NONDESTRUCTIVE EVALUATION OF PLAIN AND POLYMER CONCRETE

by

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A Dissertation Submitted to the faculty of the
DEPARTMENT OF CIVIL ENGINEERING AND ENGINEERING MECHANICS

In Partial Fulfillment of the Requirements

For the Degree of
DOCTOR OF PHILOSOPHY

In the Graduate College
THE UNIVERSITY OF ARIZONA
2014
THE UNIVERSITY OF ARIZONA

GRADUATE COLLEGE

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Entitled: Nondestructive Evaluation of Plain and Polymer Concrete.

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SIGNED: __________________________

Ehsan Mahmoudabadi
ACKNOWLEDGEMENTS

I would like to thank all those who stood behind me and supported me throughout my life. First of all, I would like to thank God. I would like to thank my parents, Seyed Hassan Mahmoudabadi and Zohreh Abdali, for all their love and continuous encouragement and support during my study.

I wish to express my gratitude to my advisors, Professor Tribikram Kundu and Professor Hamid Saadatmanesh, for their teaching, guidance, and support during the course of this research. This dissertation would not have been possible without my supervisors who not only served as my advisors but also encouraged and challenged me throughout my academic program. I would like to thank Professor Kundu and Professor Saadatmanesh for supporting me financially during these years and for the use of the constitutive modeling laboratory, helping me with the laboratory test.

I would like to thank the members of my committee, Prof. G. Frantziskonis and Prof. John M. Kemeny for their advice and criticism, which helped to improve my work. Also I would like to thank Professor L. Zhang to let us use the device in his laboratory.

Special thanks are due my friends and colleagues Mr. Abdullah Al-Hussein, Umar Amjad, Jesus Eiras, Richard Kaiser, Anshul Agarwal, Vahab Toufigh and Vahid Toufigh for their assistance in the laboratory tests. I would like to thank all my friends and my family for their support.
DEDICATION

I would like to dedicate this dissertation to my father, Seyed Hassan, my mother, Zohreh

Also I want to dedicate this dissertation to my sister, Nafiseh and my brother, Ershad

and their children Elina and Amirhossein
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ABSTRACT

Nondestructive measurement of the concrete strength is an important topic of research. Among different nondestructive testing (NDT) methods the ultrasonic pulse velocity (UPV) technique is the most popular method for concrete strength estimation. The device which is commonly used for finding this property is called “PUNDIT”. Many studies have been conducted to find the relationship between the concrete strength and its UPV (ultrasonic pulse velocity) using this device.

The purpose of this study is to gain a better understanding of nondestructive evaluation of plain and polymer concretes to determine their properties using a new method. To achieve this goal, instead of using PUNDIT an alternative method has been used to generate the ultrasonic waves. The main difference of this new method in comparison to the earlier methods followed by other researchers is that in this method a linear chirp signal with different frequencies is transmitted through the material unlike the single frequency signal as PUNDIT does. This new method is applied to both plain and polymer concrete samples.

While measuring concrete strengths by the UPV method almost all researchers have neglected the effect of applied stress or load on the concrete member. When the specimen is tested under stress, its behavior is quite different from when it is tested without any load. In this investigation attempts have been made to properly incorporate the effect of the applied load on the strength prediction of plain and polymer concrete specimens from the UPV generated values. After applying the load on the specimen in multiple steps - at 20%, 40%, 60% and 80% of its failure strength $f'_c$ - the time of flight
(TOF) was measured for every loading step. From regression analyses the best equations which can be used to find the applied stress on the structure from the velocity values have been derived.

Under cyclic loadings pre-existing cracks inside both conventional concrete and polymer concrete may grow and cause catastrophic failure of the structure. Structural failure under cyclic loading is also known as the fatigue failure. Another aim of this study is to investigate the behavior of plain and polymer concretes under the cyclic loading or fatigue. The specimens used for this study were subjected to compressive loads for different numbers of cycles. The applied load was gradually increased. After the loading-unloading cycles, nonlinear impact resonance acoustic spectroscopy (NIRAS) test was carried out on the samples using instrumented impact hammer. The degree of nonlinearity in the concrete specimens was measured by recording the shift in the resonance frequency as the impact energy increased. Experimental results from the conventional plain concrete and polymer concrete specimens were compared to investigate which type of concrete exhibits more nonlinear behavior under fatigue.
CHAPTER 1

INTRODUCTION

1.1 General

In this Chapter physical and mechanical properties of two types of widely used materials in civil engineering, plain and polymer concrete, are reviewed. The nondestructive methods used to find these materials’ properties are also discussed. The main goal of this study is to use new nondestructive methods to find plain and polymer concrete’s compressive strength, and to find the effects of applied load on the nondestructive measurements output. Nonlinear behavior of these two materials under cyclic loading is also studied.

1.2 Plain and Polymer Concrete

Concrete is the most commonly used man-made product in the world. It is a composite construction material made primarily of aggregate, cement, and water. The technology of concrete was known by the Ancient Romans and was widely used within the Roman Empire. After the Roman Empire period, the use of concrete was limited until the technology was re-pioneered in the mid-18th century [1].

A variety of concrete with different compositions and properties are used in the construction industry. Concrete is widely used for making architectural features in structures, foundations, brick/block walls, pavements, bridges, roads, runways, parking structures, dams, pools, pipes, footings for gates, fences and poles and even boats.
Concrete structures can have a long service life. Since concrete has a high thermal mass and very low permeability, it can be used for energy efficient housing [2].

By changing the proportions of concrete’s main ingredients, which are aggregates, cement and water, many types of concrete can be made. In this manner, or by substituting the cementitious and aggregate parts with materials with certain properties, finished product can be made for very specific applications such as for its chemical and thermal resistance, specific weight, abrasion resistance, etc.

Aggregate constitute the largest portion of materials in a concrete mix, and it generally consist of a coarse gravel or crushed rocks such as limestone, or granite, along with finer materials such as sand. Cement serves as the binding agent for the aggregate. It is generally Portland cement or combined with other cementitious materials such as fly ash and slag cement. Water is then mixed with this dry composite, which enables it to initiate the chemical hydration reaction in the cement binder and to convert the materials into a plastic state that can be formed into the desired shape. The water reacts with the cement creating a jell-like substance that upon drying bonds the other components together, eventually creating a strong stone-like material.

Concrete has a relatively high compressive strength, but much lower tensile strength when compared to its compressive strength. For this reason, it is usually reinforced with materials that are strong in tension like steel rebars. The stiffness of concrete is relatively constant at low stress levels, but it starts to decrease at higher stress levels as matrix cracking develops.
Tests can be conducted to ensure that the properties of concrete correspond to the specifications needed for an application. Different mixes of concrete ingredients produce different strengths, measured in psi or MPa. Different strengths of concrete are used for various purposes. Very low-strength (2000 psi or less) concrete may be used when the concrete must be lightweight [3]. Lightweight concrete is often achieved by adding air, foams, or lightweight aggregates. For most routine applications, 3000 to 4000 psi concrete is used. 5000 psi concrete is also commercially available as a more durable but a more expensive option. The 5000 psi concrete is often used for larger civil structures [4].

Strengths above 5000 psi are generally used for specific building elements. For example, the lower floor columns of high-rise concrete buildings may use concrete strength of 12,000 psi or more, to keep the size of the columns relatively small. In bridges, concrete strength of 10,000 psi may be used to allow construction of longer spans [5, 6]. Other structural needs may also require high strength concrete. If a structure must be very rigid, concrete of very high strength may be specified; it can be stronger than what is required to bear the service loads. Strengths as high as 19,000 psi have been used in different applications.

Concrete is the most popular building material because it is relatively inexpensive and can easily be shaped into various forms. Concrete has many desirable properties, but it also has many limitations. Its low tensile strength, poor durability in certain environments and susceptibility to sulfate and acid attack have restricted its use. These problems have been solved for some applications. Concrete has been reinforced with embedded steel in regions subjected to tensile forces. Air-entraining agents have been used when the concrete had to resist severe freezing and thawing. Special cements have
been developed for concrete subjected to sulfate attack and protective coatings have been placed on concrete exposed to acids. These preceding treatments have resolved some of deficiencies of concrete, but there has not been a single solution which would improve or solve all four major problem areas. Since 1966 a new approach has been investigated that shows great potential of improving the properties of concrete by the formation of a composite material called "polymer concrete" [7]. By substituting the cement part of concrete with epoxy, a special kind of concrete called Polymer Concrete can be made.

ACI Committee 548 deals with various aspects of polymer concrete use since 1971 [7]. Polymer Concrete is a composite material consisting of a polymeric resin and aggregate fillers. This kind of concrete is strong, durable, non-absorbent and chemical-resistant. These properties make it desirable over conventional materials like concrete, metals, fiberglass and plastic. The most widely-recognized use of polymer concrete is in solid surface countertops for modern kitchens and baths. With its decorative finishes and excellent resilience, polymer concrete is ideal for many decorative applications including park benches, outdoor furniture, and waste containers. It also has great potential in industrial and construction applications and has been used for pump bases, flooring blocks, chemical containment, trench drains and more.

Polymer concrete also may be used for new construction or repairing of old concrete. The adhesion properties of polymer concrete allow patching for both polymer and cementitious concretes. The low permeability of polymer concrete allows it to be used in swimming pools, sewer pipes, drainage channels, electrolytic cells for base metal recovery, and other structures that contain liquids. It can also be used as a replacement for asphalt pavement, for higher durability and higher strength. It is a robust, cost-saving
material that can be shaped to meet even complex part requirements. The use of polymer concrete as a building material can save both material and labor cost.

In polymer concrete, thermosetting resins are used as the principal polymer component due to their high thermal stability and resistance to a wide variety of chemicals. Polymer concrete is also composed of aggregates that include silica, quartz, granite, limestone, and other high quality materials. The aggregate must be of good quality, free of dust and other debris, and dry. Failure of these criteria can reduce the bond strength between the polymer binder and the aggregate.

When compared with ordinary concrete, typical polymer concrete exhibits the following improvements [7]:

- Compressive strength is increased four times;
- Modulus of rupture is improved by 256 percent;
- Freezing/thawing resistance is increased by 300 percent;
- Hardness-impact is increased by 73 percent;
- Water permeability is virtually eliminated;
- Water absorption is decreased up to 95 percent; and
- Resistance to many corrosive materials and conditions is greatly improved.

These impressive improvements are attributed to the polymer, since the exposure of plain concrete to radiation or to thermal catalytic treatment gives no improvements. The extent of the improvement depends, at least in part, on the monomer material used.
and the technique employed to polymerize. In tests reported jointly by the Bureau of Reclamation and Brookhaven National Laboratory, it was found that of the monomers tested, methyl-methacrylate appeared most promising in improving concrete properties. These tests showed that concrete polymerized by heat-catalyst had strengths 7 to 15 percent lower than concrete polymerized by radiation [7].

### 1.3 Strength of concrete

Strength is probably the most important single property of concrete, since the first consideration in structural design is that the structural elements must be capable of carrying the imposed loads. The maximum value of stress in a loading test is usually taken as the strength, even though under compressive loading the test piece is still whole (but with substantial internal cracking) at this stress, complete breakdown subsequently occurs at higher strains and lower stresses. Strength is also important because it is related to several other important properties which are more difficult to measure directly, and a simple strength test can give an indication of these properties [8].

When used in structures concrete is subject to different types of loading, resulting in different modes of failure. Knowledge of the relevant strength is therefore important. For example, in columns or reinforced concrete beams, the compressive strength is required. For cracking of a concrete slab the tensile strength is important. Other situations may require torsional strength, fracture or impact strength or strength under multi-axial loading [8].
1.3.1 Compressive strength of concrete

As discussed above, one of the most important properties of concrete is its compressive strength. Based on the ingredients used in plain or polymer concrete, the strength of the resulting material is changed. Compressive strength is the capacity of a material or structure to withstand axially directed pushing forces. It provides data (or a plot) of force versus deformation for the conditions of the test method. When the limit of compressive strength is reached, brittle materials are crushed. Concrete can be made to have high compressive strength, e.g. many concrete structures have compressive strengths in excess of 50 MPa, whereas a material such as soft sandstone may have a compressive strength as low as 5 or 10 MPa.

Concrete differs from other construction materials because it can be made from an infinite number of combinations of suitable materials, and its final properties depend on the treatment it undergoes after it arrives at the job site. The efficiency of the consolidation and the effectiveness of curing procedures are critical for attaining the full potential of a concrete mixture. Although concrete is known for its durability, it is susceptible to a range of environmental degradation factors that can limit its service life.

Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens is described in ASTM C39 [9]. This method is destructive, that means to find the compressive strength of concrete, the sample must be loaded up to its failure load.

There has always been a need for test methods to measure the in-situ properties of concrete for quality assurance and for evaluation of existing conditions. Ideally, these
methods should be nondestructive so that they do not harm the structure and also permit retesting at the same locations to evaluate changes in properties with time.

1.4 Destructive and Nondestructive evaluation of concrete’s compressive strength

Most of the society’s infrastructure supporting certain sectors of human activity is based on cementitious materials. Bridges, highways, water intake facilities and other structures are made of concrete. These structures sustain external functional loads, their own weights, as well as deterioration due to temperature cycles and the attack of the environmental agents during their useful life span. The number of civil infrastructures built more than 50 years ago is estimated to be several hundreds of thousands worldwide [10]. The operational efficiency of these structures is of primary importance for economic reasons, but mostly for human safety.

Concrete structures have ceased to be considered maintenance-free [10]. They should be inspected at regular intervals; their damage level should be evaluated and when necessary, be repaired. Maintenance or repair projects should be based on prioritization as to the importance of the structure and its damage status. Therefore, economic, fast and reliable characterization schemes are highly demanded. In most of the currently available methods of assessment, elastic modulus of material and strength characteristics are the primary criteria of consideration. It often becomes necessary that mechanical tests be conducted on samples acquired from the target structure, through extraction of such samples often inflicts more damage to the already-degraded target structure. In addition, considering the nature of these exercises, which are usually conducted at selected
locations in the structure, they result in only limited assessment of the structure and often not representative of the entire structure.

Recently, extensive research has been reported on nondestructive techniques resulting in reliable assessment of structural condition in-situ concrete materials and structures. The characterization of concrete structures by Non Destructive Testing (NDT) produces reveals mainly qualitative, but very important results concerning damage.

Various destructive and non-destructive test (NDT) methods have been developed for determining the compressive strength. In non-destructive tests, the sample is not destroyed and the test is very useful in determining the strength of existing buildings or structures, whereas in destructive tests a sample is made and then destroyed to find out the strength of concrete.

Non-destructive testing of concrete can be defined as the test method used to examine the properties of concrete used in the actual structure. These test methods are also called in-situ tests or in-place tests. Traditionally these tests are said to be non-destructive, although some minor damage to the structure may occur. An important feature of a non-destructive test is that the place where the test is conducted can be used for re-testing. The direct determination of the strength of concrete, as destructive tests, implies that concrete specimens must be loaded to failure. Therefore, the determination of concrete strength requires special specimens to be taken, shipped, and tested at laboratories. This procedure may give the actual strength of concrete, but may cause harm in existing structures and delay in evaluating them. Leshchinsky [11] summarized the following advantages of nondestructive tests, as compared to core testing:


1. A reduction in the labor consumption of testing.

2. A decrease in labor consumption of preparatory work (such as tedious work associated with determining the location and diameters of reinforcement bars).

3. A smaller amount of structural damage in testing.

4. A lower probability of such structural damage which may cause the need for reinforcement.

5. A possibility of testing concrete strength in structures where cores cannot be drilled (thin-walled, densely-reinforced, etc.).

6. An application of less expensive testing equipment.

The use of non-destructive tests has increased the safety level of construction and also has helped to improve the scheduling of construction. It increases the speed of construction besides being economically beneficial. Non-destructive tests give a relation between various properties of the structure and its strength. In NDT for concrete, the strength of concrete is assessed and not measured. An experimental relationship between the property being measured and compressive strength of concrete is determined in the laboratory and then used to convert the NDT measurements into strength values. An understanding of the physical relationship between the measured NDT parameters and the compressive strength is essential and often an engineering judgment should be exercised to interpret the test results. These tests are usually done in case of disputes between the parties which are involved in construction of any type of structure.
Compared with the development of NDT methods for steel structures, the development of NDT methods for concrete has progressed at a slower pace, because concrete is more difficult to test nondestructively than steel. Concrete is highly heterogeneous, it is electrically nonconductive but usually contains significant amount of steel reinforcement, and it is often used in thick members. Thus, it has not been easy to transfer NDT technologies developed for steel to the evaluation of concrete. In addition, there has been little interest among those from the traditional NDT community (physicists, electrical engineers, mechanical engineers) to develop test methods for concrete. Since 1980s, however, advances have occurred in NDT of concrete as a result of the microcomputer revolution and the development of powerful signal-processing techniques. No standard definition exists for nondestructive test as applied to concrete. For some people, it is any test that does not damage the properties of concrete. For others, it is a test that does not alter the function of a structure, in which case the drilling of cores maybe considered to be a NDT method. For some others, it is a test that does less damage to the structure than what drilling of cores does. Additional information on the application of these methods is available in ACI 228.1R [12], ACI 228.2R [13], Malhotra and Carino [14], and Bungey and Millard [15]. The review of different NDT methods applicable to concrete structures (Table 1.1) was given in the Report of ACI Technical Committee 228-Non-Destructive Testing of Concrete [12,13], in which the capabilities, limitations and potential applications of various NDT methods were presented. The ACI Reports, ACI 364.1R-94 [16] and ACI 228.2R-98 [13], have been published together in 1999 as “The Concrete Repair Manual.”
Table 1.1 Nondestructive methods for evaluation of concrete structures

<table>
<thead>
<tr>
<th>Method Scheme</th>
<th>Principle:</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rebound Hammer</td>
<td>Measurement of rebound height after striking concrete surface with spring loaded hammer; correlation between rebound number and compressive strength is determined</td>
</tr>
<tr>
<td>Ultrasonic Pulse Velocity (UPV)</td>
<td>Principle:</td>
</tr>
<tr>
<td>Measurement of the travel time of ultrasonic P-wave, over a known path length, calculation of pulse velocity in concrete, regression analysis of relationship between pulse velocity and concrete properties (mainly compressive strength)</td>
<td></td>
</tr>
<tr>
<td>Method Scheme</td>
<td>Principle</td>
</tr>
<tr>
<td>---------------</td>
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</tr>
<tr>
<td><strong>Ultrasonic pulse echo, UP-E</strong></td>
<td>Propagation of a short pulse of ultrasonic wave; measurement of travel time to boundaries separating materials with different densities and elastic properties; by knowing the wave speed the distance to the reflecting interface is detected.</td>
</tr>
<tr>
<td><strong>Spectral Analysis of Surface Waves, SASW</strong></td>
<td>Analysis of the spectrum of the disperse generalized Rayleigh surface wave in a layered system; the received signal is analyzed to obtained the dependence of phase velocity on the frequency</td>
</tr>
<tr>
<td><strong>Ground-Penetrating Radar, GPR</strong></td>
<td>Non-contact method; method analogous to UP-E techniques, except that pulses of electromagnetic waves are used instead of stress waves, results are registered as a water plot</td>
</tr>
<tr>
<td>Method Scheme</td>
<td>Principle:</td>
</tr>
<tr>
<td>---------------</td>
<td>------------</td>
</tr>
<tr>
<td><strong>Infrared Thermography, IT</strong></td>
<td>Measurement of surface temperature differences – thermographic image</td>
</tr>
<tr>
<td><strong>Electro-magnetic (covermeters)</strong></td>
<td>Interaction between the steel reinforcement and low-frequency electromagnetic fields; two principles are used: magnetic reluctance and eddy currents</td>
</tr>
<tr>
<td><strong>Radioactive (radiometric, radiographic)</strong></td>
<td>High-energy electromagnetic radiation (X-ray, gamma, neutron); concrete evaluation on the basis of changes in detected intensity of radiation; two types of methods based on the type of sensor used have been developed: radiometric (detector) and radiographic (photographic film)</td>
</tr>
</tbody>
</table>
NDT methods are applied to concrete structures for 4 main reasons:

- Quality control of new structures,
- Unexpected problems with new construction,
- Evaluation of existing structures, including evaluation prior to repair,
- Quality control of concrete repair.

In general, considering the measured parameters, NDT methods can be divided into the following categories [17]:

- Rebound hammer,
- Acoustic methods – stress wave propagation (ultrasound, acoustic emission, impact echo, etc.),
- Radiation methods (X-ray, gamma ray, neutron emission, etc.),
- Electromagnetic methods,
- Others – e.g. infrared thermography.

Another classification takes into account the aim of nondestructive evaluation. Two main categories can be recognized [17]:

- Evaluation of concrete strength and its homogeneity (e.g. rebound hammer, ultrasonic pulse velocity),
- Evaluation of structural integrity
- Detection of various types of defect in concrete, detection and evaluation of steel reinforcement (e.g. visual inspection, stress wave propagation, impact-echo, infrared thermography, radiation methods, electromagnetic methods).

Among the available nondestructive methods, the rebound hammer and the ultrasonic pulse velocity testers are the most commonly used ones for the evaluation of concrete structures. This is true in many developing countries where the lack of technology and funds requires the optimization of available methods and techniques. Also, in many developing countries, records of tested concrete constituents may not be available, or the available data lack some of the requirements for strength estimation by means of ultrasonic pulse velocity (or any other nondestructive testers) [17]. These two common methods are described in detail in the following.

1.4.1 Rebound Number:

The rebound hammer test is described in ASTM C805 [18] and BS 1881: Part 202 [19]. A Rebound hammer, also known as a Swiss hammer or a Schmidt hammer, is a device to measure the elastic properties or strength of concrete or rock, mainly surface hardness and penetration resistance.

In 1984, Ernst Schmidt, a Swiss engineer, developed a device for testing concrete based on the rebound principle [14]. As was the case with earlier indentation tests, the motivation for this new device came from tests developed to measure the hardness of metals. In this case, the new device was an outgrowth of the Scleroscope test, which involved measuring the rebound height of a diamond tipped hammer, or mass that is dropped from a fixed height above the test surface. As noted by Kolek [21], when
concrete is struck by a hammer, the degree of rebound is an indicator of the hardness of the concrete. Schmidt standardized the hammer blow by developing a spring-loaded hammer and devised a method to measure the rebound of the hammer. Several different models of the device were built [22], and Figure 1 provides a schematic of the model that was eventually adopted for field use. The essential parts of the Schmidt rebound hammer are the outer body, the hammer, the plunger, the spring, and the slide indicator. To perform the test, the plunger is extended from the body of the instrument, which causes a latch mechanism to grab hold of the hammer (Figure 1.1a). The body of the instrument is then pushed toward the concrete surface which stretches the spring attached to the hammer and the body (Figure 1.1b). When the body is pushed to the limit, the latch is released and the hammer is propelled toward the concrete by a combination of gravity and spring forces (Figure 1.1c). The hammer strikes the shoulder of the plunger and it rebounds (Figure 1.1d). The rebound distance is measured on a scale by a slide indicator. The rebound distance is expressed as a rebound number, which is the percentage of the initial extension of the spring [21]. Currently, various models of the instrument that are available differ in the mass of the hammer and the stiffness of the spring; thus, different impact energies can be used for different materials.
Due to its simplicity and low cost, the Schmidt rebound hammer is by far the most widely used nondestructive test device for concrete. Although the test appears simple, there is no simple relationship between the rebound number and the strength of concrete. In principle, the rebound is affected by the small penetration (elastic and inelastic) of the end of the plunger in contact with the concrete. The more the end of the plunger penetrates, the lower is the rebound; thus, the rebound number is likely to be influenced by the elastic stiffness and the strength of the concrete. Because the rebound number is indicative of the near-surface properties of the concrete, it may not be indicative of the bulk concrete in a structural member. The report of ACI Committee 228 [12] outlines some of the factors that may result in rebound numbers that are not representative of the bulk concrete. They include:
• The moisture condition of the surface concrete affects the rebound number; a dry surface results in a higher rebound number.

• The presence of a surface layer of carbonation increases the rebound number.

• The surface texture affects the rebound number, with smooth hard-troweled surfaces giving higher values than rough-textured surfaces.

• The rebound number is affected by the orientation of the instrument in relation to the direction of gravity (approximate correction factors are available).

The test method starts by the careful selection and preparation of the concrete surface to be tested. Once the surface is chosen, it should be prepared by an abrasive stone so that the test surface has a smooth finish. Then, a fixed amount of energy is applied by pushing the hammer against the test surface. The plunger must be allowed to strike perpendicularly to the surface. The angle of inclination of the hammer affects the result. After impact, the rebound number should be recorded. Since the rebound number is affected by the near-surface conditions, erratic results may occur if the plunger is located directly over a coarse aggregate particle or a subsurface air void. To account for these possibilities, ASTM C 805 (ASTM, 2002b) [23] requires that ten rebound numbers be taken for a test. If a reading differs by more than six units from the average, that reading should be discarded and a new average should be computed based on the remaining readings. If more than two readings differ from the average by six units, the entire set of readings is discarded. There is no unique relation between hardness and strength of concrete but an empirical relation can be obtained for a given concrete from the experimental data. However, this relationship is dependent on several factors.
affecting the concrete surface such as degree of saturation, carbonation, temperature, surface preparation, location, and type of surface finish. The result is also affected by the type of aggregate, mix proportions, hammer type, and hammer inclination. Areas exhibiting honeycomb, scaling, rough texture, or high porosity must be avoided. Concretes being tested must be approximately of the same age, and have similar moisture content and same degree of carbonation (note that carbonated surfaces yield higher rebound values).

The rebound hammer was constructed and tested extensively at the Swiss Federal Materials Testing and Experimental Institute in Zurich. A correlation was developed between the compressive strength of standard cubes and the rebound number, and this correlation was provided with the instrument. As other investigators began to develop correlations between strength and rebound number, it became evident that a unique relationship did not exist between strength and rebound number [21]. The current recommended practice (ACI Committee 228) is to develop the strength relationship using the same as used in construction. Without such a correlation, the rebound hammer is useful only for detecting gross changes in concrete quality throughout a structure.

In summary, the rebound number method is recognized as a useful tool for performing quick surveys to assess the uniformity of concrete. Since many factors besides concrete strength can affect rebound number, the sole use of this method is not recommended where reliable strength estimates are needed.
1.4.2 Methods used in the ultrasonic measurement technique

The most commonly used ultrasonic methods for concrete testing are the pulse velocity method, the echo method, and the resonance method. Visual and holographic methods, used for direct visualization of the ultrasonic field in the given medium, are of lesser use.

1.4.2.1 Ultrasonic pulse velocity

The ultrasonic pulse velocity method (also known as the transmission method) is one of the oldest and simplest methods of materials testing. The method consists of determining the travel time, over a known path length of the longitudinal ultrasonic wave after its transmission through the tested medium (Figure 1.2).

![Figure 1.2 Ultrasonic pulse velocity method:
 a) direct method, b) semi-direct method, c) indirect (surface) method](image)

Both the emitting and receiving transducers are usually placed on the opposite sides of the tested sample (coaxially if possible). Other transducer arrangements are also used in concrete testing (Figures 1.2b, c). They can be placed on the perpendicular surfaces (Fig.2b) or on the same side of the tested member (Figure 1.2c).

This test is described in ASTM C597 [24] and BS 1881: Part 203 [25]. When the surface of a semi-infinite solid is excited by a time varying mechanical force, energy is
radiated from the source as three distinct types of elastic wave propagation. The fastest of these waves has particle displacements in the direction of travel of the disturbance and is called the longitudinal, compression or P-wave. The longitudinal wave velocity $C_p$ is defined as:

$$C_p = \sqrt{\frac{\lambda + 2\mu}{\rho}}$$  \hspace{1cm} (1.1)$$

Where $\lambda$ and $\mu$ are the lame’s first and second constants and $\rho$ is the mass density.

By writing the relationship between $\lambda$, $E$ and $\nu$, the above equation can also be defined as a function of the Young’s modulus $E$, the Poisson’s ratio $\nu$, and the mass density $\rho$ [26], and is given by:

$$\lambda = \frac{\nu E}{(1+\nu)(1-2\nu)}$$  \hspace{1cm} (1.2)$$

From equations 1 and 2:

$$C_p = \sqrt{\frac{E(1-\nu)}{\rho(1+\nu)(1-2\nu)}}$$  \hspace{1cm} (1.3)$$

It is clear from this equation that the velocity is dependent on the modulus of elasticity of concrete. Monitoring the modulus of elasticity of concrete by measuring pulse velocity is generally not recommended. On the other hand, it has been shown that the strength of concrete and its modulus of elasticity are related (it will be shown in the literature review). In the test described in BS 1181: Part 203 [25], the time the pulses take to travel through concrete is recorded. Then, the velocity is calculated as:

$$V = \frac{L}{T}$$  \hspace{1cm} (1.4)$$
Where \( V = \text{pulse velocity (m/s)}, \ L = \text{length (m)}, \) and, \( T = \text{effective time (s)}, \) which is the measured time minus the zero time correction. The zero time correction is equal to the travel time between the transmitting and receiving transducers when they are pressed firmly together. The ultrasonic pulse velocity results can be used to:

- Check the uniformity of concrete,
- Detect cracking and voids inside concrete,
- Control the quality of concrete and concrete products by comparing results to a similarly made concrete,
- Detect the condition and deterioration of concrete,
- Detect the depth of a surface crack, and,
- Determine the strength if previous data are available.

The quality of concrete shown in table 1.2 based on the Ultrasonic Pulse Velocity (UPV) number was reported by Whitehurst in 1951 [27]:

<table>
<thead>
<tr>
<th>Pulse Velocity (m/s)</th>
<th>Condition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Above 4570</td>
<td>Excellent</td>
</tr>
<tr>
<td>3660–4570</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>Below 2130</td>
<td>Very poor</td>
</tr>
</tbody>
</table>

Since strength is the major property in structural concrete, measured velocity was related to strength, and plots of velocity vs. strength were obtained in many references (Figure 1.3)
However, the test result is sensitive to surface properties, presence of steel reinforcement, presence of voids and cracks, properties of aggregates, and mix proportions.

As was mentioned earlier, The UPV method is a stress wave propagation method based on measuring the travel time, over a known path and length, of a pulse of ultrasonic compression wave (stress waves associated with normal stress). The pulses are introduced into the concrete by a piezoelectric transducer, and a similar transducer acts as receiver to monitor the surface vibration caused by the arrival of the pulse. A grease or gel is applied to the faces of the transducers to ensure good coupling with the surfaces. A timing circuit is used to measure the time it takes for the pulse to travel from the transmitter to the receiver. Figure 1.4 provides a schematic of the ultrasonic pulse velocity technique.
1.4.2.2 Ultrasonic Echo Method

The ultrasonic echo method is often used for defect detection in metal members. The method consists of generating a short impulse of the ultrasonic wave by the transmitting transducer (Figure 1.5).

![Figure 1.4 Schematic of the ultrasonic pulse velocity technique](image)

**Figure 1.4** Schematic of the ultrasonic pulse velocity technique

**1.4.2.2 Ultrasonic Echo Method**

The ultrasonic echo method is often used for defect detection in metal members. The method consists of generating a short impulse of the ultrasonic wave by the transmitting transducer (Figure 1.5).

![Figure 1.5 Testing the concrete by ultrasonic echo method](image)

**Figure 1.5** Testing the concrete by ultrasonic echo method:

a) single transmitting-receiving transducer, b) double transducers
After reflection by the material’s structural heterogeneity or by the limiting surface, the impulses are recorded by the receiving transducer (dual transmitting-receiving transducers are also available). Part of the ultrasonic wave is reflected by the material defect, returns to the receiving transducers and is recorded as the defect’s echo. Another part of the wave passes by the defect and reaches the opposite wall of the tested material, where it is reflected and returns to the receiver with some delay as the bottom echo. The depth of the defect or the reflecting surface is determined on the basis of the travel time of the impulse and the ultrasonic wave velocity. A small grain size of the tested material is necessary for the echo method to be efficient. The grain size should be significantly smaller than the searched-for defects; if not, then any defect echo will be overlapped by the echoes formed by the grain boundaries.

### 1.4.2.3 The resonance method

The resonance method consists of introducing an ultrasonic wave into the tested medium, which has a constant thickness $g$. A resonant standing wave, of wavelength $\lambda$, is formed under the condition:

$$g = n \cdot \frac{\lambda}{2} = \frac{nc}{2f}$$  \hspace{1cm} (1.5)

Where $n$ = an integer that defines the harmonic number.

Unlike the echo method, in the resonance method the interference of the incident and reflected waves is observed. A continuous wave is usually emitted in this method. The demonstration of the resonance of the continuous wave requires a large area of contact of the transducer with the tested material. The transducers usually have a
diameter of about 30 mm and should be well pressed against a smooth surface of the material. The main problem of this technique is finding the resonance frequency for \( n = 1 \). The other limitation is that the practical use of the method is really only possible in the laboratory, not in the field.

### 1.4.3 Application of ultrasonic methods for concrete testing

At present, two ultrasonic methods are used for concrete testing: the pulse velocity method and the echo method. These methods enable the evaluation of concrete strength and homogeneity. In a limited range, the ultrasonic pulse velocity method is also applied for determination of the elastic constants [25], detection of crack geometry [25, 28], and the evaluation of the degree of concrete degradation, e.g. deterioration due to freeze-thaw attack.

The structure of concrete, as defined by the maximum aggregate size, requires low frequency ultrasonic waves, since the wavelength should be larger than the grain size for minimizing losses caused by dispersion. Ultrasonic waves are diffracted by discontinuities smaller than the wave length. Assuming the pulse velocity in concrete, \( C_L \), is 3 km/s to 5 km/s, the wave length, \( \lambda \), ranges from 75 mm to 125 mm at a frequency of 40 kHz. The maximum diameter of aggregate grains, \( D \), does not usually exceed 32 mm. In this case, the ratio \( D/\lambda \) ranges from 0.25 to 0.41 [29]. As the frequency increases, the wave length decreases and it becomes close to the grain diameter. This implies that other types of wave dispersion can occur. Therefore, the resolution in the concrete testing is worse than in the case of metals. In practice [30], frequencies from 100 kHz to 1 MHz are used for testing concrete samples or members smaller than 0.5 m thick. Members larger than 0.5 m in thickness require low frequencies (below 100 kHz). Usually ultrasonic waves in the frequency range of 40-50 kHz are used for such structures.
The direct pulse velocity method is most often used to assess concrete structures. The indirect pulse velocity method is used rarely, only for some specific applications. Longitudinal ultrasonic waves are most commonly used in practice, because transverse waves are difficult to generate in concrete and are strongly attenuated in this material. The pulse velocity method is used mainly for evaluation of the compressive strength vs. time as well as for evaluation of structural homogeneity.

The common procedure for evaluation of cement concrete properties with the pulse velocity method (Figure 1.6) consists of regression analysis of the experimental relationship between the pulse velocity and selected mechanical properties (mainly compressive strength), leading to the development of suitable reference curves (also known as calibration curves, correlation curves or ISO-strength curves) [17,31]. There are many recommendations and national standards for the ultrasonic evaluation of concrete compressive strength. They define the type of reference concrete as well as the material parameters that can be varied to develop the reference curve. Komlos et al. [32] have analyzed the standards concerning the rules for ultrasonic testing by the pulse velocity method. In general, three methods of reference curve development can be recognized:

a) Calibration curve developed for cube concrete specimens with the same composition and cured in the same way as the concrete in the investigated structure (Figure 1.6a). The number of specimens needed to develop the curve depends on its universality (range of strength variability); as the universality of the reference curve increases, the number of samples necessary to develop it increases. To develop reference curves for a wide range of strength variability it is recommended to change the quantity
of mixing water, the degree of compaction, age of the concrete, the curing or storage conditions, and if necessary, the proportion of fine material and cement content,

b) Calibration curve experimentally established from samples taken from the structures from zones of different pulse velocity (Figure 1.6a); at least three individual transit time measurements should be carried out in each location and cores should be taken from the same location to obtain the compressive strength; the number of cores depends on the concrete volume.

c) Calibration curve established with inversion procedure (Figure 1.6b) using the reference curve for the concrete with similar composition and specimens taken from structures (number of specimens is lower than in case (b) and it depends on the concrete volume - at least three). This procedure is often used in practice for structures with unknown concrete composition or higher aged concrete and for structures where the possibility of coring is limited. To obtain a recalculated reference curve, the inverse coefficient should be determined. The compressive strength is calculated from the following equation:

\[ f_{ce}^{ef} = C_i^{exp} . f_{c}^{ref} \]  \hspace{1cm} (1.6)

where:

\( f_{ce}^{ef} = \) Effective compressive strength of tested concrete,

\( f_{c}^{ref} = \) Compressive strength determined from a reference curve on base of ultrasonic measurements,

\( C_i^{exp} = \) Total coefficient of influence obtained from tests on cores.
Most standards and guidelines recommend two regression equations for description of the relationship between pulse velocity and concrete strength:

- Linear: \( f_c = a_0 + a_1 \)  \hspace{1cm} (1.7)

- Exponential: \( f_c = a_0 \exp(a_1 c_p) \)  \hspace{1cm} (1.8)

where:

\( f_c \) = compressive strength,

\( c_p \) = longitudinal pulse velocity, \( a_0 \) and \( a_1 \) are regression coefficients.

However, others types are also allowed, i.e.:

\[ f_c = a_0 + a_1 c_p + a_2 c_p^2 \]  \hspace{1cm} (1.9)
\[ f_c = a_0 c_p^{a_1} \]  

(1.10)

where the symbols are the same as in Eqs. 1.7 and 1.8.

The measure of the accuracy of strength estimation from a reference curve is the coefficient of standard deviation, \( C_d \):

\[
C_d = \left( \frac{1}{n-1} \sum_{i=1}^{n} \left[ \frac{(f_{c,i}^{\text{ref}} - f_{c,i}^{\text{exp}})}{f_{c,i}^{\text{exp}}} \right]^2 \right)^{1/2} \times 100\%  
\]

(1.11)

where:

- \( f_{c,i}^{\text{ref}} \) = Strength determined from reference curve from ultrasonic measurements on specimen i,
- \( f_{c,i}^{\text{exp}} \) = Measured strength of specimen i,
- \( n \) = Number of specimens tested.

If \( C_d \) is less than 12\%, the estimation of compressive strength with the ultrasonic method is considered satisfactory.

Although the ultrasonic pulse velocity method is commonly used for compressive strength estimation, many authors have stressed that various factors can affect the pulse velocity and thus can overshadow the changes resulted from the strength variations [17, 33]. Komlos et al. [32] have also concluded that applications of longitudinal waves for concrete evaluation may be classified in the following way, according to decreasing precision of measurement:

- Monitoring of how concrete properties change with time,
- Control of homogeneity of the structure of concrete (possible disturbances of the signal from the reinforcement),

- Estimation of the compressive strength necessary for calibration,

- Determination of the elastic constants (doubtful as concrete is a heterogeneous composite),

- Detection of defects – least attractive of all (possibility of obtaining faulty results with dangerous and expensive consequences).

They have also stressed the necessity of enhancing ultrasonic measurement techniques in the following proposed directions:

- Using waves others than longitudinal, e.g. surface wave, plate waves,

- Using wave parameters other than wave propagation velocity,

- using advanced methods for the analysis of the ultrasonic signals.

Recently, many institutions have focused on the improvement of the ultrasonic test methods, e.g. using surface waves and advanced signal processing [34, 35]

The ultrasonic echo method is rarely used for concrete strength measurement, and mostly used for flaw detection. This method gives lower resolution in concrete compared to metal testing. Impulse duration is long at a frequency of 100 kHz. Double transducers cannot be applied here due to the small directivity of the ultrasonic beams emitted with low frequencies and the possibility of “cross-talk” between them, as well as the reflection of the wave by the surfaces parallel to the beam axis. In the case of concrete testing using
the echo method, the results are difficult to interpret due to the multiple echoes caused by material heterogeneities such as the presence of coarse aggregates and steel reinforcement. Recently, some work on improving the echo method has been carried out, focusing on using the ultrasonic waves at higher frequencies and a new data processing procedure, called split spectrum processing [36].

1.4.4 General Approach to Ultrasonic Evaluation of Polymer Concrete

From the engineering point of view, the nondestructive assessment of the properties of polymer concrete should be developed for three main types of field application:

- Repair,
- Protective coatings (including industrial floors),
- Precast elements.

Usually, different procedures for nondestructive evaluation should be used because of different purposes for evaluating polymer concrete materials or the systems in which they are used in.

1.4.4.1 NDT evaluation of polymer concrete elements

Due to the similarity of the geometrical features of the microstructure of both cement concrete and polymer concrete, it would seem possible that experience using ultrasonic techniques on cement concrete can be implemented into polymer concrete technology. The ultrasonic pulse velocity can be used for this purpose. This implies that reference curves should be determined for a given type of PC. However, the differences
in properties should be taken into account, especially differences in elastic properties between cement paste and resin binder, which can affect ultrasonic wave propagation in polymer concrete. Ultrasonic wave propagation depends generally on material composition and composite microstructure. In the specific case of polymer concrete composites, the ultrasonic wave propagation is influenced by: type of the binder and filler, content and grain-size distribution of the aggregate, and micro filler content and porosity. The adhesion between resin binder and aggregate is also important. For example, using a wet aggregate can result in a lack of adhesion, and by using coupling agents the adhesion can be increased [37, 38].

1.5 In situ stress measurement of concrete structures

Trustworthy information about the in-situ state of stress in the existing concrete structures can be critical for structural health monitoring. This monitoring may be made as part of a load rating determination, or it may be performed to find if repair or replacement of the structure is required.

Research on non-destructive or semi-destructive methods of determining in-situ stress in a variety of engineering materials has been ongoing for over 80 years. Mathar [39] was one of the first researchers to propose that stresses in an object could be determined by measuring displacements around a hole drilled in that object. His research was especially important because he recognized that the drilled hole must be small enough in both diameter and depth so that the object being tested could still perform its planned function. In the following sections, two general well-developed in-situ stress determination methods will be reviewed.
1.5.1 ASTM Hole Drilling Method

Since the invention of non-destructive hole-drilling techniques in the 1930s, a large number of researchers have contributed to improve the accuracy of this method. Because of the number of researchers involved and the time period over which they have worked, the amount of literature on hole-drilling methods is extensive. Most of the research done on hole-drilling techniques has been distilled into an ASTM standard. ASTM E837-01e1 2008 [40] provides a standardized method by which residual and surface stresses can be determined by drilling a small (1-2mm diameter) hole in the center of a specialized strain gauge rosette, shown in Figure 1.7.

![Figure 1.7 ASTM standard strain-gauge rosette](ASTM E837-01e1 2008)

The rosette measures radial strain relaxation around the hole. The measured strain can then be used to compute the stress state in the object.

The simplest hole-drilling case is when a hole that penetrates the object completely is drilled in an object that has the following properties:

1. Isotropic, homogeneous, linearly-elastic material;
2. The specimen is large in comparison to the hole size;  
3. Existing stresses do not exceed 50% of the object’s yield stress;  
4. No variation of stress with depth;  
5. No variation of stress around the area of the hole;  
6. Plane stress conditions exist.  

The ASTM method is reported to give excellent results when these conditions are satisfied [41, 42]. Conditions 1, 2, and 6 are particularly important, as the ASTM standard uses the theory of elasticity to solve the hole-drilling problem as a through-hole in an infinite thin plate with a uniform stress through depth shown in Figure 1.8.

\[ \varepsilon = (A^* + B^* \cos \alpha) \sigma_{max} + (A^* - B^* \cos(2\alpha)) \sigma_{min} \tag{1.12} \]

where

\[ A^* = -\frac{1-v}{2E} \left( \frac{a}{r} \right)^2 \tag{1.13} \]

\[ B^* = -\frac{1-v}{2E} \left[ \frac{4}{1+v} \left( \frac{a}{r} \right)^2 - 3 \left( \frac{a}{r} \right)^4 \right] \tag{1.14} \]
And \( a \) is the hole radius, \( r \) is the distance at which the strain is measured, \( E \) is the modulus of elasticity, \( \nu \) is Poisson’s ratio, and \( \alpha \) is the angle (measured positive clockwise) from the x-axis to the direction of maximum principal stress. The three unknowns \( (\sigma_{\text{max}}, \sigma_{\text{min}} \text{ and } \alpha) \) in previous equation can be solved by using the three measured strains from the rosette.

\[
\sigma_{\text{max}}, \sigma_{\text{min}} = \frac{\varepsilon_1 + \varepsilon_3}{4A^*} \pm \sqrt{\frac{(2\varepsilon_2 - \varepsilon_1 - \varepsilon_3)^2 + (\varepsilon_1 - \varepsilon_3)^2}{4B^*}}
\]

\[
\beta = \frac{1}{2} \tan^{-1} \left( \frac{2\varepsilon_2 - \varepsilon_1 - \varepsilon_3}{\varepsilon_1 - \varepsilon_3} \right)
\]

where \( \varepsilon_1, \varepsilon_2 \text{ and } \varepsilon_3 \) are the strains from the rosette and \( \beta \) is the angular measurement from gauge 1 of the rosette to the direction of maximum principal stress.

**1.5.2 The Incremental Core-drilling method (ICDM)**

The incremental core-drilling method (ICDM) is a technique to assess in-situ stresses that vary as a function of depth in concrete structures. In this method, a core is drilled into a concrete structure incrementally. The displacements which occur locally around the perimeter of the core at each increment are measured and related to the in-situ state of stress by elasticity theory and a matrix technique known as the influence function (IF) method. Because the ICDM has the ability to account for non-uniform stress distributions, it is potentially applicable to a wide variety of structural members and stress states. The ICDM is classified as a non-destructive method because it allows the structure to perform its intended function after the investigation is complete [43].
1.6 Scope of this study

As explained above, it was concluded that to find the final strength of the concrete structures by the ultrasonic pulse velocity (UPV) method, using the Portable Ultrasonic Nondestructive Digital Indicating Tester device (PUNDIT), generates and receives the wave with single frequency and the results can be unreliable. To improve the predicted results in this research the single frequency is substituted by the chirp signals containing multiple frequencies. In this method a linear chirp signal is transmitted through the material instead of a signal having only one frequency as PUNDIT does. Since the ingredients of concrete samples are different from one sample to another, by this method, we can generate many frequencies and then choose the appropriate time windowing of transient signals to interpret the results. The graph of strength versus UPV can be drawn for every time windowing.

Since the properties of polymer concrete are different from plain concrete, the graphs obtained for plain concrete cannot be used for finding the strength of polymer concrete. The proposed new method was applied for both plain concrete and polymer concrete.

The unique feature of this investigation is improving the strength prediction of plain and polymer concretes using UPV method. All attempts that have been made so far have neglected the effects of applied stress or load on the member while finding its strength. Attempts are made here to properly incorporate the effects of the applied load (stress) on the velocity value measurements in the structure.
Under cyclic loadings pre-existing cracks inside both conventional concrete and polymer concrete may grow and cause catastrophic failure of the structure. Structural failure under cyclic loading is also known as fatigue failure. This study investigated the behavior of plain and polymer concrete under cyclic loading or fatigue. Experimental results from conventional concrete and polymer concrete specimens were compared. Nonlinear behavior in polymer concrete and plain concrete were also compared.
Chapter 2

LITERATURE REVIEW

2.1 General

Condition assessment of building materials is critical when monitoring existing structures, since material aging can result in performance loss, degradation of safety, and higher maintenance costs. For these reasons, the use of nondestructive testing (NDT) has become more common to assess the condition of existing concrete structures.

Many investigations have aimed to improve NDT techniques and data processing. Some standards have been developed for individual techniques and reference texts have been produced on individual problems as mentioned in Chapter 1. Some authors such as Bungey and Millard [44] and Uemoto [45] have also tried to synthesize the capabilities of various techniques for a given problem. The general agreement is that the quality of assessment is limited by various uncertainties caused by the testing method, systematic interferences with the environment, random interference (due to intrinsic variability of materials), human factors, and data interpretation [46]. Thus, improved assessment can be achieved by reducing any of these sources of uncertainty. In the following sections, attempts are made to describe some of the studies that have been conducted to find the plain and polymer concrete strengths.

2.2 Evaluation of in-situ concrete compressive strength

Some of the first methods to evaluate the in-situ strength of concrete were adaptations of the Brinell hardness test for metals, which involved pushing a high-strength steel ball into the test piece with a given force and measuring the area of the
notch. In the metals test, the load was applied by a hydraulic loading system. Modifications were required to use this type of test on concrete structures. In 1934, Gaede in Germany reported the use of a spring-driven impactor to supply the force to drive a steel ball into the concrete [20]. A nonlinear, empirical relationship was obtained between cube compressive strength and indentation diameter. In 1936, Williams in England reported on a spring-loaded, pistol-shaped device in which a 4-mm ball was attached to a plunger [20]. The spring was compressed by turning a screw, a trigger released the compressed spring, and the plunger was propelled toward the concrete. The diameter of the indentation produced by the ball was measured with a magnifying glass and scale.

In 1938, a landmark paper by Skramtajev [47], of the Central Institute for Industrial Building Research in Moscow, summarized 14 different techniques for estimating the in-place strength of concrete. Skramtajev divided the test methods into two groups: 1) those that required installation of test hardware prior to placement of concrete, and (2) those that did not require pre-installation of hardware. Among all the methods that didn’t need pre-installation, two relatively simple methods were discussed in this chapter.

2.2.1 Literature Review on Schmidt hammer use:

As was discussed earlier, the Schmidt hammer is used to measure the strength of concrete using a rebound number. This number is very sensitive to the environment where the sample is tested. The conditions like the temperature, moisture, carbonation and also the type of aggregate in concrete can affect the results.
Amasaki [48] presented the effect of carbonation on the rebound number. Grieb [49] showed the effect of the type of aggregate on the rebound number and hence the estimated strength. Willetts [50] presented the effect of moisture content of concrete on the results of the rebound hammer test by comparing dry and wet tested samples. Based on the Neville’s research, it was concluded that it was necessary to take 10 to 12 readings over the area to be tested because the test was sensitive to the presence of aggregate and voids immediately underneath the plunger [51]. It is clear then that the rebound number reveals the condition of only the surface of concrete. BS 1881: Part 202 [19] suggests that the measured number is an indication of about the first 30-mm depth of concrete. According to Teodoru [52], the results obtained are only representative of the outer concrete layer with a thickness of 30±50 mm. Due to the difficulty of acquiring the appropriate correlation data in a given instant, the rebound hammer is most useful for rapidly surveying large areas of similar types of concrete in a structure.

Neville [53] presented the benefits of using the rebound hammer in concrete and stated that this test alone was not a very reliable strength test and the exaggerated claims of its use as a replacement for compression test should not be accepted.

In 2009, Liu et al. [54] tried to increase the accuracy of calculating the strength, using surface hardness rebound values, material design parameters and regression analysis. The strength of the concrete specimens was 130-480 kgf/cm², and their ages were 7-38 days. In total, 166 standard specimens of concrete were grouped into 146 training examples and 20 test examples to estimate concrete compressive strength. Regression analysis was performed to establish a mathematical formula. Study results
indicated that the correlation coefficient may reach 0.9622, indicating that the proposed
method had a good accuracy.

2.2.2 Ultrasonic Pulse Velocity:

The most widely used feature in elastic wave NDT is pulse velocity, which is
correlated with the degree of damage. Despite its sometimes inconsistent results, pulse
velocity monitoring of large concrete structures is of great significance since it offers a
general estimate of the condition and enables proper repair action. Empirical correlations
between pulse velocity and damage or strength have been exploited for a long time.

The development of field instruments to measure the pulse velocity occurred
nearly simultaneously in Canada and in England by Whitehurst in 1967 [55]. These
developments were outgrowths of earlier successful work by the U.S. Army Corps of
Engineers to measure the speed of a mechanical stress pulse through concrete by Long et
al. in 1945 [56]. The Army Corps of Engineers’ approach involved attaching two
receivers to the concrete surface. A horizontal hammer blow was applied in line with the
receivers, and a specially designed electronic interval timer measured the time for the
pulse to travel from the first to the second receiver. The major purpose of this technique
was to calculate the in-situ modulus of elasticity.

In 1946 and 1947, engineers at the Hydro-Electric Power Commission of Ontario
(Ontario Hydro) worked on the development of a device to investigate the extent of
cracking in dams [57]. The device they developed for this purpose was called the
Soniscope. It had a 20-kHz transmitting transducer, was capable of penetrating up to 15
m of concrete, and could measure the travel time with an accuracy of 3%. The stated
purposes of the Soniscope were to identify the presence of internal cracking, determine the depth of surface cracks openings, and determine the dynamic modulus of concrete. It was further stated that the fundamental measurement was the travel time. The amplitude of the received signal was said to be of secondary importance because the transfer of energy between the transducers and the concrete could not be controlled. It was also emphasized that the interpretation of the results required knowledge of the history of the structure being investigated.

Early uses of the Soniscope on mass concrete emphasized measuring the pulse velocity rather than estimating the strength or calculating the modulus of elasticity. Based on velocity readings on a grid marked on the surface, the presence of distressed concrete could be easily detected. In 1953, Parker reported Ontario Hydro’s early attempts to develop relationships between pulse velocity and compressive strength. They investigated 46 mixtures involving the same aggregate, different cement types, and different admixtures. The results indicated no significant differences in the velocity–strength relationships for different mixtures; therefore, the results were treated as one group, and the best-fit relationship was determined. Figure 2.1 shows the relationship between the estimated strength and the pulse velocity.

Figure 2.1 also shows that the change in pulse velocity per unit change in strength decreases with increasing strength. This means that pulse velocity is relatively insensitive to strength for matured concrete.
While work on the Soniscope was in progress in Canada, Jones and coworkers at the Road Research Laboratory (RRL) in England were involved in independent research to develop ultrasonic testing equipment [59]. The RRL researchers were interested in testing the quality of concrete pavements, which involved shorter path lengths compared with the work at Ontario Hydro. As a result, the device that was developed operated at a higher frequency than the Soniscope, and it was called the ultrasonic concrete tester. Transducers with resonant frequencies from 60 to 200 kHz were used, depending on the desired penetration [60]. In addition to using a different operating frequency, the RRL device used a different approach than the Soniscope to measure travel time. This was necessary because of the shorter path lengths in the RRL work. It was reported that the ultrasonic concrete tester could measure travel times to within ±2 μs. Jones reviewed the research carried out with the newly developed ultrasonic concrete tester [61].

Among his findings were the following:
• An investigation into the variations in pulse velocity with height in standard cube specimens and with depth in slabs was one of the first to document the “top-to-bottom” effect that is often mentioned as a problem when planning and interpreting in-place tests (ACI Committee 228, 2003).

• An investigation into the influence of water–cement ratio, aggregate type, and aggregate content on pulse velocity demonstrated the importance of aggregate type and aggregate content on pulse velocity.

• An investigation of the relationships between pulse velocity and compressive strength demonstrated that, for a given mixture under uniform conditions, there was good correlation between strength and pulse velocity.

Thus, Jones revealed the problems inherent in using pulse velocity to estimate concrete strength. Despite these early findings, numerous researchers sought to establish correlations between pulse velocity and strength, and many reached the same conclusions as Jones’ [62].

In the United States, a Soniscope was developed in 1947 at the Portland Cement Association in cooperation with Ontario Hydro, and field applications were reported by Whitehurst in 1951. In his summary of the industry’s experience in the United States, Whitehurst published a tentative classification table for using pulse velocity as an indicator of quality. This table was quoted by others in many subsequent publications. Whitehurst warned, however, that these values were established on the basis of tests of normal concrete having a density of about 2400 kg/m³ and that the boundaries between the conditions could not be sharply drawn. He mentioned that, rather than using these
limits, a better approach would be to compare velocities with the velocity in a portion of the structure that is known to be of acceptable quality. Nevertheless, inexperienced investigators often used this table (table 1.2) as the sole basis for interpreting test results.

After the publication of these landmark papers in the late 1940s and early 1950s, a flurry of activities occurred worldwide, and efforts were made to develop standard test methods. In the United States, a proposed ASTM test method was published in 1955 by Leslie [63], but it was not until 1967 that it was adopted as a tentative test method (ASTM, 2002a). In Europe, the International Union of Testing and Research Laboratories for Materials and Structures (RILEM) organized a working group to study nondestructive testing. In 1969, draft recommendations for testing concrete by the ultrasonic pulse velocity method were published [64]. In Eastern Europe, the method was used extensively as a quality control tool in precast concrete plants.

During the 1960s and 1970s, considerable attention was devoted to gaining more knowledge about the effects of different factors on pulse velocity. Researchers continued to explore the relationship between compressive strength and pulse velocity. They appear, however, to have reached an agreement that there is no unique relationship. Numerous studies showed that the type and quantity of aggregate have major effects on pulse velocity but not on strength. Significant effort was also devoted to examining whether or not attenuation measurements could provide additional information about concrete strength. These results were, in general, found to be impractical in field situations because of difficulties in achieving consistent coupling of the transducers, which is critical for measuring attenuation.
Perhaps the most significant advances during the 1960s and 1970s were in the development of improved field instrumentation. Due to advances in microelectronic circuitry, the cumbersome instruments developed in the 1940s and 1950s gave way to compact portable devices. In the late 1960s, TNO (Netherlands Organization for Applied Scientific Research) in Delft, Netherlands, developed a portable, battery-operated pulse velocity device that incorporated a digital display of the travel time. In the earlier devices, travel time was measured by examination of oscilloscope displays, which was a time-consuming process. The portable instrument had a resolution of 1 μs, which resulted in low accuracy for short path lengths, and it had limited penetrating ability. At about the same time, R.H. Elvery of University College, London, developed a similar portable device named the *PUNDIT* (Portable Ultrasonic Nondestructive Digital Indicating Tester). It weighed 3.2 kg, had a resolution of 0.5 μs, and could be powered by rechargeable batteries. These and other relatively low-cost, portable devices simplified testing and resulted in a worldwide increase in the number of consultants and researchers who could perform this type of testing. Later models of these devices had resolutions of 0.1 μs, and some provided an optional output channel to allow the received signal to be displayed on an oscilloscope.

After invention of PUNDIT device, many investigations were conducted to find the relationship between concrete strength and the ultrasonic pulse velocity (UPV).

In 2011, Lawson et al. [65] investigated the relationship between Ultrasonic Pulse Velocity (UPV) and the compressive strength of concrete. The specimens used in the studies were made of concrete with a paste content of 18% and the constituents of the specimens varied in different water-cement ratios (w/c). The UPV measurement and
compressive strength tests were carried out at the concrete ages of 2, 7, 15 and 28 days (Figure 2.2). Materials used for making specimen included ordinary Portland Limestone type 2.5N, standard sand, Coarse Aggregate (CA) of 10 mm grain size and water. The water-cement ratio (w/c) used in this study ranged from 0.35 to 0.75. The cement paste occupied approximately 18% of the total concrete volume. Five concrete specimens were produced for each mixture proportion. All the specimens were cast in steel molds and kept in their molds for approximately 24 hrs. in the laboratory. After removing the molds, three concrete cubes were tested at the age of 2 days and all other concrete cubes were cured in water at 20°C (68°F) and tested at ages of 7, 15 and 28 days, respectively. At each age, the pulse velocity and compressive strength of the five specimens were measured according to the specifications of the British standard. Five simulation curves of the relationship between UPV and compressive strength of the hardened concrete were proposed for concrete with w/c of 0.35, 0.4, 0.5, 0.6 and 0.75 (Figure 2.3). These curves were verified to be suitable for prediction of hardened concrete strength with a measured UPV value.

Results indicated that concrete with high w/c (w/c = 0.75) at the age of 2 days had a UPV of about 89% of that at 28 days, but the strength was only about 60%. At the age of 2 days, concrete with low w/c (w/c = 0.35) had a UPV that was approximately 97% of that at 28 days and the strength was about 30%. To sum up, the UPV and strength growth rates of high and low w/c concrete are significantly different at an early age. As a result, the relationship between UPV and strength of concrete becomes unclear when age and mixture proportion are taken into consideration simultaneously. This observation
suggested that it was better to separately consider the effects of age and mixture proportions on UPV and strength relationship.

![Figure 2.2 UPV value vs age for different w/c][Lawson 2011]

Typical relationship curves between UPV and compressive strength are shown below for concrete with a given w/c ratio.
The equations for the curves shown above are given below, respectively:

\[
\sigma(0.35) = 1E - 15e^{0.008v} \\
\sigma(0.40) = 0.022e^{0.001v}
\]
\[ \sigma(0.50) = 0.053e^{0.001\nu} \]
\[ \sigma(0.60) = 0.097e^{0.001\nu} \]
\[ \sigma(0.70) = 0.205e^{0.001\nu} \]

where \( \sigma \) and \( \nu \) represent the compressive strength (MPa) and the ultrasonic pulse velocity (m/s), respectively.

For the five w/c ratios (0.35, 0.40, 0.50, 0.60 and 0.75) the correlation between the UPV and the compressive strength of concrete is pretty good for this particular mix proportion with a very high coefficient of determination, R-squared of 0.866, 0.981, 0.888, 0.994 and 0.989, respectively.

2.2.3 Combined UPV and rebound number methods

When variation in properties of concrete affect the test results, (especially in opposite directions), the use of one method alone would not be sufficient to study and evaluate the required property. Therefore, the use of more than one method yields more reliable results. For example, the increase in moisture content of concrete increases the ultrasonic pulse velocity but decreases the rebound number [53]. Hence, using both methods together will reduce the errors produced by using one method alone.

In 2000, Qasrawi tried to investigate the concrete strength by combined methods of nondestructive testing [66]. Both the traditional well-known rebound number and the ultrasonic pulse velocity tests were used in his study.

From this study it was concluded that:
1. The use of rebound hammer alone is not suitable to estimate or predict the strength of concrete. High variations are obtained, which makes engineering judgment quite difficult.

2. When compared to the rebound number, the ultrasonic pulse velocity method seems to be more efficient in predicting the strength of concrete. However, the use of such method alone would not give reliable prediction of the strength of concrete.

3. The use of combined methods produces results that lie close to the true values when compared with other methods.

4. The lower strengths of concrete showed higher prediction intervals and hence, less predictable strength by the combined methods.

5. The use of the combined methods yields more reliable and closer results to the actual strength. The 95% prediction interval is quite narrow, which enhances engineering judgment.

6. Better prediction results of strength are obtained for crushing cube strengths exceeding 20 MPa.

7. The method can be extended to test existing structures by taking direct measurements on concrete elements.

Attempts have been made to relate rebound number and ultrasonic pulse velocity to concrete strength [67, 68, 69]. An equation for predicting concrete strength by means of combined methods is shown in Ref. [67]. Unfortunately, the equation requires previous knowledge of concrete constituents in order to obtain reliable and predictable
results. Some of the empirical equations used for the estimation of compressive strength of concrete are summarized in Table 2.1.

### Table 2.1 Empirical equations used for compressive strength estimation (Erdal 2009)

<table>
<thead>
<tr>
<th>Eq. No.</th>
<th>Equations</th>
<th>Explanations</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Single-variable equations</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>$f_c = 1.2 \times 10^{-3} \times V^{1.74e7}$</td>
<td>$f_c$ [MPa], $V$ [km/s]</td>
<td>Kheder 1 (1998)</td>
</tr>
<tr>
<td>2</td>
<td>$f_c = 0.4030 \times R^{2083}$</td>
<td>$f_c$ [MPa]</td>
<td>Kheder 2 (1998)</td>
</tr>
<tr>
<td>3</td>
<td>$f_c = 36.72 \times V - 129.077$</td>
<td>$f_c$ [MPa], $V$ [km/s]</td>
<td>Qasrawi 1 (2000)</td>
</tr>
<tr>
<td>4</td>
<td>$f_c = 1.353 \times R - 17.393$</td>
<td>$f_c$ [MPa]</td>
<td>Qasrawi 2 (2000)</td>
</tr>
<tr>
<td><strong>Multi-variable equations</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>$f_c = -25.568 + 0.000635 \times R^3 + 8.397V$</td>
<td>$f_c$ [MPa], $V$ [km/s]</td>
<td>Bellander (1979)</td>
</tr>
<tr>
<td>6</td>
<td>$f_c = -24.668 + 1.427 \times R + 0.0294V^4$</td>
<td>$f_c$ [MPa], $V$ [km/s]</td>
<td>Meynink at al. (1979)</td>
</tr>
<tr>
<td>7</td>
<td>$f_c = 0.743 \times R + 0.951 \times V - 0.544$</td>
<td>$f_c$ [MPa], $V$ [km/s]</td>
<td>Tanigawa et al. (1984)</td>
</tr>
<tr>
<td>8</td>
<td>$f_c = [R/(18.6 + 0.019 \times R + 0.515 \times V)]$</td>
<td>$f_c$ [kg/cm²], $V$ [km/s]</td>
<td>Postacioglu (1985)</td>
</tr>
<tr>
<td>9</td>
<td>$f_c = 18.6 \times e^{(0.019/R - 0.515 V)}$</td>
<td>$f_c$ [kg/cm²], $V$ [km/s]</td>
<td>Arioglu et al. (1991)</td>
</tr>
<tr>
<td>10</td>
<td>$f_c = 10^{3.140 [\log R^3 \times V^3] - 5.890}$</td>
<td>$f_c$ [kg/cm²], $V$ [km/s]</td>
<td>Arioglu et al. (1994)</td>
</tr>
<tr>
<td>11</td>
<td>$f_c = -39.570 + 1.532 \times R + 5.0614 \times V$</td>
<td>$f_c$ [MPa], $V$ [km/s]</td>
<td>Ramyar et al. (1996)</td>
</tr>
<tr>
<td>12</td>
<td>$f_c = 0.00153 \times (R^3 \times V)^{0.611}$</td>
<td>$f_c$ [MPa], $V$ [km/s]</td>
<td>Arioglu et al. (1996)</td>
</tr>
<tr>
<td>13</td>
<td>$f_c = 0.0158 \times V^{0.4254} \times R^{-1.171}$</td>
<td>$f_c$ [MPa], $V$ [km/s]</td>
<td>Kheder 3 (1996)</td>
</tr>
</tbody>
</table>

Where $f_c$ = Compressive strength; $V$ = ultrasonic pulse velocity; $R$ = rebound number.

In summary, the ultrasonic pulse velocity test method is a relatively simple test to perform on-site testing when it is possible to gain access to both sides of the member. Although UPV tests can be performed with the transducers placed on the same surface, the results are not easy to interpret, and this method of measurement is not recommended. Care must be exercised to ensure that good and consistent coupling with the concrete surfaces is achieved. Other important factors, besides concrete strength, that can affect the measured ultrasonic pulse velocity, and that should be considered are discussed in the report of ACI Committee 228 in 2003 [12]. These factors include:

- Moisture content: An increase in moisture content increases the pulse velocity.
• Presence of reinforcement oriented parallel to the pulse propagation direction: The pulse may propagate through the bars and result in an apparent pulse velocity that is higher than that propagating through concrete.

• Presence of cracks and voids: Cracks and voids can increase the length of the travel path and result in a longer travel time.

Because of these factors, only experienced individuals should use the ultrasonic pulse velocity method for estimating concrete strength. Like the rebound number test, the pulse velocity method is useful for assessing the uniformity of concrete in a structure. It is often used to locate regions in a structure where other tests should be performed or where cores should be drilled.

Most of the efforts that have been performed so far relate to finding the velocity-strength relationship (table 2.1). The effect of stress on the structure is neglected in these studies. Researchers in this field have all tested their samples under no-stress condition. In the present investigation, attempts were made to use new methods of finding the strength of plain and polymer concrete and also finding the effect of applied load (stress) on the UPV value.
CHAPTER 3
DISCRIPTION OF MATERILAS

3.1 General

In this chapter the materials that have been used for this research are described. These materials are:

- Aggregates
  - Aggregates used to make plain concrete
  - Aggregates used to make polymer concrete
- Cement
- Water
- Epoxy resin

3.2 Aggregates

3.2.1 Plain Concrete Aggregate

In this research, to make plain concrete two types of aggregate were used: Fine aggregate and coarse aggregate. Natural gravel and sand that were dug or dredged from pits or rivers were washed and used in the study.

The coarse aggregate was $\frac{3}{4}$ inch maximum in size with an oven dry specific gravity of 2.57, and absorption of 1.9% (moisture content at SSD condition). Fine aggregate was natural sand with oven dry specific gravity of 2.53 and absorption of 0.9%. Figure 3.1 shows the aggregate used for plain concrete.
3.2.2 Polymer Concrete Aggregate

The importance of using the right type and quality of aggregates cannot be ignored as they generally occupy about 80% of the polymer concrete volume and strongly influence the fresh and hardened concrete behavior.

In this research two different combinations of aggregates were used to make polymer concrete. The first combination which was used to make samples for fourth objective of this research; that is, the evaluation of the effects of the existing stress on the compressive strength of concrete contained:

- Gravel size between 2.36 mm and 6.3 mm
- Gravel size between 1.18 mm and 2.36 mm
- Sand

The second combination which was used to make samples for fifth objective, that is, the nonlinear evaluation of polymer concrete contained:
- Gravel size of 2.36-4.75 mm (0.093-0.19 in)
- Gravel size of 1.18-2.36 mm (0.046-0.093 in)
- mortar sand

The three different types of aggregates for polymer concrete considered in this study are shown in Figure 3.2.

![Figure 3.2 Different types of aggregates for polymer concrete under this study](image)

3.3 Epoxy Resin

In chemistry, epoxy or polyepoxide is a thermosetting epoxide polymer that cures (polymerizes and crosslinks) when mixed with a catalyzing agent or hardener. Most common epoxy resins are produced from a reaction between epichlorohydrin and bisphenol-A. The first commercial attempts to prepare resins from epichlorohydrin were made in 1927 in the United States. Credit for the first synthesis of bisphenol-A-based epoxy resins is shared by Dr. Pierre Castan of Switzerland and Dr. S.O. Greenlee of the United States in 1936. Dr. Castan's work was licensed by Ciba, Ltd. of Switzerland, which went on to become one of the three major epoxy resin producers worldwide. Ciba's
Epoxy business was spun off and later sold in the late 1990s and is now the advanced materials business unit of the Huntsman Corporation of the United States. Dr. Greenlee's work was for the firm of Devoe-Reynolds of the United States. Devoe-Reynolds, which was active in the early days of the epoxy resin industry, was sold to Shell Chemical (now Hexion, formerly Resolution Polymers and others).

Epoxy resin usually combines with carbon and glass reinforcement and produces fiber composite laminates with exceptional strength, durability and chemical resistance. Epoxy resin is a 100% solid formation with low toxicity and low odor during cure. The cure time depends on temperature and usually takes 72 hours; therefore, the curing time is nine times faster than concrete. The density of the epoxy resin is approximate 1.1kg/L. In practice, a hardener is referred to as part B and the resin is part A. In this study, a proportion of two parts component “A” to one part component “B” by volume was mixed, and the properties the cured epoxy resin are shown in Table 3.1. The properties for Parts A and B are shown in Figure3.3.
Table 3.1 Properties of epoxy resin

<table>
<thead>
<tr>
<th>EPOXY PROPERTIES</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Color</td>
<td>Part A is pigmented syrup</td>
</tr>
<tr>
<td></td>
<td>Part B is amber liquid</td>
</tr>
<tr>
<td>Viscosity at 25°C</td>
<td>1,400–1,700 cps</td>
</tr>
<tr>
<td>Pot Life at 25°C</td>
<td>3–4 hours (thin film set time)</td>
</tr>
<tr>
<td>Full cure time</td>
<td>3 days</td>
</tr>
<tr>
<td>Density at 20°C</td>
<td>Part A: 9.4 lbs/gal (1.13 kg/L)</td>
</tr>
<tr>
<td></td>
<td>Part B: 8.3 lbs/gal (1.00 kg/L)</td>
</tr>
<tr>
<td>Tensile Strength (ASTM D-638)</td>
<td>8,915 psi (61.5 MPa)</td>
</tr>
<tr>
<td>Tensile Modulus (ASTM D-638)</td>
<td>376,228 psi (2,608 MPa)</td>
</tr>
<tr>
<td>Compressive Strength (ASTM D-695)</td>
<td>14,011 psi (96.6 MPa)</td>
</tr>
<tr>
<td>Compressive Modulus (ASTM D-695)</td>
<td>618,706 psi (4,266 MPa)</td>
</tr>
<tr>
<td>Flexural Strength (ASTM D-790)</td>
<td>17,583 psi (121.2 MPa)</td>
</tr>
<tr>
<td>Flexural Modulus (ASTM D-790)</td>
<td>415,234 psi (2,863 MPa)</td>
</tr>
<tr>
<td>Shear Strength (ASTM D-3165)</td>
<td>Unable to force a shear failure mode; samples fail in tension.</td>
</tr>
<tr>
<td>Water absorption (% gain) in 24 hours</td>
<td>&lt; 1%</td>
</tr>
<tr>
<td>Expansion Coefficient [-37.4° to 40.1°C]</td>
<td>78 × 10⁻⁶ m/m°C</td>
</tr>
<tr>
<td>Expansion Coefficient [120° to 222°C]</td>
<td>151.8 × 10⁻⁶ m/m°C</td>
</tr>
</tbody>
</table>

3.4 Cement

In this research, normal cement (Portland Cement Type II) was used. The specific gravity of the Portland cement was 3.1. The necessary amount of the cement for the entire study was ordered and kept in a dry place to prevent deterioration. All batches of concrete were made within a week to eliminate any possible change in cement property.

3.5 Water

Water containing less than 2000 ppm of total dissolved solids can generally be used for making concrete. Normal Tucson tap water was used in the concrete mixture.
CHAPTER 4

SPECIMEN PREPARATION AND TEST SETUP

4.1 General

This chapter covers specimen preparation and laboratory test setup. Over fifty plain and fifty polymer concrete samples were made to investigate the non-destructive measurements of plain and polymer concrete properties such as compressive strength, and the effects of the applied load on the compressive strength.

4.2 Plain Concrete Samples

In this research, plain concrete samples were made with different compressive strengths according to the Lab Manual at the University of Arizona [70]. Table 4.1 shows the amount of each material used to make plain concrete specimens of different groups. Values are given in pound per cubic foot of concrete.

<table>
<thead>
<tr>
<th>Group#</th>
<th>CA</th>
<th>FA</th>
<th>C</th>
<th>W</th>
</tr>
</thead>
<tbody>
<tr>
<td>Group 1</td>
<td>64.8</td>
<td>51.4</td>
<td>21.2</td>
<td>12.1</td>
</tr>
<tr>
<td>Group 2</td>
<td>64.8</td>
<td>47.4</td>
<td>25.1</td>
<td>12.1</td>
</tr>
<tr>
<td>Group 3</td>
<td>64.8</td>
<td>43.1</td>
<td>29.4</td>
<td>12.1</td>
</tr>
<tr>
<td>Group 4</td>
<td>64.8</td>
<td>36</td>
<td>36.5</td>
<td>12.1</td>
</tr>
</tbody>
</table>

After mixing plain concrete materials together, the fresh concrete was placed into the mold with a size of 3×6 in (7.62×15.24cm) and left at room temperature for one day. After one day, the samples were removed from the mold by air pressure and left in a moisture room for 28 days to reach the maximum compressive strength. After 28 days,
the samples were cut at the top and bottom of the specimen to create a smooth surface. Figure 4.1 shows plain concrete samples used in this study.

It should be noted that the plain concrete specimens were made based on the mixture design given in Table 4.1. These samples were used to carry out the tests for objectives 1, 3 and 5 which will be explained in detail in Chapter 5.

4.3 Polymer Concrete

The polymer concrete samples were made with two different mix designs:

- First: to carry out the tests for objectives 2 and 4.
- Second: to carry out the tests for objective 5.
4.3.1 Preparation of polymer concrete for objectives 2 and 4 of this study

The samples that were prepared by the mix design explained in this section were used in objectives 2 and 4 of this study which will be explained in details in Chapter 5. These samples were used to find the relationship between change in strength of polymer concrete and change in the ultrasonic pulse velocity. These specimens were also used to find the effects of applied stress on the polymer concrete compressive strength.

Different combinations of aggregates and resin were used to make the polymer concrete specimens with different compressive strengths ($f'_{pc}$). Aggregate and epoxy resin were mixed with a volume ratio of 4:1. The aggregate mix was obtained from Table 4.2. As shown in Table 4.2, five different aggregate combinations were made to have different compressive strengths ($f'_{pc}$). In Table 4.2, “A” represents the total aggregate weight.

<table>
<thead>
<tr>
<th>Group #</th>
<th>Fine aggregates (FA)</th>
<th>Sand (S)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2.36-6.3 mm</td>
<td>1.18-2.36 mm</td>
</tr>
<tr>
<td>Group 1</td>
<td>60%A</td>
<td>0%A</td>
</tr>
<tr>
<td>Group 2</td>
<td>30%A</td>
<td>30%A</td>
</tr>
<tr>
<td>Group 3</td>
<td>42%A</td>
<td>18%A</td>
</tr>
<tr>
<td>Group 4</td>
<td>40%A</td>
<td>0%A</td>
</tr>
<tr>
<td>Group 5</td>
<td>20%A</td>
<td>20%A</td>
</tr>
</tbody>
</table>

Part A and part B of epoxy resin were mixed with an electrical drill for approximately three minutes. The weight of aggregate and epoxy resin were then measured with a scale and then combined. Then the epoxy resin and the aggregates were mixed again for 5 minutes. The fresh epoxy resin and aggregate (polymer concrete) was
then placed into 7.62x15.24 cm (3x6 in) cylindrical plastic molds. The inside of these molds were coated with oil to ease sample extraction. The mold was filled in thirds with each layer being rodded twenty five times. The side of the mold was tapped in order to prevent large surface voids. The top surface of the specimen was smoothed and marked for identification. The samples were then allowed to cure at room temperature for five days. After the five days of curing, a small hole was drilled at the bottom of the mold to apply air pressure in order to remove the samples from the molds. A compressed air hose was inserted into the holes and the samples were extracted. Figure 4.2 shows the polymer concrete used in this study. For uniform distribution of the load on the samples surfaces, the bottom and top of these samples were saw cut.

![Image of polymer concrete samples](image.png)

**Figure 4.2 Polymer concrete samples used in this study**

### 4.3.2 Preparation of polymer concrete for objective 5 of this study

To have the final mix design for making the samples in this section, attempts were made to find the maximum dry bulk density of aggregates based on previous research at the University of Arizona [71]. Three different aggregate sizes were used to make the
samples. These three aggregate types were Gravel8 (G8), Gravel16 (G16) and sand. G8 and G16 were obtained from a sieve analysis indicated in (Figure 4.3). The fine aggregates used were obtained from storage bins and measured accordingly. The sieves (#4 #8 #16) were cleaned with a brush and were inspected for excessive wear and damage before being used. The specified sieves were stacked in order (Top-down) from largest to smallest. The sample was placed in the top sieve and a cover plate was attached to the top. The sieves were locked in place by screwing down the cover attached to the mechanical sieve shaker. The mechanical sieve shaker was turned on for approximately five minutes. Each aggregate size from the sieve was taken out from the mechanical shaker and stockpiled in buckets. The part of the sample remaining on sieve #8 was deemed as G8 and likewise the sample which remained on sieve #16 was G16. The sand for this investigation was obtained locally.

Figure 4.3 Sieve test machine
Three types of aggregates were mixed together as shown below according to ASTM C29 [72]:

1. G8 and G16
2. G8 and Sand
3. G16 and Sand
4. G8, G16 and Sand

According to Figure 4.4, among all these combinations, the mix design of 40% of sand and 60% of G8, had the highest dry density compared to other mix designs of G8, G16 and sand. Therefore in this section, this mix design (40% sand and 60% G8) was used for making polymer concrete.

![Figure 4.4 Dry density vs. percent of fine aggregate for mix design [Toufigh 2009]](image)

Aggregate and epoxy resin were mixed together in a volume ratio of 4:1, respectively. The samples were prepared in the same manner as that described in section 4.3.1. The fresh epoxy resin and aggregate (polymer concrete) were placed in plastic
molds in the same fashion as that in section 4.3.1. The samples stayed at room temperature for five days to cure. After removing the samples from the molds with high air pressure, the top and bottom of the samples that were to undergo compression testing were saw cut to create a smooth surface.
CHAPTER 5
DESIGN OF EXPERIMENTS AND ANALYSIS OF RESULTS

5.1 General

The specimens were tested with the following five objectives in mind.

Objective 1: Find a new method to measure the compressive strength of plain concrete

Objective 2: Find a new method to measure the compressive strength of polymer concrete

Objective 3: Find the effects of applied load (stress) on UPV measurements of compressive strength of plain concrete.

Objective 4: Find the effects of applied load (stress) on UPV measurements of compressive strength of polymer concrete.

Objective 5: Compare the nonlinear behavior of plain and polymer concretes under cyclic loading.

Tests were designed and conducted to achieve the above goals.
5.2 Objective 1: A New method to measure the compressive strength of plain concrete using ultrasonic waves

5.2.1 General concepts

Measuring the strength of the concrete by nondestructive testing (NDT) method is an interesting topic for researchers. Among all NDT methods, ultrasonic pulse velocity is the most popular and probably the simplest method. The device which is developed for this purpose is called “PUNDIT” and is shown in Figure 5.1. This instrument has a time resolution of 0.1 μs. Many studies were developed to find the relationship between the concrete compressive strength and its UPV (ultrasonic pulse velocity).

![Figure 5.1 PUNDIT instrument](image)

5.2.2 Experimental investigation and results

In this investigation, attempts were made to develop a new method for finding concrete compressive strength using ultrasonic wave speed. Instead of using PUNDIT, an alternative method was used to generate the ultrasonic waves of different frequencies simultaneously. The ultrasonic wave was generated inside the concrete by the
piezoelectric transducers (made of Lead-Zirconate-Titanate ceramic or commonly called PZT).

The main difference of this method in comparison to the earlier methods is that in this method a linear chirp signal with different frequencies is transmitted through the material unlike the single frequency signal as PUNDIT does. Since the ingredients of concrete samples vary from one sample to another, exciting them with signals having multiple frequencies have the advantage of choosing the right time window of transient signals and right frequency for easier interpretation of the results. Another advantage of this method is that the results derived from the PUNDIT device is obtained from the first arrival time only, while for the new method any time window inside the received transient signal can be chosen. Arrival times are obtained for the chosen time window.

To achieve our goal under this objective, the following procedures were undertaken:

- Cylindrical concrete specimens of 3 inch diameter and 6 inch in length were fabricated with different strengths, $f'_{c}$. Four groups of such specimens with four different strengths were fabricated. Five concrete specimens were produced for each group.

- The appropriate time window in the transient signal was chosen.

- For the specified time window, one sample was chosen as the reference sample. All settings were based on the reference sample. Changes in the ultrasonic wave velocity in other samples compared to this velocity in the reference sample were measured. This process was repeated for all twenty samples. Every time one
sample was chosen as the reference sample and the other sample properties were compared with this reference sample.

- After conducting the nondestructive test, the strength values for all samples were measured by standard destructive compressive tests.

- The relationship between the variation in strength [Δ(strength)] and the variation in the wave velocity [Δ(velocity)] was obtained and shown graphically.

The test setup and procedures are explained in details in the following:

Five concrete specimens produced for each group should have close values for the compressive strength $f'_{c}$ that was verified by the standard destructive compressive tests. Twenty specimens were tested to find the relationship between the compressive strength and the value of ΔV. ΔV is the change in velocity due to the change in strength.

To make plain concrete specimens, two types of aggregates and regular Portland cement were used with the properties mentioned in Chapter 4. The samples were divided into four different groups with 4 different values of $f'_{c}$. The measured compressive strength values for each group are tabulated below (Table 5.1):

<table>
<thead>
<tr>
<th>Sample # of Group</th>
<th>Sample1 strength (psi)</th>
<th>Sample2 strength (psi)</th>
<th>Sample3 strength (psi)</th>
<th>Sample4 strength (psi)</th>
<th>Sample5 strength (psi)</th>
<th>Average strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Group 1</td>
<td>3855.1</td>
<td>3749.3</td>
<td>3788.2</td>
<td>3606.7</td>
<td>3554.4</td>
<td>3711</td>
</tr>
<tr>
<td>Group 2</td>
<td>4303</td>
<td>4280</td>
<td>4178</td>
<td>4183</td>
<td>4299</td>
<td>4249</td>
</tr>
<tr>
<td>Group 3</td>
<td>4709</td>
<td>4888</td>
<td>4793</td>
<td>4952</td>
<td>5011</td>
<td>4871</td>
</tr>
<tr>
<td>Group 4</td>
<td>5831</td>
<td>5967</td>
<td>5782</td>
<td>5861</td>
<td>5663</td>
<td>5821</td>
</tr>
</tbody>
</table>
After making the samples, the surface of the specimens were prepared to be attached to the transducers. To do so, a saw was used to make the surface aligned and smooth. After cutting, the surface was cleaned so that there was no dust and debris on the surface. In Figure 5.2, the saw which was used to cut the specimens is shown.

![Figure 5.2 Saw machine used to cut the surface of the plain concrete](image)

Broadband Conrad transducers (EPZ-27MS44F, Item Number 712930 - 62) as shown in the Figure 5.3 were used as transmitters and receivers ([www.CONRAD.de](http://www.CONRAD.de)). These thin circular disk type transducers were fabricated by attaching a ceramic disk of 20 mm diameter to a metal disk of 27 mm diameter.

![Figure 5.3 Broadband Conrad transducers](image)
Two thin disk type PZT transducers were mounted on the two ends of the samples, as shown in Figure 5.4.

Figure 5.4 PZT glued to the sample

A linear chirp was used to excite the transmitter placed on the samples in the frequency range varying from 45 kHz to 175 kHz. A Handy-scope HS-3 provided by ASI (Analog Speed Instruments) of Germany shown in Figure 5.5 was used for exciting the transducer. The Handy-scope HS-3 is a computer controlled measuring instrument that integrated the functions of three instruments - oscilloscope, arbitrary waveform generator and data logger. An Arbitrary Function Generator (HS3) generates an electric pulse controlled by a computer. The generated electric pulse is then converted to an ultrasonic pulse by the transmitter. The ultrasonic pulse is then transmitted and propagated through the cylindrical concrete specimen. At the receiving end, the ultrasonic pulse is detected and then converted back to the electric signal by a second transducer or detector. The received electric signal is then sent back to the HS3 unit which sends the signal to the Processing Unit (Computer) for further analysis.
Figure 5.5 Handyscope HS3, a professional USB oscilloscope

By connecting the transducers to the Handy-scope HS-3, the chirp signal was generated through the sample.

The recorded signals by the receiving transducer were processed in order to obtain ultrasonic signatures that are capable of revealing changes in specimens’ strength. The wave generator triggered the signal with 12 volts amplitude and was recorded with a sampling rate of 50 MHz. The received signal data was acquired by the Handy-scope HS-3 and transmitted to the computer.

To measure the change in time of flight due to the change in plain concrete compressive strength, a cross correlation technique was adopted. The cross-correlation between the two time histories can accurately measure the small difference in the time-of-flight (TOF). In contrast to other procedures we do not correlate the measured signal pulse with the sent one to get the absolute TOF. Instead the received signal pulse measured at time $t_0$ is correlated with the received signal pulses at time $t_0 + t$, where $t_0$ represents the beginning of the data acquisition and $t$ is an integer running from 0 to $t_{\text{final}} - t_0$ ($t_{\text{final}}$ is the time when the measurement is finished) times the sampling period. This has the advantage that imperfections of the transducer and the dispersion inside the
material are already included in the reference signal, but instead of the absolute TOF a differential TOF, the difference in the TOF between the two pulses, is measured.

It must be mentioned that in this section for cross correlation technique the received transient signal is windowed into 3 major windows which are shown in Figure 5.6:

Window 1: this window, which starts from the first arrival time in the transient signal and covers the maximum amplitude of the signal, has the time period from 50μs to 150μs.

Window 2: this window covers another part of the transient signal with less amplitude and has the time period between 150μs and 250μs.

Window 3: this window has the time period between 250μs to 350μs.
Figure 5.6 Time windows in the transient signal
To carry out the test, from the twenty samples, one sample was chosen as the reference sample for adjusting the set up window. The measured TOF for the reference sample is denoted as the reference TOF ($TOF_0$). Then the other nineteen samples were tested with the reference setting and the shift in TOF was measured for every sample compared to the reference sample. The same procedure was carried out for all nineteen samples.

After testing every sample in the same way as the reference sample and finding the value of TOF shift in all other samples, the value of compressive strength were measured for all specimens by standard destructive tests (Table 5.1).

A compression test determines the behavior of materials under crushing loads. The compressive strength of a material that fails by a shattering fracture can be defined within fairly narrow limits as an independent property. However, the compressive strength of materials that do not shatter in compression must be defined differently. It may be defined as the amount of stress required to cause significant distortion in the material. Compressive strength is calculated by dividing the maximum load by the original cross-sectional area.

To measure the maximum strength of the samples in compression, ASTM C 39/C 39M (Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens) was used. This test method covers determination of compressive strength of cylindrical concrete specimens such as molded cylinders and drilled cores.

The test method consisted of applying a compressive axial load to a plain concrete cylinder at a constant rate within a prescribed range. The specimens were 3x6 in
cylinders with smooth and leveled top and bottom surfaces. The leveled and smooth top and bottom surfaces allow the load to be uniformly applied across the cross section of the sample. To apply the compressive stress on the samples, an MTS 311 Load Frame was used. Figure 5.7 shows the MTS 311 Load Frame with a sample. To measure the compressive strength, the load was applied on the top and bottom of the specimens with a 0.025 mm/sec (0.001 in/sec) load rate.

Since finding the relationship between the change in strength and the change in ultrasonic wave velocity is of interest in this study the value of the shift in TOF must be changed to $\Delta(velocity)$. If the distance between the transducers is known, then the TOF can be converted to velocity using the simple wave equation:
The strength, TOF and velocity values of the reference sample were denoted by $f_{c0}$, $TOF_0$ and $V_0$. When the other samples’ strength, TOF and velocity were measured, to find the change in these values, one can easily subtract them from the corresponding reference values. Then by knowing the $\Delta$(strength) and $\Delta$(velocity) the graphs can be plotted. Figure 5.8 shows the $\Delta$(strength) - $\Delta$(velocity) relationship for all samples in window 1 of the transient signal. For windows 2 and 3, the data points were very scattered and are not shown.

The graph shown in Figure 5.8 can be used for any concrete sample with the compressive strength varying between 3500 psi and 6000 psi. To use this graph, the zero point on the vertical axis must be the real strength of the reference sample and the strength of the other samples can be calculated by adding or subtracting the $\Delta$(strength value) to the reference strength based on the value of change in velocity which is the difference of the unknown strength sample velocity and the reference sample velocity.

$$Velocity = \frac{Distance\ between\ PZTs}{(TOF)_{\text{chosen\ window}}} \quad (5.1)$$
To find the equation that can be used to express the data points in Figure 5.8, two types of curves were fitted to the data: linear fit and exponential fit. Figures 5.9 a and b shows these graphs, respectively.

![Graphs showing linear and exponential fits](image)

**Equation**

\[ y = a + b^x \]

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Standard Error</th>
</tr>
</thead>
<tbody>
<tr>
<td>( a )</td>
<td>0.75171</td>
<td>32.87466</td>
</tr>
<tr>
<td>( b )</td>
<td>43.47263</td>
<td>0.91693</td>
</tr>
</tbody>
</table>

**Equation**

\[ y = A_1 \exp(a_1 x) + y_0 \]

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
<th>Standard Error</th>
</tr>
</thead>
<tbody>
<tr>
<td>( y_0 )</td>
<td>-31750.8486</td>
<td>24250.66</td>
</tr>
<tr>
<td>( A_1 )</td>
<td>31767.7708</td>
<td>24250.66</td>
</tr>
<tr>
<td>( a_1 )</td>
<td>-7300.542</td>
<td>5273.14</td>
</tr>
</tbody>
</table>

**Figure 5.9 Δstrength-Δvelocity plots – experimental points and fitted curves**

**a) Linear fit**, **b) Exponential fit**
As can be seen here, the linear curve fits the data points well. The equation for the relationship between Δstrength-Δvelocity is shown below (Eq. 5.2) with R-square value equal to 0.95495.

\[
\Delta \text{Strength} = 8.75171 + 43.47263 \times \Delta \text{velocity}
\] (5.2)

Where, Δ Strength is Compressive strength difference of two samples and Δ Velocity is velocity difference of two samples.

To clarify the use of these graphs, an example is provided here. Suppose that sample 1 has the compressive strength of \( f_{c1}' \) and velocity of \( V_1 \) for the chosen window. By measuring the other sample velocity, \( V_2 \), the change in velocity (ΔV) can be calculated.

Since the value of ΔV is known, by using the graph of Figure 5.9 or Equation 5.2, the value of Δ strength can be derived. By knowing this value the compressive strength of the second sample can be calculated.

\[
f_{c2}' = f_{c1}' + \Delta \text{(strength)}
\] (5.3)

By taking Fast Fourier Transform (FFT) of the transient signal (time window), data can be expressed in the frequency domain. In other words, after taking FFT the time domain signal changes to the frequency domain. In the frequency domain, the behavior of samples in different groups is shown for window 1 in Figure 5.10.
Figure 5.10 Transient signal for different concrete groups, (left) Time domain, (right) Frequency domain

When the sample is excited by a chirp signal, several resonant modes are observed in the frequency spectra. By comparing the frequency domain of different
groups of samples, since material properties were changing from one sample to another
sample, the shift in propagation frequency was observed. Therefore, it is hard to interpret
the results to find the compressive strength based on the frequency response. This is the
reason why the time of flight was considered instead of the frequency to find the change
in strength. TOF approach gives a clear picture of TOF change (velocity change) with the
change in strength. As it was explained before, to measure the change in the time of flight
because of the change in the plain concrete compressive strength, a cross correlation
technique was adopted. The cross-correlation between the two time histories can
accurately measure the small difference in the time-of-flight (TOF).

The feature extraction is an important component of Structural Health Monitoring
(SHM). The process of feature extraction involves identifying damage sensitive
properties from the dynamic response of a structure facilitating the assessment of the
health of the structure. The following investigation of Time-Frequency Representation
(TFR) was carried out for different groups of concrete with different $f_c'$ values.

Fourier analysis in its modified form overcomes the limitation of loss of temporal
information forming basis for Short Time Fourier Transform (STFT) technique. In this
 technique the time information within a small segment of the signal (window) is
transformed into its frequency content. The STFT thus presents a compromise between
time and frequency resolution [73]. It provides a tool to choose the needed precision in
either time or in frequency domain. Mathematically, the STFT of a time history signal
x(t) is defined as:

$$X(\omega, \tau) = \int_{-\infty}^{\infty} x(t)w(t-\tau)e^{-j\omega t}dt$$  \hspace{1cm} (5.4)
Where, $w(t)$ is the window function and $x(t)$ is the signal to be transformed. In STFT the wide window size gives better frequency resolution at the cost of time resolution and vice versa. The drawback is once a window size is selected, that remains the same for all frequencies. Figure 5.11 shows the Short Time Fourier Transform (STFT) of time history computed with a 128 Hanning window for different groups of concrete samples.

![Figure 5.11 Interpolated contour plots for STFT of different groups of concrete](image)

It can be observed in STFT results that for all groups, the maximum amplitude is in the time period of 50µs to 150µs. Therefore, the first window was the best window that could be used to find the TOF change due to the change in the compressive strength.
5.3 Objective 2: A new method to measure the strength of polymer concrete using ultrasonic waves

5.3.1 General Concepts

Ultrasonic methods are well-known and standardized for traditional building materials, like metals, cement concrete, and rocks. For polymer concrete composites, ultrasonic methods are at their infancy. Because of the similarity of the geometrical features of the microstructure of both cement concrete and polymer concrete, it seems logical that experience with ultrasonic techniques on cement concrete can be extended to polymer concrete.

5.3.2 Experimental investigation and results

The aim here is the analysis of the usability of ultrasonic wave method for assessment of polymer concrete compressive strength.

To achieve our goal under this objective the same approach as explained in section 5.2 was performed. Fifteen cylindrical polymer concrete specimens of 3 inch diameter and 6 inch length with five different strength values were fabricated. Three types of aggregates (G8, G18 and sand) and epoxy resin were used with properties mentioned in Chapter 4. The measured compressive strength value of every sample is tabulated in Table5.2.
Table 5.2 Final strength of each group, $f_{pc}'$ (psi)

<table>
<thead>
<tr>
<th>Group #</th>
<th>Sample 1 strength (psi)</th>
<th>Sample 2 strength (psi)</th>
<th>Sample 3 strength (psi)</th>
<th>Average strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Group 1</td>
<td>7997</td>
<td>7793</td>
<td>7624</td>
<td>7804</td>
</tr>
<tr>
<td>Group 2</td>
<td>9340</td>
<td>9120</td>
<td>9289</td>
<td>9250</td>
</tr>
<tr>
<td>Group 3</td>
<td>9882</td>
<td>9734</td>
<td>9919</td>
<td>9845</td>
</tr>
<tr>
<td>Group 4</td>
<td>8479</td>
<td>8949</td>
<td>8520</td>
<td>8649</td>
</tr>
<tr>
<td>Group 5</td>
<td>9199</td>
<td>9160</td>
<td>8949</td>
<td>9102</td>
</tr>
</tbody>
</table>

The surface of the polymer concrete samples was prepared so that the transducers could be attached. To do so, as before the top and bottom surfaces of the samples were saw cut. The prepared surfaces are shown in Figure 5.12.

![Figure 5.12 (left) saw machine used to cut the surface of the polymer concrete, (right) polymer concrete with prepared surface](image)

The same type of transducers (Broadband Conrad transducers EPZ-27MS44F) used for plain concrete characterization were mounted on the two ends of the sample as shown in Figure 5.13.
By connecting the transducers to the Handy-scope HS-3, the chirp signal was generated through the samples.

A cross correlation technique, as explained before, was adopted to measure the change in the time of flight due to change in polymer concrete compressive strength.

The received transient signal was windowed into three major windows (Figure 5.15) for the cross correlation technique. These windows were:

Window 1: this window, which started from the first arrival time in the transient signal and covered the maximum amplitude of the signal, had the time period from 50µs to 150µs.

Window 2: this window covered another part of the transient signal with lower amplitude and had the time period between 150µs and 250µs.

Window 3: this window had the time period between 250µs to 350µs.
The same procedure as explained in section 5.2 was carried out for fifteen polymer concrete specimens. After making every sample as the reference sample and finding the value of TOF shift for all other samples, the values of compressive strength
were measured for all samples by destructive tests (Table 5.2), using MTS 311 load frame shown in Figure 5.15. The load was applied on the top and bottom of the specimens with a 0.025 mm/sec (0.001 in/sec) load rate.

![Figure 5.15 Polymer concrete sample under MTS Load Frame 311](image)

By measuring the shift in TOF (change in velocity) and corresponding compressive strength value of every sample, the appropriate graphs can be plotted. Figure 5.16 shows the $\Delta$(strength) - $\Delta$(velocity) relationship for the tested samples in windows 1, 2 and 3 of the transient signal.
Figure 5.16 $\Delta$strength-$\Delta$velocity relationship data for polymer concrete in window 1, 2 and 3
These graphs can be used for any polymer concrete sample with the compressive strength varying between 7500 psi and 10000 psi. To use these graphs, the zero point on the vertical axis must be the real strength of the reference sample and the strength of the other samples can be calculated by adding or subtracting the Δ(strength) value to the reference strength based on the value of change in velocity which is the difference of the unknown strength sample velocity and the reference sample velocity.

To find the equation that can be used to express the data points in Figure 5.16 for each window, two types of curves were fitted to the data: linear fit and exponential fit. Figures 5.17 to 5.19 show these graphs.
Figure 5.17 Δstrength-Δvelocity plots – experimental points and fitted curves for window 1

a) Linear fit

b) Exponential fit
Figure 5.18 $\Delta$strength-$\Delta$velocity plots – experimental points and fitted curves for window 2

a) Linear fit

b) Exponential fit
Figure 5.19 Δstrength-Δvelocity plots – experimental points and fitted curves for window 3

a) Linear fit

b) Exponential fit
The equations of the relationship between Δstrength-Δvelocity are tabulated in Table 5.3 for all three windows. It is obvious in the Figures that the linear curve can be a good fitting curve for the data points. This result can be justified by comparing the R-square values in Table 5.3.

<table>
<thead>
<tr>
<th>Window #</th>
<th>Type of curve fitting</th>
<th>Equation</th>
<th>R-square value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Window 1</td>
<td>Linear fit</td>
<td>Δstrength = 13.20638 + 9.13508ΔV</td>
<td>0.97448</td>
</tr>
<tr>
<td></td>
<td>Exponential fit</td>
<td>Δstrength = 17744.6 exp\left( \frac{ΔV}{1944.55} \right) - 17745.96</td>
<td>0.94447</td>
</tr>
<tr>
<td>Window 2</td>
<td>Linear fit</td>
<td>Δstrength = -5.88231 + 14.25509ΔV</td>
<td>0.76918</td>
</tr>
<tr>
<td></td>
<td>Exponential fit</td>
<td>Δstrength = 24595.2 exp\left( \frac{ΔV}{1723.85} \right) - 24608.6</td>
<td>0.76448</td>
</tr>
<tr>
<td>Window 3</td>
<td>Linear fit</td>
<td>Δstrength = 7.85974 + 10.38507ΔV</td>
<td>0.48711</td>
</tr>
<tr>
<td></td>
<td>Exponential fit</td>
<td>Δstrength = 891.97 exp\left( \frac{ΔV}{8615} \right) - 1016.29</td>
<td>0.5201</td>
</tr>
</tbody>
</table>

By comparing the R-square values of each window, it can be seen that window 1 has more reliable data compared to windows 2 and 3. This result also can be noticed by observing the data points in Figures 5.17 to 5.19. As it was observed, the data points are more scattered in window 2 compared to that in window 1. The same pattern was detected for windows 2 and 3.

By expressing the data in the frequency domain, the behavior of samples in different groups is shown for window 1, 2 and 3 in Figure 5.20.

Several resonant modes were observed in the frequency spectra when the sample was excited by a chirp signal. By comparing the frequency domain results of different groups of samples in different windows it was found that the amplitude of FFT graph was the highest in window 1 and it was reduced by changing the window to windows 2 and 3. This can be a reason for the scatter in the data points obtained for Δstrength-ΔV graphs when the window was changed from 1 to 2 and 3.
Figure 5.20 Frequency domain of transient signal for different concrete groups
window 1, window 2 and window 3
Another way to see this behavior is to compare the STFT results. This technique was explained in section 5.2. Figure 5.21 shows the Short Time Fourier Transform (STFT) of time history computed with a 128 Hanning window for different groups of polymer concrete specimens.

![Figure 5.21 Interpolated contour plots for STFT of different groups of polymer concrete](image)

Figure 5.21 Interpolated contour plots for STFT of different groups of polymer concrete
It can be observed that in STFT results of different groups of polymer concrete that, for all groups, the maximum amplitude is in the time period of 50μs to 150μs. Also some part of the second window has the high amplitude. Therefore in addition to selecting window 1, window 2 with the time period of 150μs to 250μs was chosen to see the behavior of polymer concrete samples for this specific window. Therefore, the first window for the polymer concrete experiments was the best window that could be used to find the TOF change due to the change in the compressive strength and second and third windows were the next best windows with less R-square values.
5-4 Objective3: The effect of applied load on UPV value

5.4.1 General

As was discussed earlier, The UPV method is a stress wave propagation method based on measuring the travel time, over a known path length, of a pulse of ultrasonic compression wave (stress waves associated with normal stress). The pulses are introduced into the concrete by a piezoelectric transducer, and a similar transducer acts as a receiver to monitor the surface vibration caused by the arrival of the pulse. A grease or gel is applied to the faces of the transducers to ensure good coupling with the surfaces. A timing circuit is used to measure the time it takes for the pulse to travel from the transmitter to the receiver. Figure 5.22 provides a schematic of the ultrasonic pulse velocity technique.

![Figure 5.22 Schematic of the ultrasonic pulse velocity technique](image-url)
In the test described in BS 1181: Part 203 [25], the time that the pulses takes to travel through concrete is recorded. Then, the velocity is calculated as:

\[ V = \frac{l}{t} \]  \hspace{1cm} (5.7)

Where \( V \) = pulse velocity (m/s), \( l \) = length (m), and, \( t \) = effective time (s), which is the measured time minus the zero time correction. The zero time correction is equal to the travel time between the transmitting and receiving transducers when they are pressed firmly together.

To find the velocity using the test described in BS 1181: Part 203, the procedure followed by researchers during the experiment included the following steps:

- Various concrete mix designs were tested.
- All concrete samples were immersed under water for a minimum period of 24 hours prior to testing.
- Just before testing, the specimens were rubbed with a clean dry cloth in order to obtain a saturated surface dry sample.
- Upon drying, the surface of every sample was prepared for the ultrasonic pulse velocity test, as described in the specifications.
- Once nondestructive testing on every sample was completed, the specimen was loaded to failure and the maximum load was recorded.

Since compressive strength is the major mechanical property in structural concrete, the measured ultrasonic wave velocity was related to strength, and plots of velocity vs. strength were obtained as indicated in many references (Figure 5.23).
Figure 5.23 Schematic of typical relationship between pulse velocity and compressive strength of a given concrete mixture

As can be seen here, to obtain a graph which shows the relationship between UPV and $f'_c$, the effects of applied load on the structure did not come into the picture. Researchers used the PUNDIT device to predict the final strength of the concrete, when the samples were absent from axial load. However, this method is not accurate because when the specimen is tested under stress, its behavior is quite different from when it is tested without any applied load. This conclusion was obtained as a result of this investigation after testing many samples.

5.4.2 Experimental investigation and results

This study aims to investigate the effect of applied load (stress) on the ultrasonic wave velocity passing through the plain concrete specimens. To carry out the test, by applying the load on the specimen up to almost 70% of specimens’ final strength ($f'_c$), the time of flight (TOF) was measured in each step of loading and it was compared with the TOF of stress-free samples. The results showed that the time of flight measured in each step of axial loading is different. Therefore, to find the strength of the concrete
based on the UPV value, the applied load on the sample must be considered as an important factor that cannot be neglected.

To reach our goal for this objective, the following procedures were undertaken:

- Cylindrical concrete specimens of 3 inch diameter and 6 inch length (shown in Figure 5.24) were made with different $f_c'$. Four groups of samples were fabricated having four different values of $f_c'$ as verified by standard destructive compressive tests.

- For every group of specimens, the change in velocity ($\Delta V$) was measured for varying applied axial loads. For example, if $f_c'$ of one group is A, then an applied stress of $10\% A$ on the specimen and the change in velocity ($\Delta V$) was measured by comparing the measured value with that for the stress-free sample. The next step was to apply $20\% A$ stress on the sample and repeat the procedure. These steps were continued up to almost $70\% A$ stress for each group.

- Different experimental graphs were plotted for the clarification of the results.

Since for every group with a specified $f_c'$ value for every load step different $\Delta V$ values existed appropriate graphs were plotted to discern signature responses that could help understanding of the effects of existing loads on the UPV values.

Figure 5.24 Plain concrete sample under this study
To make plain concrete, two types of aggregate and regular Portland cement were mixed with the properties mentioned in Chapter 4. The samples were under four different groups with four different $f'_{c}$ values. The measured strength values of every group are tabulated below (Table 5.4):

**Table 5.4 Final strength of each group, $f'_{c}$ (psi)**

<table>
<thead>
<tr>
<th>Group #</th>
<th>Sample1 strength (psi)</th>
<th>Sample2 strength (psi)</th>
<th>Sample3 strength (psi)</th>
<th>Sample4 strength (psi)</th>
<th>Sample5 strength (psi)</th>
<th>Average strength (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Group 1</td>
<td>3621.94</td>
<td>3473.39</td>
<td>3638.64</td>
<td>3599.54</td>
<td>3748.34</td>
<td>3616.37</td>
</tr>
<tr>
<td>Group 2</td>
<td>4102.66</td>
<td>4293.93</td>
<td>4368.76</td>
<td>4195.6</td>
<td>4355.47</td>
<td>4263.2844</td>
</tr>
<tr>
<td>Group 3</td>
<td>4897.01</td>
<td>4722.59</td>
<td>4951.66</td>
<td>4901.33</td>
<td>5025.19</td>
<td>4899.556</td>
</tr>
<tr>
<td>Group 4</td>
<td>5926.86</td>
<td>5736.16</td>
<td>5799.02</td>
<td>5832.88</td>
<td>5504.27</td>
<td>5759.838</td>
</tr>
</tbody>
</table>

Each sample group was tested by the ultrasonic wave technique as shown in Figure 5.25.

![Figure 5.25 (left) photo of the experimental setup, and (right) Schematic for experimental setup](image)

After making the specimens with different final or ultimate strength values, the surface of concrete was prepared before attaching the PZTs. For this purpose, a special
small grinding machine was used as shown in Figure 5.26. Making the surface smooth helped to attach the transducers to the specimen.

![Figure 5.26 Special grinding machine for surface preparation](image)

Broadband Conrad transducers (EPZ-20MS64, Item Number 712918-62) as shown in the Figure 5.27, were used as transmitters and receivers ([www.CONRAD.de](http://www.CONRAD.de)). These thin, circular, disk-type transducers were fabricated by attaching a ceramic disk of 15 mm diameter to a metal disk of 20 mm diameter.

![Figure 5.27 Broadband Conrad transducers](image)

Two thin disk type PZT transducers were glued to the sample as shown in Figure 5.28.
A linear chirp was used to excite the transmitter placed on the samples operating in a frequency range varying from 45 kHz to 175 kHz. By connecting the transducers to the Handy-scope HS-3, the chirp signal was generated (Figure 5.29).

The recorded signals by the receiving transducer were processed in order to obtain ultrasonic signatures that were capable of revealing changes in the material properties due to applied load (stress). The same technique (cross-correlation), as explained in section 5.2, was used to carry out the test and to obtain the time difference.
By generating and receiving chirp signal for each group of samples under the specified applied load, the change in TOF was measured. To measure the change in TOF, three different windows were chosen on a received transient signal (time history):

The first window was the window near the first arrival.

The second window was the window that covered the maximum amplitude of the transient signal.

The third window was the window farthest from the first arrival time and had lower amplitude compared to the second window. These windows are shown in Figure 5.30.

![Figure 5.30](image)

**Figure 5.30** (a) First window, (b) Second window, (c) Third window
The first arrival of the signal in the transient window (time window) was between 30 µs and 50µs. The 85µs to 105µs window corresponded to the maximum amplitude of the transient signal, and the 125µs to 175µs time interval covered the third window.

Twenty plain concrete specimens were tested with four different values for the final strength ($f_c'$). These samples were selected for the specified window testing based on Table 5.5.

**Table 5.5 Selected plain concrete samples to be tested for different windows**

<table>
<thead>
<tr>
<th>Group #</th>
<th>First window</th>
<th>Second window</th>
<th>Third window</th>
</tr>
</thead>
<tbody>
<tr>
<td>Group 1</td>
<td>M1-1</td>
<td>M1-2 &amp; M1-5</td>
<td>M1-3 &amp; M1-4</td>
</tr>
<tr>
<td>Group 2</td>
<td>M2-1</td>
<td>M2-2 &amp; M2-5</td>
<td>M2-3 &amp; M2-4</td>
</tr>
<tr>
<td>Group 3</td>
<td>M3-1</td>
<td>M3-2 &amp; M3-5</td>
<td>M3-3 &amp; M3-4</td>
</tr>
<tr>
<td>Group 4</td>
<td>M4-1</td>
<td>M4-2 &amp; M4-5</td>
<td>M4-3 &amp; M4-4</td>
</tr>
</tbody>
</table>

5.4.2.1 First window (window of first arrival) results

As mentioned before, by selecting the time window of transient signal for each sample at no load, the transient signal and the corresponding TOF were recorded. By choosing this TOF as the reference time of flight, we could easily find the change in TOF due to the applied load for every step of loading.

Based on the information given in Table 5.5, and to carry out the test for this specified window of transient signal, four samples were selected: M1-1, M2-1, M3-1 and M4-1. These specimens were from different groups of concrete with different $f_c'$ values.

In Figure 5.31, the relationship between the applied load and the change in TOF is shown. It is clear in the Figure that by considering the first window, which covers the
first arrival of the received signal, no significant change is observed in TOF by applying the load. In other words, applied load (stress) didn’t have any significant effect on the time of flight shift or velocity change. Since the measured UPV value was based on the first arrival time collected by PUNDIT device, most of the researchers neglected the effect of applied load on the UPV value.

![Figure 5.31 Change in TOF (ΔTOF) versus applied load](image)

The equation 5.8 shows the relationship for a linear fit of the data with R-square value of 0.99728.

\[
\Delta \text{TOF} = 0.00117 \times \text{applied load} - 6.99729 \times 10^{-4}
\]

It is clear in Figure 5.31 or Equation 5.8, that by applying a load of almost 25 kips, the TOF increased less than 0.03 µs. This increase is due to the Poisson’s ratio. By applying the longitudinal compressive load, there is contraction in the direction of the applied load and an expansion in the direction perpendicular to the direction of the
applied load. This lateral expansion increases the distance between the PZTs and, due to this distance change, there is an increase in TOF.

By considering the other two windows (window 2 and window 3) the change in the TOF was found to be significant due to the applied load (stress). The use of these two windows was emphasized to reveal the effects of the applied load (stress) on the ultrasonic pulse velocity value.

### 5.4.2.2 Second Window (Window of maximum amplitude) results

Based on the information given in Table 5.5, to carry out the test for this window (Figure 5.32), eight samples were selected: M1-2, M1-5, M2-2, M2-5, M3-2, M3-5, M4-2 and M4-5. These samples were from different groups of concrete with different $f_{c'}$ value.

![Figure 5.32 Second Window (window of maximum amplitude signal)](image)

As shown in Figure 5.33, by choosing the window related to the maximum amplitude of the transient signal, with increasing applied load (stress), the time of flight (TOF) decreases.
Figures 5.34 to 5.37 show the relationship between the applied load and the change in TOF for groups 1, 2, 3 and 4 specimens, respectively.

By performing regression analysis on the collected data, three different types of curve were suggested. The accuracy of these curves was compared by their R-square values. The results are summarized in Table 5.6 for each group of specimens.

Table 5.6 Applied load versus ΔTOF for all groups

<table>
<thead>
<tr>
<th>Group #</th>
<th>Type of fitted curve</th>
<th>Equation</th>
<th>R-square</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Linear</td>
<td>$\text{load} = 13.20638 + 9.13508\Delta\text{TOF}$</td>
<td>0.92793</td>
</tr>
<tr>
<td>Group 1</td>
<td>Polynomial</td>
<td>$\text{load} = 1.1049 - 2.026\Delta\text{TOF} + 6.39029(\Delta\text{TOF})^2$</td>
<td>0.96109</td>
</tr>
<tr>
<td></td>
<td>Exponential</td>
<td>$\text{load} = 2.671 \exp\left(\frac{-\Delta\text{TOF}}{0.75}\right) - 1.381$</td>
<td>0.97195</td>
</tr>
<tr>
<td></td>
<td>Linear</td>
<td>$\text{load} = -1.269 - 12.368\Delta\text{TOF}$</td>
<td>0.92981</td>
</tr>
<tr>
<td>Group 2</td>
<td>Polynomial</td>
<td>$\text{load} = 0.9638 - 1.7978\Delta\text{TOF} + 5.9638(\Delta\text{TOF})^2$</td>
<td>0.96382</td>
</tr>
<tr>
<td></td>
<td>Exponential</td>
<td>$\text{load} = 2.683 \exp\left(\frac{-\Delta\text{TOF}}{0.7837}\right) + 1.565$</td>
<td>0.97503</td>
</tr>
<tr>
<td></td>
<td>Linear</td>
<td>$\text{load} = -0.52271 - 12.82301\Delta\text{TOF}$</td>
<td>0.93458</td>
</tr>
<tr>
<td>Group 3</td>
<td>Polynomial</td>
<td>$\text{load} = 1.18526 - 2.538\Delta\text{TOF} + 6.3275(\Delta\text{TOF})^2$</td>
<td>0.96524</td>
</tr>
<tr>
<td></td>
<td>Exponential</td>
<td>$\text{load} = 3.08534 \exp\left(-\frac{\Delta\text{TOF}}{0.7837}\right) - 1.73$</td>
<td>0.9764</td>
</tr>
<tr>
<td></td>
<td>Linear</td>
<td>$\text{load} = -0.95413 - 12.6780\Delta\text{TOF}$</td>
<td>0.93019</td>
</tr>
<tr>
<td>Group 4</td>
<td>Polynomial</td>
<td>$\text{load} = 1.11528 - 1.98655\Delta\text{TOF} + 6.32443(\Delta\text{TOF})^2$</td>
<td>0.96197</td>
</tr>
<tr>
<td></td>
<td>Exponential</td>
<td>$\text{load} = 2.62709 \exp\left(-\frac{\Delta\text{TOF}}{0.749}\right) - 1.326$</td>
<td>0.97489</td>
</tr>
</tbody>
</table>

For every group of concrete, the best fit curve was an exponential fit with a high R-square value (Table 5.6).
Figure 5.34 Applied load versus change in time of flight relationship for group 1 specimens
Figure 5.35 Applied load versus change in time of flight relationship for group 2 specimens
Figure 5.36 Applied load versus change in time of flight relationship for group 3 specimens
Figure 5.37 Applied load versus change in time of flight relationship for group 4 specimens
To express the results based on the applied stress and velocity change, some mathematical manipulations must be performed. TOF and velocity have an inverse relationship \((V=x/t)\). Therefore, if the TOF decreases the velocity increases. The results showed that increasing the stress on the structure led to an increase in the velocity value.

To calculate the value of \(\Delta V\), the exact value of distance between transducers at each load step must be measured accurately. Since the transducers were attached to the circumference of the sample, by applying the load, this distance was subject to change. To correct the transducers’ distance value used in measuring the velocity in each step of loading, an extensometer was placed in the middle of the sample to measure the strain in the direction of the applied load, as shown in Figure 5.38. Then by knowing the strain in the direction of applied load, the circumference strain could be calculated using Poisson’s ratio:

\[
\nu = -\frac{\varepsilon_{\text{lateral}}}{\varepsilon_{\text{axial}}} \quad \text{or} \quad \varepsilon_{\text{lateral}} = -\nu \varepsilon_{\text{axial}}
\]  

(5.9)

Since the circumferential strain is related to the strain along the diameter, the PZTs distance can be corrected in each step of loading.

Figure 5.38 Extensometer used to find the strain in the direction of the applied load
By multiplying the lateral strain at each load step to the measured PZT to PZT distance value at the load value of zero, the exact value of transducers’ distance can be calculated at any load step. To find the velocity at any load step, Equation 5.10 is used:

\[
(velocity)_{at\ each\ load\ step} = \frac{(exact\ value\ of\ PZT\ to\ PZT\ distance)_{at\ each\ load\ step}}{(TOF)_{at\ each\ load\ step}} \quad (5.10)
\]

If the velocity of the sample at zero kips load is denoted by \( V_0 \), then the change in velocity can be calculated from:

\[
\Delta V_{at\ each\ load\ step} = V_{at\ each\ load\ step} - V_0 \quad (5.11)
\]

In Figures 5.39 to 5.41, the relationship between applied stress on the specimen and the change in velocity for the samples in group 1 were studied. To draw the graphs, the effect of Poisson’s ratio and multiple types of curve fitting methods were considered. To see the effect of Poisson’s ratio on the results, stress versus change in velocity graphs are given for “No Poisson’s ratio”, “Poisson’s ratio 0.2” and “Poisson’s ratio 0.25”. To fit the best curve through the data, three different types of curves were used: Linear, polynomial and exponential fit. The accuracy of the curves was compared by their \( R^2 \) values.
Figure 5.39 Stress versus change in velocity graphs of group 1 samples (linear fit),
a) No Poisson’s ratio, b) Poisson’s ratio 0.2, c) Poisson’s ratio 0.25
Figure 5.40 Stress versus change in velocity graphs of group 1 samples (Polynomial fit),

a) No Poisson’s ratio, b) Poisson’s ratio 0.2, c) Poisson’s ratio 0.25
Figure 5.41 Stress versus change in velocity graphs of group 1 samples (Exponential fit),

a) No Poisson’s ratio, b) Poisson’s ratio 0.2, c) Poisson’s ratio 0.25
For group 1 samples with an average strength of 3616.37 psi, the equations that can be used to find the applied stress on a sample, based on the velocity change are tabulated in Table 5.7.

By comparing the results, it can be seen that the best curve fit to express the relationship between the stress and the change in velocity is an exponential fit with the highest R-square value. Also, by observing the graphs for each type of curve, the Poisson’s ratio does have an effect on the results. However, this effect is not significant.

<table>
<thead>
<tr>
<th>Group 1 SPECIMENS</th>
<th>Applied stress (psi) versus Change in velocity (ft/sec)</th>
<th>R-square</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Type of fitted curve</strong></td>
<td><strong>Poisson’s ratio</strong></td>
<td><strong>Equation</strong></td>
</tr>
<tr>
<td>Linear</td>
<td>No Poisson’s ratio</td>
<td>$\Delta stress = -113.41 + 77.45 \Delta V$</td>
</tr>
<tr>
<td></td>
<td>Poisson’s ratio 0.2</td>
<td>$\Delta stress = -113.14 + 75.815 \Delta V$</td>
</tr>
<tr>
<td></td>
<td>Poisson’s ratio 0.25</td>
<td>$\Delta stress = -113.057 + 75.416 \Delta V$</td>
</tr>
<tr>
<td>Polynomial</td>
<td>No Poisson’s ratio</td>
<td>$\Delta stress = 158.613 + 13.29 \Delta V + 1.65 (\Delta V)^2$</td>
</tr>
<tr>
<td></td>
<td>Poisson’s ratio 0.2</td>
<td>$\Delta stress = 156.63 + 13.969 \Delta V + 1.55 (\Delta V)^2$</td>
</tr>
<tr>
<td></td>
<td>Poisson’s ratio 0.25</td>
<td>$\Delta stress = 156.11 + 14.139 \Delta V + 1.5299 (\Delta V)^2$</td>
</tr>
<tr>
<td>Exponential</td>
<td>No Poisson’s ratio</td>
<td>$\Delta stress = 398.33 \exp\left(\frac{\Delta V}{17.826}\right) - 213.423$</td>
</tr>
<tr>
<td></td>
<td>Poisson’s ratio 0.2</td>
<td>$\Delta stress = 429.916 \exp\left(\frac{\Delta V}{18.839}\right) - 249.85$</td>
</tr>
<tr>
<td></td>
<td>Poisson’s ratio 0.25</td>
<td>$\Delta stress = 438.05 \exp\left(\frac{\Delta V}{19.079}\right) - 259.185$</td>
</tr>
</tbody>
</table>

By observing the graphs, it can be seen that in the linear fit the data point corresponding to the higher stress values cannot be covered by the suggested linear curve while the exponential curve covers those data points perfectly. Results showed that the change in $\Delta V$ per unit change in stress decreases with increasing stress. This means that change in velocity is relatively insensitive to stress for plain concrete under higher values of stress. By performing the same procedures on the samples in other groups of specimens (groups 2, 3 and 4), the relationship between applied stress and change in velocity were obtained. These results are given in Figures 5.42 to 5.50, and the respective
equations that can be used to find the applied stress based on the velocity change are tabulated in Tables 5.8 to 5.10.

Figure 5.42 Stress versus change in velocity graphs of group 2 samples (Linear fit), a) No Poisson’s ratio, b) Poisson’s ratio 0.2, c) Poisson’s ratio 0.25
Figure 5.43 Stress versus change in velocity graphs of group 2 samples (Polynomial fit),

a) No Poisson’s ratio, b) Poisson’s ratio 0.2, c) Poisson’s ratio 0.25
Figure 5.44 Stress versus change in velocity graphs of group 2 samples (Exponential fit),
a) No Poisson’s ratio, b) Poisson’s ratio 0.2, c) Poisson’s ratio 0.25
Figure 5.45 Stress versus change in velocity graphs of group 3 samples (Linear fit),
a) No Poisson’s ratio, b) Poisson’s ratio 0.2, c) Poisson’s ratio 0.25
Figure 5.46 Stress versus change in velocity graphs of group 3 samples (Polynomial fit),

a) No Poisson ratio, b) Poisson ratio 0.2, c) Poisson ratio 0.25
Figure 5.47 Stress versus change in velocity graphs of group 3 samples (Exponential fit),

a) No Poisson’s ratio, b) Poisson’s ratio 0.2, c) Poisson’s ratio 0.25
Figure 5.48 Stress versus change in velocity graphs of group 4 samples (Linear fit),
a) No Poisson’s ratio, b) Poisson’s ratio 0.2, c) Poisson’s ratio 0.25
Figure 5.49 Stress versus change in velocity graphs of group 4 samples (Polynomial fit),
   a) No Poisson’s ratio, b) Poisson’s ratio 0.2, c) Poisson’s ratio 0.25
Figure 5.50 Stress versus change in velocity graphs of group 4 samples (Exponential fit),

a) No Poisson’s ratio, b) Poisson’s ratio 0.2, c) Poisson’s ratio 0.25
Table 5.8 Applied stress versus ΔV for group 2 specimens

<table>
<thead>
<tr>
<th>Type of fitted curve</th>
<th>Poisson’s ratio</th>
<th>Equation</th>
<th>R-square</th>
</tr>
</thead>
<tbody>
<tr>
<td>Linear</td>
<td>0.2</td>
<td>Δstrength = -173.16 + 73.63ΔV</td>
<td>0.93443</td>
</tr>
<tr>
<td></td>
<td>0.25</td>
<td>Δstrength = -173.16 + 73.25ΔV</td>
<td>0.93516</td>
</tr>
<tr>
<td></td>
<td>0.2</td>
<td>Δstrength = -136.58 + 11.82ΔV + 1.52(ΔV)^2</td>
<td>0.9638</td>
</tr>
<tr>
<td></td>
<td>0.25</td>
<td>Δstrength = -133.62 + 12.86ΔV + 1.42(ΔV)^2</td>
<td>0.96722</td>
</tr>
<tr>
<td></td>
<td>0.2</td>
<td>Δstrength = 399.03 exp(ΔV/19.67) - 240.58</td>
<td>0.97523</td>
</tr>
<tr>
<td></td>
<td>0.25</td>
<td>Δstrength = 429.97 exp(ΔV/19.69) - 276.26</td>
<td>0.97734</td>
</tr>
<tr>
<td></td>
<td>0.2</td>
<td>Δstrength = 437.93 exp(ΔV/19.95) - 285.397</td>
<td>0.97782</td>
</tr>
<tr>
<td></td>
<td>0.25</td>
<td>Δstrength = 437.93 exp(ΔV/19.95) - 285.397</td>
<td>0.97782</td>
</tr>
</tbody>
</table>

Table 5.9 Applied stress versus ΔV for group 3 specimens

<table>
<thead>
<tr>
<th>Type of fitted curve</th>
<th>Poisson’s ratio</th>
<th>Equation</th>
<th>R-square</th>
</tr>
</thead>
<tbody>
<tr>
<td>Linear</td>
<td>0.2</td>
<td>Δstrength = -71.19 + 77.95ΔV</td>
<td>0.93597</td>
</tr>
<tr>
<td></td>
<td>0.25</td>
<td>Δstrength = -71.795 + 75.87ΔV</td>
<td>0.93958</td>
</tr>
<tr>
<td></td>
<td>0.2</td>
<td>Δstrength = 169.98 + 16.35ΔV + 1.6(ΔV)^2</td>
<td>0.96581</td>
</tr>
<tr>
<td></td>
<td>0.25</td>
<td>Δstrength = 166.71 + 17.27ΔV + 1.48(ΔV)^2</td>
<td>0.96896</td>
</tr>
<tr>
<td></td>
<td>0.2</td>
<td>Δstrength = 461.58 exp(ΔV/18.81) - 267.5</td>
<td>0.97658</td>
</tr>
<tr>
<td></td>
<td>0.25</td>
<td>Δstrength = 429.97 exp(ΔV/19.67) - 276.26</td>
<td>0.97734</td>
</tr>
<tr>
<td></td>
<td>0.2</td>
<td>Δstrength = 507.33 exp(ΔV/20.22) - 319.38</td>
<td>0.97885</td>
</tr>
<tr>
<td></td>
<td>0.25</td>
<td>Δstrength = 507.33 exp(ΔV/20.22) - 319.38</td>
<td>0.97885</td>
</tr>
</tbody>
</table>

Table 5.10 Applied stress versus ΔV for group 4 specimens

<table>
<thead>
<tr>
<th>Type of fitted curve</th>
<th>Poisson’s ratio</th>
<th>Equation</th>
<th>R-square</th>
</tr>
</thead>
<tbody>
<tr>
<td>Linear</td>
<td>0.2</td>
<td>Δstrength = -132.005 + 77.16ΔV</td>
<td>0.93163</td>
</tr>
<tr>
<td></td>
<td>0.25</td>
<td>Δstrength = -130.552 + 75.12ΔV</td>
<td>0.93554</td>
</tr>
<tr>
<td></td>
<td>0.2</td>
<td>Δstrength = 159.53 + 12.99ΔV + 1.61(ΔV)^2</td>
<td>0.96261</td>
</tr>
<tr>
<td></td>
<td>0.25</td>
<td>Δstrength = 156.66 + 14.99ΔV + 1.51(ΔV)^2</td>
<td>0.96541</td>
</tr>
<tr>
<td></td>
<td>0.2</td>
<td>Δstrength = 429.97 exp(ΔV/19.69) - 276.26</td>
<td>0.97734</td>
</tr>
<tr>
<td></td>
<td>0.25</td>
<td>Δstrength = 437.62 exp(ΔV/19.327) - 258.97</td>
<td>0.97748</td>
</tr>
</tbody>
</table>
For the remaining groups of concrete with different compressive strength values, the results showed that the best curve fit is the exponential curve that has the highest R-square value, as compared to other types of curves.

To see the effect of $f'_c$ on the stress-$\Delta V$ relationship in the maximum amplitude window (Second window), all stress-$\Delta V$ relationship data points from each group of concrete were collected on one graph. Results showed that, although the values of $f'_c$ are different in different groups, it is still possible to express the relationship between stress and $\Delta V$ with good accuracy with only one exponential function. In Figures 5.51 to 5.53, results are shown for three different values of Poisson’s ratio.

![Figure 5.51 Stress versus change in velocity graphs for all groups with No Poisson’s ratio](image-url)
Since the value of $f'_c$ doesn’t have that much effect on the data points, one can simply express the relationship between the stress and ΔV using only one equation for all groups of specimens with different compressive strengths ($f'_c$). Also, by comparing the accuracy of different fitted curve types, it was observed that for all groups of specimens, the exponential curve was the most appropriate fit. Therefore, the results shown in Figures 5.54 to 5.56 are simply the exponential fitting curves for different values of Poisson’s ratio.
Figure 5.54 Stress versus change in velocity for all the groups in second window, No Poisson’s ratio

Figure 5.55 Stress versus change in velocity for all the groups in second window, Poisson’s ratio 0.2

Figure 5.56 Stress versus change in velocity for all the groups in second window, Poisson’s ratio 0.25
The equations that can be used to find the stress based on the velocity changes are given in Table 5.11.

**Table 5.11 Applied stress versus ΔV for all groups in window 2**

<table>
<thead>
<tr>
<th>Samples</th>
<th>Poisson’s ratio</th>
<th>Equation</th>
<th>R-square</th>
</tr>
</thead>
<tbody>
<tr>
<td>All groups of Plain concrete samples</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>No Poisson’s ratio</td>
<td>Δstrength = 493.01exp(ΔV/19.77) – 327.47</td>
<td>0.97128</td>
<td></td>
</tr>
<tr>
<td>0.2</td>
<td>Δstrength = 522.254exp(ΔV/20.756) – 359.997</td>
<td>0.97354</td>
<td></td>
</tr>
<tr>
<td>0.25</td>
<td>Δstrength = 529.85exp(ΔV/21.01) – 368.43</td>
<td>0.97407</td>
<td></td>
</tr>
</tbody>
</table>

Up to this point, the aforementioned results corresponded to the first arrival and maximum amplitude windows. To see if the effect of applied stress on the change in velocity is the same as for previously chosen windows, another window that was far from the first arrival time in the transient signals was selected, with less amplitude compared to the maximum amplitude window. The results of this window are explained in greater detail below.

5.4.2.3 Third Window (Window far from the first arrival time)

Based on the information given in Table 5.5, to carry out the test for this specific window, eight samples were selected: M1-3, M1-4, M2-3, M2-4, M3-3, M3-4, M4-3 and M4-4. These samples were from different groups of concrete with different $f_{c}'$ values.

The chosen window in the transient signal and also the effect of applied load on the change in TOF are shown in Figures 5.57 and 5.58, respectively.
Figure 5.57 Third window (window far from first arrival time)

Figure 5.58 TOF shift due to applied load for third window

It can be seen from Figure 5.58, by choosing the window that was far from the first arrival, and had less amplitude in transient signal, the pattern on the change in TOF didn’t change. With increasing applied load (stress), the time of flight (TOF) decreased.

The test was performed for different groups of concrete choosing the third window as the appropriate window. Figures 5.59 to 5.62 show the relationship between the applied load and the change in TOF in window 3 for groups 1, 2, 3 and 4 of the specimens, respectively.
Figure 5.59 Applied load versus change in time of flight relationship for group 1 specimens
Figure 5.60 Applied load versus change in time of flight relationship for group 2 specimens
Figure 5.61 Applied load versus change in time of flight relationship for group 3 specimens
Figure 5.62 Applied load versus change in time of flight relationship for group 4 specimens
If the TOF decreases, the velocity increases. The test results showed that increasing the stress on the structure led to an increase in the velocity, the same as what happened in the second window. The relationship between stress and change in velocity are shown in Figures 5.63 to 5.74 for groups 1, 2, 3 and 4 specimens respectively.

![Diagram](image1.png)

**Figure 5.63 Stress versus change in velocity relationship for group 1 specimens (Linear fit)**

a) No Poisson’s ratio

b) Poisson’s ratio 0.2

c) Poisson’s ratio 0.25
Figure 5.64 Stress versus change in velocity relationship for group 1 specimens (Polynomial fit)
Figure 5.65 Stress versus change in velocity relationship for group 1 specimens (Exponential fit)
Figure 5.66 Stress versus change in velocity relationship for group 2 specimens (Linear fit)
Figure 5.67 Stress versus change in velocity relationship for group 2 specimens (Polynomial fit)
Figure 5.68 Stress versus change in velocity relationship for group 2 specimens (Exponential fit)
Figure 5.69 Stress versus change in velocity relationship for group 3 specimens (Linear fit)
Figure 5.70 Stress versus change in velocity relationship for group 3 specimens (Polynomial fit)
Figure 5.71 Stress versus change in velocity relationship for group 3 specimens (Exponential fit)
Figure 5.72 Stress versus change in velocity relationship for group 4 specimens (Linear fit)
Figure 5.73 Stress versus change in time of flight relationship for group 4 specimens (Polynomial fit)
Figure 5.74 Stress versus change in time of flight relationship for group 4 specimens (Exponential fit)
For groups 1, 2, 3, and 4 samples with the average strength of 3616, 4263, 4899 and 5759 psi, respectively, the equations that can be used to find the applied stress on the sample based on the velocity change for window 3 are tabulated in Tables 5.12 to 5.15.

Comparing the results shows that the best fit to express the relationship between stress and change in velocity is an exponential curve with the highest R-square value. Also by observing the graphs for each type of curve, the Poisson’s ratio does have an effect on the results, but this effect is not significant.

### Table 5.12 Applied stress versus ΔV for group 1 specimens, Window3

<table>
<thead>
<tr>
<th>Type of fitted curve</th>
<th>Poisson’s ratio</th>
<th>Equation</th>
<th>R-square</th>
</tr>
</thead>
<tbody>
<tr>
<td>Linear</td>
<td>No Poisson’s ratio</td>
<td>$\Delta \text{strength} = -54.898 + 171.57\Delta V$</td>
<td>0.98185</td>
</tr>
<tr>
<td></td>
<td>Poisson’s ratio 0.2</td>
<td>$\Delta \text{strength} = -53.88 + 168.83\Delta V$</td>
<td>0.98294</td>
</tr>
<tr>
<td></td>
<td>Poisson’s ratio 0.25</td>
<td>$\Delta \text{strength} = -53.63 + 165.68\Delta V$</td>
<td>0.9832</td>
</tr>
<tr>
<td>Polynomial</td>
<td>No Poisson’s ratio</td>
<td>$\Delta \text{strength} = 42.118 + 115.69\Delta V + 3.76(\Delta V)^2$</td>
<td>0.98864</td>
</tr>
<tr>
<td></td>
<td>Poisson’s ratio 0.2</td>
<td>$\Delta \text{strength} = 40.61 + 114.96\Delta V + 3.45(\Delta V)^2$</td>
<td>0.98938</td>
</tr>
<tr>
<td></td>
<td>Poisson’s ratio 0.25</td>
<td>$\Delta \text{strength} = 40.24 + 113.65\Delta V + 3.37(\Delta V)^2$</td>
<td>0.98956</td>
</tr>
<tr>
<td>Exponential</td>
<td>No Poisson’s ratio</td>
<td>$\Delta \text{strength} = 2813.41\exp\left(\frac{\Delta V}{23.81}\right) - 2774.8$</td>
<td>0.98894</td>
</tr>
<tr>
<td></td>
<td>Poisson’s ratio 0.2</td>
<td>$\Delta \text{strength} = 2668.256\exp\left(\frac{\Delta V}{22.1}\right) - 2627.87$</td>
<td>0.98966</td>
</tr>
<tr>
<td></td>
<td>Poisson’s ratio 0.25</td>
<td>$\Delta \text{strength} = 2813.41\exp\left(\frac{\Delta V}{23.81}\right) - 2774.8$</td>
<td>0.98983</td>
</tr>
</tbody>
</table>

### Table 5.13 Applied stress versus ΔV for group 2 specimens, Window3

<table>
<thead>
<tr>
<th>Type of fitted curve</th>
<th>Poisson’s ratio</th>
<th>Equation</th>
<th>R-square</th>
</tr>
</thead>
<tbody>
<tr>
<td>Linear</td>
<td>No Poisson’s ratio</td>
<td>$\Delta \text{strength} = -136.707 + 165.49\Delta V$</td>
<td>0.97487</td>
</tr>
<tr>
<td></td>
<td>Poisson’s ratio 0.2</td>
<td>$\Delta \text{strength} = -134.22 + 160.2\Delta V$</td>
<td>0.97626</td>
</tr>
<tr>
<td></td>
<td>Poisson’s ratio 0.25</td>
<td>$\Delta \text{strength} = -133.62 + 159.83\Delta V$</td>
<td>0.97659</td>
</tr>
<tr>
<td>Polynomial</td>
<td>No Poisson’s ratio</td>
<td>$\Delta \text{strength} = 12.269 + 88.49\Delta V + 4.98(\Delta V)^2$</td>
<td>0.98898</td>
</tr>
<tr>
<td></td>
<td>Poisson’s ratio 0.2</td>
<td>$\Delta \text{strength} = 11.017 + 87.92\Delta V + 4.59(\Delta V)^2$</td>
<td>0.98973</td>
</tr>
<tr>
<td></td>
<td>Poisson’s ratio 0.25</td>
<td>$\Delta \text{strength} = 10.71 + 87.79\Delta V + 4.5(\Delta V)^2$</td>
<td>0.98993</td>
</tr>
<tr>
<td>Exponential</td>
<td>No Poisson’s ratio</td>
<td>$\Delta \text{strength} = 1548.15\exp\left(\frac{\Delta V}{15.56}\right) - 1540.31$</td>
<td>0.98973</td>
</tr>
<tr>
<td></td>
<td>Poisson’s ratio 0.2</td>
<td>$\Delta \text{strength} = 1617.43\exp\left(\frac{\Delta V}{16.67}\right) - 1610.7$</td>
<td>0.99044</td>
</tr>
<tr>
<td></td>
<td>Poisson’s ratio 0.25</td>
<td>$\Delta \text{strength} = 1634.84\exp\left(\frac{\Delta V}{16.9}\right) - 1628.38$</td>
<td>0.99061</td>
</tr>
</tbody>
</table>
Table 5.14 Applied stress versus ΔV for group 3 specimens, Window3

<table>
<thead>
<tr>
<th>Type of fitted curve</th>
<th>Poisson’s ratio</th>
<th>Equation</th>
<th>R-square</th>
</tr>
</thead>
<tbody>
<tr>
<td>Linear</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>No Poisson’s ratio</td>
<td></td>
<td>Δstrength = −60.43 + 165.31ΔV</td>
<td>0.98819</td>
</tr>
<tr>
<td>Poisson’s ratio 0.2</td>
<td></td>
<td>Δstrength = −58.76 + 160.73ΔV</td>
<td>0.98877</td>
</tr>
<tr>
<td>Poisson’s ratio 0.25</td>
<td></td>
<td>Δstrength = −58.36 + 159.62ΔV</td>
<td>0.98891</td>
</tr>
<tr>
<td>Polynomial</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>No Poisson’s ratio</td>
<td></td>
<td>Δstrength = 18.42 + 124.29ΔV + 2.65(ΔV)²</td>
<td>0.99262</td>
</tr>
<tr>
<td>Poisson’s ratio 0.2</td>
<td></td>
<td>Δstrength = 18.53 + 121.62ΔV + 2.46(ΔV)²</td>
<td>0.99305</td>
</tr>
<tr>
<td>Poisson’s ratio 0.25</td>
<td></td>
<td>Δstrength = 18.55 + 120.97ΔV + 2.42(ΔV)²</td>
<td>0.99315</td>
</tr>
<tr>
<td>Exponential</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>No Poisson’s ratio</td>
<td></td>
<td>Δstrength = 3843.59 exp (ΔV/30.27) − 3825.82</td>
<td>0.99279</td>
</tr>
<tr>
<td>Poisson’s ratio 0.2</td>
<td></td>
<td>Δstrength = 3940.07 exp (ΔV/31.75) − 3922.12</td>
<td>0.99322</td>
</tr>
<tr>
<td>Poisson’s ratio 0.25</td>
<td></td>
<td>Δstrength = 3964.15 exp (ΔV/32.12) − 3946.16</td>
<td>0.99332</td>
</tr>
</tbody>
</table>

Table 5.15 Applied stress versus ΔV for group 4 specimens, Window3

<table>
<thead>
<tr>
<th>Type of fitted curve</th>
<th>Poisson’s ratio</th>
<th>Equation</th>
<th>R-square</th>
</tr>
</thead>
<tbody>
<tr>
<td>Linear</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>No Poisson’s ratio</td>
<td></td>
<td>Δstrength = −99.66 + 161.47ΔV</td>
<td>0.97311</td>
</tr>
<tr>
<td>Poisson’s ratio 0.2</td>
<td></td>
<td>Δstrength = −97.51 + 157.09ΔV</td>
<td>0.97465</td>
</tr>
<tr>
<td>Poisson’s ratio 0.25</td>
<td></td>
<td>Δstrength = −96.98 + 156.03ΔV</td>
<td>0.97501</td>
</tr>
<tr>
<td>Polynomial</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>No Poisson’s ratio</td>
<td></td>
<td>Δstrength = 40.46 + 88.66ΔV + 4.69(ΔV)²</td>
<td>0.98685</td>
</tr>
<tr>
<td>Poisson’s ratio 0.2</td>
<td></td>
<td>Δstrength = 38.83 + 88.26ΔV + 4.31(ΔV)²</td>
<td>0.98769</td>
</tr>
<tr>
<td>Poisson’s ratio 0.25</td>
<td></td>
<td>Δstrength = 38.44 + 88.15ΔV + 4.22(ΔV)²</td>
<td>0.98789</td>
</tr>
<tr>
<td>Exponential</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>No Poisson’s ratio</td>
<td></td>
<td>Δstrength = 1635.48 exp (ΔV/16.52) − 1600.36</td>
<td>0.98745</td>
</tr>
<tr>
<td>Poisson’s ratio 0.2</td>
<td></td>
<td>Δstrength = 1713.11 exp (ΔV/17.62) − 1679.34</td>
<td>0.98824</td>
</tr>
<tr>
<td>Poisson’s ratio 0.25</td>
<td></td>
<td>Δstrength = 1732.6 exp (ΔV/17.87) − 1699.16</td>
<td>0.98843</td>
</tr>
</tbody>
</table>

For this window, which covers a time range that is far from the first arrival time, all the data points for the stress-ΔV relationship from each group are drawn in one graph to see if a change in $f'_c$ has any effect on the fitted curve. The same results were obtained as explained for window 2. Although the value of $f'_c$ is different in each group, the relationship between stress and ΔV can still be expressed by one exponential equation. In Figures 5.75 to 5.77 the results are shown for three different values of the Poisson’s ratio.
Figure 5.75 Stress versus change in velocity graphs for all groups with No Poisson’s ratio

Figure 5.76 Stress versus change in velocity graphs for all groups with Poisson’s ratio 0.2
Since the value of $f_c'$ doesn’t have much effect on the data points, we can simply express the relationship between stress and $\Delta V$ by using just one equation for all groups of specimens with different compressive strength ($f_c'$) values. Also, by comparing the accuracy of different curve types, it was observed that for all groups of specimens, an exponential curve fit was the best. Therefore, the results shown in Figure 5.78 are just for the exponential curve fit for different values of Poisson’s ratio. The results are summarized in Table 5.16.
Figure 5.78 Stress versus change in velocity for all the groups in third window for different values of Poisson's ratio
Table 5.16 Applied stress versus $\Delta V$ for all specimens in window 3

<table>
<thead>
<tr>
<th>Type of fitted curve</th>
<th>Poisson’s ratio</th>
<th>Equation</th>
<th>R-square</th>
</tr>
</thead>
<tbody>
<tr>
<td>Exponential fit</td>
<td>No Poisson’s ratio</td>
<td>$Stress = 2322.28 \exp \left( \frac{\Delta V}{20.68} \right) - 2296.92$</td>
<td>0.98587</td>
</tr>
<tr>
<td></td>
<td>Poisson’s ratio 0.2</td>
<td>$Stress = 2410.2 \exp \left( \frac{\Delta V}{21.84} \right) - 2387.64$</td>
<td>0.98666</td>
</tr>
<tr>
<td></td>
<td>Poisson’s ratio 0.25</td>
<td>$Stress = 2432.29 \exp \left( \frac{\Delta V}{22.14} \right) - 2409.92$</td>
<td>0.98685</td>
</tr>
</tbody>
</table>

For window 2, corresponding to the maximum amplitude with time period between 85µs to 105µs, a relationship between stress and $\Delta V$ can be found. For the third window, with time period between 125µs to 175µs (which was far from the first arrival time and had less amplitude compared to window 2), a similar characteristics in the relationship between stress and $\Delta V$ could be obtained. If the results of these two windows are combined and shown in one graph (Figures 5.79 to 5.81), one can observe the differences in the relationship curves.

Figure 5.79 Stress versus change in velocity for 2 different windows, No Poisson’s ratio
By examining the above graphs, it can be seen that if a constant value of stress is chosen, by following the second window with more amplitude in transient signal, there would be more change in the velocity compared to the third window (which had less amplitude).

As explained in greater details in section 5.2, it can be seen that the distinction of the dataset between the applied load (stressed) condition and unloaded condition is
clearer in the frequency domain than in time domain. This is because the change in magnitude of FFT results is more obvious.

When the sample is excited by a chirp signal, several resonant modes are observed in the frequency spectra. In frequency domain, the behavior of samples due to the applied load is shown in Figure 5.82.

![Figure 5.82 Frequency domain of plain concrete sample under different applied load steps](image)

**Table 5.17 Dominant frequency magnitude due to applied load**

<table>
<thead>
<tr>
<th>Applied load (kips)</th>
<th>Magnitude</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>69.81</td>
</tr>
<tr>
<td>3</td>
<td>54.66</td>
</tr>
<tr>
<td>6</td>
<td>50.25</td>
</tr>
<tr>
<td>9</td>
<td>41.66</td>
</tr>
<tr>
<td>12</td>
<td>41.2</td>
</tr>
<tr>
<td>15</td>
<td>37.6</td>
</tr>
<tr>
<td>18</td>
<td>26.8</td>
</tr>
<tr>
<td>21</td>
<td>22.96</td>
</tr>
<tr>
<td>24</td>
<td>19.37</td>
</tr>
</tbody>
</table>
This figure clearly indicates that, when the load is increased, the dominant frequency magnitude is reduced and the magnitude of the other frequencies is increased. The reduction value in dominant frequency can be obtained in tabular form, as given in Table 5.17. This can be a sign of increasing nonlinearity in the concrete (stress-train relationship) due to the applied load.
5.5 Objective 4: The effect of applied load on UPV value for polymer concrete

5.5.1 General

As was observed for plain concrete, the applied load (stress) has an effect on the measured velocity. Therefore, to find the strength of the concrete based on the UPV value, the applied load on the sample should be considered as an important factor that cannot be neglected. With this as the primary objective, attempts were made to find the effect of applied stress on the velocity change in polymer concrete specimens.

5.5.2 Experimental investigation and results

To achieve this goal, five samples of polymer concrete were tested in order to investigate the effect of applied load on UPV value for polymer concrete where three different windows were chosen in the transient signal, as shown in Figure 5.83.

The first window was the window near the first arrival time with a time period of 30μs to 50μs. The second window, which covered the maximum amplitude of the transient signal, was within the time period of 50μs to 75μs. Finally, the third window was the window far from the first arrival time with the time period of 100μs to 120μs.

To carry out the test, by applying the load on the specimen up to almost 70% of specimens’ compressive strength ($f_{pc}^\prime$), the time of flight (TOF) was measured at every step of loading, and it was compared with the TOF of stress-free samples. Results showed that the time of flight measured in each step of axial loading is different. Therefore, to find the strength of the concrete based on the UPV value, the applied load on the sample must be considered.
To achieve our goal under this objective, five cylindrical polymer concrete samples (Figure 5.84), were made with the properties given in Chapter 4.
The measured strength value of each sample is tabulated below (Table 5.18):

<table>
<thead>
<tr>
<th>sample #</th>
<th>M1-1</th>
<th>M2-1</th>
<th>M3-1</th>
<th>M4-1</th>
<th>M5-1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample strength (psi)</td>
<td>7645</td>
<td>9304</td>
<td>9927</td>
<td>8702</td>
<td>9253</td>
</tr>
</tbody>
</table>

To carry out the test, each group of samples was tested by the ultrasonic wave technique (Figure 5.85) as explained in detail in section 5.4.
The surface of the polymer concrete was prepared before attaching the PZTs (Figure 5.86).

![Figure 5.86 Special grinding machine for surface preparation](image)

The same type of transducer as explained in section 5.4 for testing plain concrete was used. Broadband Conrad transducers (EPZ-20MS64, Item Number 712918-62) were used as transmitters and receivers. Two thin disk type PZT transducers were glued to the sample as shown in Figure 5.87.

![Figure 5.87 PZT glued to the sample](image)

A linear chirp was used to excite the transmitter placed on the samples in the frequency range varying from 45 kHz to 175 kHz. By generating and receiving chirp
signals for every group of samples under the specified applied load, the change in TOF was measured for each time window of the transient signals.

To apply the compressive load on the polymer concrete specimens, the MTS 311 Load Frame (Figure 5.88) was used with a 0.025 mm/sec (0.001 in/sec) load rate.

Figure 5.88 Polymer Concrete specimen with Extensometer under MTS Load Frame 311

5.5.2.1 First window (window of first arrival) results

As previously mentioned, by selecting the time window of transient signal for the sample at zero kip load, the transient signal and the corresponding TOF were recorded. By choosing this TOF as the reference time of flight, we could easily find the change in TOF due to the applied load in each step of loading.

To carry out the test for this specific window of transient signal, sample M1-1 was selected. This specimen had the $f_{pc}'$ value of 7645 psi.
In Figure 5.89 the relationship between applied load and the change in TOF is shown. It is clear in the Figure that by considering the first window which covered the first arrival of the received signal, no significant change is observed in TOF by applying the load. Therefore, applied load (stress) didn’t have any significant effect on the time of flight shift or velocity change.

The linear curve fit equation, given in Equation 5.12, had the R-square value of 0.992.

\[
\Delta TOF = 0.00108 \times \text{applied load} + 3.75237 \times 10^{-4}
\] (5.12)

It is clear in Figure 5.89 or Equation 5.12 that by applying almost 45 kips of load the TOF increased less than 0.05 µs. This increase is due to the Poisson’s effect. Because, by applying the longitudinal compressive load, there is an expansion in the direction
perpendicular to the direction of applied load. This lateral expansion increases the distance between the PZTs and due to this distance change the TOF increases.

5.5.2.2 Second Window (Window of maximum amplitude) results

The specimens used to carry out the test for this specific window were M2-1 and M3-1 polymer concrete samples.

![Figure 5.90 Second Window (window of maximum amplitude signal)](image)

It is clear in Figure 5.90 that by choosing the window related to the maximum amplitude of transient signal, with increasing applied load (stress), the change in the time of flight didn’t follow any significant pattern.

By observing the stress-ΔV relationship graph, it can be concluded that by applying the load (stress) on the specimens, when the applied load is increased to 30% of the final load capacity of the sample, there is no change in velocity. By increasing the
applied load up to 70%, it is observed that the more stress there is, the lower the velocity. This result is shown in Figure 5.91.

Another observation from the above graph is that a change in the value of $f_{pc}'$ doesn’t have any effect on the stress-$\Delta V$ relationship.

Up to this point, the aforementioned results corresponded to the first arrival and the maximum amplitude windows. Unfortunately, no pattern was observed between the stress and $\Delta V$ value. To see if the effect of applied stress on the change in velocity is the same as previously selected windows, a different window far from the first arrival time in transient signal was selected with smaller amplitude compared to the maximum amplitude. The results of this window are given below in more detail.
5.5.2.3 Third Window (Window far from first arrival time) results

To see the behavior of polymer concrete samples in this time window of transient signal, two specimens were tested: M4-1 and M5-1.

The selected window in transient signal and also the effect of the applied load on the change in TOF are shown in Figures 5.92 and 5.93.

![Figure 5.92 Window 3 far from first arrival time](image1)

![Figure 5.93 TOF shift due to applied load for third window](image2)

From Figure 5.93, by choosing a window which is far from first arrival and has less amplitude in transient signal, a good pattern to express the relationship between applied load and change in TOF can be found. It was found that with increasing applied load (stress), the time of flight (TOF) decreased.
This test was performed in the third window for two polymer concrete specimens with different compressive strengths. Figure 5.94 shows the relationship between the applied load and change in TOF for this window.

By converting the load to stress and finding the ΔV values from change in TOF, the relationship between stress and ΔV can be plotted as shown in Figures 5.95 and 5.96. By increasing the stress, the change in velocity value increased.

It should be mentioned that these graphs are plotted for two different Poisson’s ratios: zero Poisson’s ratio and Poisson’s ratio of 0.3. Also, to fit the appropriate graph on the data points, polynomial and exponential curves were used. Results showed that the best curve is the exponential fit type with the highest R-square value, which can cover the data points of high stress values.
Figure 5.95 Stress versus change in velocity relationship for window 3 (Polynomial fit)

a) No Poisson’s ratio

b) Poisson’s ratio 0.3
To generate the above graphs, the collected data points were obtained from two different samples with different compressive strength. As can clearly be seen in the graphs, the value of $f_{pc}'$ doesn’t have any effect on the stress-ΔV relationship.
The equations of the curves drawn based on the best fit are summarized in Table 5.19.

**Table 5.19 Applied stress versus ΔV for window 3 specimens**

<table>
<thead>
<tr>
<th>Type of fitted curve</th>
<th>Poisson’s ratio</th>
<th>Equation</th>
<th>R-square</th>
</tr>
</thead>
<tbody>
<tr>
<td>Polynomial</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>No Poisson’s ratio</td>
<td></td>
<td>$\text{Stress} = 212.89 - 13.93 \Delta V + 17.26(\Delta V)^2$</td>
<td>0.93698</td>
</tr>
<tr>
<td>Poisson’s ratio 0.3</td>
<td></td>
<td>$\text{Stress} = 186.8 + 5.98 \Delta V + 11.57(\Delta V)^2$</td>
<td>0.96176</td>
</tr>
<tr>
<td>Exponential</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>No Poisson’s ratio</td>
<td></td>
<td>$\text{Stress} = 195.29 \exp\left(\frac{\Delta V}{4.886}\right) + 58.985$</td>
<td>0.95611</td>
</tr>
<tr>
<td>Poisson’s ratio 0.3</td>
<td></td>
<td>$\text{Stress} = 374.04 \exp\left(\frac{\Delta V}{7.24}\right) - 179.95$</td>
<td>0.97454</td>
</tr>
</tbody>
</table>
5.6 Objective 5: Nondestructive Detection of Nonlinear Behavior of Plain and Polymer Concrete under Cycling Loading

5.6.1 General

One of the characteristic parameters that can be compared for plain and polymer concrete is the behavior under cyclic loading. Under cyclic loadings, pre-existing cracks inside both plain and polymer concrete may grow and cause catastrophic failure of the structure. This study investigates the behavior of polymer concrete under cyclic loading or fatigue.

To see the behavior of plain and polymer concrete under cyclic loads, the NIRAS (Nonlinear Impact Resonant Acoustic Spectroscopy) technique was followed. This technique is explained in detail below.

In general, concrete as a result of its intrinsic heterogeneities, can be classified as a Nonlinear Mesoscopic Elastic (NME) material [74]. Cement-based materials exhibit nonlinear elastic behavior in their constitutive relations due to their heterogeneities at different scales, like porosity, microcracks, etc. In the dynamic case, the nonlinear response manifests as generation of anharmonicities, which can be described by the classical theory [75], and as non-equilibrium nonlinear dynamics effects termed as fast dynamics, slow dynamics and conditioning [76]. The NME behavior shown as a frequency shift in their resonant frequencies is known as the fast dynamic effect. Experimental findings have demonstrated that fast dynamic effect is related to hysteresis in the strain-stress relationship [77]. The equation of state describing nonlinear non-classical phenomena (only fast dynamics) can be written as [78].
\[ E = E_o \left[ 1 + \beta \varepsilon + \delta \varepsilon^2 + \alpha (\Delta \varepsilon + \text{sign}(\dot{\varepsilon})) \right] \] (5.16)

Where \( E_o \) is the linear elastic modulus, \( \beta \) and \( \delta \) are the cubic and quartic anharmonicities, \( \varepsilon \) is strain, \( \Delta \varepsilon \) is the strain amplitude, \( \dot{\varepsilon} \) is the strain rate due to hysteresis, and sign is the sign function which is equal to 1 if \( \dot{\varepsilon} > 0 \), -1 if \( \dot{\varepsilon} < 0 \) and 0 if \( \dot{\varepsilon} = 0 \). The hysteresis nonlinearity parameter \( \alpha \) is a measure of the material hysteresis and is related to the fast dynamics as follows [79].

\[ \frac{f_o - f}{f_o} = \alpha \Delta \varepsilon \] (5.17)

Where \( f_o \) is the linear resonance frequency, and \( f \) is the resonance frequency with increasing strain amplitude. Therefore, the resonance frequency becomes strain amplitude dependent as the nonlinearity behavior increases with damage.

NIRAS measurement allows monitoring of a number of resonance frequencies and shift in these frequency values with increasing level of impact energy. The main advantage of obtaining the resonance frequencies by an impact hammer is that the impact has a wide range of frequencies and therefore it excites several modes of vibration simultaneously and their corresponding dynamic nonlinear parameter \( \alpha \) can be obtained. It has been demonstrated that the dynamic nonlinear parameter \( \alpha \) is a sensitive indicator of damage in cement based materials subjected to compressive mechanical damage [80].

As was indicated before, this study aims to investigate the behavior of plain and polymer concrete under the cyclic loading or fatigue. After the loading-unloading cycles on the samples, NIRAS test was carried out on the specimens using instrumented impact
hammer. NIRAS response from conventional concrete and polymer concrete specimens were recorded to compare their nonlinear behaviors.

5.6.2 Experimental investigation

Cylindrical plain and polymer concrete specimens of 3 inch diameter and 6 inch in length were made as shown in Figure 5.97.

Polymer concrete contains aggregates and epoxy resins. The importance of using the right type and quality of aggregates cannot be ignored as they generally occupy about 80% of the polymer concrete volume and strongly influence the fresh and hardened concrete behavior. Three different types of aggregate were used to find the maximum dry density. The three aggregate types were Gravel8 (G8), Gravel16 (G16) and sand.

To have the maximum dry density many combinations of these three types of aggregate were used. Among all these combinations, the combination which contained 40% sand and 60% G8 had the maximum dry density and it was used as the aggregate part of the polymer concrete in this study.
The epoxy resin which was used had two components or ingredients. Two parts of component “A” to one part of component “B” by volume were mixed to obtain the epoxy resin. The epoxy resin after its curing time becomes a solid with low toxicity and had low odor during the curing process. The curing time depends on the temperature and this time is usually 72 hours; therefore, the curing time for polymer concrete is nine times faster than that of the plain concrete. The density of the epoxy resin is approximately 1.1kg/L. In practice, a hardener is referred to as part B and the resin is part A. Aggregate and epoxy resin were mixed together with a volume ratio of 4:1. Also to make plain concrete, two types of aggregates and regular Portland cement were used. The properties and specification of the material used to make plain and polymer concrete for this objective were explained in detail in Chapter 4. A total of seven polymer concrete and seven plain concrete specimens were used to carry out the tests. Among the seven samples used in this study three were loaded under MTS311 machine to find the compressive strength of the samples. The remaining 4 specimens were used to carry out the NIRAS tests.

The specimens were subjected to compressive load for various numbers of cycles. The applied load was gradually increased up to 36 kips. In order to find the strength of the specimens, standard destructive tests were performed using MTS311 device. After applying the load up to 3, 9, 18 and 36 kips in every step; the samples were unloaded and reloaded a number of times as dictated by the pre-set values of the number of cycles.

After the loading-unloading cycles (2, 7 and 19 cycles) NIRAS test was conducted on plain and polymer concrete specimens using instrumented impact hammer. An impact hammer (Bruel & Jaer 8206, 22.7 mV/N) with an aluminum tip shown in
Figure 5.98 struck the cylindrical sample, in order to excite the resonance frequencies of vibration.

Figure 5.98 An impact hammer with an aluminum tip

The frequency-amplitude dependency is obtained by exciting the resonance frequencies at ten impact levels. An instrumented hammer mounted in a frame acts as a pendulum under the force of gravity and strikes the sample. An accelerometer (0.956 mV/m/s²) then senses the excitation. The computer program acquired 8192 points, with a sampling frequency of 50 kHz. The signals were zero-padded to obtain a frequency step resolution of 0.5 Hz. The test configuration is shown in Figure 5.99.

Figure 5.99 Experiment setup for NIRAS.
The degree of nonlinearity in the concrete specimens was measured by recording the shift in the resonance frequency as the impact energy increased.

The Eigen value problem for the cylindrical specimen of plain and polymer concrete was solved numerically using ANSYS 13.0 to obtain different Eigen-frequencies or resonance frequencies of the specimen considering concrete as a linear isotropic material to extract vibration modes in the frequency range 0 to 15 kHz. The results are shown in Figure 5.100.

<table>
<thead>
<tr>
<th></th>
<th>Plain</th>
<th>Polymer</th>
</tr>
</thead>
<tbody>
<tr>
<td>F1</td>
<td>7509.29 Hz</td>
<td>5942 Hz</td>
</tr>
<tr>
<td>F2</td>
<td>7640.51 Hz</td>
<td>6046 Hz</td>
</tr>
<tr>
<td>F3</td>
<td>11738.3 Hz</td>
<td>9288 Hz</td>
</tr>
<tr>
<td>F4</td>
<td>13588.5 Hz</td>
<td>10755 Hz</td>
</tr>
</tbody>
</table>

Figure 5.100 Eigen-frequencies for plain concrete

(E=29 GPa, Poisson’s ratio=0.2, density= 2300 kg/m³) and polymer concrete (E=15 GPa, Poisson’s ratio=0.2, density= 1900 kg/m³)

considering it to be isotropic material

By comparing the frequency of undamaged sample derived from NIRAS technique and the frequency of the sample derived by modeling the specimen in ANSYS, it is concluded that in the NIRAS method the flexural mode is generated. The resonance frequency from NIRAS technique is close to the resonance frequency corresponding to the flexural vibration mode obtained from ANSYS modeling.
From resonance tests the dynamic modulus of bending \( E_d \) can be obtained [14] knowing the density \( \rho \), the length \( L \), the radius of gyration \( t \) of the sample and the resonance flexural frequency \( F_f \).

\[
E_d = \frac{4\pi L^4 F_f^2 \cdot \rho \cdot 1.401}{4.73^4 \cdot t^2}
\]  

(5.18)

Table 5.20 lists the \( E_d \) values obtained for plain and polymer concrete. By comparing the results for plain and polymer concrete, it can be said that plain concrete is 1.43 times stiffer than polymer concrete.

<table>
<thead>
<tr>
<th>Batch</th>
<th>Density ((\text{kg/m}^3))</th>
<th>( F_f ) ((\text{Hz}))</th>
<th>( E_d ) ((\text{GPa}))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plain concrete</td>
<td>2365</td>
<td>7428.9</td>
<td>29.15</td>
</tr>
<tr>
<td>Polymer concrete</td>
<td>1927</td>
<td>5957.6</td>
<td>20.37</td>
</tr>
</tbody>
</table>

Figures 5.10a and 5.10b show the frequency spectra obtained by NIRAS for plain and polymer concrete and for undamaged and loaded specimens. After applying load two main observations could be made: (i) The flexural frequency decreased when the load protocol was applied to plain concrete samples, while it remained almost unchanged for polymer concrete, and (ii) the resonance frequency for the loaded plain concrete sample decreased with increasing amplitude indicating increasing material nonlinearity due to the accumulated damage, while for polymer concrete sample this did not happen.

For all levels of applied load in polymer concrete no dependency between resonance frequency and excitation amplitude was observed implying the material remained almost linear (Figures 5.101b and 5.102b). The results obtained suggest that the
fracture mechanisms of plain concrete are different from those of the polymer concrete. During cyclic loading, micro-cracks are expected to occur in plain concrete increasing the mechanisms that activate fast dynamic effect and material nonlinearity. However such micro-cracks do not occur in polymer concrete and it remains linear. Micro-cracks also reduced the stiffness of the plain concrete sample that created about 0.5 kHz drop in the resonance frequency.

![Figure 5.101 NIRAS frequency spectra for undamaged and damaged samples](image1)

**Figure 5.101** NIRAS frequency spectra for undamaged and damaged samples

(a) Plain concrete, b) Polymer concrete.

![Figure 5.102 Resonance frequency-amplitude dependence for undamaged and damaged samples](image2)

**Figure 5.102** Resonance frequency-amplitude dependence for undamaged and damaged samples

(a) Plain concrete, b) Polymer concrete.
When the samples are loaded, the constitutive properties of the material are modified. The Chaboche & Lemaitre model can be considered to describe the damage process [82].

\[ E = E_0 (1 - D) \]  

Where \( E \) is the Young’s modulus of the damaged concrete, \( E_0 \) is the modulus of the undamaged specimen and \( D \) is a parameter that is equal to zero when the sample is undamaged.

Assuming that Poisson’s ratio does not vary between the load steps, the above expression can be expressed as a function of the fundamental resonance frequency as:

\[ D = 1 - \frac{f_{n, \text{cycles}}^2}{f_{0, \text{cycles}}^2} \]  

The squared ratio between damaged \( (f_{n, \text{cycles}}) \) and undamaged resonance frequencies \( (f_{0, \text{cycles}}) \) is also known as the relative modulus of elasticity [81].

In Figures 103a and 103b, the changes in D values with the number of cycles are presented. From these results one can see that changing the number of load cycles has an effect on plain concrete but its effect on polymer concrete is insignificant.
Figure 5.10 Damage parameter $D$, computed as $D = 1 - \left( \frac{f_{n\text{ cycles}}}{f_{0\text{ cycle}}} \right)^2$ for

(a) Plain concrete, b) Polymer concrete.

From NIRAS tests the nonlinear hysteretic behaviour of the material can be ascertained by means of the hysteretic parameter $\alpha$. In this study $\alpha$ has been estimated from the acceleration amplitude ($A$) since it is proportional to the strain amplitude ($\Delta\varepsilon$). Hence $\alpha'$ is proportional to $\alpha$. 
\[
\frac{\Delta f}{f_0} \propto \alpha'. A 
\]  

(5.21)

A non-monotonic evolution of the hysteretic parameter \(\alpha'\) with increasing cycles is shown in Figure 104a. Similar trends (first increasing and then decreasing) of \(\alpha'\) were also observed when NIRAS were applied to address alkali-silica reaction [ASR] in concrete samples [82]. The accumulated damage, computed as the integrated curve over the number of cycles is presented in Figure 104b. As were approached for ASR damage [82] the cumulative nonlinearity curve discriminates between different levels of damage.

\begin{figure}
\centering
\includegraphics[width=\textwidth]{figure5104}
\caption{Nonlinear hysteretic evolution for plain concrete, a) Normalized hysteretic parameter, b) Cumulative normalized parameter}
\end{figure}
CHAPTER 6

CONCLUSION

Conclusions from this study are summarized in this section.

Strength development monitoring of different concrete mixes using new ultrasonic method (chirp signal) was the first objective. From the results of this investigation, it was concluded that finding the compressive strength of plain concrete by the conventional ultrasonic pulse velocity method is not very reliable. In other words, the first arrival time is not the best indicator of the concrete strength. The analysis of the entire ultrasonic signal rather than just the first arrival time can give more useful details that can help in determining the concrete strength.

Since the ingredients of concrete samples vary from one sample to another, exciting them with signals having multiple frequencies (chirp signal) have the advantage of choosing the right time window and frequency of transient signals for easier interpretation of the results compared to the single frequency excitation as the PUNDIT device does. Another advantage of the chirp signal method is that the results derived from the PUNDIT device is obtained from the first arrival time only while for the new method any time window inside the received transient signal can be chosen.

When the sample was excited by a chirp signal, several resonance modes were observed in the frequency spectra. By comparing the frequency domain results of different groups of samples a shift in the signal frequency was observed as the samples’ material properties changed. Therefore, it was difficult to interpret the results to find the compressive strength based on the frequency response. This is the reason why the time of
flight was analyzed in this research instead of the frequency to find the change in strength. TOF approach gave a clear relation between the TOF change (velocity change) with the change in strength. To measure the change in the time of flight because of the change in the plain concrete compressive strength, a cross correlation technique was adopted. The cross-correlation between the two time histories can accurately measure the small difference in the time-of-flight (TOF). The results showed that an increase of the compressive strength gives an increase of the wave velocity for the appropriate time window in the transient signal.

By knowing the values of the velocity change \([\Delta(\text{velocity})]\) and the corresponding strength change \([\Delta(\text{strength})]\), appropriate graphs were plotted for the first window of the transient signal in the time domain between 50 µs and 150 µs. By performing the regression analysis, two different types of curves were fitted to the data points: Linear fit and Exponential fit. The R-square values of these two curve types were compared. It was concluded that the linear fit was the best curve to express the relationship between \([\Delta(\text{strength})]\) and \([\Delta(\text{velocity})]\). The results derived using the new method were more accurate compared to the results derived by the conventional PUNDIT device.

Elastic properties of the cement paste and the resin binder are different. Therefore, the ultrasonic wave velocities in polymer concrete are different from those values in plain concrete. As a result the derived graphs relating the velocity change with the strength change for plain concrete cannot be used for polymer concrete. Finding the relationship between the change in strength and the change in velocity of polymer concrete using the chirp signal was the second objective of this research. For polymer concrete specimens, three windows were considered in the transient signal and the results were compared. It
was concluded that the first window which covered the maximum amplitude in the transient signal gave more reliable results compared to the second and third windows that had less amplitude. This conclusion was drawn by comparing the R-square values from the regression analyses of the data in three windows. Also in the frequency domain, the amplitude of the FFT graph is highest in window 1. This can be a reason for why the data points obtained for Δstrength-ΔV relationship graphs are scattered when the window is changed from 1 to 2 or 3. Short Time Fourier Transform (STFT) technique also justified these results.

Researchers have used PUNDIT device to predict the final strength of the concrete, when the samples were free of axial load. This method is not reliable when the specimen is tested under stress because its behavior is quite different from when it is tested without any load. This result was obtained in this investigation by testing many plain and polymer concrete samples. Therefore, this study aimed to investigate the effect of applied load (stress) on the ultrasonic wave velocity passing through the plain and polymer concrete specimens – these were the third and the fourth objectives.

By choosing three different windows in transient signal and comparing the results for these windows, it was concluded that by considering the first window which covers the first arrival of the received signal, for both plain and polymer concrete, no significant change was observed in TOF by applying the load. In other words, applied load (stress) didn’t have significant effect on the time of flight or velocity. Since the measured UPV value was based on the first arrival time collected by PUNDIT device, most researchers involved in the concrete strength measurement neglected this effect of applied load on the UPV value.
By considering the other two windows, window 2 and window 3, the change in the TOF was found to be significant due to the applied load (stress) in plain concrete samples. Use of these two windows was emphasized to study the effects of the applied load (stress) on the ultrasonic wave velocity value. By performing the regression analysis and comparing the accuracy of each type of curve (linear-polynomial and exponential), the exponential fit with the highest R-square value was concluded to be the best fitted curve to express the relationship between stress and change in velocity. Also by observing the graphs for each type of curve fitting, the effects of Poisson’s ratio was taken into the consideration, but this effect was not found to be significant.

To see the effect of \( f'_c \) value on the stress-\( \Delta V \) relationship in windows 2 and 3, all stress-\( \Delta V \) relationship data points from every group of concrete were collected in one graph. It was concluded that although the values of \( f'_c \) were different in different groups, it was still possible to express the relationship between stress and \( \Delta V \) by only one exponential function with good accuracy for every window.

For polymer concrete samples, by choosing the window related to the maximum amplitude of the transient signal (window 2), with increasing applied load (stress), the change in the time of flight did not follow any defined pattern. Even after the applied load increased up to 30% of the final load capacity of the sample, there was no change in velocity. By increasing the applied load to 70%, it was observed that increasing the stress results in decreasing velocity. But choosing the third window which was far from the first arrival and had lower amplitude in the transient signal followed a good pattern as explained for plain concrete.
As the last objective of this study, linear and nonlinear behaviors of plain and polymer concretes were investigated by NIRAS technique. After inducing mechanical damage by applying the cyclic loads of various magnitudes for different numbers of cycles, the following observations were made:

- By applying the load and increasing the number of cycles, the resonance frequency is decreased in plain concrete. Increase of load and number of cycles generate more microcracks in plain concrete and it reduces the stiffness of the samples. As a result, the resonance frequency is reduced. In the polymer concrete, the microcracks are not generated so its stiffness and resonance frequencies do not change under the applied cyclic load.
- In plain concrete, the resonance frequency reduction was observed after two cycles. After two cycles the rate of reduction of the resonance frequency goes down.
- The undamaged plain concrete is found to be a linear material but after a few loading and unloading cycles it becomes a nonlinear material because of the generation of micro-cracks. However, polymer concrete remains linear even after several loading-unloading cycles. The material nonlinearity is measured from the shift in the resonance frequency peaks with the increasing levels of excitation.
- The hysteretic nonlinear parameter $\alpha$ can be reliably estimated from simple laboratory tests on the samples. This parameter can be used to estimate internal damage. In fact the relative changes in the hysteretic nonlinear parameter are far more sensitive to internal damage in concrete than linear parameters such as fundamental resonance frequency.
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