "geophone," can be connected to the top of the pile for integrity tests which, as discussed in Section 2.2, do not involve large displacements. The low-pass frequency for 5% tolerance is typically 5 to 30 Hz.

Larger displacement velocity transducers consist of a permanent, rod-shaped magnet that needs to be connected to the pile head and that moves through a coil. In the system proposed by Marchetti (1979) (see Figure 17), the coil is left free to fall under gravity. Further refinements of the velocity transducer setup were suggested by Holeyman (1984), involving a stationary suspension system or adding a coil support released upon impact.

Displacement transducers are used to obtain high-reliability measurements of the maximum transient displacement under high-strain tests and to calibrate the results of the integration of acceleration and/or velocity measurements. Because the mechanical connections required to operate common relative displacement transducers (such as LVDTs) are cumbersome and impractical when applied to a pile dynamic test, one can overcome these difficulties by using an optical connection. Two types of systems have been reported, one involving an optoelectric transducer and the other one a laser beam. As shown in Figure 18, the optoelectric transducer consists of a cathodic tube that projects a beam of light on a half-reflective, half-absorbing target attached to the pile head (Lepert, 1986). The amplitude of the reflected light depends on the position of the target relative to the beam. The reflected target is picked up by a sensor that then produces an electrical signal proportional to the vertical movement of the target. The frequency range of the instrument can be 0 to 1,000 Hz. The laser system is based on a phase shift of coherent light when reflected by a target attached to the pile head. Alternatively, a laser-sensitive transducer can be used to perform an electrical signal dependent on the location where the stationary laser beam hits the transducer. Transient measurements are valid until ground vibrations generated by the impact reach the tripod securing the beam alignment. As a result of that limitation, typically only the first 30 to 50 ms of the transient signal can be used; the measurement of the permanent penetration can be estimated long after the impact, assuming that the tripod station has regained its initial position.

Applying stress-wave theory to piles, velocity can be estimated from the separation of the downward and upward stress waves afforded by the strain measurement at two sections on the pile (Lundberg and Henchoz, 1977). Referring to subscripts 1 and 2 for the location of the measurements and to T as the time difference for the wave to travel from Section 1 to Section 2 (see Figure 19), the particle velocity v_1, is related to the strain ϵ_1 and ϵ_2 through the difference equation:

$$\frac{v_1(t)}{c} = \frac{v_1(t-2T)}{c} - \epsilon_1(t) - \epsilon_1(t-2T) + 2\epsilon_2(t-T) \qquad (2)$$

Although an interesting way to obtain velocity from strain measurements, this procedure finds little application for pile dynamic testing where a sufficiently deeper portion of the shaft (typically 0.5 m deeper than Section 1) is not accessible.

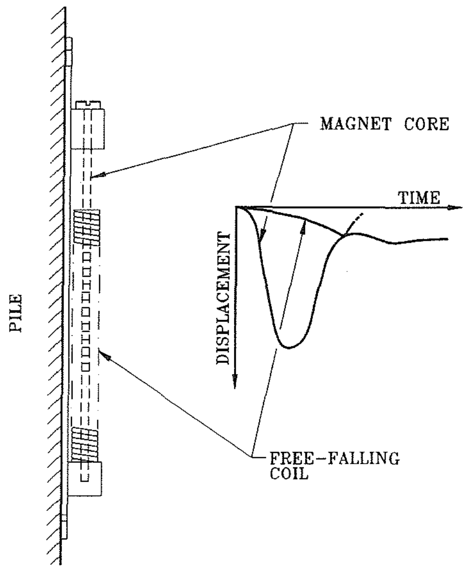


Figure 17 Velocity Transducer Setup (Marchetti, 1979)

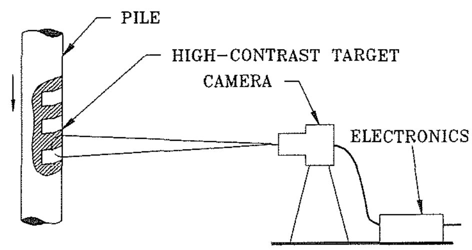


Figure 18 Optoelectric Transducer (Lepert, 1986)

4.3 *Acquisition*

Manual recording of the transient displacement of the pile head using a pen and hard paper attached to the pile has been practiced based on earlier piling textbooks (Chellis, 1961). This practice has been almost abandoned for safety reasons. Zeitlen et al. (1988) have developed a mechanical device that facilitates the safer recording of set and quake of a pile head during an impact.

The electrical signals generated by modern transducers are acquired and stored by use of transient recorders. In the early stages of experimental dynamic monitoring, a trace of the signals could be recorded on a high-speed, ultraviolet sensitive paper (Glanville, 1938). Later, magnetic tape recorders were used to play back the signals (Goble and Rausche, 1970). In the early 1980s, the wider availability of silicon chips and digital processing opened a new era of signal recording and processing. The key element of this revolution is the Analog to Digital (or A/D) conversion by which a continuous function of time is converted into a discrete succession of numerical values. Each value is digitized into a set of bits (0 or 1), which defines the resolution of the A/D converter. The earlier A/D converters expressed measured voltage in 8 bits, relative to a full-scale voltage (VFS), which meant that the resolution of the conversion was $VFS/2^8 \simeq 0.4\%$ VFS.

The frequency with which the analog signal is converted to a numerical value is termed "sampling rate." It can be shown that, to avoid aliasing of the signal, a sampling rate of at least five times the frequency of the signal is desirable. Therefore, sampling rates of 20 and 200 kHz for each channel of measurement are satisfactory for high- and low-strain testing, respectively. By the late 1980s, the commercially affordable digital resolution reached 12 bits (Reiding et al., 1984 and 1988); currently, 16 bits resolution are used (Likins, 1991). It should be noted, however, that 12 and 16 bits correspond to a resolution of 0.024% and $2\ 10^{-6}$, respectively, which by now far exceed the accuracy of transducers.

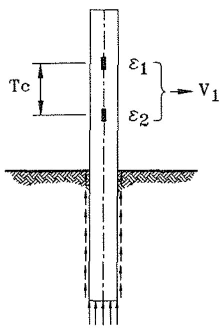


Figure 19 Two-point Measurement Technique (Lundberg and Henchoz, 1977)

Earlier electrical signals were captured by digital storage transient recorders, that could play back the analog equivalent of the signal for visualization on an oscilloscope. Nowadays, computers which are driving the A/D conversion boards come with a screen that replaces the oscilloscope and include a keyboard or touch screen for other data entry. A/D conversion boards are commercially available for IBM-PC compatible computers (AT, 386 or 486).

One of the noteworthy aspects of the acquisition system is its reduction in size and power requirements that make it portable and autonomous and therefore very practical to use on construction sites. An example of this miniaturization is embodied in the PIT collector (Rausche et al., 1992), which measures approximately 10 x 15 x 8 cm, has a mass of 0.5 kg, a touch screen, operates for 8 hours on rechargeable batteries, and can store approximately 360 records in solid state memory. The data can be downloaded to an IBM PC-compatible computer with an RS 232 interface for further processing and interpretation.

One of the benefits of having a computer-driven acquisition system is that peripheral information can be entered for easier referencing of the blows. Name of project, date, pile label, blow number, calibration and amplification factors are memorized as the transient events are recorded, thus minimizing the risk of mundane errors that can otherwise take up critical efforts that should be devoted to the interpretation of the collected signals.

5 INTERPRETATION

Observations made and measurements collected during a dynamic test can be interpreted in several ways. Major classes of interpretation can be identified such as visual inspection, application of pile driving formulas, signal processing, numerical models, and frequency domain analysis.

5.1 *Visual Inspection*

All signals that require interpretation should be visually inspected to verify that they appear to be logical: signals should be checked for amplitude, duration, echo at the end of the pile, integrity, and match of the force trace with the velocity x pile impedance trace.

Integrity test signals can be interpreted visually by recognizing some patterns indicative of defects. However, visual interpretations may be subjective and of limited power when attempting to quantify the magnitude of the defects.

5.2 *Application of Pile Driving Formulas*

In spite of the formidable advances offered by the stress-wave theory, some practitioners still hope to use pile driving formulas with the help of improved measurements such as the Enthru, elastic set and maximum transient displacement. Pile driving formulas that are still popular include Hiley's formula (Broms and Puay Chao, 1988) and Janbu's (or so-called Danish) formula.

5.3 *Signal Processing*

Processing of the collected signals, using simple operations such as the addition and subtraction of simultaneous or phase-delayed signals, is a quick and powerful method of interpretation. In the case of high-strain testing, signal processing may lead to the evaluation of shaft, toe, and total resistance, while in the case of low-strain testing, signal processing may lead to the impedance profile of the shaft. The various processing approaches discussed below result from the direct application of the stress-wave theory to piles and the formulation of certain hypotheses regarding soil resistance mobilization.

As discussed in Section 1.0 and as illustrated in Figure 2, the skin friction $F_\ell(z^*)$ mobilized down to depth z* of a prismatic pile can be readily derived from the difference between the force and velocity signals:

$$F_\ell(z^*) = \int_0^{z^*=ct/2} Q_F(z)dz =$$

$$F(t) - I.v(t) \quad for \quad t = 2z^*/c < 2L/c \tag{3}$$

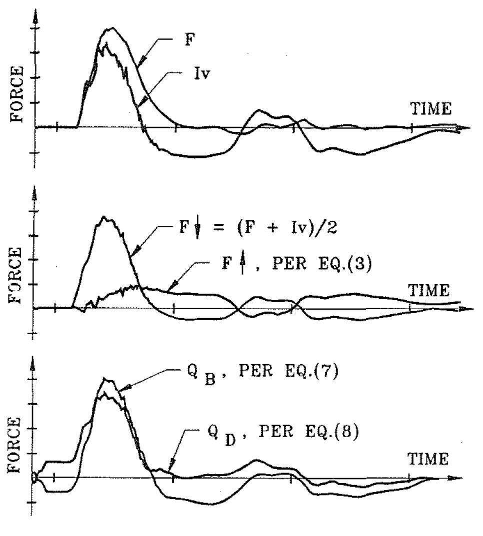


Figure 20 Signal Processing

An example of application of Equation (3) to measured force and velocity signals is illustrated in Figure 20. Equation (3) will provide reliable evaluations in as much as the assumption of a constant skin friction can be validated within the time and depth domain covered by the interpretation. It can be observed that radiation damping along the shaft is generally sufficiently high to produce a plastic deformation at moderate velocities, i.e., that the condition

$$v\sqrt{\rho_s G_s} > \tau_f \tag{4}$$

is satisfied, with v being the velocity, ρ_s the soil density, G_s the soil shear modulus, and τ_f the dynamic ultimate unit shaft resistance. As a result, the argument can be made that during the first 2L/c, most of the shaft resistance is in a condition of plasticity and that equation (3) should produce reliable results. Equation (3) underscores the capability of evaluating shaft resistance distribution as a function of z* (depth resolution).

Ultimate static shaft resistance $F\ell_s$ may then be derived from the dynamic resistance using a velocity-dependent formula of the type

$$F\ell = F\ell_s(1 + J.v^N) \tag{5}$$

with J and N depending on the nature of the soil (Gibson and Coyle, 1968; Heerema, 1979; Litkouhi and Poskitt, 1980). In the absence of specific indications, the writer has used N = 0.2 and $J=1s^{0.2}m^{-0.2}$. An experimental verification that the plastic condition is satisfied and that the shaft resistance is velocity-dependent can be obtained by plotting the results of Equation (3) as a factor of v for several blows of increasing magnitudes (Holeyman, 1984; Corté and Bustamante, 1984). A diagram of shaft resistance versus time and depth has been suggested by Paquet (1988) to verify the extent of mobilization during the impact (see Figure 21).

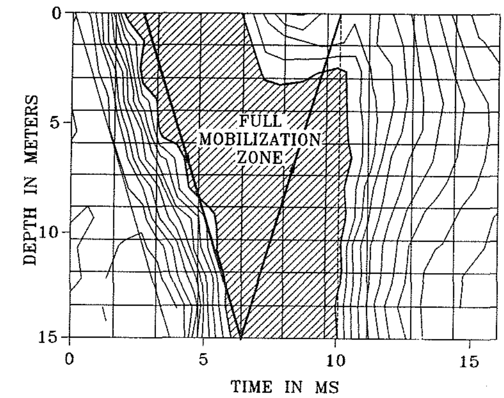


Figure 21 Diagram of Shaft Resistance versus Time and Depth (Paquet, 1988)

Table 4 Resistance Mobilization Conditions

	Typically Required for Static Mobilization @ Shaft	Toe	Typical High-Strain Dynamic Test
Displacement (% diameter)	2%	10%	2.5%
Displacement (1)	8 mm	40 mm	10 mm
Velocity (1)	0.13 m/s	15 m/s	5 m/s
Notes:	(2)	(3)	

(1) for diameter of 0.4 m
(2) assuming τ_f = 50 kPa and G = 80 MPa
(3) Assuming q_{bf} = 10 MPa and E_c = 200 MPa

Signal processing is not so easy when it comes to evaluating the toe resistance, mainly because the assumption of a rigid-plastic behavior at the pile toe does not make sense. It has been observed from static load tests and demonstrated in theory that displacements required to fully mobilize the toe resistance are much larger than those required to fully mobilize the shaft resistance. Under dynamic conditions, this difference is exacerbated by the contrast in radiation damping. Whereas condition (4) is easy to satisfy, the equivalent condition at the toe

$$v\sqrt{\rho_s E_c} > q_{bf} \qquad (6)$$

with E_c = soil compression modulus at the toe and q_{bf} = ultimate unit toe resistance, would require tremendous velocities to be satisfied, as shown in Table 4. Nonetheless, if a rigid-plastic condition is being assumed at the toe for reason of simplicity, the following expression can provide some insight into the development of the toe resistance Q_B:

$$Q_B=\frac{1}{2}[F(t^+)+I.v.(t^+)]-\frac{1}{2}[F(t^+ +\frac{2L}{c}-I.v.(t^+ +\frac{2L}{c})] \qquad (7)$$

where t^+ is a time selected to obtain the maximum value of Q_B within the interval $2L/c < t^+ < 4L/c$.

Experimental verification that Equation (7) provides a toe resistance that is not rigid-plastic can be obtained by plotting the values of Q_B versus V_{max} (at the toe) (or S_{max} or Enthru) for several blows of increasing magnitude (Holeyman, 1984; Corté and Bustamante, 1984). Such a plot will generally show a steady increase of Q_B with other parameters characterizing the magnitude of the blow. The fact that Q_B increases with V_{max}, S_{max} and the acceleration level at the toe renders the separation of the influence of those terms difficult. However, it can be concluded that the ultimate toe resistance will not be readily available for dynamic tests unless performed beyond their typical levels that are shown in Table 4.

The CASE Method (Goble et al., 1975) is a combination of Equations (3) and (6) and was developed to evaluate the total resistance Q_D (shaft and toe) based on the assumption of a rigid-plastic behavior of all terms contributing to the resistance

$$Q_D=\frac{1}{2}[F(t^+ +\frac{2L}{c})+F(t^+)]+\frac{M_p.c}{2L}[v(t^+ +\frac{2L}{c})-v(t^+)] \qquad (8)$$

with M_p = mass of pile, and t^+ time corresponding to the maximum of v.

Based on the observation that for a theoretical pile showing no resistance, F↓ is reflected at the pile toe in an equal and opposite wave $F\uparrow_{free}$, Paquet (1988) proposed the following simplified formula:

$$Q_D = F\uparrow - F\uparrow_{free} \qquad (9)$$

The static resistance Q_S is obtained from a second CASE formula:

$$Q_S = Q_D - J_c\,[F(t^+) + I.v.(t^+) - Q_D] \qquad (10)$$

with J_c being the Case damping, a nondimensional empirical factor. Empirical charts correlating J_c with soil nature have been published (Goble et al., 1975).

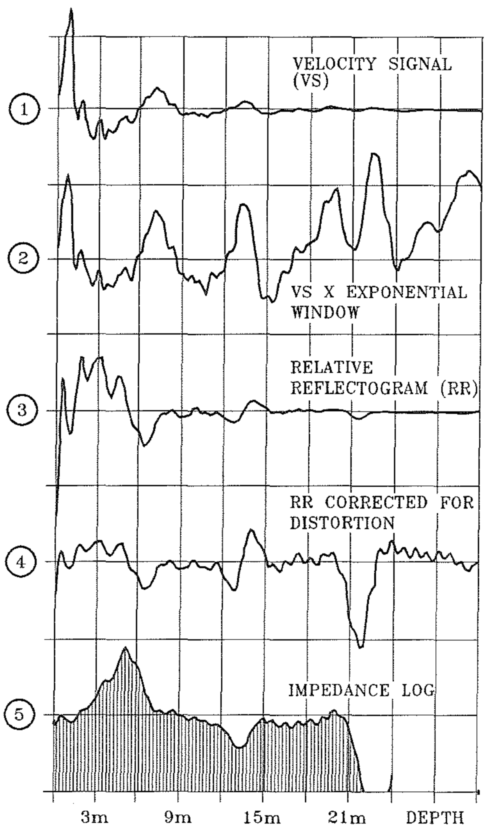


Figure 22 Impedance Log Processing (Paquet, 1991)

These contain large discrepancies that reflect the dependency of the J_c factor on other factors such as pile impedance, length, perimeter, and toe area. As observed for the toe resistance, blows of higher magnitude produce higher estimates of the total resistance as evaluated by the CASE formula (Legrand, 1985).

A common treatment is to amplify the low-strain velocity signal using an exponential function increasing with time, such as e^{At} (Reiding et al., 1984). This treatment is aimed at counteracting the attenuation of the signal as it travels through the shaft. The law governing the attenuation was established by Briard (1970)

$$A = \frac{1}{r}\frac{\rho_s}{\rho} x \frac{v_s}{c} \qquad (11)$$

where r = radius of pile and $v_s = \sqrt{G_s/\rho_s}$ = shear wave speed in the soil.

The velocity signal collected during a low-strain, integrity test may also be processed to more clearly identify "reflectors." Using a free nominal pile as a reference, Paquet (1991) has suggested that the velocity signal be processed to obtain "reflectograms" by difference. By subtracting the observed signal from that of the reference pile, the upward wave allows a clear discrimination between the effects due to soil resistance and those due to a variation in the impedance of the shaft. The final reflectogram summarizes solely shaft effects (Figure 22) and is interpreted by using an integration of the so-called Beta method [β = (1-s)/(1+s)] where s is the amplitude of the reflection relative to the initial peak signal).

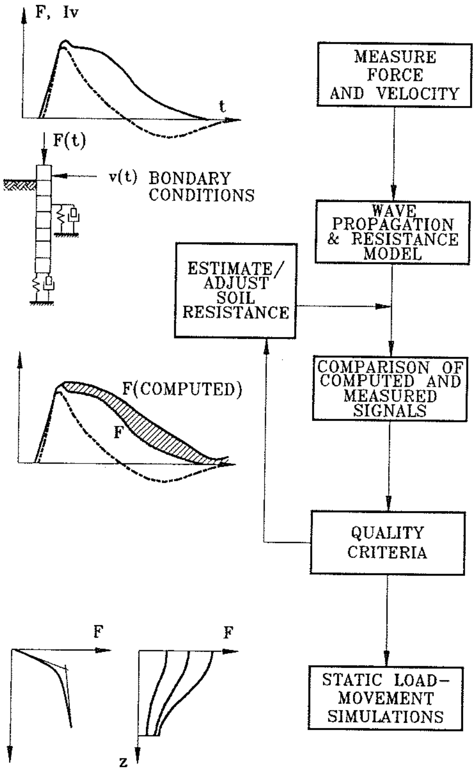


Figure 23 NUSUMS Procedure

5.4 *Numerical Models*

Pile and soil behavior can be modelled using a computer simulation of the event recorded during a dynamic test, as illustrated in Figure 23. The adjustment of the model parameters is guided by matching a calculated signal with a reference measured signal, while the model is subjected to a stimulus (measured signal) imposed at a boundary. Either dynamic (force) or kinetic (velocity) measurement can be used as a forcing function at the boundary of the model with the other measurement used as a reference for matching the calculated model response.

A number of proprietary procedures have been developed: CAPWAP (Rausche et al., 1972), TNOWAVE (Middendorp, 1987), and SIMBAT (Paquet, 1988). The writer suggests that these procedures be referred to as NUSUMS for NUmerical Simulations Using Measured Signals. The models used for that purpose can be classified into two broad categories: (1) lumped parameter, finite element, and finite difference models, and (2) models based on differential equation characteristics. While methods based on lumped parameters are widely used because they are associated with the early numerical development of the wave equation (Smith, 1960), they have been criticized for their lack of accuracy when it comes to sharp variations of section. Models based on characteristics, or systems of waves propagating up and down the pile, offer a greater simplicity and lend themselves to the simulation of refractors or reflectograms, as suggested by Paquet (1991). One

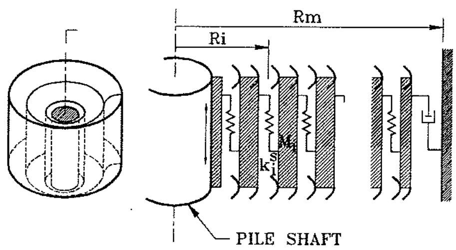


Figure 24 One-dimensional Skin Friction Model (Holeyman, 1985a)

difficulty of models based on characteristics appears to be the necessity to model the mechanical elements located above the measuring section.

Soil models that are used within the framework of NUSUMS can vary in complexity, starting from a single degree of freedom system (Smith, 1960), passing through more elaborate one-dimensional models, such as the one shown in Figure 24 (Holeyman, 1985a and 1985b) and ending up with full three-dimensional finite elements representations (Chow, 1981). Models used for interpretation of high-strain tests generate a soil mobilized resistance that depends on displacement, velocity, and possibly acceleration, while models used for the interpretation of low-strain tests generate a soil mobilized resistance that depends generally only on velocity. The development, merits, and limitations of these soil models, which hold the key to the interpretation of the tests in terms of static resistance have been discussed in the past on several occasions (Holeyman, 1986, 1988, and 1990) and are discussed at this conference in Key-Note Lecture No. 1a (Randolph, 1992).

5.5 *Frequency Analysis*

The analysis of signals collected during pile dynamic tests can be interpreted in the frequency domain by Fourrier transforming the velocity and force signals and calculating their ratio, as illustrated in Figure 25. Resulting from these operations, the mechanical impedance (F/v) or cyclic mobility (v/F) is then interpreted with regards to several of the factors following (Paquet, 1968; Davis and Guillermain, 1979):

- The average value of the impedance, which provides an overall confirmation of the section and material properties of the shaft,
- The frequency difference between maxima and minima, which provides a confirmation of the dominant and secondary lengths of the pile, and
- The slope of the cyclic mobility function at low frequency, which provides an evaluation of the pile stiffness under static loading.

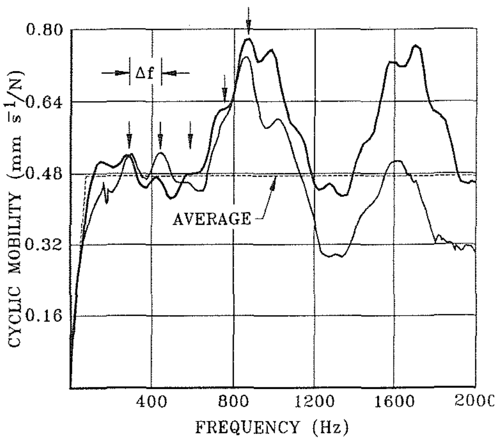


Figure 25 Frequency Analysis

The frequency analysis is generally reserved for low-strain testing signals, including reflectograms (Paquet, 1991). However, pile head stiffness at larger displacements can be obtained from the frequency analysis of high-strain signals (Holeyman, 1987a).

6 CONCLUSIONS

The technology of pile dynamic testing has become more sophisticated and encompassing. In the area of loading procedures, the salient facts are (1) the wider recognition that larger energies lead to higher mobilized soil resistances, and therefore more reliable predictions, and (2) the emergence of kinetic loading approaches that by-pass the dynamic interpretation of the waves but do not benefit from the depth resolution. Improvements are still needed in the area of transient motion, particularly velocity measurements. Improved frequency range of accelerometers may help in the usual integration of the acceleration to obtain the velocity. Improvements are needed in the area of loading procedures for integrity testing, particularly, to obtain a better repeatability of collected signals.

Major progress has been made in the development of soil models that promote the use of physical parameters and that represent more realistically the dynamically mobilized soil resistance. Nevertheless, in the absence of a new technology of dynamic pile loading that would generate much higher velocities and displacements, the confidence in soil resistance predictions will be limited to the ultimate shaft resistance distribution and the beginning of the loading curve at the pile toe. Finally, recently developed procedures for interpreting low-strain tests in terms of an impedance log (Paquet, 1991) should be applied more widely to better assess their power and limitations.

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