MONITORING THE INTEGRITY OF CONCRETE USING STRESS-WAVES

by

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A thesis submitted in conformity with the requirements for the degree of Masters of Applied Science in Civil Engineering Graduate Department of Civil Engineering University of Toronto

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ABSTRACT


Concrete setting was monitored using both penetration resistance and stress-wave testing. Although velocity is increasing exponentially, there is no distinct change at the time of final set. Concrete strength development was monitored over a period of 28 days and equations were developed between strength and various characteristics of stress-waves. The equations were validated by casting new concrete and monitoring it over a period of 90 days. The accuracy of the estimate was found to be within 5% using the combined method of velocity and $Q$-factor for ultrasonic through-transmission and within 6% using only velocity for impact-echo technique.

In monitoring deterioration due to alkali-silica reaction and alternate freezing and thawing cycles, the attenuation indices (except transmission frequency shift) showed little sensitivity with deterioration. However, velocity and transmission frequency shift both showed sensitivity to deleterious expansion. For large field-exposed concrete blocks, all 4-analysis indices showed sensitivity with expansion.
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Without the love, support and encouragement of my family—Tesfamariam Tseggai, Letteselasse Berhe, and Lula Tesfamariam— I wouldn’t have completed my thesis.

I would like to dedicate my thesis to my mom, Letteselasse Berhe.
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NOMENCLATURE

$\alpha$  attenuation coefficient
$\Delta t$  sampling interval
$\Delta f$  frequency resolution
$\varepsilon$  expansion
$\theta$  angle of the side of the cone created by the spreading of the distant field
$\rho$  mass density
$\phi_n$  Fourier series: phase coefficient
$K$  Fourier series: constant
$\lambda$  wave length
$\nu$  Poisson's ratio
$\omega_0$  Natural angular frequency
$a$, $b$, and $c$  constants
$A_i$  Amplitude of incident wave
$A_n$  Fourier series: cosine coefficient
$A_{refracted}$  amplitude of refracted wave
$A_{reflected}$  amplitude of reflected wave
$b_1$  length of the near field
$B_n$  Fourier series: sine coefficient
$C_n$  Fourier series: amplitude
$C_P$  P-wave velocity
$C_{P1,2}$  P-wave velocity in medium 1 and 2
$C_R$  R-wave velocity
$C_s$  S-wave velocity
$C_{S1,2}$  S-waves velocity in medium 1 and 2
$ASR$  alkali-silica reaction
$D$  diameter of the sphere
$D$  diameter of transmitter
$D_n$  Defraction coefficient for normal incident
$D_{CP}$  damage coefficient of velocity
$D_\alpha$  damage coefficient of attenuation coefficient
$D_Q$  damage coefficient of Q-factor
$D_{PF}$  damage coefficient of peak frequency shift
$E_d$  dynamic modulus of elasticity
FFT  Fast Fourier Transform
$f_c'$  concrete strength.
$f_{w}'$  standard cube strength (water-cured)
$f_{a}'$  standard cube strength (air-cured)
$f_P$  P-wave resonance frequency between the top and bottom of a two layer solid system
$f_{max}$  maximum useful frequency (a frequency with sufficient energy)
$f_2$ and $f_l$  frequencies at 70.7 % of the spectral peak amplitude
F/T  freezing and thawing
$G$  shear modulus of elasticity
<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$h$</td>
<td>drop height</td>
</tr>
<tr>
<td>$i$</td>
<td>angle of incidence</td>
</tr>
<tr>
<td>$I_d$</td>
<td>intensity at a distance $d$</td>
</tr>
<tr>
<td>$I_0$</td>
<td>intensity at a distance 0</td>
</tr>
<tr>
<td>$I_t$</td>
<td>intensity at time $t$</td>
</tr>
<tr>
<td>$k$</td>
<td>constant dependent on compaction, moisture content and mix proportions</td>
</tr>
<tr>
<td>$n$</td>
<td>number of samples to be recorded</td>
</tr>
<tr>
<td>$P_1$</td>
<td>length of the transit zone</td>
</tr>
<tr>
<td>$Q$-factor</td>
<td>Quality factor</td>
</tr>
<tr>
<td>$r$</td>
<td>angle of refraction</td>
</tr>
<tr>
<td>$R$</td>
<td>angle of reflection</td>
</tr>
<tr>
<td>$R_n$</td>
<td>Reflective coefficient for normal incident</td>
</tr>
<tr>
<td>SWT</td>
<td>stress-wave techniques</td>
</tr>
<tr>
<td>$t$</td>
<td>travel time</td>
</tr>
<tr>
<td>$t_{cc}$</td>
<td>impact duration or contact time</td>
</tr>
<tr>
<td>TFS</td>
<td>transmission frequency shift</td>
</tr>
<tr>
<td>UPV</td>
<td>ultrasonic pulse velocity</td>
</tr>
<tr>
<td>$V$</td>
<td>wave velocity</td>
</tr>
<tr>
<td>$V_a$</td>
<td>standard cube pulse velocity (air-cured)</td>
</tr>
<tr>
<td>$V_w$</td>
<td>standard cube pulse velocity (water-cured)</td>
</tr>
<tr>
<td>$X$</td>
<td>stress wave propagation distance</td>
</tr>
<tr>
<td>$Z$</td>
<td>acoustic impedance</td>
</tr>
<tr>
<td>$Z_{1,2}$</td>
<td>Specific acoustic impedance of media 1 and 2</td>
</tr>
</tbody>
</table>
Increasing incidences of deteriorating concrete highlight the need for non-destructive testing (NDT) techniques to estimate the internal condition of concrete structures. A non-destructive, non-intrusive, testing method is defined as a testing method that causes no structurally significant damage to the test object (ACI 228.2R-98). The various applications of NDT range from early age to deteriorated concretes, and may be summarized as follows:

- Estimating the start and termination of setting (e.g. for determining slip forming rates),
- Quality control,
- Formwork removal times,
- Time for transfer of prestressing or application of post tensioning,
- Determining the likely in-situ strength,
- Locating areas of weakness or deteriorated concrete,
- And to “assess the safety margin of structures that have deteriorated and to formulate measures for repairing, rehabilitating and strengthening them.” (Blight, 1996)

Various NDT techniques are available for carrying out the above tasks. These NDT techniques range from visual observation to sophisticated instrumentation. Each of the NDT techniques used in the evaluation of concrete structures are summarized by ACI 228.2R-98 as follows: visual inspection, ultrasonic pulse velocity, ultrasonic-echo, impact-echo, spectral analysis of surface waves, sonic-echo, impulse-response, impedance logging, crosshole sonic logging, parallel
seismic, direct transmission radiometry, backscatter radiometry, radiography, gamma-gamma logging, covermeter, half-cell potential, polarization methods, penetrability methods, infrared thermography, and radar.

Each NDT technique has its unique advantages and inherent limitations. The choice of a particular technique is dependent on the level of detail required and the accessibility of the test object. Some of these testing methods, include the use of stress-waves and these are the focus of this thesis. Commonly used stress-wave techniques (SWT) include ultrasonic-through transmission (UTT), ultrasonic echo (PE) (pulse echo) and impact-echo (IE). This method is based on the principle that stress-waves are generated on a test object using a transducer or an impact source. The stress-wave that propagates through the test object is collected using a receiving transducer and stored in the time-domain. (The various stress-wave techniques are further illustrated in Chapter 2). The ultrasonic-through transmission and ultrasonic echo both utilize a resonant frequency transducer as a source with a frequency at or above 20 kHz. Therefore, the two techniques are referred as ultrasonic because they use sound waves with frequencies above the audible range (above 20 kHz).

The use of stress-waves for evaluating structures dates back to the 1940’s at Ontario Hydro in Canada (Leslie and Cheesman, 1949) and at the Road Research Laboratory in the UK (Jones, 1949). The emphasis of Jones’ (1949) research was the measurement of the thickness of concrete road slabs and in-situ concrete quality using an instrument called the Ultrasonic Concrete Tester. Whereas in Canada, the focus of the research of Leslie and Cheesman (1949) was determining depth of cracks in monolithic concrete structures using an instrument called the Soniscope. The
two instruments, the Ultrasonic Concrete Tester and the Soniscope, were developed at around the same time. The Ultrasonic Concrete Tester utilized a higher transmission frequency and had superior repeatability compared with the Soniscope. However, the higher transmission frequency was more prone to attenuation and therefore, the maximum range of transmission was 2m, whereas the maximum range of transmission of the Soniscope was 15m (Whitehurst and Parker, 1954).

Whitehurst (1951b) carried out one of the earliest tests on the use of stress-waves to measure the setting time of concrete using the Soniscope. Jones (1953) further applied stress-wave techniques for monitoring strength gain and different deterioration mechanisms. Most of the experimental work performed was limited to laboratory-cast concrete and hence the practicality of applying it to existing structures was limited. Tomsett (1980) overcame this limitation by introducing the paste efficiency concept. This concept is based on taking the UPV reading of an existing structure and correlating it with the UPV reading of a lab-cured concrete of similar composition. However, if the composition of the concrete in the structure is not known, or there are no similar lab-cast cylinders available, Tomsett’s paste efficiency concept applicability is limited.

Early works using stress-waves were limited to measuring the time of first arrival and hence calculating the ultrasonic-pulse velocity (UPV). To date, the UPV method is standardized and given in ASTM C 597-83 (Reapproved 1991). The UPV method has gained increasing application with time. Reportedly it has been used for estimating concrete strength, flaw detection, estimating setting time, measuring layer thickness, estimating elastic modulus, Poisson’s ratio, homogeneity and degree of deterioration (Naik and Malhotra, 1991).
With the advancement of stress-wave-testing instrumentation, researchers have used different analysis indices. Examples of these new analysis indices include transmission frequency shift, attenuation coefficient ($\alpha$), and quality factor ($Q$-factor). These analysis indices are affected by the attenuation of the stress-wave signal at the aggregate cement interface, voids and microcracks.

With research carried out at the U.S. National Bureau of Standards and Cornell University, an Impact-echo technique was introduced to the concrete industry in the mid-80s (Sansalone and Streett, 1997). Although the initial research was mainly focused on flaw detection (Sansalone, 1986), different applications of the technique have been published since.

Early work has shown that with the limitations of instrumentation and analysis software, analysis was restricted to only one or two variables. However, this thesis describes an evaluation of state-of-the-art stress-wave testing instrumentation, the AndecScope™, for monitoring setting of concrete, strength development and deterioration due to freeze/thaw and alkali-silica reaction. Each of the analysis indices was used separately and in combination. The following section presents the research overview.

1.1 Research Overview

This research is mainly divided into three different sections:

1. Literature review, on stress-wave techniques for the non-destructive evaluation of concrete,

2. Use of stress waves to monitor setting and strength development,
3. Use of stress-waves to evaluate concrete affected by ASR and freezing and thawing cycles.

1.1.1 Literature review

The literature review, Chapter 2, is focused on the principles of stress-wave propagation and generation techniques. The stress-wave propagation principle (2.1), reflection and refraction (2.2), attenuation and dispersion (2.3), and analyses techniques (2.6) are reviewed. Further, three stress-wave propagation techniques, ultrasonic through transmission (2.7.1), ultrasonic-echo (2.7.2) and impact echo (2.7.3), are reviewed. After covering the three SWT techniques, cases where the stress-wave techniques have been applied to the estimation of setting time (2.8.1), strength development (2.8.2), and deteriorated concrete (2.8.3) are reviewed.

1.1.2 Setting and Strength development

The setting and strength development section, Chapter 3, covers the experiments carried out to investigate existing and new techniques used for estimating setting time and in-situ concrete strength in structures. This chapter presents the experimental setup and the results obtained using regression analysis.

1.1.3 Deteriorated Concrete

This section covers the effects of ASR, and freezing and thawing on the response of concrete to stress waves. This was carried using laboratory-cast concrete specimens, (Chapter 4), and large concrete blocks stored under field conditions (Chapter 5).
The use of NDT techniques in the concrete construction and repair industry is widespread. For example, it is used to estimate early-age concrete compressive strength, for formwork removals; and condition surveying of existing structures, for developing repair strategies. Different techniques are available to carry out these tasks. The choice of the particular technique is dependent on the availability of equipment, level of detail required and the accessibility of the test object. Existing NDT techniques used in practice range from visual observations to computerized methods using sophisticated instrumentation. Each technique has its own unique advantages and limitations. The techniques are described in ACI 228.2R-98. Since the focus of this thesis is on the use of stress-waves, this chapter will be limited to stress-wave techniques (SWT) for evaluating concrete setting and strength development; and the condition of deteriorated structures. Before reviewing the literature for cases where such techniques have been applied, it is imperative that the principles behind the theory of stress-wave propagation and the SWT are understood. This is covered in Sections 2.1–2.7, and cases where the SWT have been applied are reviewed in Section 2.8.
2.1 Stress-Wave Propagation Principle

When stress is released on an object by an impact or a transducer, the stress wave propagates in a manner analogous to that of the propagation of a sound wave. The stress-wave propagation is affected by the properties of the medium through which it travels; such properties include: modulus of elasticity, density, and Poisson's ratio. The dependence of the stress waves on the material can be used to infer the properties of the material. The stress wave propagates in three different modes (Krautkramer and Krautkramer, 1990):

- Dilatational, compression (P-wave),
- Distortional, shear (S-wave),
- Rayleigh wave (R-wave).

A schematic presentation of the P- and S-wave propagation is shown in Figure 2-1. Each of the propagation techniques is described in the following subsections.

2.1.1 Compression Wave

The P-waves travel parallel to the direction of the particle motion of the material. This propagation is characterized by alternating regions of compression and dilatation, i.e. compression wave (Blitz and Simpson, 1996).

For infinite, homogeneous, isotropic and uniform material the P-wave velocity, $C_P$, is computed by the following equation (Krautkramer and Krautkramer, 1990):

$$C_P = \sqrt{\frac{E_d(1-\nu)}{\rho(1+\nu)(1-2\nu)}}$$

Eq. [2-1]

where $E_d$ is the dynamic modulus of elasticity, $\rho$ is the mass density and $\nu$ is the Poisson's ratio.
Chapter 2 - Literature Review

For rod-like structures such as piles, the P-wave speed is independent of the Poisson ratio if the rod diameter is much less than the wavelength of the propagating wave and the P-wave velocity can be determined by (Sansalone and Carino, 1991):

\[ C_p = \sqrt{\frac{E_d}{\rho}}. \]  

Eq. [2-2]

Although concrete is a heterogeneous and anisotropic media and the rigorous application of Eq. 2-1 may not be valid, the P-wave velocity is still considered to be strongly influenced by the material stiffness density and Poisson's ratio.

2.1.2 Shear Wave

S-waves travel perpendicular to the direction of the particle motion. This creates a shear stress in the propagating medium. Therefore, a shear wave can propagate only in solids, except for very short distances in highly viscous liquids (Blitz and Simpson, 1996).

The S-wave propagation propagates at a slower S-wave velocity, \( C_S \), and may be computed by (Krautkramer and Krautkramer, 1990):

\[ C_S = \sqrt{\frac{E_d}{\rho} \frac{1}{2(1+\nu)}} = \sqrt{\frac{G}{\rho}}, \]  

Eq. [2-3]

where \( G \) is the shear modulus of elasticity.

2.1.3 Rayleigh Wave

An R-wave propagates along the surface of the solid. It has an elliptical oscillatory motion (Krautkramer and Krautkramer, 1990). At a depth of one wavelength of the particles, the
Figure 2-1 Successive stages in the deformation of a block of a material by compressive and shear waves and their particle motion. The sequences in progress with time from top to bottom. (Sadri, 1996)

a) The block of material, b) Compressive waves, and c) Shear waves.

amplitude of the oscillation approaches zero (Krautkramer and Krautkramer, 1990). Thus for a pure R-wave to exist, its wavelength has to be smaller than the depth of the material.
The R-wave speed, $C_R$, is approximated by (Krautkramer and Krautkramer, 1990):

$$C_R = \frac{0.87 + 1.12\nu}{1 + \nu} \sqrt{\frac{E_d}{\rho} \frac{1}{2(1 + \nu)}}.$$  

Eq. [2-4]

2.2 Reflection and Refraction

The propagation of stress waves in a non-homogeneous medium, such as concrete, is subject to reflection and refraction. When a stress wave is incident obliquely at a boundary, the layer in the vicinity is subjected to both compression and shear stresses (Blitz and Simpson, 1996). The incident wave may be reflected and refracted (complete change of propagation medium) as P- and S-waves. However, the S-wave is observed only when the medium of propagation supports shear waves. For example, since water does not have shear strength, the S-wave can not propagate in it and hence it can not be observed in that medium. A schematic presentation of a P-wave incident at an interface of two mediums, and subsequent reflection, refraction and mode change is shown in Figure 2–2.

The wave interaction shown in Figure 2–2 obeys Snell’s Laws. This interaction is shown as:

$$i_P = R_P$$  

Eq. [2-5]

and,

$$\frac{\sin i_P}{C_{P1}} = \frac{\sin R_P}{C_{P1}} = \frac{\sin R_S}{C_{S1}} = \frac{\sin r_P}{C_{P2}} = \frac{\sin r_S}{C_{S2}},$$  

Eq. [2-6]
where $R$ is the angle of reflection, $i$ is the angle of incidence, $r$ is the angle of refraction, $C_{P1,2}$ and $C_{S1,2}$ are the velocity of P- and S-waves velocity in medium 1 and 2, respectively. Since S-waves propagate at a lower velocity than P-waves, they will reflect and refract at angles less than the angles of reflection and refraction for P-waves as shown in Figure 2–2.

The reflection of the incident waves at the interface between two media is dependent on the specific acoustic impedance of each medium. The acoustic impedance, $Z$, is calculated by (Krautkramer and Krautkramer, 1990):

$$Z = \rho V,$$

Eq. [2-7]

where $\rho$ is the density of the material, and $V$ is the wave velocity. As shown in Eq. 2-7, acoustic impedance is directly dependent on density and velocity. Thus different materials with varying
density have different acoustic impedances. The acoustic impedances and the P-waves of
different materials are summarized in Table 2–1 (Sansalone and Carino, 1991).

<table>
<thead>
<tr>
<th>Material</th>
<th>Density (kg/m$^3$)</th>
<th>P-wave velocity (m/s)</th>
<th>Specific acoustic impedance (kg/(m$^2$-s))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Air</td>
<td>1.205</td>
<td>343</td>
<td>0.413</td>
</tr>
<tr>
<td>Concrete</td>
<td>2300</td>
<td>3000-4500</td>
<td>6.9–10.4 x 10$^6$</td>
</tr>
<tr>
<td>Granite</td>
<td>2750</td>
<td>5500-6100</td>
<td>15.1–16.8 x 10$^6$</td>
</tr>
<tr>
<td>Limestone</td>
<td>2690</td>
<td>2800-7000</td>
<td>7.5–18.8 x 10$^6$</td>
</tr>
<tr>
<td>Marble</td>
<td>2650</td>
<td>3700-6900</td>
<td>9.8–18.3 x 10$^6$</td>
</tr>
<tr>
<td>Quartzite</td>
<td>2620</td>
<td>5600-6100</td>
<td>14.7–16.0 x 10$^6$</td>
</tr>
<tr>
<td>Soils</td>
<td>1400-2150</td>
<td>200-2000</td>
<td>0.28–4.3 x 10$^6$</td>
</tr>
<tr>
<td>Steel</td>
<td>7850</td>
<td>5940</td>
<td>4.3 x 10$^6$</td>
</tr>
<tr>
<td>Water</td>
<td>1000</td>
<td>1480</td>
<td>1.48 x 10$^6$</td>
</tr>
</tbody>
</table>

The amplitude of the reflected ray relative to the amplitude of the incident ray, can be
determined by (Krautkramer and Krautkramer, 1990):

$$R_n = \frac{A_{\text{reflected}}}{A_i} = \frac{Z_2 - Z_1}{Z_2 + Z_1},$$  \hspace{1cm} Eq. [2-8]

where

$R_n$ = Reflective coefficient for normal incident,
$A_{\text{reflected}}$ = Amplitude of reflected wave,
$A_i$ = Amplitude of incident wave,
$Z_{1,2}$ = Specific acoustic impedance of media 1 and 2, respectively.
The amplitude of the refracted ray relative to the amplitude of the incident ray, can be determined by (Krautkramer and Krautkramer, 1990):

\[
D_n = \frac{A_{\text{refracted}}}{A_i} = \frac{2Z_2}{Z_2 + Z_1},
\]

Eq. [2-9]

where

\[D_n = \text{Defraction coefficient for normal incident},\]
\[A_{\text{refracted}} = \text{Amplitude of reflected wave.}\]

There are three cases where Eqs. 2-8 and 2-9 are applied, which are when \(Z_2<<Z_1, Z_2>>Z_1\) and \(Z_2\approx Z_1\). When \(Z_2<<Z_1\), a case where the stress waves is propagating in one medium (for example concrete) and encounters by a second medium that is empty space (for example the acoustic impedance of air is approximately \(10^7\) times less than that of concrete). And the \(R_n\) in Eq. 2-8 has a value of -1. The magnitude and the change in polarity indicates the stress wave has is completely reflected with the same amplitude as the incident stress wave, and there is a phase change; if the incident wave was in tension, it is reflected back in compression and vice versa.

When \(Z_2>>Z_1\), the reflected wave has the same magnitude as the incident wave and there is no phase change. The amplitude of the refracted wave has twice the magnitude of the incident wave.

In the impact-echo technique, this is encountered when the stress waves are propagating from a concrete medium to a steel medium (Sansalone and Streett, 1997).

The third case is when \(Z_2\approx Z_1\). This is the case where the amplitude of the reflected wave is zero and amplitude of the refracted wave is 1.
Eq. 2-8 is valid only for normal incident angle (where \( i_p = 0 \)). If the angle of incidence is different from normal, the reflection coefficient can be determined by formulae given in Krautkramer and Krautkramer (1990). The relationship for the concrete/air interface derived by using the formulas in Krautkramer and Krautkramer (1990) is shown in Figure 2–3.

![Reflection coefficients diagram](image)

Figure 2–3. Reflection coefficients at a concrete/air interface for an incident P-wave as a function of the incident P-Wave and the incident angle (Poisson’s ratio=2) (Sansalone and Carino, 1991)

The assumption made in Figure 2–3 is that the amplitude is equal to unity. The figure is divided into three different graphs. The graph at the top left gives the coefficient of \( R_P \), for the reflected P-wave. The graph at the lower right gives the coefficient of \( R_S \), for the mode-converted S-wave. The graph at the top right gives the angular relationship between the incident wave and the mode-converted wave, which is governed by Snell’s law. The drawing at the lower left corner is an example of how the graphs are used.
2.3 ATTENUATION AND DISPERSION

So far the wave propagation has been described for an ideal material where the stress-wave propagation energy is unaffected by the material. However, in a heterogeneous material like concrete, the energy of the stress waves decreases with increasing path-length. The decrease in wave energy is due to coupled effects of both attenuation and divergence. Divergence is a phenomenon where the sound wave diverges away from the centre of the beam and hence the amplitude decreases. The pulse amplitude at a distance, \( d \), from the source can be described as (Krautkramer and Krautkramer, 1990):

\[
P_d = P_o \cdot 2 \sin \left( \pi \left( \sqrt{\frac{D^2}{4} + d^2} - d \right) / \delta \right),
\]

where, \( P_d \) is the diverged pulse amplitude at distance \( d \), \( P_o \) is the initial pulse amplitude at the source, \( \delta \) is the wavelength of the sound beam, \( D \) is the diameter of the transducer, and \( r \) is the distance from the source. If the distance is far from the source, which is \( d >> D^2/4\delta \), it is considered to be a far field problem (described in Section 2.4.1), and the pulse amplitude can be calculated by (Krautkramer and Krautkramer, 1990):

\[
P_d = \frac{P_o \pi D^2}{4\delta d}.
\]

The second type of stress-wave attenuation is caused by the absorption and scattering of the wave energy. Absorption and scattering losses can give information about the structure and physical properties of the medium (Blitz and Simpson, 1996). Absorption is a result of conversion of stress wave energy to heat, caused by the friction as a result of the particle
movement. Scattering in concrete structures is a result of wave reflection, refraction, diffraction, and mode conversion due to dissimilar materials, micro-cracks and air voids.

For a plane sinusoidal wave, the attenuation due to the material can be calculated by (Krautkramer and Krautkramer, 1990):

\[ P_d = P_0 e^{-\alpha d} \]

Eq. [2-12]

where \( \alpha \) is the attenuation coefficient.

Dispersion is a frequency dependent phenomenon. The propagating stress-waves having higher frequency (lower \( \lambda \)) are more affected by the anomalies in the concrete than low frequency waves.

2.4 Stress-Wave Generation

In SWT techniques, various methods are used to generate stress waves. Some of the stress-wave generators are, standard ultrasonic pulsers, impactors, tone burst systems, laser generation, and lithotripsy (Popovics et al., 1993). Each technique has its unique advantage and inherent difficulty. The choice of an appropriate stress wave generator takes into consideration, the input control, penetrating strength, and simplicity (Popovics et al., 1993).

Since in this research stress waves are generated by using only a transducer and an impactor, the following subsections will be limited to these two techniques.
2.4.1 Transducer

The most commonly used ultrasonic generators are made up of a piezoelectric plate. A piezoelectric crystal is a material which has the ability of producing electric charge if it is deformed by external mechanical pressure, and on the other hand, has the potential to change its form if an electric potential is applied (Krautkramer and Krautkramer, 1990). The first property is used for receiving stress waves and the second property is used for generating stress waves. The most commonly used piezoelectric material is lead zincronate-titanate (PZT) (Krautkramer and Krautkramer, 1990).

At a point of application of an ultrasonic transmitter, the radiating ultrasonic field is divided into three parts: near field, transit zone and far field (Galan, 1990). This is depicted in Figure 2–4. The near field is computed by Galan (1990):

\[ b_1 = \frac{D^2}{4\lambda}, \]  

Eq. [2-13]

where \( D \) is the diameter of transmitter, \( b_1 \) is length of the near field, \( \lambda \) is the wave length. Eq. 2-13 is valid for \( \lambda >> D \).

![Figure 2–4 Graphical representation of the approximate shape of the ultrasonic fields. (Galan, 1990)](image-url)
The length of the transit zone is also computed by Galan (1990):

\[ p_i = \frac{\pi D^2}{4\lambda}, \]

Eq. [2-14]

where \( p_i \) is length of the transit zone.

The angle \( \theta \) in Figure 2–4 is represented by Galan (1990):

\[ \sin \theta = \frac{C_i V}{D f}, \]

Eq. [2-15]

where \( f \) is the natural frequency of transmitter. The values of \( C_i \) are given as 1.22, 1.00, 0.76, and 0.56 when the amplitude of ultrasonic vibration decreases by 100, 10, 30, and 50%, respectively (Galan, 1990).

The distance field is represented by means of radiation (directional) characteristics. As shown in Figure 2–5, the P-wave propagates perpendicular to the transmitter. The amplitude of the P-wave is maximized when \( \lambda >> D \). At \( \lambda \approx D \), the P-wave is shown with minimal amplitude. The S-wave with the largest amplitude propagates at an angle 45°, and the R-wave propagates on the surface.
Figure 2–5 Representation of the radiation characteristics of the distant ultrasonic field (Galan, 1990).

2.4.2 Impactor

Stress waves can be generated by a mechanical impact on the surface. The use of an impactor to generate stress waves dates back to the 1970's (ACI 228.2R-98). The most commonly used impactors, for SWT techniques, are a steel ball or a hammer. Impactors have advantages over transducers because of the small contact area where there is no need for grinding the surface to get good contact. However, impactors generate a range of frequencies and may cause different undesirable vibrations.
The propagation of the P- and S-waves generated by an impactor is shown in Figure 2–6.

As shown in Figure 2–6, the maximum P-wave propagates at 0° from the impact source. The S-wave is a maximum at 45° from the source (for a Poisson’s ratio of 0.25) (Sadri, 1996). Zero amplitude is observed at angle given by \( \sin \theta = C_s / C_p \), which for a Poisson’s ratio of 0.25 is 35.2° (Sadri, 1996).

2.5 SIGNALIZATION

The stress wave generated is collected in a digital format in the time-domain (see Figure 2–7a). The signal in the time-domain format is transformed to the frequency-domain format (see Figure 2–7b) using a Fourier transform function, and is based on the principle that any time-dependant
function can be represented as a sum of sine curves of different amplitudes and frequencies (Cartwright, 1990). The Fourier transform is carried out by numerical calculations on a digitized waveform, using a technique known as fast Fourier transform (FFT). The mathematical transformation algorithm of the Fourier transform is shown as (Cartwright, 1990):

\[ x(t) = K + \sum_{n=1}^{\infty} \left( A_n \cos n\omega_o t + B_n \sin n\omega_o t \right), \quad \text{Eq. [2-16]} \]

\[ x(t) = K + \sum_{n=1}^{\infty} \left( C_n \cos(n\omega_o t + \varphi_n) \right), \quad \text{Eq. [2-17]} \]

\[ A_n = \frac{2}{T} \int_{-T}^{T} x(t) \cos(n\omega_o t) \, dt, \quad \text{Eq. [2-18]} \]

\[ B_n = \frac{2}{T} \int_{-T}^{T} x(t) \sin(n\omega_o t) \, dt, \quad \text{Eq. [2-19]} \]

\[ C_n = \sqrt{A_n^2 + B_n^2}, \quad \text{Eq. [2-20]} \]

\[ \varphi_n = \tan^{-1} \frac{B_n}{A_n}, \quad \text{Eq. [2-21]} \]

where,
\( A_n \) = Fourier series: cosine coefficient,
\( B_n \) = Fourier series: sine coefficient,
\( C_n \) = Fourier series: amplitude,
\( K \) = Fourier series: constant,
\( \varphi_n \) = Fourier series: phase coefficient,
\( \omega_o \) = Natural angular frequency,

Choosing the correct signal is crucial for accurate presentation of the desired results. The two most important parameters for optimizing the data acquisition are: the sampling interval, \( \Delta t \) (the interval of time between successive digital points), and the number of samples to be recorded, \( n \) (Sansalone and Streett, 1997).
The first variable, the sampling interval, determines the highest frequency that can be shown in the frequency domain. The reciprocal of sampling interval, \(1/\Delta t\), denotes the sampling frequency (Sansalone and Streett, 1997). The accuracy of specific frequency, \(f\), can be enhanced by using a sampling rate of ten times the desired specific frequency (Sansalone and Streett, 1997). This can be shown as:

\[
\Delta t_{\text{max}} \approx \frac{1}{10f}.
\]

Eq. [2-22]

For example, for a velocity of 4,000 m/s and sample thickness of 0.3m, the desired peak frequency of the P-wave reflection occurs at 6.67 kHz. Therefore, according to Eq. 2-22, the maximum sampling interval selected should be 15 \(\mu\)s.

The product of the second variable, sample length, \(n\) and \(\Delta t\) gives the total length of time which the waveform is recorded (Sansalone and Streett, 1997). If the total length of time recorded exceeds the appropriate value, it has two major effects: first, the time where the useful data is collected could be exceeded, and hence unwanted signals will be collected; and second, unwanted reflections from the edges could be collected (Sansalone and Streett, 1997).
Since in the impact-echo testing technique, the peak frequency is used for calculating the P-wave velocity, the resolution in the frequency domain has to be enhanced. This can be achieved by
increasing the successive points in the frequency-domain (Sansalone and Streett, 1997). The frequency resolution, \( \Delta f \), can be calculated by,

\[
\Delta f = \frac{1}{n \Delta t}.
\]

Eq. [2-23]

The frequency resolution described using Eq. 2-23 can be enhanced by increasing \( n \) and \( \Delta t \). However, as mentioned earlier, there is a practical limitation to the product of \( n \) and \( \Delta t \). For example, in impact-echo testing, for a specimen less than 1m thick, a sampling interval of 1 to 4 \( \mu s \), and sample numbers of 1024 or 2048 has been shown to provide acceptable results (Sansalone and Streett, 1997).

### 2.6 Analysis Techniques

There are different analysis techniques used for analyzing signals collected during stress-wave testing. The analysis techniques are carried out both in the time-domain, using velocity and attenuation coefficients, and in the frequency-domain using parameters such as quality factor and transmission frequency shift. The following sections will give a brief introduction on each of the analysis techniques.

#### 2.6.1 Velocity

The use of ultrasonic pulse velocity (UPV) in concrete structures dates back to the 1940’s at Ontario Hydro in Canada for finding the depth of cracks (Leslie and Cheesman, 1949) and at the Road Research Laboratory in the UK, for measuring the thickness of pavements (Jones, 1949). The calculation of the P-wave velocity from the time of first arrival and the peak frequency for
ultrasonic through-transmission, pulse-echo and impact-echo are given by Eqs. 2-24, 2-25 and 2-26, respectively:

\[ C_p = \frac{X}{t}, \quad \text{Eq. [2-24]} \]

\[ C_p = \frac{2X}{t}, \quad \text{Eq. [2-25]} \]

\[ C_p = 2Xf_p, \quad \text{Eq. [2-26]} \]

where \( C_p \) is the P-wave velocity; in Eq. 2-24, \( X \) is the distance that the P-wave propagates between source and receiver; and in Eqs. 2-25 and 2-26, \( X \) is the distance between the surface where the signal is generated at the surface form which it is reflected; \( t \) is travel time and \( f_p \) is the P-wave thickness frequency between the top and bottom of a two layer solid system.

A general criterion proposed by Whitehurst (1951a) for assessing the condition of concrete based on pulse velocity is shown in Table 2–2.

**Table 2–2 Concrete Condition Assessment using Pulse Velocity**

<table>
<thead>
<tr>
<th>Pulse Velocity (m/s)</th>
<th>Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>&gt; 4570</td>
<td>Excellent</td>
</tr>
<tr>
<td>3600-4570</td>
<td>Generally good</td>
</tr>
<tr>
<td>3050-3660</td>
<td>Questionable</td>
</tr>
<tr>
<td>2130-3050</td>
<td>Generally poor</td>
</tr>
<tr>
<td>&lt; 2130</td>
<td>Very poor</td>
</tr>
</tbody>
</table>

2.6.2 Attenuation Coefficient

It is shown by Krautkramer and Krautkramer (1990) that the stress wave intensity over a distance \( d \) can be described by an exponential function as follows:
where $I_d$ and $I_o$ are the intensity at distances $d$ and $0$, and $\alpha$ is known as the attenuation coefficient. For the same distance, the stress-wave propagation can be represented in the time-domain as:

$$I_t = I_o e^{-\alpha t},$$

where $I_t$ is the intensity at time $t$. A schematic representation of this equation is shown in Figure 2-8.

In the literature review section, various techniques are described for calculating the attenuation coefficient. The equation described in Eq. 2-28 is used for calculating the attenuation coefficient in this thesis. Galan (1967, 1990) and Kesner et al. (1998) have used similar techniques. For a better understanding of the literature, the different techniques will be summarized as follows.

Popovics et al. (1994) and Tharmaratnam and Tan (1990) have used the ratio of amplitudes for calculating the attenuation coefficient. The equation can be described as:

$$\alpha = 20 \log_{10} \left( \frac{Amp_1}{Amp_2} \right),$$

where $\alpha$ is the attenuation coefficient in decibels (dB), $Amp_1$ is the amplitude of the first wave arrival when the transmitter and receiver are in direct contact, and $Amp_2$ is the amplitude recorded after the pulse passed through the concrete at any particular time.
Similarly, Ismail et al. (1996) has used Eq. 2-29. However, the two parameters in Eq. 2-29, $Amp_1$ and $Amp_2$, are amplitudes of the P-wave and the S-wave, respectively.

### 2.6.3 Quality Factor

Another analysis technique used in the investigation of attenuation in concrete structures is the quality factor (Q-factor). The Q-factor is a measure of the spectral width for the particular frequency. The equation for the calculation of the Q-factor is given by (Krautkramer and Krautkramer, 1990) as:

$$Q = \frac{f_r}{f_2 - f_1},$$

where $f_r$ is the particular frequency and $f_2$ and $f_1$ are the frequencies at 70.7% of the spectral peak amplitude. A schematic representation of this analysis technique is shown in Figure 2-9.

### 2.6.4 Transmission Frequency Shift

Attenuation is frequency dependent. Stress-waves are attenuated more as the transmitting frequency increases (Whitcomb et al., 1993) because the aggregate and micro-cracks absorb and scatter the higher frequency components of the signal. The high-frequency signals have short wavelengths and are more affected by the composition of the concrete than the low-frequency (long-wavelength) signals. Therefore, the higher frequency components of the signal are more attenuated in comparison to the lower frequency components of the signal. For a fixed path-length, the transmitted pulse frequency shifts to a lower value as the signal is received by a receiving transducer (Whitcomb et al., 1993).
The resonance frequency shift in the impact-echo technique is related to period of the P-wave arrival between the two layers and \( f_p \) is inversely proportional to time. Attenuation causes an increase in the periodic arrival of P-wave to the receiving transducer, thus the thickness frequency shifts to the lower value.

2.7 Stress-Wave Propagation Techniques

In the previous sections, stress-wave generation and propagation principles were described. This section outlines various stress-wave propagation techniques. There are seven different SWT currently used and described in the ACI 228.2R-98 report. The seven different SWT are: the ultrasonic through-transmission method, also known as ultrasonic pulse velocity; ultrasonic-echo, also known as pulse-echo; impact-echo; spectral analysis of stress waves; sonic-echo; impulse-response; impedance logging; and crosshole sonic logging. For each SWT, the underlying methods, principles and applications are summarized in Table 2-2 (ACI 228.2R-98). The section number shows the section where the technique is described in this thesis.

### Table 2-2 Summary of Stress-wave methods

<table>
<thead>
<tr>
<th>Section No.</th>
<th>Method and Principle</th>
<th>Applications</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.7.1</td>
<td>Ultrasonic pulse velocity – measures the travel time of a pulse of ultrasonic waves over a known path length</td>
<td>Determines the relative condition of concrete based on measured pulse velocity</td>
</tr>
<tr>
<td>2.7.2</td>
<td>Ultrasonic-echo – a transducer emits a short pulse of ultrasonic waves which is reflected by the opposite side of the member or an internal defect; arrival of reflected pulse is recorded by an adjacent receiver or the same receiver, and the round-trip travel time is determined</td>
<td>Locates delaminations and voids in relatively thin elements; primarily a research tool</td>
</tr>
<tr>
<td>2.7.3</td>
<td>Impact-echo – an impactor is used to generate a stress wave and a receiver adjacent to the impact point monitors the arrival of stress waves as they</td>
<td>Locates a variety of defects within concrete elements, such as delaminations,</td>
</tr>
<tr>
<td>Method</td>
<td>Description</td>
<td>Notes</td>
</tr>
<tr>
<td>---------------------------------------------</td>
<td>---------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------</td>
<td>-------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------</td>
</tr>
<tr>
<td>Spectral analysis of surface waves</td>
<td>An impact is used to generate a surface wave, and two receivers monitor the surface motion; signal analysis allows the determination of wave speed as a function of wavelength; an inversion process determines elastic constants of layers.</td>
<td>Determines the stiffness profile of a pavement system. Also used to determine depth of deteriorated concrete.</td>
</tr>
<tr>
<td>Sonic-echo</td>
<td>A hammer impacts the surface and a receiver monitors reflected stress wave. Time-domain analysis is used to determine travel time.</td>
<td>Determines the length of deep foundations (piles and piers); determine the location of cracks or constrictions (neck-in).</td>
</tr>
<tr>
<td>Impulse-response</td>
<td>Test is similar to sonic-echo method except the signal processing involves frequency-domain analysis of the received signal and the impact force history.</td>
<td>Determines the length of deep foundations (piles and piers), location of cracks or constrictions (neck in). Provides information on the low-strain dynamic stiffness of the shaft/soil system.</td>
</tr>
<tr>
<td>Impedance logging</td>
<td>Test is similar to sonic-echo or impulse-response, but the use of more complex signal analysis (time and frequency domains) allows reconstructing the approximate shape of the deep foundation.</td>
<td>Determines the approximate 2-dimensional shape of the deep foundation.</td>
</tr>
<tr>
<td>Crosshole sonic logging</td>
<td>Analogous to the ultrasonic pulse velocity test, but transducers are positioned within tubes cast into the deep foundation or holes drilled after construction.</td>
<td>Determines the location of low-quality concrete along the length of the shaft and between transducers. With drilled holes permits direct determination of shaft length.</td>
</tr>
</tbody>
</table>

### 2.6.1 Ultrasonic through-transmission method

The ultrasonic through-transmission (UTT) method is the oldest non-destructive stress-wave method used for concrete inspection. It is based on measuring the time of propagation for ultrasonic pulses over a known distance: the velocity is calculated by dividing the distance
propagated by the time taken. The method has been used by Jones (1949) for concrete quality assessment and Leslie and Cheesman (1949) for crack detection. Its application has increased with the passing of time. To date, UTT has been used for: estimating concrete strength, determining the homogeneity of concrete, determining the dynamic modulus of elasticity and Poisson’s ratio, studying the hydration of cement (e.g. setting time), studying the durability of concrete, and measuring the location and depth of cracks (Naik and Malhotra, 1991). A test method for determining the pulse velocity through concrete was first standardized by ASTM in 1967 (C597-67T). The current edition of the standard test, ASTM C 597-97, differs little from the originally published version.

2.6.1.1 Principles of Ultrasonic through-transmission method

The testing set-up or principle behind the UTT method is shown in Figure 2–10. Ultrasonic pulse signals are transmitted and received using piezoelectric transducers. The transducers have to be coupled with the test object in order to avoid possible error due to an air gap at the transducer concrete interface. Different couplants are commonly used: e.g. petroleum jelly, grease, liquid soap, and kaolin-glycerol paste (Naik and Malhotra, 1991). This minimizes the attenuation of ultrasonic pulses at the concrete-transducer interface. The signals collected using the receiving transducers may be stored in a time-domain format. The time-domain signals are used for estimating the time of first arrival, predominantly P-wave or ultrasonic wave, and hence calculating the ultrasonic pulse velocity using Eq. 2-24.
NDT Test: Through-Transmission

As shown schematically in Figure 2–10, the signal is stored in the time-domain and may be converted to the frequency-domain using FFT techniques. Therefore, the techniques described in Section 2–5 can be carried out on the time- and frequency-domain signals.

There are three different arrangements for locating transducers on the surfaces of concrete structures, as shown in Figure 2–11.

Direct transmission (Figure 2–11a) is the most sensitive, and indirect transmission (Figure 2–11c) is the least sensitive (Komlos et al., 1996). This is because with direct transmission, maximum pulse energy is transmitted and received (Naik and Malhotra, 1991). In the semi-direct transmission (Figure 2–11b) the S-wave is predominantly collected.
2.7.2 Ultrasonic-echo method

The use of experimental ultrasonic-echo or pitch-catch systems started in the early 1960s (Sansalone and Carino, 1991). If the transmitter is used as a receiver, it is known as ultrasonic-echo, whereas if a different receiving transducer is used, it is known as the pitch-catch method. With currently available transducers, the true ultrasonic-echo method, with a single transducer functioning as a transmitter and a receiver, is not used in practice because of possible signal overlapping. In the true ultrasonic-echo test, if the transducer is not damped, the vibration of the piezoelectric transducer will make it unresponsive to echoes (Sansalone and Carino, 1991). However, if the transducer is heavily damped, its sensitivity to displacement will be compromised. Thus, there is a need for careful judgement in the trade off between sensitivity and vibration of the transducer. To overcome the problem of one single transducer in the true ultrasonic-echo method, a separate transmitter and a receiver located adjacent to the transducer is used. This method is called pitch-catch (ACI 228.2R-98), and it is used for measuring the...
thickness of thin slabs, pavements, and walls; measuring the length of piles and surface opening cracks (Sansalone and Carino, 1991).

2.7.2.1 Principles of Ultrasonic-Echo

The ultrasonic-echo method works on the principle that an ultrasonic pulse is transmitted to the test object, reflected by the object, and received through a receiving transmitter. The displacement received is converted into a time-domain signal. A schematic presentation of the pitch-catch method is shown in Figure 2–12.

**NDT Test: Pulse-Echo**

![Diagram of Pulse-Echo method]

\[ C_p = \frac{2X}{t} \]

*Figure 2–12 Pulse-Echo method*
As shown schematically in Figure 2-12, which depicts the setup for the pitch-catch method, for a stress wave propagation distance, \(2X\), and the first time of arrival, \(t\), read from the time-domain graph, the propagation speed can be calculated using Eq. 2-25.

However, if there is any flaw in the concrete, and if the speed of propagation is known, using the shorter first time of arrival, \(t\), the location of the flaw (i.e. depth beneath the surface) can be calculated by

\[
X = \frac{t \cdot C_p}{2}.
\]

**Eq. [2-31]**

### 2.7.3 Impact-echo method

Stress wave techniques using an impactor dates back to the 1970s (ACI 228.2R-98). This method used to be referred to as seismic-echo (or sonic-echo), and it has been used to evaluate concrete piles and drilled shaft foundations. The impact-echo method for the testing of concrete was introduced in the mid 1980s by the U.S. National Bureau of Standards and Cornell University (Sansalone and Streett, 1997). The research was mainly focused on flaw detection in concrete using transient stress waves (Sansalone, 1986). To date, the method has found a wide area of applications. It can be used to determine thickness, to locate cracks, voids and other defects in different structures (Sansalone and Streett, 1997); to estimate setting time, and concrete strength (Pessiki and Carino, 1987; Pessiki and Carino, 1988). A thorough review of the impact-echo technique was presented by Sansalone and Streett (1997) and in the following subsection, brief summaries of the principles are reviewed.
2.7.3.1 Principles of Impact-Echo

Impact-echo works on the principle that an elastic impact is used to generate a transient stress wave in the test object. A schematic diagram of the test method is shown in Figure 2-13. A small spherical steel ball (or hammer) is used to generate low-frequency stress waves. The stress waves are reflected back from internal flaws or external surfaces. The reflected waves are collected using a displacement sensitive broadband transducer located adjacent to the impact source. The response received by the transducer is saved as a digital wave format in the time-domain. The signal from the time-domain is converted to the frequency-domain using the Fast Fourier Transfer function (See Section 2.4).

Multiple reflections of stress waves between the impact surface and flaws and/or the external surface give rise to transient resonances, which can be identified in the frequency-domain. This is the underlying principle of the impact-echo technique. The response in the frequency-domain is used to evaluate the integrity of structures and determine the location of flaws. For circular and rectangular columns as well as rectangular, I-, and T-beams; impact-echo tests produce distinctive time- and frequency-domain response, and hence the dominant patterns are easily recognized (Sansalone and Streett, 1997).

Since impact-echo is based on the principle of collecting reflected waves from interfaces, the coefficient of reflection, $R_m$, in Eq. 2-8 is a crucial parameter. For the reflected stress wave to have a distinct amplitude in the frequency-domain, the minimum coefficient of reflection required is 0.24 (Lin and Sansalone, 1994a,b). For example, a stress wave generated on the surface of the test object travels through the concrete and is reflected back at the concrete soil
NDT Test: Impact-Echo

interface. For concrete with an acoustic impedance, $Z$, value of $7 \times 10^6$ and soil with a $Z$ value of $3 \times 10^6$, the $R_n$ value, according to Eq. 2-8, is -0.40. Therefore, since the calculated $R_n$ value of 0.40 is over the minimum required 0.24, there will be distinct amplitude for the P-wave thickness frequency in the frequency-domain.

In the impact-echo testing, the frequency (or wavelength) of the propagating stress waves is crucial for identifying defects and measuring thickness. In detecting flaws, the wavelength ($\lambda$) has to be smaller than the lateral dimension of the flaw and smaller than twice the depth to the flaw (Sansalone and Streett, 1997). However, if $\lambda$ is too large, the periodic displacement pattern in the waveform will not correspond to the period between the successive P-wave arrival.
2.7.3.2 Impact Duration

As shown in Figure 2–13, stress waves are generated by a mechanical impactor, usually a steel sphere. During contact, the kinetic energy of the steel sphere is transformed into elastic wave energy in the concrete (Sansalone and Streett, 1997). Thus the kinetic energy determines the displacement of the resulting stress waves. The impact duration or contact time, $t_c$, is determined by (Sansalone and Streett, 1997):

$$t_c = \frac{0.0043D}{h^{0.1}}, \quad \text{Eq. [2-32]}$$

where $t_c$ is in seconds and $D$, the diameter of the sphere, and $h$, the drop height, are in metres. However, since the drop height is small, the dependence of $t_c$ on $h$ is weak. Thus Eq. 2-32 is approximated by (Sansalone and Streett, 1997):

$$t_c = 0.0043D. \quad \text{Eq. [2-33]}$$

It was shown by Sansalone and Streett (1997) that when the force-time graph is converted using a Fast Fourier Transform (FFT) function into the frequency domain, at a frequency of $1.5/t_c$, the amplitude is zero. Thus the maximum useful frequency (a frequency with sufficient energy), $f_{\text{max}}$ (in Hz), is set at $1.5/t_c$. This can be approximated by (Sansalone and Streett, 1997):

$$f_{\text{max}} = \frac{291}{D}, \quad \text{Eq. [2-34]}$$

where $f_{\text{max}}$ is in Hz and $D$ is in metres. For a diameter of sphere 0.015 m, a useful frequency is produced up to 19.4 kHz.
2.8 Stress-Wave Application

2.8.1 Setting Time

Whitehurst (1951b) carried out one of the earliest tests on the use of UPV to measure the setting time of concrete. He used 100mm x 100mm x 400mm concrete beams with different cement types. Stiff concrete mixtures were used having slumps of 3 to 9 mm. These stiff mixes allowed early demoulding of the ends of the beams for testing purposes. Whitehurst reported that successful measurements were not possible from 2 to 4 hr. after casting, depending on the cement used. The measured velocity was as low as 1,200 m/s. He found that during the period 4.5-8.5 hr. after the initial mixing, there was a rapid rate of increase in velocity. After this, the rate of change in velocity decreased sharply. Whitehurst showed that the time of set is defined by the intersection of the tangent lines drawn before and after the rate of change in velocity decreased sharply. The setting times were in good agreement with the setting time as determined on cement pastes using Gillmore needles (ASTM 1102-94).

Popovics et al. (1994) studied ultrasonic properties in fresh concrete. Freshly placed concrete samples in 100 mm x 200 mm plastic moulds were used. The test was started as early as 10 min. from the time of casting. Popovics et al. (1994) reported velocity readings as low as 1,000 m/s, lower than the velocity of sound in water (1,500 m/s). This is due to the suspended state of cement and aggregate particles in fresh concrete. The authors showed that after this stage, there is a dormant period with a maximum velocity of 1,100-1,200 m/s where there is little change in the plasticity of the paste despite the intensive chemical reactions between cement and water. Following this period, a decrease in velocity ending in small dip in the velocity-time graph is
reported as the concrete undergoes its initial set (3 to 4 hours after casting). After the initial set, the velocity increases rapidly until the age of 24 hours.

Popovics *et al.* (1994) used another analysis index, the attenuation (\(\alpha\)) coefficient. The \(\alpha\) coefficient is calculated by taking the ratio of the first arrival peak amplitude of the test through the specimen and the transducers in contact, respectively (see Eq. 2-29). They showed that \(\alpha\) has good correlation with velocity, (correlation factor = 0.94). They suggested that this shows UPV and \(\alpha\) may be co-variables in concrete. An \(\alpha\) factor of 70 dB was recorded at an age of 10 minutes followed by a decrease to 63–65 dB at an age of one hour. An increase in \(\alpha\) to around 80 dB was observed at an age of 2 to 5 hours after casting. This is around the set time of the concrete. Following this period, \(\alpha\) decreased sharply and leveled off after 24 hrs. They showed that there is less regularity in \(\alpha\) than in velocity. They gave two possible explanations: first, amplitude measurement is less reliable than the measurement of the time of first arrival; and second, \(\alpha\) may be more sensitive than UPV to changes in the internal structure. They showed that at a later stage, the values of \(\alpha\) for the three mixes approached each other due to the probable homogenization of concrete with continued hardening.

Pessiki and Carino (1988) carried out tests to study the feasibility of the impact-echo method in measuring the setting time of concrete. The P-wave velocity method was compared with the ASTM C 403 standard test method for time of setting of concrete. Cylindrical specimens were cast with dimensions 152 mm x 160 mm using different concrete mixes, these were: low w/c ratio, low w/c ratio with retarding and accelerating admixtures, and high w/c ratio. For most of
the mixes, the velocity increased after 3 hr. at a well-defined line. Pessiki and Carino presented two methods for estimating the setting time: the time at the start of the increase in P-wave velocity, and the time to reach a specified P-wave velocity. In the first method, the time at the start of the increase in P-wave velocity was determined to correspond with a velocity of 300 m/s. They showed that the relationship between the initial setting times measured by penetration resistance and the time the P-wave velocity began to increase had good correlation, (correlation coefficient = 0.996). They showed that it is sometimes difficult to pick the time where the P-wave velocity starts to increase. In the second method, the time to reach a specified P-wave velocity of 670 m/s was used. Pessiki and Carino (1988) explain that although this method is easy to use, the velocity is significantly affected by the mix composition. For example, they found that the 3 mixes with a w/c ratio of 0.40 had a correlation coefficient of 1.00 between target P-wave velocity and initial set measured by penetration resistance. However, they showed that when this curve was used for a mix with a w/c ratio of 0.80, the P-wave velocity underestimated the setting time.

2.8.2 Strength development

Research in the use of UPV to estimate concrete strength dates back to the 1950’s at the Road Research Laboratory (Jones, 1953). Jones was correlating compressive strength of concrete with UPV. To date this method has been used for estimating compressive strength by Carette and Malhotra (1984), Pessiki and Carino (1988), Ravindrarajah (1997), Sturrrup et al. (1984), Swamy and Al-Hamed (1980), and Tomsett (1980). This method is standardized and given in ASTM C 597-83 (Reapproved 1991) “Standard Test Method for Pulse Velocity Through Concrete.” Komlos et al. (1996) have summarized standards used in different countries.
The strength of concrete is affected by the interaction of the elastic and viscoelastic components (aggregate, sand, and cement paste) and the inelastic components (voids and cracks). Therefore, in the use of SWT to estimate concrete strength, both the elastic and inelastic components should be considered. Research along this line was first published by Galan (1967). Galan used UPV, to estimate the elastic component, and \( \alpha \), to estimate the inelastic component. To date, this method has been used by Galan (1990), Ismail et al. (1996), and Tharmatnam and Tan (1990).

Another NDT technique based on the combined results of UPV and the rebound hammer test, known as SONREB, was developed by Facaoaru (1984). This method is based on the principle that the UPV-test models the strength of the internal part of the concrete structure and the rebound-test models the strength of the external surface of the concrete structure. It is assumed that the combined model of the two gives a good representation of the concrete. This method has been used by Carette and Malhotra (1984), Facaoaru (1984), Di Maio (1995), Teodoru (1988), Samarín and Dhir (1994), Tanigawa et al. (1984), and Yun et al. (1988).

Since the main objective of this thesis is the use of stress-waves in concrete, the following section will present a literature review on the application of stress-waves to estimate concrete strength using UPV; and UPV plus attenuation indices.
2.8.2.1 Ultrasonic pulse velocity

Ultrasonic pulse velocity (UPV) is one of the oldest SWT methods for assessing concrete quality. The $C_p$ of concrete in relation with the $E_d$ of the material is given in Eq. 2-1. It was shown by Neville (1997) that the concrete strength is related with $E_d$ by:

$$E_d = a\sqrt{f'_c}.$$  \hspace{1cm} \text{Eq. [2-35]}

where $a$ is a constant and $f'_c$ is concrete strength.

Combining Eq. 2-1 and Eq. 2-35, $f'_c$ is related to $C_p$ by:

$$f'_c = b.C_p^4,$$ \hspace{1cm} \text{Eq. [2-36]}

where $b$ is a constant.

Generally, many researchers (Facaoaru (1984), Galan (1967, 1990), Samarin and Dhir (1984) and Teodoru (1988)), have expressed the relationship between concrete strength and velocity by

$$f'_c = a.e^{kC_p}.$$ \hspace{1cm} \text{Eq. [2-37]}

Based on the principle of Eq. 2-37, Tomsett (1980) introduced the paste efficiency concept. This method is used to estimate an in-situ compressive strength of concrete based on the compressive strength of lab-cured concrete and comparing the velocity readings for water-cured and in-situ concrete (Tomsett, 1980)

$$\ln\left(\frac{f'_c}{f'_a}\right) = b(V_w - V_a) = kV_w(V_w - V_a),$$ \hspace{1cm} \text{Eq. [2-38]}

where,
\( f'_w \) = standard cube strength (water-cured),
\( f'_s \) = standard cube strength (in-situ concrete),
\( V'_w \) = standard cube pulse velocity (water-cured),
\( V'_s \) = standard cube pulse velocity (in-situ concrete),
\( k \) = constant dependent on compaction, moisture content and mix proportions.

Tomsett (1980) showed that the values of \( k \) range from 0.005 to 0.025 for good compaction and poor compaction, respectively. For normal-strength concrete, Tomsett (1980) suggested a \( k \) value of 0.019. Swamy and Al-Hamed (1984) further verified the validity of Eq. 2-39. They found that the values of \( k \) ranged from 0.015 to 0.027.

Ravindrarajah (1992) applied the paste efficiency concept for the estimation of compressive strength of high-strength concrete. He found that the value of \( k \) at 28 days ranged from 0.008 to 0.012 with an average of 0.010, which is less than the value of 0.019 for normal-strength concrete (Tomsett, 1980; Swamy and Al-Hamed, 1984). Ravindrarajah (1992) showed that the reason for this smaller \( k \) value of 0.010 occurred because high-strength concrete, which has a lower water cement ratio, retains most of its moisture when it is air cured.

### 2.8.2.2 Combined Method

Due to the heterogeneous properties of concrete, the propagation of stress waves is affected both by the elastic and inelastic components. Research carried out thirty years ago by Galan (1967), used UPV to estimate the elastic components (aggregate and cement paste) and \( \alpha \), to estimate inelastic components (air voids and cracks). This was a breakthrough for researchers in the NDT field. Galan (1967) showed the exponential relationship between strength and the combined method of UPV and \( \alpha \) by
\[ f' = aC_p^b \alpha^c, \]  
Eq. [2-35]

where \( a, b, \) and \( c \) are constants.

Galan (1967, 1990) carried out tests using a wide range of concrete mixes. The results show a correlation coefficient of up to 0.99 using the combined method given in Eq. 2-35. The \( \alpha \) coefficient is calculated using the exponential decay curve of the time domain graph of amplitude vs. time.

Tharmaratnam and Tan (1990) used the combined method, shown in Eq. 2-35, to estimate the strength of a mortar mix. They achieved a correlation coefficient of up to 0.93 using this method. The attenuation coefficient, \( \alpha \), was calculated using the peak amplitude ratio of the first wave arrival when the stress wave propagates through the material and when the transducers are in contact, respectively (see Eq. 2-29). Since the distance of propagation affects the peak amplitude, they developed a correction factor to account for the difference.

Ismail et al. (1996) also used the combined equation for concrete strength estimation. Tests were conducted on hundreds of 150-mm cube samples prepared using ordinary Portland cement. With the combined method, a +/- 5% difference between the actual and estimated concrete strength was achieved. However, when the stress-wave parameters, UPV and \( \alpha \), were used separately, a larger deviation of between 20 to 35% was recorded. The \( \alpha \) coefficient was calculated using the ratio of the first-arrival wave to the first-arrival shear wave (see Eq. 2-29).
2.8.3. Deteriorated Concrete

2.8.3.1 Alkali-Silica Reaction

Alkali-silica reaction (ASR) is a reaction between alkali hydroxides from cement and amorphous or microcrystalline silica in certain aggregates. First described over 50 years ago, this deleterious reaction manifests itself as expansion, and eventual cracking of concrete. ASR was first identified as a deterioration mechanism in concrete in California in the late 1930s (Stanton, 1940). However, the published use of NDT to evaluate ASR-affected concrete is a relatively recent development (Swamy and Wan, 1993). Various studies have been conducted on lab-cast concrete, cores taken from structures, and structures in service.

It is well established that expansion due to ASR leads to a reduction in the stiffness of concrete (Pantazopoulou & Thomas, 1996), (Thomas, 1997), and consequently the P-wave velocity (Thomas, 1997; Thomas et al., 1997).

Tests done on concrete core samples may not give an actual representation of the structure in service. Okada (1986) showed that the UPV tends to be larger in drilled cores if the cores do not have large defects. Akashi et al. (1986) observed that the UPV of ASR-affected cores decreased until it reached a minimum, and then the UPV increased to more than that of the undamaged test specimen. They attributed the recovery of the UPV to silica gel filling the cracks.

The location where the concrete core sample is taken affects the results significantly. Akashi et al. (1986) showed that the UPV velocity of ASR-affected concrete cores within reinforcement
cages decreased much more than that of cover concrete cores. Jones et al. (1994) described this phenomenon as follows: when the concrete is cored, an immediate release of restraint causes the ASR-affected concrete to expand. Jones et al. (1994) suggested testing the cores as soon as possible after coring.

Blight and Alexander (1986) found that on a large, badly cracked foundation that a low UPV indicated severe deterioration. However, the deterioration did not extend beyond a 300-mm depth. When they used UPV from the bottom of the core holes, there was no deterioration.

Similar results were found by Imai et al. (1986). Tests were done on cores taken both from damaged and sound piers in the Hanshin Expressway. Applying UPV measurement, they found a 35% reduction in velocity for an ASR-affected concrete pier. Nevertheless, the stiffness and load carrying capacity of an ASR-affected concrete pier was the same as that of a sound pier. This is because heavily reinforced, ASR-affected concrete is slightly enhanced due to the pre-stressing effect resulting from restrained expansion (Thomas, 1997).

Various analytical indices are used to analyze ASR-affected concretes. Akashi et al. (1986) used frequency-domain analysis. They found that, as the degree of deterioration increased, frequencies higher than 20 kHz were attenuated.

Al-Akras et al. (1996) used frequency analysis in the power spectrum (frequency spectrum). They found that at very low expansion, there was a shift in the high frequency components to a lower value, whereas, the UPV showed no significant change.
Kesner et al. (1998) used the $\alpha$ coefficient in the impact-echo method to study distributed damage mechanisms: delayed ettringite formation (DEF) and alkali-silica reactivity (ASR). They found that as the damage increases, $\alpha$ decreases. Kesner et al. developed a correlation between $\alpha$ and the crack density quantified using neutron radiography.

2.8.3.2 Effects of Freezing and Thawing

Deterioration due to freezing and thawing cycles is a primary concern for structures in cold climates. Freezing and thawing is a process where water in the capillary pores freezes and expands. When the tensile pressure exerted exceeds the tensile force of the cement paste, cracking results. As a consequence, during subsequent thawing, the water migrates further and causes more damage during freezing. Observation of cracking via optical microscopy reveals that lab specimens damaged by freezing and thawing contain uniformly distributed microcracking (Sellick et al., 1998).

The standard test method for measuring the resistance of concrete to freezing and thawing cycles is ASTM C666. This method is used to monitor the dynamic modulus of elasticity, $E_{dt}$, of concrete specimens subjected to cycles of freezing and thawing whilst in a saturated condition. Although this method is reliable, its application in-situ is not possible. Hence, there is a need for using a NDT method in-situ.

In monitoring structures affected by freezing and thawing, it is important to know before hand if there was an extended thawing or curing of the concrete. Jones (1953) has reported that during
an extended thawing, the UPV significantly improved. Similarly, for lab specimens, Jacobsen et al. (1996) reported for concrete deteriorated by freezing and thawing cycles, the UPV significantly improved during subsequent storage under water. However, Jacobsen et al. (1996) reported that the increase in the compressive strength was unacceptable.

Selleck et al. (1998) used the UTT method for monitoring deterioration due to freezing and thawing cycles. The signals were analyzed using an attenuation index. They reported that the attenuation index was sensitive despite a variation in measurement.

Apart from tests in the time domain, Selleck et al. (1998) reported the sensitivity of peak frequency measurements. El-Korchi et al. (1989) reported the sensitivity of the $Q$-factor of the resonance peak frequency; during freezing and thawing deterioration, the $Q$-factor decreased significantly.

2.8.3.3 Effect of Loading

Jones (1953), the first to carry out research on the effect of stress-induced cracking on the ultrasonic-pulse velocity, showed that the velocity measurement in the direction of loading is not appreciably affected. Jones (1953) showed that the decrease in velocity was only 20% prior to failure. Sauris and Fernando (1987) reported a similar result where the decrease in velocity, prior to failure, was only about 12%. Daponte et al (1990), and Popovics and Popovics (1991) have shown that velocity was only affected above around 70-80% of the ultimate load.
It is interesting to see that pulse-velocity up to about 25% of the ultimate load gradually increased (Popovics and Popovics, 1991). This is thought to be due to the initial closure of the voids before the onset of cracking. Wu and Lin (1988) have illustrated this hypothesis further. They found that the UPV measurement for a mortar remained constant despite increasing the stress. However, the UPV measurement for a concrete increased with increase in stress. The mortar has fewer interfaces because of the absence of coarse aggregate.

Researchers have reported different analysis parameters thought to be more sensitive than velocity. Sauris and Fernando (1987) measured ultrasonic attenuation and reported that at 90% of the peak stress, the reduction in amplitude was up to 30%, and at 99% of the peak stress, the reduction in amplitude was up to 70%. Daponte et al. (1990) showed that the attenuation index was sensitive to stress at loads as low as 20-30% of the ultimate load.

Apart from studies in the time-domain, researchers have studied the effect of stress in the frequency-domain. Daponte et al. (1990) reported that the transmission frequency shift to a lower value was sensitive to loading as low as 5-10% of the ultimate load. Drabkin and Kim (1994) have reported similar sensitivity to frequency. Drabkin and Kim reported that the high frequency component of the stress-wave signal is attenuated, leaving only signals in the lower frequency range.
CHAPTER 3
SETTING AND STRENGTH DEVELOPMENT

This chapter reports the findings from a laboratory test program aimed at evaluating the use of stress waves for monitoring concrete setting and strength development. To carry out this task, concretes were cast with three different water/cement ratios and monitored both for setting and strength development. Results collected from the standard ASTM tests for setting time (penetration resistance) were compared with the results from non-destructive testing and relationships were developed between various parameters. The validity of the relationships for estimating strength development were evaluated by casting new concrete specimens of similar composition and testing them over a period of 90 days.

3.1 Materials

It was shown by Sturrup et al. (1984) and Jones (1953) that varying the concrete constituents will affect the relationship between concrete strength and stress wave velocity. One of the principal constituents that influence this relationship is the coarse aggregate as both the type and content of aggregate have been shown to have a significant effect (Sturrup et al., 1984). In the present study, three mixes were cast with the same water content but varying cement contents. This produced a range in both the water/cement ratio (from 0.4 to 0.6) and aggregate/cement
ratio (3.9 to 6.3) of the concretes tested. For the purpose of this program the type of coarse (crushed limestone) and fine aggregate (glacier sands with mixed carbonate and siliceous) were the same for all three mixes. The mix designs used are shown in Table 3-1.

Table 3-1 Composition of the three concrete mix designs

<table>
<thead>
<tr>
<th></th>
<th>Mix 1</th>
<th>Mix 2</th>
<th>Mix 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Portland cement-St. Mary’s T10 (kg/m³)</td>
<td>450</td>
<td>360</td>
<td>300</td>
</tr>
<tr>
<td>Coarse Aggregate (10-20 mm)</td>
<td>709.71</td>
<td>740.57</td>
<td>761.14</td>
</tr>
<tr>
<td>Coarse Aggregate (5-10 mm)</td>
<td>354.86</td>
<td>370.29</td>
<td>380.57</td>
</tr>
<tr>
<td>Fine Aggregate – Lab Sand</td>
<td>709.71</td>
<td>740.57</td>
<td>761.14</td>
</tr>
<tr>
<td>Water</td>
<td>180</td>
<td>180</td>
<td>180</td>
</tr>
<tr>
<td>25 XL, ml/m³ (water reducer)</td>
<td>1350</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Density (kg/m³)</td>
<td>2404</td>
<td>2391</td>
<td>2383</td>
</tr>
<tr>
<td>Water/cement ratio</td>
<td>0.40</td>
<td>0.50</td>
<td>0.60</td>
</tr>
<tr>
<td>Aggregate/cement ratio</td>
<td>3.94</td>
<td>5.14</td>
<td>6.34</td>
</tr>
</tbody>
</table>

3.2 SPECIMEN PREPARATION

The mixtures shown in Table 3-1 were cast according to the ASTM C 192/C 192M – 95 Standard Practice for Making and Curing Concrete Test Specimens in the Laboratory. The coarse aggregate used was washed and sieved to provide the specified gradation. The mixing procedure was as follows:

- The mixing pan was wetted with a damp cloth,
- The coarse aggregate and sand were added and mixed for one min.,
- The cement was added and mixed further for two minutes,
- After the dry components were thoroughly mixed, water was added over a period of 1 minute. As soon as the water touched the mix, the timer was started,
The concrete was mixed for 3 min. and allowed to rest for a further 2 min. Then it was mixed another 2 min. and rested for 1 min. During this rest period the slump was measured. Following the rest period, it was mixed for a final 1 min.

After completing the mixing procedure, for the SWT setting development test, fresh concrete was placed in a 150-mm diameter x 300 mm cylinder in three layers and each layer was hand rodded 25 times. Following the final rodding, the cylinder was placed in lab air until the time of test.

For the ASTM C403/C 403M – 95 Time of Setting of Concrete Mixtures by Penetration Resistance, the fresh concrete mix was wet-sieved through a 4.75-mm sieve. The sieved mortar was thoroughly mixed by hand prior to placing it in 150-mm diameter x 200-mm cylindrical moulds in layers. The cylindrical moulds were sealed with caps and stored in lab air until the time of test.

As for the strength development test, the fresh concrete mix was placed in 150-mm diameter x 300 mm cylinders in three layers and each layer was hand rodded 25 times. After completing the final rodding, the surface was finished and all cylinders were sealed with plastic caps. The cylinders were covered with wet burlap and plastic sheets for 24 hrs. After 12 hours one cylinder from each mix was demolded for testing. All other cylinders were demolded at 24 hours, end ground, and subsequently stored in the fog room at 23°C. For each mix design, seven concrete cylinders (152 x 305 mm) were cast. One of these cylinders was tested at 12 hrs, 1, 2, 3, 7, 14, and 28 days.
3.3 Specimen Testing

3.3.1 Setting Development

For the setting development test, two SWT tests, impact-echo (IE) and ultrasonic through-transmission (UTT), were carried out. The test setup used for the setting development test in the UTT test is shown in Figure 3–1. The tests were carried out on a 150 mm diameter x 300 mm concrete specimen that was cast into a special testing rig to facilitate ultrasonic-pulse and impact-echo testing at early ages. A schematic diagram of the test apparatus is shown in Figure 3–2.

Before the concrete was placed, the transmitting transducer was inserted in the bottom plate and held in place by the bottom adjustable supporter. Following this procedure, the cylindrical mould, with a hole at the bottom to accommodate the transducer, was placed over the UTT transmitting transducer on to the support plate. The transducers had specially made conical shape attachments, 40-mm diameter by 40-mm height, to facilitate testing at the early age. To prevent leakage of the concrete at the bottom cylindrical hole and transducer interface, the perimeter of the hole was sealed by plasticine. Following this, the fresh concrete was placed as described in Section 3.2. When the final layer of the concrete was compacted, the transducer at the top plate was clamped in place. In carrying out the UTT test, the transducer at the bottom plate is used as a
transmitter and the one at the top plate is used as a receiver. This helps in using the receiver for both UTT and IE. In performing the IE test, a plastic sheet was placed over the fresh concrete and the impact with the steel ball was made through the sheet.

Concurrently with SWT tests, the ASTM C403/C 403M – 95 Time of Setting of Concrete Mixtures by Penetration Resistance was carried out on the mortar mixes.
3.3.2 Strength Development

For the strength development tests, for each concrete cylinder, two SWT tests, IE and UTT, were carried out, following which the compressive strength of the same cylinder was determined. Hence each data point recorded is the result of the NDT testing and compressive strength test on the same cylinder. The test setup used for the UTT testing technique is shown in Figure 3–3. A schematic drawing of the testing rig is shown in Figure 3–4.

In carrying out the UTT test for strength development, two transducers were used, one as a receiver and the other as a transmitter. The transducers were clamped to the adjustable transducer support frames. The specimen was placed on a specimen support. The support frames were moved until the two transducers touched the specimen and the wing nuts were tightened.
3.3.3 Stress-waves technique

The SWT testing equipment used was the AndecScope™, a SWT instrument capable of carrying out tests in the IE, UTT, and UPE. The instrument was manufactured by Andec Mfg. Ltd. See Appendix A for further detail.

For the UTT testing technique, a 50-kHz piezoelectric transducer was used for transmission and a flat broadband 150-kHz piezoelectric transducer was used as a receiver. To amplify the signal, an external preamplifier was attached to the receiving transducer. To ensure good surface contact, petroleum jelly (Vaseline) was used as a couplant. The sampling rate used was 2 MHz and the sample length analyzed was 1024 points. For each test carried out, 100 signals were averaged to improve the signal to noise ratio.
For the IE testing technique, a steel ball was used to generate a stress wave and a flat broadband 150-kHz transducer was used as a receiver. A preamplifier was attached at the receiving end of the transducer. The sampling rate used was 10 MHz and the sample length analyzed was 16,000. For each test carried out, 5 signals were averaged to improve the signal to noise ratio. A sample of the signal captured by the IE technique in the time-domain and the corresponding contact time is shown in Figure 3-5. The shaded area in Figure 3-5 shows the area that is magnified in the bottom figure.

Samples of the signals captured by the UTT technique are shown in Figure 3-6 for time-domain, and in Figure 3-7 for frequency-domain. The IE signals are shown in Figure 3-8. The associated data analyses are as follows.

For the UTT technique, velocity is calculated using Eq. 2-24. For a cylinder tested with a length of 0.300m, the time of first arrival from Figure 3-6a is 67.3 μs. Hence the velocity is 4460 m/s. The peak frequency is 31.25 kHz.

The attenuation coefficient and quality factor are calculated on the filtered signal. The filtering process is demonstrated in Figure 3-7a. This is carried out first by the identifying the peak frequency amplitude, the signals before and after the peak frequency are filtered using a high pass and low pass filter, respectively. In this example, only the high-pass filter is performed. The time- and frequency-domains of the filtered signal are shown in Figures 3-6b and 3-7b, respectively.
The attenuation coefficient is calculated using Eq. 2-28. By taking the successive peak amplitude starting the first peak in Figure 3-6b, a best-fit curve using Eq. 2-28 is superimposed. The exponential decay constant, \( \alpha \), from the best fit curve given by Eq. 2-28 is ±2.44. It is expected that as deterioration increases, the \( \alpha \) coefficient will become more negative. For example, with an ideal material where there is no attenuation, the exponential decay in Figure 3-6b will not be exhibited. Hence, the associated \( \alpha \) coefficient of the exponential decay curve will be 0.

The \( Q \)-factor is calculated using Eq. 2-30. The calculation is carried out on the filtered frequency-domain signal. As shown in Figure 3-7b, the peak frequency is 31.25 kHz. At 70\% of the peak amplitude, the corresponding frequencies, \( f_2 \) and \( f_1 \) are 32 and 28.7 kHz, respectively. Substituting the corresponding frequencies in Eq. 2-30, the \( Q \)-factor is 9.46. It is expected, with deterioration that the \( Q \)-factor would decrease. This is because of the decrease in the peak frequency compounded by an increase in the spectral width (\( f_2 - f_1 \)).

For the IE technique, the \( \alpha \) coefficient and \( Q \)-factor are calculated the same way as for the UTT technique. Velocity is calculated using Eq. 2-26, which is dependent on the P-wave thickness frequency between the top and bottom of the cylinder. As shown in Figure 3-8a, the time between the successive P-wave arrival, \( \Delta t \), is 149 \( \mu \)s. Since frequency is \( 1/\Delta t \), the P-wave thickness frequency is 6.71 kHz. This is shown in Figure 3-8b as a peak frequency. Hence using Eq. 2-26, the velocity for a P-wave thickness frequency of 6.71 kHz and thickness of 0.300m is 4029 m/s.
Figure 3-5 Typical waveform collected using a steel ball on a cylinder of 300 mm thickness
Figure 3–6 UTT, Time domain graph a) Unfiltered signal b) Filtered signal
Figure 3-7 UTT, Frequency domain graph
a) Unfiltered signal b) Filtered signal
Figure 3-8 IE signal a) Time domain b) frequency domain
3.4 Setting Time Determination

Concurrently with the SWT tests, the ASTM C403/C 403M - 95 (Time of Setting of Concrete Mixtures by Penetration Resistance) was carried out on the mortars extracted from the concretes. Before carrying out the penetration test, the bleed water was removed from the mortar specimen. Following this the bearing surface of the appropriate needle and the mortar surface were brought into contact. Then a downward vertical force was gradually applied until the needle penetrated the mortar to a depth of 25 mm. Results of the penetration resistance vs. elapsed time, for the three w/c ratios, are shown in Figure 3–9.

**ASTM Setting Time Test - PR vs. Time**

![Graph showing PR vs. Time for different w/c ratios](image)

*Figure 3–9 ASTM penetration resistance (PR) vs. Time*
A best-fit curve, using a power relationship, is superimposed on each set of points. According to the ASTM C403/C 403M – 95 (Time of Setting of Concrete Mixtures by Penetration Resistance), initial and final set are determined at 3.5 MPa and 27.6 MPa, respectively. The results of the time of initial and final set corresponding to these values are summarized in Table 3–2 below.

<table>
<thead>
<tr>
<th>W/c ratio</th>
<th>Initial set (min.)</th>
<th>Final set (min.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.40</td>
<td>280</td>
<td>372</td>
</tr>
<tr>
<td>0.50</td>
<td>336</td>
<td>448</td>
</tr>
<tr>
<td>0.60</td>
<td>345</td>
<td>484</td>
</tr>
</tbody>
</table>

As shown in Table 3–2, an increase in the w/c ratio is accompanied by an increase in the time of both initial and final set.

3.5 **Compressive Strength Determination**

Immediately after the NDT testing, the cylinders were tested in compression as described in ASTM C 39 - 94 (Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens) standard. Each specimen was loaded at a rate of 4.5 kN per second. In computing the failure compressive stress of each specimen, $f'_c$, the cross-sectional area was taken as 0.01815 m², based on the nominal diameter of the cylinder. The compressive strength results for the three different w/c ratios (0.4, 0.5, and 0.6) are shown in Figure 3–10. As shown in Figure 3–10, an increase in the water cement ratio is accompanied by a decrease in the compressive strength.
Destructive compressive strength vs. Time

![Graph showing the relationship between time and destructive compressive strength for different w/c ratios.](image)

Figure 3-10 Destructive testing compressive strength vs. time

3.6 RESULTS OF SETTING TIME ANALYSIS

This section presents the SWT results of the four analysis indices (velocity, $\alpha$, $Q$-factor and transmission frequency shift) from the setting time tests. Further, the correlation between the ASTM setting time test and velocity is carried out. All signals collected from the SWT (in both time- and frequency-domain) used for calculation of the analysis indices are shown in Appendices B-1 and B-2.

The $\alpha$ coefficients of IE results were not obtained because of difficulty carrying out the readings. This is due to the inadequate sampling length used and hence the $\alpha$ coefficient profile could not be established.
3.6.1 Analysis indices: Setting Time Tests

This section summarizes the results of the four analysis indices, vs. time, both for UTT and IE techniques. It should be mentioned that some of the very early-age signals presented in Appendices B-1 and B-2, of the setting time tests, are very noisy.

The results of velocity vs. time for UTT and IE, for the three different w/c ratios (0.40, 0.50 and 0.60), are shown in Figures 3-11 and 3-12, respectively. As shown in Figure 3-11, velocity is increasing with time. However, there is no significant difference in velocity for the three water cement ratios. The results from the IE testing techniques in Figure 3-12 show
more discernible differences in velocity.

The results of $Q$-factor vs. time for UTT and IE are shown in Figures 3–13 and 3–14, respectively. As shown in Figure 3–13, the $Q$-factor for the UTT exhibits no consistent trend with time. This may be due to the higher frequency attenuation at earlier ages. However, for the IE technique, shown in Figure 3–14, the overall trend of the $Q$-factor with time is increasing.

The results of $\alpha$ coefficient vs. time for UTT is shown in Figure 3–15. As shown in Figure 3–15, there is no discernible pattern for the $\alpha$ coefficient with time.

The results of peak frequency vs.
time for UTT is shown in Figure 3–16. The overall trend of the peak frequency with time is increasing up to 300 min., after the time of casting, and following this period, it stabilizes at around 23 kHz. It is interesting to note that the time where the frequency is stabilizing is around the time of initial set. It should be mentioned that at the early ages, a frequency as low as 14 kHz was recorded.

Figure 3–15 Setting time test, UTT – $\alpha$ coeff. vs. time

Figure 3–16 Setting time test, UTT – Peak freq. vs. time
3.6.2 Setting time determination

This section presents the four different SWT analysis techniques in estimating the initial and final setting time. The analysis indices are plotted against the ASTM penetration resistance test for initial and final setting times. The results of ASTM penetration resistance vs. velocity, for both UTT and IE tests, are shown in Figures 3–17 and 3–18, respectively. In both figures, a best-fit curve is superimposed on the points. As shown in Figure 3–17, the results of penetration resistance vs. velocity for UTT, there is a distinct difference between the three different w/c ratios. At a given penetration resistance, the velocity increases with increasing the w/c ratio. Similarly, Figure 3–18 shows a distinct difference between w/c ratio and velocity. The velocities at initial and final set are summarized in Table 3–3.

Table 3–3 SWT velocity of initial and final set

<table>
<thead>
<tr>
<th>w/c ratio</th>
<th></th>
<th></th>
<th>Initial Set</th>
<th></th>
<th></th>
<th>Final Set</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>UTT</td>
<td>IE</td>
<td></td>
<td>UTT</td>
<td>IE</td>
</tr>
<tr>
<td>0.40</td>
<td></td>
<td></td>
<td>900</td>
<td>640</td>
<td></td>
<td>1625</td>
<td>1100</td>
</tr>
<tr>
<td>0.50</td>
<td></td>
<td></td>
<td>1425</td>
<td>850</td>
<td></td>
<td>1904</td>
<td>1375</td>
</tr>
<tr>
<td>0.60</td>
<td></td>
<td></td>
<td>1525</td>
<td>890</td>
<td></td>
<td>2076</td>
<td>1550</td>
</tr>
</tbody>
</table>

It is interesting to note that the velocity from the UTT testing is consistently higher than the velocity from the IE testing. The reason for this is that the velocity of UTT is a phase velocity, where the velocity is propagating in a specified direction; and the velocity of IE is a group velocity, where the velocity is inferred from the peak frequency in the frequency-domain (Blitz...
and Simpson, 1996). Therefore, in the IE testing, the successive P-wave arrivals are subjected to increasing attenuation due to increasing path length, and hence a decrease in the peak frequency.

As shown in Figures 3–17 and 3–18, after the initial set, the rate of increase in penetration resistance is high. However, at final set, there is no discernible pattern for the rate of increase in penetration resistance.

The results of penetration resistance vs. Q-factor for both UTT and IE are shown in Figures 3–19 and 3–20, respectively. The results of the UTT test show no distinct pattern between the two variables. This may be due to the early age noisy signals. The results of the IE technique shown in Figure

![Graph](image)

**Figure 3–17 ASTM penetration res. vs. velocity, UTT**

![Graph](image)

**Figure 3–18 ASTM penetration res. vs. velocity, IE**
3–20 show distinct change. However, for the initial and final times of set, the order of magnitude in the change of the \( Q \)-factor is only 2. The practicality of applying this technique, considering the error margin, will be limited. This is because the calculated \( Q \)-factor is based on the spectral width of the peak frequency. Since the peak frequency in the IE technique is dependent on the thickness of the test object, any changes in the thickness of the test object gives a different value of \( Q \)-factor.

The result of penetration resistance vs. peak frequency is shown in Figure 3–21. Although frequency is increasing with the penetration resistance, there is no discernible pattern which would allow

\[
\text{UTT-PR vs. } Q \text{-factor} \\
w/c = 0.40, 0.50 & 0.60
\]

\[
\begin{array}{c}
\text{w/c = 0.40} \\
\text{w/c = 0.50} \\
\text{w/c = 0.60}
\end{array}
\]

\[
\begin{array}{c}
\text{Final Set} \\
\text{Initial Set}
\end{array}
\]

\[
Q \text{-factor}
\]

\[
PR (\text{MPa})
\]

\[
0 \quad 5 \quad 10 \quad 15
\]

\[
0 \quad 10 \quad 20 \quad 30 \quad 40 \quad 50 \quad 60
\]

\[
0 \quad 3 \quad 5 \quad 8 \quad 10
\]

\[
0 \quad 10 \quad 20 \quad 30 \quad 40 \quad 50 \quad 60
\]

\[
Q \text{-factor}
\]

\[
PR (\text{MPa})
\]

\[
0 \quad 3 \quad 5 \quad 8 \quad 10
\]

\[
0 \quad 10 \quad 20 \quad 30 \quad 40 \quad 50 \quad 60
\]

\[
Q \text{-factor}
\]

\[
PR (\text{MPa})
\]

\[
0 \quad 3 \quad 5 \quad 8 \quad 10
\]

\[
0 \quad 10 \quad 20 \quad 30 \quad 40 \quad 50 \quad 60
\]

\[
Q \text{-factor}
\]

\[
PR (\text{MPa})
\]

\[
0 \quad 3 \quad 5 \quad 8 \quad 10
\]

\[
0 \quad 10 \quad 20 \quad 30 \quad 40 \quad 50 \quad 60
\]

\[
Q \text{-factor}
\]

\[
PR (\text{MPa})
\]

\[
0 \quad 3 \quad 5 \quad 8 \quad 10
\]

\[
0 \quad 10 \quad 20 \quad 30 \quad 40 \quad 50 \quad 60
\]

\[
Q \text{-factor}
\]

\[
PR (\text{MPa})
\]

\[
0 \quad 3 \quad 5 \quad 8 \quad 10
\]

\[
0 \quad 10 \quad 20 \quad 30 \quad 40 \quad 50 \quad 60
\]

\[
Q \text{-factor}
\]

\[
PR (\text{MPa})
\]

\[
0 \quad 3 \quad 5 \quad 8 \quad 10
\]

\[
0 \quad 10 \quad 20 \quad 30 \quad 40 \quad 50 \quad 60
\]

\[
Q \text{-factor}
\]

\[
PR (\text{MPa})
\]

\[
0 \quad 3 \quad 5 \quad 8 \quad 10
\]

\[
0 \quad 10 \quad 20 \quad 30 \quad 40 \quad 50 \quad 60
\]

\[
Q \text{-factor}
\]

\[
PR (\text{MPa})
\]

\[
0 \quad 3 \quad 5 \quad 8 \quad 10
\]

\[
0 \quad 10 \quad 20 \quad 30 \quad 40 \quad 50 \quad 60
\]

\[
Q \text{-factor}
\]
estimation of initial and final setting.

![UTT-PR vs. Peak frequency](chart.png)

**Figure 3–21 ASTM penetration resistance (PR) vs. UTT Peak frequency**

3.7 **RESULTS OF COMPRESSION STRENGTH ANALYSIS**

This section presents the results of the four analysis indices (velocity, $\alpha$, $Q$-factor and transmission frequency shift) collected from the SWT. The signals collected in the time– and frequency–domain are presented in Appendix C. Further, this section discusses the regression analysis carried out on the experimental results obtained both from the destructive testing ($f'_c$) and the SWT testing. The SWT techniques utilized both the IE and UTT testing techniques. The results were analyzed using different analysis indices. The analysis indices represent both the elastic components, (e.g. velocity), and the inelastic components, (e.g. attenuation).
Chapter 3 – Setting and Strength Development

The results are correlated using least squares regression analysis. The following sections summarize the results of the four analysis indices and present the discussion for each analysis technique.

3.7.1 Analysis indices: Strength Development

This section summarizes the results of the four analysis indices both for UTT and IE techniques. The results of compressive strength vs. velocity for UTT and IE, of the three different w/c ratios (0.40, 0.50 and 0.60), are shown in Figures 3–22 and 3–23, respectively. It is clear from the two figures that an increase in compressive strength is accompanied by an increase in velocity. Moreover, at a given velocity, the compressive strength increases with a decrease in the w/c ratios. It should be mentioned that the results of the IE technique in Figures 3–22 show velocity in a single columns. This is due to the incorrect sampling rate used. At a sampling rate of 10 MHz and a sample length of 16,000, the frequency resolution is 625 Hz. For the sample thickness of 0.295m, the velocity reading is every 370 m/s (2 x 625 x 0.295 = 370 m/s).

The results of compressive strength vs. $Q$-factor for UTT and IE are shown in Figures 3–24 and 3–25, respectively. As shown in Figures 3–24 and 3–25, the $Q$-factor is increasing with an increase in the compressive strength.
Compressive Strength vs. UTT-Velocity

![Graph showing Compressive Strength vs. UTT-Velocity with different w/c ratios](image)

Figure 3-22 Compressive Strength vs. UTT - Velocity

Compressive Strength vs. IE - Velocity

![Graph showing Compressive Strength vs. IE-Velocity with different w/c ratios](image)

Figure 3-23 Compressive Strength vs. IE - Velocity
Compressive strength vs. UTT - Q-factor

Figure 3-24 Compressive Strength vs. UTT - Q-factor

Compressive strength vs. IE - Q-factor

Figure 3-25 Compressive Strength vs. IE - Q-factor

75
The results of compressive strength vs. $\alpha$ coefficient for UTT is shown in Figure 3–26. The overall trend of the $\alpha$ coefficient is a decrease with decreasing compressive strength. This is expected since $\alpha$ is an attenuation coefficient. However, the pattern of the $\alpha$ coefficient is not discernible.

The results of compressive strength vs. peak frequency for UTT is shown in Figure 3–27. The overall trend of peak frequency is increasing with compressive strength. It is interesting to note that peak frequency is showing sensitivity to the difference in the w/c ratios. At a given frequency, the compressive strength of the concrete with a w/c of 0.40 is higher than the concretes with w/c ratios of 0.50 and 0.60. Similarly, the compressive strength of the concrete with a w/c 0.50 is higher than the concrete with a w/c ratio of 0.60.
Compressive strength vs. UTT - \( \alpha \) coefficient

\[ \begin{align*}
\bullet & \ w/c = 0.40 \\
\bullet & \ w/c = 0.50 \\
\Delta & \ w/c = 0.60
\end{align*} \]

\( \alpha \) coefficient (1/s)

Figure 3–26 Compressive Strength vs. UTT - \( \alpha \) coefficient

Compressive strength vs. UTT - Peak Frequency

\[ \begin{align*}
\bullet & \ w/c = 0.40 \\
\bullet & \ w/c = 0.50 \\
\Delta & \ w/c = 0.60
\end{align*} \]

Peak frequency (kHz)

Figure 3–27 Compressive Strength vs. UTT - Peak frequency
3.7.2 Regression model: Velocity

The results of the compressive strength vs. velocity, for both UTT and IE, are shown in Figures 3-22 and 3-23, respectively. Since the intent of this section is to develop equations that would be valid with a range of w/c ratios, the results of the three w/c ratios are combined in the calculation of the regression analysis. Consistent with all compressive strength results, at a given compressive strength, it is shown that velocity of IE is always smaller than UTT.

It is clear from Figures 3-22 and 3-23 that the strength-velocity data appear to follow a power relationship in agreement with theory. At low strengths, an increase in strength is accompanied by a significant increase in velocity, however, as the strength continues to increase, these increases in velocity becomes proportionately less.

The power relationship between velocity and compressive strength is represented by

\[ f_c' = a(C_p)^b, \]  
Eq. [3-1]

where \( f_c' \) is the uniaxial destructive compressive strength, in MPa, \( C_p \) is P-wave velocity, in m/s, and \( a \) and \( b \) are constants. The constants were found by regression analysis and are reported in Table 3-4 for the two SWT techniques.

The constant terms, \( a \) and \( b \), were calculated using linear regression in log space. The best fit values of \( a \) and \( b \), in the UTT testing, are \( 10^{-16.288} \) and 4.834, respectively, and the corresponding \( r^2 \) value is 0.798. Similarly, the best fit values of \( a \) and \( b \), in the impact-echo testing, are \( 10^{-18.072} \) and 5.413, respectively, with the corresponding \( r^2 \) value being 0.822. The value of the constant
term $b$, 4.834, for the ultrasonic through-transmission testing, is in close agreement with Eq. 2-31, i.e. a 4th power relationship. Pessiki and Carino (1988) reported a similar 4th power relationship. On the other hand, it was reported by Popovics et al. (1990) that the 4th power relationship is not valid.

As shown in Figures 3-28a and 3-29a, for both UTT and IE testing techniques, the estimated and actual compressive strengths are in good agreement for the lower compressive strength range. However, at higher compressive strengths, there is more scatter. This may be attributed to the insensitivity of velocity to strength at later ages. Reportedly other researchers have shown that there exists a high $r^2$ value for $f'_c$ vs. velocity relationship. For instance, Carette and Malhotra (1984) reported an $r^2$ value of 0.92 for three days, and Ravindrarajah (1997) reported an $r^2$ value of 0.96 for an early age of 5 hrs using high-strength concrete. However, for long term $f'_c$ vs. UPV, Yun et al. (1988) has shown an $r^2$ value of 0.84. This is in good agreement with the average 28 day $r^2$ value, 0.81, obtained in this work.

Besides $r^2$, the goodness of fit was analyzed using regression residuals (or residuals). A residual is the difference between the actual and estimated mean compressive strength. For any given set of values, it is assumed that random error has a normal probability distribution with a mean equal to 0 and the random errors are probabilistically independent (McClave and Diethrich, 1991). Thus it is expected that the random errors be randomly distributed. The residuals for the UTT are showed in Figure 3-28b. As shown in Figure 3-28b, the residuals are randomly distributed between the positive and negative sides. Similarly, residuals of the IE testing technique shown in Figure 3-29b display a random distribution about zero.
Figure 3-28 Regression Model (UTT - Velocity) (a) Compressive strength vs. UTT - Velocity (b) Residuals (UTT - velocity)
Figure 3–29 Regression Model (IE - Velocity) (a) Compressive strength vs. IE – Velocity (b) Residuals (IE - velocity)
3.7.3 Regression model: Combined Method

In this section the experimental results collected in the UTT techniques are analyzed using both the elastic (velocity) and inelastic ($\alpha$ and $Q$-factor) components of the signal. The combined methods of the UPV and attenuation indices are assumed to have a power relationship. The power relationships for each attenuation index are described below.

For a combined method of UPV and $\alpha$ the relationship may be represented by Galan (1990):

$$f_e' = a(C_p)^b(\alpha)^c,$$

Eq. [5-3]

and the constant terms, $a$, $b$, and $c$, can be calculated in the log space from:

$$\log f_e' = \log a + b \log C_p + c \log \alpha.$$

Eq. [5-4]

For a combined method of UPV and $Q$-factor the relationship may be represented by:

$$f_e' = a(C_p)^b(Q)^c,$$

Eq. [5-5]

and the constant terms, $a$, $b$, and $c$, can be calculated in the log space from:

$$\log f_e' = \log a + b \log C_p + c \log Q.$$

Eq. [5-6]

The results of the regression analyses for UTT testing techniques are summarized in Table 3-4.
Table 3-4 Summary of regression equations

<table>
<thead>
<tr>
<th>Variables</th>
<th>Equation</th>
<th>$r^2$</th>
<th>Equation</th>
<th>$r^2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>F(V)</td>
<td>$f_c(V) = 10^{-16.285} \cdot V^{4.835}$</td>
<td>0.798</td>
<td>$f_c(V) = 10^{-18.072} \cdot V^{5.431}$</td>
<td>0.822</td>
</tr>
<tr>
<td>F(V, $\alpha$)</td>
<td>$f_c(V) = 10^{-16.110} \cdot V^{4.796} \cdot \alpha^{0.011}$</td>
<td>0.784</td>
<td></td>
<td></td>
</tr>
<tr>
<td>F(V, $Q$)</td>
<td>$f_c(V) = 10^{-10} \cdot V^{2.955} \cdot Q^{0.616}$</td>
<td>0.881</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

From the equation summarized in Table 3-4, the addition of $\alpha$ did not have any significant improvement on the $r^2$ value as compared with velocity. Moreover, $\alpha$ being an attenuation coefficient, is expected to have a negative exponent with increase in strength. But, Galan (1990) has shown that the $r^2$ value increases with the use of the combined results of UPV and $\alpha$ coefficient. This discrepancy may be attributed to the inadequate sampling length used and hence the profile for the calculated $\alpha$ could not be established.

It is interesting to note that the use of $Q$-factor has improved the $r^2$ value of the equation to 0.881. Although the $Q$-factor is an attenuation index, the ease of calculating the result may give more consistent readings.

The results of actual and estimated compressive strengths for the combined methods are shown in Figures 3-30 and 3-31. Figure 3-30a shows the result of combining UPV and $\alpha$. The corresponding residuals for velocity and $\alpha$ are shown in Figure 3-30a and 3-30b, respectively.
Figure 3-31a shows the result of combining the UPV and $Q$-factor. The corresponding residuals for velocity and $Q$-factor are shown in Figures 3–31b and 3–31c, respectively.

As shown in Figure 3–30a, for a combination of UPV and $\alpha$, the actual and estimated compressive strengths are closely located around the line of unity. The residuals for velocity and $\alpha$ are shown in Figures 3–30b and 3–30c, respectively. The residuals for velocity show random distribution. However, the residuals for $\alpha$ are exhibiting a bias on the positive side. This explains the decrease in the adjusted $r^2$ value of 0.784 for the combined method of UPV and $\alpha$, as compared with the $r^2$ value to 0.798 for velocity.

As shown in Figure 3-31a, the actual and estimated compressive strengths for the combined method of UPV and $Q$-factor are closely located around the line of unity. The use of $Q$-factor index has improved the estimation of the long-term concrete strength. This is clearly demonstrated by the randomness of residuals for the combination of velocity and $Q$-factor, Figure 3-31b. The residual value has improved significantly, as compared with the residual value of only velocity. This shows that the combination of velocity and $Q$-factor is a better estimating tool for long term strength.
Figure 3-30 Regression Model ($f(V,\alpha)$ UTT) (a) Actual $f_c'$ vs. Estimated (b) Residuals (UTT - Velocity) (c) Residuals (UTT - $\alpha$)
Figure 3–31 Regression Model (f(V,Q-factor) UTT) (a) Actual fc' vs. Estimated (b) Residuals (UTT - Velocity) (c) Residuals (UTT - Q-factor)
Chapter 3-Setting and Strength Development

3.4 LONG TERM COMPRESSIVE STRENGTH ESTIMATE

The aim of this section is to demonstrate the accuracy of the regression equations developed in Section 3.7. The equations developed using the best-fit linear regression analysis are summarized in Table 3-4.

The validity of the equations were examined by casting concrete and testing it over a 90 days period. The mix proportions of this mix were the same as those used previously for the concrete with a w/c ratio = 0.50 and are given in Table 3-1.

The destructive and estimated compressive strength results from the UT test are shown in Figure 3-25. As shown in Figure 3-25, the results of all equations provide a good estimate on the first day of testing, day 3. However, as expected, at later ages, both estimated strength results from UPV and $\alpha$ deviate from the actual compressive strength. It is interesting to note that the estimates for strength using velocity are deviating as early as 7 days. Other researchers have also shown the insensitivity of velocity at later ages. The long term estimated compressive strength using only velocity at 87 days is off by 25%.

Interestingly, the result of the combining velocity and with the $Q$-factor improved the strength estimate significantly. The estimated concrete strength at 87 days is off only by 5%. It should be noted that since the $Q$-factor is an attenuation index, improper testing techniques could lead to erroneous results. For example, insufficient coupling could lead to excessive attenuation and error in estimation of the $Q$-factor. For instance, at an age of 34 days, the actual compressive
strengths were 35 and 37 MPa. However, the corresponding estimated compressive strengths were 38 and 33 MPa, respectively. Thus a slight change in the velocity and $Q$-factor reading may cause this problem.

![Long Term Compressive Strength Estimation (UTT)](image)

**Figure 3-32 UTT, compressive strength vs. time**

The result of the impact-echo method for long term strength estimation is shown in Figure 3–33. It is interesting to see that the destructive and estimated compressive strengths are in good agreement.
In this lab study UTT and IE were used to monitor the setting and strength development in concrete. This was carried out by casting concrete with varying w/c ratios (0.40, 0.50 and 0.60). The SWT results were analyzed using velocity and attenuation indices ($\alpha$, $Q$-factor, and peak frequency). It is interesting to note that the four analysis techniques have shown sensitivity to the setting of concrete. However, there is no discernible pattern with varying the w/c ratios. Further, the different analysis indices were plotted against the ASTM penetration resistance test. Although, the different analysis indices increased with the penetration resistance, of all the analysis indices, only velocity showed distinct differences for different w/c ratio.
The estimation of concrete strength using combined results is promising. The various analysis indices have shown sensitivity with concrete strength. By correlating the concrete strength with the various analysis indices, relationships were developed using least square regression analysis. It is interesting to note that a combined result of velocity and $Q$-factor, in the UTT technique, produced promising estimates of long-term strength. Similarly, in the IE technique, the regression equation using just velocity was promising.

One of the important parameters that affects the properties of concrete is aggregate. The results developed in this section will be applicable only to concrete with the same aggregate content and type. This limits the practical application of SWT to estimate in-situ concrete strength without prior knowledge of the concrete in the structures. However, this method, after developing correlations for the prevailing concrete constituents, could be used for estimating concrete strength.
This chapter reports the findings from a laboratory test program aimed at monitoring concrete prisms deteriorated due to alkali-silica reactions (ASR) and freezing and thawing (F/T) cycles. To carry out these tasks, both laboratory-cast (lab-cast) and 12-year-old lab-cast prisms supplied by the Ministry of Transportation Ontario (MTO) were used. For the lab-cast concrete, the zero-day reading was taken 24 hours after casting. Subsequently, the deterioration due to ASR was monitored every 4 weeks for 42 weeks (Section 4.3.1). Two of the lab-cast prisms were tested after F/T cycles (Section 4.3.2). The MTO series of prisms were tested using both dynamic modulus and SWT techniques. Later, three prisms from the MTO series of testing were selected and tested after F/T cycles. The response of field-exposed concrete to SWT are presented in Chapter 5.

4.1. MATERIALS

4.1.1 Lab-Cast Prisms

To study the response of stress waves at different level of deterioration, both for ASR-affected and F/T cycles, concretes were cast with the mix design given in Table 4–1. The cement used
was high-alkali cement (0.94% Na₂O₆) from the Blue Circle's Bowmanville plant. The aggregate used was reactive New Mexico coarse aggregate (Shakespeare quarry) and non-reactive fine aggregate, (lab sand). The materials selected and the mix design used were in accordance with the standard of CSA A23.2-14A Concrete Prism Expansion Test except that NaOH was not added to augment the cement alkalis. This concrete was non-air entrained.

<table>
<thead>
<tr>
<th>Table 4-1 ASR Mix Design</th>
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<tbody>
<tr>
<td></td>
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<tr>
<td>0.42</td>
</tr>
</tbody>
</table>

4.1.2 MTO prisms

A series of 12-year-old concrete prisms (0.75 x 0.75 x 0.350) were supplied by MTO for this program. The prisms were cast using a range of different cements, water-cement ratios and types of aggregate. All prisms are non-air entrained.

<table>
<thead>
<tr>
<th>Table 4 - 2 MTO prisms</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
</tr>
<tr>
<td>748</td>
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<tr>
<td>713</td>
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<td>730</td>
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<td>750</td>
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<td>725</td>
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<tr>
<td>751</td>
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</tbody>
</table>
The relevant details of the specimens used for the SWT tests are given in Table 4-2 together with measured lengths after the ends containing the measuring pins had been trimmed using a diamond saw blade. The Nelson aggregate is a non-deleterious reactive crushed carbonate, the Spratt is a reactive siliceous limestone, and the Pittsburg is an alkali-carbonate reactive argillaceous dolostone. All aggregate originated from quarries in Ontario.

4.2 Specimen Preparation

4.2.1 Lab-Cast Prisms

The material were mixed according to the ASTM C192/C 192M – 95 (Standard Practice for Making and Curing Concrete Test Specimens in the laboratory) standard. Six different prisms (0.75 x 0.75 x 0.306) were cast. Four prisms were used for monitoring deterioration due to ASR and the other two were used for F/T cycles. Of the prisms used for ASR tests, two had studs embedded in at the ends for measuring expansion, and two were cast without studs for SWT tests. Casting was carried out in accordance with CSA A23.2-14A (Concrete Prism Expansion Test, Oct 19, 1992):

- The concrete was placed in the moulds in two layers and each layer was tamped with a tamping rod,
- Excess concrete was struck off and the surface of prism brought to finish,
- Finally, the prisms were covered with wet burlap and a polyethylene sheet.

After 24 hrs from the time of casting, the prisms were demoulded and the initial length and SWT measurements were carried out. Following the initial measurements, the specimens were stored in sealed containers over water in a room at 38°C until testing at various ages. The F/T prisms,
after being cured in the moist room for 14 days, were kept in lab air until the time of testing (1 year from the time of casting).

4.2.1 MTO Prisms

The studs at the end of the MTO prisms were removed using a diamond saw blade and stored under water until the time of testing.

4.3. Specimen Testing

4.3.1 ASR

For the lab-cast concrete prisms, the zero day readings were carried out 24 hours after casting.

---

**ASR Prism Expansion Measurement**

![ASR Prism Expansion Measurement](image_url)

*Figure 4–1 ASR expansion results*
Subsequently, expansion measurements were performed every 4 weeks for 42 weeks. Expansion measurements were carried out in accordance with the CSA A23.2-14A Concrete Prism Expansion Test standard. The expansion results are shown in Figure 4-1. At the same time the expansion measurements were made, SWT measurements, (UTT and IE), were performed on the companion prisms without studs.

The 12-year-old MTO prisms were taken out of the moist room and kept under water prior to carrying out the test. However, since prisms in the saturated condition showed no discernible difference, they were kept in lab air to stabilize in the laboratory environment prior to carrying out further testing. Following this period, SWT (Section 4.3.3) and dynamic modulus of elasticity tests were carried out. Determination of the dynamic modulus of elasticity was carried out in accordance with the standard ASTM C 215.

4.3.2 Freeze/Thaw

The freeze/thaw tests were carried out on the lab-cast and the 12-year-old MTO prisms in accordance with ASTM C666 –92 (Standard Test Method for Resistance of Concrete to Rapid Freezing and Thawing) standard. The lab-cast prisms were stored in a lab air condition for a year prior to carrying out the tests. Seven days prior to F/T testing, the specimens were stored in saturated lime water.

4.3.3 SWT

The SWT testing was carried out on the lab-cast specimens with no studs. The ends containing the studs were trimmed from the MTO specimens. The SWT equipment used was the
AndecScope™ manufactured by Andec Mfg. Ltd. (See Appendix A for further detail of the equipment).

For the ultrasonic through-transmission technique, a 50-kHz piezoelectric transducer was used for transmission and 150-kHz flat broadband piezoelectric transducer was used as a receiver. To amplify the signal, an external preamplifier was attached to the receiving transducer. To ensure a good surface contact, petroleum jelly (Vaseline) was used as a couplant. For the UTT technique the sampling rate used was 2 MHz and the sample length analyzed was 1024.

For the IE testing technique, a steel ball was used to generate stress waves and a flat broadband 150-kHz transducer was used as a receiver. A preamplifier was attached at the receiving end of the transducer. The sampling rate used was 100 kHz and the sample length analyzed was 2048.

4.4 Summary of Results

4.4.1 ASR

4.4.1.1 Velocity

The expansion and velocity results for the lab-cast specimens, for the UTT and IE techniques are shown in Figures 4–2 and 4–3, respectively. For both UTT and IE techniques, velocity increased rapidly during the first four weeks. This is probably due to continuing curing of the concrete samples. After four weeks, the velocity suddenly decreased. This appears to coincide with measured expansions in the range 0.04% to 0.10% and the appearance of visible cracking on the prisms. For the UTT technique, the velocity continues to decrease as the prism continues to
expand. Beyond about 20 weeks the rate of expansion decreases significantly, and there is no further reduction in the velocity. The expansion that occurs between the ages of 4 to 20 weeks is approximately 0.10%. During this period the velocity decreases from a maximum of 4400 m/s (at 4 weeks) to 4300 at 20 weeks, a decrease of only 2.5%.

The velocity measured by IE follows a similar trend although a further sudden reduction in velocity occurs between 24 and 28 weeks. Regardless of this further reduction, the total drop in velocity during the testing period was from 3940 m/s at 4 weeks to 3700 m/s at 40 weeks which represents a reduction of just 6%.

The velocity results from the UTT tests on the MTO prisms are shown in Figure 4–4. Also shown in the figure are the w/c ratios of the mixes. The velocities ranged from 4200 m/s to 4800 m/s for prisms with measured expansions in the range of 0.034 to 0.481%. Although there is an overall trend of decreasing velocity with increasing expansion, there is clearly not a unique relationship between velocity and expansion. It is probable that the effects of expansion on velocity are partly masked by differences in water-cement ratio and aggregate type between prisms.

4.4.1.2 Attenuation Coefficient

For the lab-cast prisms, the attenuation coefficients (α) of UTT and IE testing techniques are shown Figures 4–6 and 4–7, respectively. It is evident from both figures that α does not exhibit sensitivity to expansion. This may be explained by the low transmission frequency (high λ) which may cause the signals to be saturated. Hence the saturated signals may propagate as a
Chapter 4 – Deteriorated Lab-Cast Concrete

guided wave and the internal properties of the material may not be accurately represented. Further tests should be performed with higher frequency signal components or bigger sample sizes.

For the MTO prisms, \( \alpha \) of the UTT testing technique is shown in Figure 4–7. On the same graph, the dynamic modulus is plotted. It is worth mentioning that both \( \alpha \) and dynamic modulus are decreasing with increasing expansion. This is in good agreement with what would be expected.

4.4.1.3 Quality Factor

The \( Q \)-factors found by UTT and IE techniques are shown in Figures 4–8 and 4–9. Both for UTT and IE techniques, with increasing expansion, contrary to what would be expected, \( Q \)-factor is increasing. The insensitivity \( Q \)-factor for this test may be attributed to the size of the specimens and the input frequency. For a given velocity of 4,300 m/s, and with an input frequency of 50 kHz, the wavelength is around 86 mm. Since the sample size is 75 x 75 mm, the signal will propagate along the surface of the specimens. Hence the internal microcracks developed by the ASR reaction will not affect the propagation signal. Further tests should be performed with higher frequency component signals or larger test specimens.

The \( Q \)-factor results for the MTO prisms are not shown because the experimental work was carried out before suitable signal processing software was available.

4.4.1.4 Transmission Frequency Shift

The transmission frequency shift is shown in Figure 4–10. The transmission frequency shift shows the same sort of behaviour as most of the other SWT parameters –i.e. a sudden change (in
the case of reduction) at around 4–8 weeks— with little or no further change beyond this time. This is due to attenuation and scattering of the higher frequency components.
Figure 4-2 UTT - Velocity and Expansion vs. Time

Figure 4-3 IE - Velocity and Expansion vs. Time
MTO prisms
(w/c ratio & UTT-velocity) vs. expansion

Figure 4-4 w/c ratio and UTT - Velocity vs. Expansion
Figure 4–5 UTT - $\alpha$ and Expansion vs. Time

Figure 4–6 IE - $\alpha$ and Expansion vs. Time
MTO Prisms
(UTT-\(\alpha\) & dynamic modulus) vs. Expansion

\[ a \text{ (1/s)} \]
\[ \text{Dynamic Modulus (GPa)} \]

Expansion (%)

0.03 0.04 0.06 0.12 0.21 0.26 0.43 0.45 0.48

Figure 4-7 UTT - \(\alpha\) and dynamic modulus vs. Expansion
Figure 4-8 UTT - Q-factor and Expansion vs. Time

Figure 4-9 IE - Q-factor and Expansion vs. Time
Figure 4-10 UTT - Peak frequency and expansion vs. time
4.4.2 Freezing and Thawing

4.4.2.1 Velocity

For the lab-cast prisms, the velocity from the UTT and IE tests are shown in Figs. 4–11 and 4–12, respectively. As shown in both figures, SWT readings were taken after 0 and 40 F/T cycles. The readings for the undamaged prisms, prior to F/T cycling, show a velocity reading of around 4,500 m/s and 4,000 m/s for UTT and IE, respectively. This high velocity is generally representative of good concrete (Whitehurst, 1951a). However, after 40 F/T cycles, both techniques exhibit a 50% drop in velocity. After this number of F/T cycles, the test was discontinued due to the physical disintegration of the prisms.

For the MTO prisms, the velocity from both the UTT and IE tests are shown in Figs. 4–13 and 4–14, respectively. Measurements were carried out after 0, 10, 40 & 59 F/T cycles. Both SWT techniques show sensitivity with the number of F/T cycles. It is interesting to note that the velocity reading at each F/T cycle is consistent with the initial expansions of the prisms. Each of the prisms designated 730, 731 and 752 were initially affected by ASR and had initial ASR-induced expansions of 0.061, 0.121, 0.207%, respectively. The prism with the least expansion, 0.061%, showed greater resistance to the subsequent FT cycles. The tests on the other two prisms were discontinued after 40 F/T cycles. This is due to the development of large cracks during testing and hence disruption of the stress-wave propagation medium.
4.4.2.2 Attenuation Coefficient

For the lab cast prisms, values of $\alpha$ measured by both the UTT and IE testing techniques are shown in Figures 4–15 and 4–16, respectively. The $\alpha$ coefficient of both SWT techniques is increasing despite an increase in the F/T cycles.

For the MTO prisms, $\alpha$ of both UTT and IE testing techniques are shown in Figures 4–17 and 4–18, respectively. The $\alpha$ coefficient of both techniques is showing similar insensitivity with increasing FT cycles. The reason could be saturation of the signal.

4.4.2.3 Quality factor

For the lab cast prisms, the values of the $Q$-factor measured by both the UTT and IE testing techniques are shown in Figures 4–19 and 4–20, respectively. It is interesting to note that $Q$-factor is decreasing with an increasing number of F/T cycles. After 40 F/T cycles, the $Q$-factor drops by about 50%.

For the MTO prisms, $Q$-factors of both UTT and IE testing techniques are shown in Figures 4–21 and 4–22, respectively. Consistent with the two SWT techniques, the overall trend of the $Q$-factor is decreasing with an increasing number of F/T cycles.

4.4.2.4 Transmission Frequency Shift

For the lab cast prisms, the transmission frequency shift (TFS) measured using the UTT testing technique is shown in Figure 4–23. It is interesting to note that after 40 F/T cycles, the TFS is significantly reduced by as much as 90%. This may be explained at high F/T cycles, by
attenuation of the high frequency component of the signal due to absorption and scattering at the concrete air interface.

For the MTO prisms, the transmission frequency shift (TFS) measured by the UTT testing technique is shown in Figure 4-24. The overall trend of the signals is decreasing with increasing F/T cycles. It is interesting to note that the prism with least expansion, 0.061, exhibits minor attenuation with the subsequent F/T cycles. However, the TFS of the other two prisms are severely attenuated with increasing F/T cycles.
FT test on Lab Cast - UTT
Velocity vs. FT cycles

Figure 4-11 Lab Cast: UTT - Velocity vs. F/T cycles

FT test on Lab Cast - IE
Velocity vs. FT cycles

Figure 4-12 Lab Cast: IE - Velocity vs. F/T cycles
Figure 4–13 MTO prisms: UTT - Velocity vs. F/T cycles

Figure 4–14 MTO prisms: IE - Velocity vs. F/T cycles
Chapter 4 – Deteriorated Lab-Cast Concrete

FT test on Lab Cast - UTT
\( \alpha \) vs. FT cycles

\[ \begin{array}{c}
\alpha (1/\alpha) \\
\hline
0 & 0 \\
-1000 & -1000 \\
-2000 & -2000 \\
-3000 & -3000 \\
-4000 & -4000 \\
-5000 & -5000 \\
-6000 & -6000 \\
-7000 & -7000 \\
-1000 & -1000 \\
0 & 0 \\
\end{array} \]

F/T cycles

Prism A
Prism B

Figure 4-15 Lab Cast: UTT - \( \alpha \) vs. F/T cycles

FT test on Lab Cast - IE
\( \alpha \) vs. FT cycles

\[ \begin{array}{c}
\alpha (1/\alpha) \\
\hline
0 & 0 \\
-200 & -200 \\
-400 & -400 \\
-600 & -600 \\
-800 & -800 \\
-1000 & -1000 \\
0 & 0 \\
\end{array} \]

F/T cycles

Prism A
Prism B

Figure 4-16 Lab Cast: IE - \( \alpha \) vs. F/T cycles
FT test on MTO Prisms - UTT
\[ \alpha \text{ vs. FT cycles} \]

\[ \begin{array}{c}
\text{Beam 730 (e=0.061\%)} \\
\text{Beam 731 (e=0.121\%)} \\
\text{Beam 752 (e=0.207\%)} \\
\end{array} \]

Figure 4-17 MTO prisms: UTT - \( \alpha \) vs. F/T cycles

FT test on MTO Prisms - IE
\[ \alpha \text{ vs. FT cycles} \]

\[ \begin{array}{c}
\text{Beam 730 (e=0.061\%)} \\
\text{Beam 731 (e=0.121\%)} \\
\text{Beam 752 (e=0.207\%)} \\
\end{array} \]

Figure 4-18 MTO prisms: IE - \( \alpha \) vs. F/T cycles
FT test on Lab Cast - UTT
$Q$-factor vs. FT cycles

Figure 4-19 Lab cast: UTT - $Q$-factor vs. F/T cycles

FT test on Lab Cast - IE
$Q$-factor vs. FT cycles

Figure 4-20 Lab cast: IE - $Q$-factor vs. F/T cycles
FT test on MTO Prisms - UTT

$Q$-factor vs. FT cycles

- Beam 730 ($e=0.061\%$)
- Beam 731 ($e=0.121\%$)
- Beam 752 ($e=0.207\%$)

Figure 4–21 MTO prism: UTT - $Q$-factor vs. F/T cycles

FT test on MTO Prisms - IE

$Q$-factor vs. FT cycles

- Beam 730 ($e=0.061\%$)
- Beam 731 ($e=0.121\%$)
- Beam 752 ($e=0.207\%$)

Figure 4–22 MTO prism: IE - $Q$-factor vs. F/T cycles
FT test on Lab Cast - UTT
Transmission Frequency Shift vs. FT cycles

Figure 4-23 Lab cast: UTT - PFS vs. F/T cycles

FT test on MTO Prisms - UTT
Transmission Frequency Shift vs. FT cycles

Figure 4-24 MTO prism: UTT - TFS vs. F/T cycles
4.5 Discussion

Tests were carried out to monitor the SWT response of concretes affected by ASR and F/T cycles. It is expected that with increasing degree of deterioration, the results from the four analysis techniques would decrease accordingly. However, in this study it is shown that some of the analysis indices increased with deterioration. This is probably due to the small sample size and relatively large $\lambda$ used. This results in a guided wave situation. It is interesting to note that, when the deterioration is severe, as with the F/T samples, the analysis indices are showing sensitivity, which is a decrease to lower values. Further tests should be carried out on larger sample sizes or using smaller SWT frequencies.
The need to quantify defects at an early stage is important for the repair and maintenance of structures. Thus analysis techniques that are sensitive to changes in the internal structure at an early age are needed. A preliminary field study was carried out to evaluate the use of various stress-wave analysis techniques for quantifying micro damages in concrete elements. The concrete specimens tested were cast over 8 years ago at the MTO Kingston Exposure Site. This chapter presents details of the materials tested and results from the different analysis techniques used.

5.1 Material

The specimens tested in this field observation were four 8-year-old, un reinforced concrete beams (0.6 x 0.6 x 2.0m) which were affected by alkali-silica reaction (ASR) to a varying degree. The concretes were cast using reactive aggregate and various combinations of cement, pozzolans and slag such that the expansion (ε) after 8 years field exposure ranged from 0.01 to 0.12% as shown in Table 5–1. The mixes all had cementitious material contents in the range 100 to 415 kg/m³ and water-cementitious material (w/c) in the range of 0.34 to 0.40. Further details
of the mix proportions and exposure conditions are presented in Afrani and Rogers (1994). Visual examination of the beams showed that Beam 4 (see Figure 5–1) exhibited no visible damage. Beam 3 (B3) (see Figure 5–2) showed some minor reactivity in the form of surface crazing. Beam 5 (See Figure 5–3) and Beam 6 (See Figure 5–4) showed from small cracks to visible map cracking, respectively. This is in good agreement with the generally accepted view that an expansion of approximately 0.04% marks the initiation of visible cracking due to ASR (Thomas, 1997).

<table>
<thead>
<tr>
<th>Beam #</th>
<th>Cementing Material, % by Mass</th>
<th>% of Expansion</th>
</tr>
</thead>
<tbody>
<tr>
<td>4*</td>
<td>24% HAPC, 51% PSFC containing 7.5% silica fume, and 25% GGBFS</td>
<td>0.01</td>
</tr>
<tr>
<td>3</td>
<td>75% HAPC and 25% GGBFS</td>
<td>0.02</td>
</tr>
<tr>
<td>5</td>
<td>100% LAPC</td>
<td>0.05</td>
</tr>
<tr>
<td>6</td>
<td>100% HAPC</td>
<td>0.12</td>
</tr>
</tbody>
</table>

*Mix 4 contained 3.8% silica fume by mass of the total cementing material content.  
Note—HAPC = high-alkali Portland cement; GGBFS = ground granulated blast-furnace slag; PSFC = Portland silica fume cement; and LAPC = low-alkali Portland cement.

5.2 RESULTS

The results of the four analysis techniques obtained form three NDT techniques; pulse-echo, through-transmission and impact-echo are shown in Figures 5–5, 5–7, 5–9, and 5–11.
A sensitivity analysis was carried out on the four analysis techniques. The sensitivity analysis was done by comparing all results to the least damaged concrete as shown by Daponte et al. (1990):

$$D = 1 - \frac{X}{X_0}$$

Eq. [5-1]
where \( D \) is the damage coefficient, \( X_0 \) is the value for the least expanded concrete (in this field test \( \varepsilon = 0.01\% \)), and \( X \) is the corresponding values for higher expansions. The damage coefficient for the four analysis techniques is represented by:

\[
\begin{align*}
D_{CP} &= \text{damage coefficient of velocity} \\
D_a &= \text{damage coefficient of attenuation coefficient} \\
D_Q &= \text{damage coefficient of } Q \text{-factor} \\
D_{pf} &= \text{damage coefficient of peak frequency shift}
\end{align*}
\]

The results of the sensitivity analysis are shown in Figures 5-6, 5-8, 5-10, and 5-11.

5.2.1 Damage Coefficient, Velocity

Values of \( C_P \) and \( (D_C)_CP \) for impact-echo and through-transmission tests are shown in Figures 5-5 and 5-6. The values of pulse-echo test are not shown because of difficulty in obtaining the time of first arrival. For the UTT techniques the sensitivity of velocity to expansion was much lower than for the IE techniques. For the UTT technique, all specimens have a measured velocity of approximately 4500 m/s, which is typical of good quality concrete (Whitehurst, 1951), despite visible map cracking. On the other hand, the P-wave velocity from the IE testing was is found to be sensitive to the micro damages. These different trends may be explained by differences in the beamwidth. For the UTT technique, the stress wave propagates in a narrow beamwidth whereas with impact-echo the hemi-spherical wave is much broader and is more affected by micro cracking (Sadri, 1996). The narrow beamwidth in the UTT technique covers only a small area and, if the test object in that medium is not significantly damaged, the sensitivity to velocity will be less. On the other hand, the broad beamwidth of the impact-echo signal tends to cover broader area and hence, the velocity will be significantly affected.
5.2.2 Damage Coefficient, Attenuation Coefficient

Values of $\alpha$ and $D_\alpha$ for the three NDT testing techniques are shown in Figures 5–7 and 5–8. It is expected that with increasing deterioration, $\alpha$ becomes more negative. As shown in the figures, there is significant reduction in $\alpha$ with expansion for the UTT and IE testing. However, the sensitivity of $\alpha$ for the IE is higher because of the high frequency component of the broad band IE signal.

5.2.3 Damage Coefficient, Quality Factor

The results for $Q$-factor and $D_Q$ are shown in Figures 5–9 and 5–10. With increasing deterioration, it is expected that the $Q$-factor will decrease. From the graphs it can be seen that three NDT techniques are sensitive to internal damage. There is significant reduction in the $Q$-factor for the UTT, and IE tests. For the PE test, the increase in $Q$-factor at 0.05% expansion is attributed to the resonance of the receiving transducer.

5.2.4 Damage Coefficient, Peak Frequency Shift

The results of the peak frequency shift are shown in Figures 5–11 and 5–12. Similar to the other analysis indices, it is expected that increasing deterioration is accompanied by a shift in the peak frequency shift to a lower value. The peak frequency shift in the two testing methods appears to be very sensitive to expansion. This is because as the expansion increases, there are more micro damages developed in the concrete sample, therefore the transmission frequency shifts towards a lower value.
Figure 5–5 Velocity vs. Expansion

Figure 5–6 Dc, Velocity vs. Expansion
Kingston Blocks - $Q$-factor vs. Expansion

$Q$-factor vs. Expansion (%)

Figure 5–9 $Q$-factor vs. Expansion

Kingston Blocks - $Dc$, $Q$-factor vs. Expansion

$Dc$, $Q$-factor vs. Expansion (%)

Figure 5–10 $Dc$, $Q$-factor vs. Expansion
Kingston Blocks - Peak frequency vs. Expansion

Figure 5-11 Peak frequency vs. Expansion

Kingston Blocks - Dc, Peak frequency vs. Expansion

Figure 5-12 Dc, Peak frequency vs. Expansion
5.3 Discussion

In this field study, the ultrasonic through-transmission, ultrasonic pulse-echo and impact-echo testing methods were used. The signals were analyzed in the time- and frequency-domain using different analysis techniques.

For the ultrasonic through-transmission testing, the three attenuation indices ($\alpha$, $Q$-factor, and transmission frequency shift) showed sensitivity to expansion. As expected, with increasing expansion, the three attenuation indices have shifted to a lower value. For the ASR-affected concrete, an increase in micro-cracking increases attenuation because of absorption and scattering at the concrete-air interface. On the other hand, it appears that velocity is less sensitive to the presence of internal micro-cracks.

From the results of impact-echo testing, all four indices are sensitive to the level of internal micro-cracking. This is ascribed to the high frequency component of the broadband impact-echo signal, which is affected by the composition of concrete. Small impactors generate signals with high frequency components (small wavelength). This method, besides being effective, has the advantage of requiring access to just one side of a structure, making it particularly suited to structures such as pavements or foundations.

For the pulse-echo test, the particular frequency shift showed correlation with expansion. The other two attenuation indices, $\alpha$ and $Q$-factor, did not show high sensitivity. This is ascribed to
the test set-up and the receiving transducer resonating. Thus the signal received may not accurately represent the properties of the material.

From the preliminary field study results, the use of SWT methods for the evaluation of structures appears promising. It is interesting to note that the results showed good correlation with expansion despite a difference in composition of the concrete tested (i.e. differences in cement type). Some of the methods appear to indicate some degree of internal damage in Beam 3 with 25% slag, which did not exhibit significant external damage beyond minor surface crazing and which had expanded by only 0.02% (generally considered to be below the level of expansion required to cause cracking). Further inspection at later ages will reveal whether ongoing damage due to ASR is occurring.
To study the application of stress waves for estimating the setting time, concrete strength and quantification of deterioration, tests were carried out on lab-cast and existing concrete elements.

6.1 SUMMARY

A series of tests were carried out to study the response of different SWT analysis indices in monitoring the setting time, strength development, and deterioration of concrete. The four analysis indices (velocity, $\alpha$, $Q$-factor and peak frequency) have shown sensitivity. However, at this point, the use of these techniques to quantify defects is not feasible. The potential errors that can be introduced in carrying out the tests may mask the change in the properties of the concrete. Through further studies and considering various concrete constituents, SWT testing methods could be used for quantifying defects and monitoring the properties of concrete.
Chapter 6 – Summary, Conclusions and Recommendations

6.2 Conclusion on Estimating of Setting and Strength Development

To estimate concrete setting time and compressive strength, tests were carried out by casting concrete having different w/c ratios. With regards to the setting development tests, although velocity and penetration resistance increase with time, there is no distinct shift at the time of initial and final set. Further tests have to be carried out by taking more readings as early as possible.

As for the compressive strength development, the results from the destructive and SWT were combined using linear regression analysis and predictive equations were developed. The $r^2$ values were as high as 87%, which is good for estimating concrete strength. To further demonstrate the validity of the equations, new concrete, with a w/c ratio of 0.5, was cast and monitored for 87 days. At the age of 87 days, estimated concrete strength using only velocity, or the combined method of velocity and $\alpha$, was off by 20%. However, the equations using the combined results of velocity and $Q$-factor significantly improved the estimation. At the age of 87 days, the estimated concrete strength was off by only 5%. This is within acceptable range of error in the field. It should be mentioned that, since $Q$-factor is an attenuation index, a slight error in testing could give erroneous estimates. As for the IE testing, the estimated concrete strength was off only by 6% using velocity alone.

6.3 Conclusion on Quantification of Defects

To study the response of stress waves to deterioration, tests were carried out on lab-cast prisms and field-exposed concrete beams. From the lab cast concrete, it can be concluded that if the input wavelength ($\lambda$) is not smaller than the size of the specimen, the stress wave may propagate
as a guided wave and hence it may not be sensitive to internal damage. Despite the guided wave
effect, both the velocity and transmission peak frequency shift have shown some limited degree
of sensitivity to increasing level of deterioration. However, both \( Q \)-factor and \( \alpha \) coefficient
showed no sensitivity with expansion and some degree of sensitivity with freezing and thawing
deterioration. It should be mentioned that sensitivity of the \( Q \)-factor and \( \alpha \) coefficient were
observed only when the specimen was severely damaged.

Tests were also carried out on field exposed concrete beams. The physical dimensions of these
beams were large enough that the signal was not be affected by the shape factor. From these
preliminary tests, the following conclusions are observed:

- For the ultrasonic through-transmission methods, \( Q \)-factor has shown the most sensitivity, to
  expansion,
- For the ultrasonic pulse-echo testing, the transmission frequency shift has shown the most
  sensitivity to expansion,
- For the impact-echo testing, \( \alpha \) has shown the most sensitivity.

6.4 Recommendations

As regards the time of set tests, different tests need to be carried out than were done here. The
mix designs should be selected in such a way that they represent the range of mix designs used in
practice. Moreover, to make the study of setting time complete, different supplementary
cementing materials should be included.

As for the tests of quantification of defects, either the size of the concrete samples should be
bigger or the input frequency has to be adjusted accordingly to avoid a guided wave situation.
It is shown in this thesis that the stress-wave analysis techniques could be affected by different SWT sampling parameters. Therefore, in carrying out large projects, small pilot project should be carried out and the appropriate sampling parameters, input frequency, and impactors sizes (for IE) should be selected.
REFERENCES

ACI 228.2R-98, 1998. *Nondestructive Test Methods for Evaluation of Concrete in Structures.* Reported by ACI Committee 228, American Concrete Institute.


References


Carette, G.G., and Malhotra, V.M. 1984. "In situ tests: variability and strength prediction at early ages." In Situ Non-Destructive Testing of Concrete (Ed. V.M. Malhotra), ACI SP-82, American Concrete Institute, Detroit, pp. 111-141.


Facaoaru, I. 1984. "Romanian achievements in nondestructive testing of concrete." In Situ Non-Destructive Testing of Concrete (Ed. V.M. Malhotra), ACI SP-82, American Concrete Institute, Detroit, pp. 35-51.


References


Tanigawa, Y., Baba, K. and Mori, H. 1984. "Estimation of concrete strength by combined nondestructive testing method." *In situ Non-Destructive Testing of Concrete* (Ed. V.M. Malhotra), ACI SP-82, American Concrete Institute, Detroit, pp. 57-76.


"Extending the Life of Bridges, Civil + Building Structures" (Ed. Prof. M.C. Forde), The Commonwealth Institute, London, 13th-15th.


APPENDIX A
ANDECSCOPE™
The AndecScope is a computerized stress-wave concrete testing system with frequency spectral analysis capabilities.

The AndecScope operates in three stress wave non-destructive evaluation (NDE) modes: ultrasonic-through transmission, ultrasonic-pulse echo, and impact-echo. This portable concrete NDE system is used to determine the quality and the integrity of concrete elements. It can detect, locate and measure the depth of cracks, flaws, delaminations, and deterioration inside concrete structures. In addition, the AndecScope is capable of measuring the thickness of virtually any concrete structure to precise accuracy. A picture of the AndecScope and the accessories are shown in Figure A-1.

![Figure A-1 AndecScope and the accessories](image)

The physical specification of the AndecScope, the automatic impactor and signal generation/amplifier specifications are present in Table A-1, -2 and -3, respectively.
### Table A-1 AndecScope physical specifications

<table>
<thead>
<tr>
<th>Dimensions</th>
<th>380x180x490 mm (WxHxL)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weight</td>
<td>11.3 Kg, not including the transducers and other cable</td>
</tr>
<tr>
<td>Operation temperature</td>
<td>0° ~ 40°C</td>
</tr>
</tbody>
</table>

### Table A-2 Computer controlled automatic impactor

<table>
<thead>
<tr>
<th>Dimensions</th>
<th>220x80 mm (LxD)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weight</td>
<td>1.8 Kg, not including the transducers and other cable</td>
</tr>
<tr>
<td>Impact-heads</td>
<td>1.6, 3.2, 6.4, 9.5, 12.7, 19.1, 25.4 mm</td>
</tr>
<tr>
<td>Operation temperature</td>
<td>0° ~ 40°C</td>
</tr>
</tbody>
</table>

### Table A-3 Signal generation/Amplifier specifications

<table>
<thead>
<tr>
<th></th>
<th>250 - 4,000 (V)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pulse Voltage</td>
<td></td>
</tr>
<tr>
<td>Toneburst frequency</td>
<td>18 kHz - 140 kHz</td>
</tr>
<tr>
<td>Chirp frequency</td>
<td>20 kHz - 200 kHz</td>
</tr>
<tr>
<td>Amplifier frequency response</td>
<td>100 Hz - 150 kHz</td>
</tr>
<tr>
<td>Amplifier gain</td>
<td>0 - 70 dB</td>
</tr>
<tr>
<td>Pre-amplifier gain (Optional)</td>
<td>50 dB</td>
</tr>
<tr>
<td>Trigger delay time</td>
<td>0 - 20,000 μs</td>
</tr>
<tr>
<td>Receiver Channel</td>
<td>2 - 10 independent</td>
</tr>
</tbody>
</table>
APPENDIX B
SETTING TIME (ST)
APPENDIX B-1
ST-ULTRASONIC-THROUGH TRANSMISSION
Appendix B – ST (UTT)

ST UTT
w/c=0.4 clap. time 424 min.

Voltage (mV)

Time (μs)

ST UTT
w/c=0.40 clap. time 424 min

Amplitude

Frequency (kHz)
APPENDIX B-2
ST - IMPACT-ECHO
Appendix B – ST (IE)

ST IE  
w/c = 0.4 elap. time 314 min.

ST IE  
w/c = 0.4 elap. time 314 min.

ST IE  
w/c = 0.4 elap. time 374 min.

ST IE  
w/c = 0.4 elap. time 374 min.

ST IE  
w/c = 0.4 elap. time 424 min.

ST IE  
w/c = 0.4 elap. Time 424 min.

B-10
APPENDIX C-1

f_c' - ULTRASONIC THROUGH TRANSMISSION
Appendix C – $f_c$ (UTT)

$f_c$ UTT
w/c=0.5 Day 0.5

$f_c$ UTT
w/c=0.5 Day 0.5

$f_c$ UTT
w/c=0.5 Day 1

$f_c$ UTT
w/c=0.5 Day 1

$f_c$ UTT
w/c=0.5 Day 2

$f_c$ UTT
w/c=0.5 Day 2

$f_c$ UTT
w/c=0.5 Day 3

$f_c$ UTT
w/c=0.5 Day 3

C-5
APPENDIX C-2

fc' - IMPACT-ECHO
Appendix C – fc' (IE)

fc' IE  
w/c=0.4 Day 0.5

fc' IE  
w/c=0.4 Day 1

fc' IE  
w/c=0.4 Day 2

fc' IE  
w/c=0.4 Day 3

C-10
APPENDIX D
RAW DATA - SETTING TIME & STRENGTH DEVELOPMENT
APPENDIX D-1
SETTING TIME - RAW DATA
### Appendix D

#### w/c = 0.40

<table>
<thead>
<tr>
<th>Time (min.)</th>
<th>PR (MPa)</th>
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<tbody>
<tr>
<td>176</td>
<td>0.0552</td>
</tr>
<tr>
<td>259</td>
<td>2.7048</td>
</tr>
<tr>
<td>259</td>
<td>2.6496</td>
</tr>
<tr>
<td>259</td>
<td>2.7048</td>
</tr>
<tr>
<td>259</td>
<td>2.8152</td>
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<tr>
<td>314</td>
<td>7.176</td>
</tr>
<tr>
<td>374</td>
<td>19.32</td>
</tr>
<tr>
<td>424</td>
<td>49.128</td>
</tr>
</tbody>
</table>

#### ASTM Setting Time Test - PR vs. Time

![Graph](image_url)

**Time (min)**

#### w/c = 0.50

<table>
<thead>
<tr>
<th>Time (min.)</th>
<th>PR (MPa)</th>
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</thead>
<tbody>
<tr>
<td>250</td>
<td>0.2484</td>
</tr>
<tr>
<td>294</td>
<td>2.208</td>
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<tr>
<td>380</td>
<td>10.626</td>
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<tr>
<td>420</td>
<td>15.18</td>
</tr>
<tr>
<td>485</td>
<td>38.64</td>
</tr>
</tbody>
</table>

#### ASTM Setting Time Test - PR vs. Time

![Graph](image_url)

**Time (min)**

#### W/C 0.60

<table>
<thead>
<tr>
<th>Time (min.)</th>
<th>PR (MPa)</th>
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</thead>
<tbody>
<tr>
<td>198.6</td>
<td>0.1449</td>
</tr>
<tr>
<td>256.4</td>
<td>0.7452</td>
</tr>
<tr>
<td>316.4</td>
<td>2.622</td>
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<tr>
<td>377.6</td>
<td>6.141</td>
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<tr>
<td>405.6</td>
<td>19.044</td>
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<tr>
<td>435.6</td>
<td>19.32</td>
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<tr>
<td>489.6</td>
<td>30.36</td>
</tr>
</tbody>
</table>

#### ASTM Setting Time Test - PR vs. Time

![Graph](image_url)

**Time (min)**

D-3
### Appendix D

**90 DAYS COMPRESSIVE STRENGTH TEST**

**Date of Cast** = February 16, 1999

**W/C ratio of 0.40**

<table>
<thead>
<tr>
<th>Date</th>
<th>27-Aug-98</th>
<th>29-Aug-98</th>
<th>30-Aug-98</th>
<th>3-Sep-98</th>
<th>10-Sep-98</th>
<th>17-Sep-98</th>
</tr>
</thead>
<tbody>
<tr>
<td>Time (days)</td>
<td>0.5</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>7</td>
<td>14</td>
</tr>
<tr>
<td>Load (KN)</td>
<td>214.4</td>
<td>434</td>
<td>535</td>
<td>586</td>
<td>639</td>
<td>731</td>
</tr>
<tr>
<td>f'c (Mpa)</td>
<td>11.59</td>
<td>23.46</td>
<td>28.92</td>
<td>31.68</td>
<td>34.54</td>
<td>39.51</td>
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</table>

**W/C ratio of 0.50**

<table>
<thead>
<tr>
<th>Date</th>
<th>28-Aug-98</th>
<th>29-Aug-98</th>
<th>30-Aug-98</th>
<th>3-Sep-98</th>
<th>11-Sep-98</th>
<th>18-Sep-98</th>
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</thead>
<tbody>
<tr>
<td>Time (days)</td>
<td>0.5</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>7</td>
<td>14</td>
</tr>
<tr>
<td>Load (KN)</td>
<td>113.5</td>
<td>212.8</td>
<td>279.7</td>
<td>377.5</td>
<td>407</td>
<td>483</td>
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<tr>
<td>f'c (Mpa)</td>
<td>6.14</td>
<td>11.50</td>
<td>15.12</td>
<td>20.41</td>
<td>22.00</td>
<td>26.11</td>
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</table>

**W/C ratio of 0.60**

<table>
<thead>
<tr>
<th>Date</th>
<th>4-Aug-98</th>
<th>5-Aug-98</th>
<th>6-Aug-98</th>
<th>7-Aug-98</th>
<th>11-Aug-98</th>
<th>18-Aug-98</th>
<th>1-Sep-98</th>
</tr>
</thead>
<tbody>
<tr>
<td>Time (days)</td>
<td>0.5</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>7</td>
<td>14</td>
<td>28</td>
</tr>
<tr>
<td>Load (KN)</td>
<td>5.6</td>
<td>144.9</td>
<td>209.5</td>
<td>241.4</td>
<td>310</td>
<td>380.8</td>
<td>445</td>
</tr>
<tr>
<td>f'c (Mpa)</td>
<td>0.3</td>
<td>7.83</td>
<td>11.32</td>
<td>13.04</td>
<td>17.2</td>
<td>21.13</td>
<td>24.05</td>
</tr>
</tbody>
</table>

### Destructive compressive strength vs. Time

- **w/c = 0.40**
- **w/c = 0.50**
- **w/c = 0.60**
- **90-DAYS f'c (w/c = 0.50)**

*Graph showing the relationship between compressive strength and time for different w/c ratios.*

**D-5**
APPENDIX E
ALKALI-SILICA REACTIONS
APPENDIX E-1
ASR - ULTRASONIC THROUGH TRANSMISSION
Appendix E – ASR (UTT)

ASR 2A UTT
Day=32 week exp=0.142

ASR 2A UTT
Day=37 weeks exp=0.172

ASR 2A UTT
Day=41 weeks exp=0.146

E-5
Appendix E - ASR (IE)

ASR 2A IE
Day = 16 weeks exp = 0.126

ASR 2A IE
Day = 21 weeks exp = 0.135

ASR 2A IE
Day = 24 weeks exp = 0.137

ASR 2A IE
Day = 28 weeks exp = 0.144
ASR 2B IE
Day=16 weeks exp=0.126

ASR 2B IE
Day=21 weeks exp=0.135

ASR 2A IE
Day=24 weeks exp=0.137

ASR 2B IE
Day=28 weeks exp=0.144
APPENDIX F
FREEZING and THAWING
APPENDIX F-1
F/T - ULTRASONIC-THROUGH TRANSMISSION
Appendix F – F/T (UTT)
APPENDIX F-2
F/T - IMPACT-ECHO