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THE DETECTION OF COMMON CONCRETE BRIDGE DECK DEFECTS USING THERMOGRAPHY, IMPACT ECHO, AND GROUND PENETRATING RADAR

by

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A Thesis
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Saleh Z. Nabulsi
The transportation infrastructure in the United States is deteriorating and will require significant improvements. A recent report by the American Society of Civil Engineers (ASCE) gave an overall grading of D+ to the nation's infrastructure. Approximately 162,000 of the nation's bridges are structurally deficient or functionally obsolete. The total estimated cost to bring conditions to acceptable levels over the next two decades would exceed $1.9 trillion. Consequently, innovations in the area of transportation infrastructure maintenance and rehabilitation are key to the health and wellness of this valuable national asset.

A major component of maintenance and rehabilitation is the ability to accurately assess the condition of the transportation infrastructure. This can be accomplished in part by using nondestructive evaluation (NDE) techniques. NDE can verify the integrity of a structure or any of its components without compromising its ability to perform in service. NDE techniques used for concrete bridge decks are studied. The three most appropriate methods are selected for further study and verification with literature. An experimental study is designed and cases mimicking concrete bridge decks defects such as voids, delaminations, and cracks are presented.

The results of the study showed that Ground Penetrating Radar (GPR), Impact Echo (IE), and Thermography (IR) are promising methods for the detection of potential problems in concrete bridge decks.
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CHAPTER ONE
INTRODUCTION

1.1 Problem Statement

Bridges are one of the most important elements in our highway and transportation network in the United States, this importance lies in the service that they provide and their high impact on safety and the high cost that comes along with building and maintaining them. Therefore, continuous maintenance and inspection of bridges is vital and should be performed according to fixed schedules to check for bridges safety and serviceability.

Today the transportation network in the United States consists of 600 thousand bridges. Most of those were built in the late 1800s and early 1900s because of the high roadway construction intensity in that period. Out of those 600 thousand bridges 13.8 % are considered functionally obsolete and 14.8 % are considered structurally obsolete, the total percentage of obsolete bridges that need to be monitored and maintained in the network is 28.5 %. This is a very high percentage that requires a high level of expertise and outrageous amounts of funds to be allocated in a systematic way that would derive the highest value and benefit from the limited funds available for this purpose.

Inspection is one of the very important tasks in a bridge operation because it provides information about the condition of bridges and their components. Many types of bridge inspection are available depending on the level of detail and the amount of information required.
Among the methods of inspection is Nondestructive testing (NDT), NDT is in its infancy and validation of some of the methods is a vital task. In addition, guidelines for the selection of nondestructive techniques for the case of concrete bridge decks are not available. Several nondestructive techniques for flaw detection in concrete bridge decks are being utilized but a comparison between these methods is not available. Ground Penetrating Radar (GPR), Impact Echo (IE), and Infrared Thermography (IR) are some of the nondestructive techniques used for concrete bridge decks. A comparison between these methods to determine which method works better for the detection of common flaws in concrete bridge decks need to be established.

1.2 Research Objectives

The objectives of the research are as follows:

- Study the different Nondestructive techniques used for concrete bridge decks.
- Select the three most applicable, most promising nondestructive techniques used for concrete bridge decks inspection.
- Validate the capabilities of the selected methods in detecting common defects in bridge decks.
- Provide some guidelines or general criteria on the selection of nondestructive techniques for bridge decks based on the type of anomaly.
- Provide a comparison between the selected methods in terms of their ability to detect common flaws.
- Develop test specimens to validate and verify the selected methods.
- Demonstrate the abilities of the methods using specimens under a controlled environment.
• Make recommendations on which tests are better and what type of flaws each of those techniques could detect.

1.3 Research Methodology

This Research involves studying the different nondestructive techniques used for concrete bridge decks. The focus of the research will be to check the ability of three methods to detect the most common bridge defects. The research will achieve its objectives through the following phases:

• Literature review phase: The first step is to investigate the methods used for concrete bridge evaluation through some literature reviews to check which of the methods are best used for bridge deck inspection. In addition, the literature survey will assist in identifying the most common concrete bridge decks defects.

• Specimen preparation phase: the second step is to prepare test specimens as bridge deck prototypes, planting different types of flaws as cracks, delaminations, and voids to test the abilities of the methods.

• Specimen testing and data collection phase: the third step is to test those prototype bridge decks under controlled environment to find out if the selected methods would be able to detect previously known anomalies.

• Result generation and recommendation phase: the fourth and final step is to study the results obtained from the tests and compare them with known as built situations. The result of the test will be validated against information obtained from the literature survey phase and the capabilities of each of the methods will be discussed.
1.4 Research Layout and Organization

This thesis covers the research findings and will be organized as follows

Chapter Two provides a literature survey that discusses inspection practices, and explains differences between destructive and nondestructive testing.

Chapter Three discusses nondestructive testing. It explains the nondestructive techniques used for concrete bridge decks and the three most appropriate methods for concrete bridge deck evaluation are selected in this chapter.

Chapter Four contained the experimental program designed to validate the selected methods.

Chapter Five covers the Impact Echo (IE) technique, providing extensive literature survey, testing on fabricated concrete specimens, discusses the obtained test results, and describes the potentials of the method for detecting common flaws.

Chapter Six covers the Ground Penetrating Radar (GPR) technique, providing an extensive literature survey. In addition, testing of the specimens using GPR is introduced and results are discussed portraying the ability of the method to detect various common flaws.

Chapter Seven covers the Infrared Thermography (IR) technique, providing a broad literature survey. Results of Thermography tests on fabricated specimens are introduced in this chapter followed by a discussion of the results describing the abilities of IR Thermography to detect several flaws.

Chapter Eight includes a summary of the results and draws conclusions from the testing phase.

Appendix section includes information about the suppliers for NDT equipment.
CHAPTER TWO
INSPECTION

2.1 Introduction

Bridges are a vital component of the transportation network as well as a large capital investment. Inspection and maintenance of the bridges are of the most important tasks related to the safe operation of a bridge. Users expect the bridge to be safe. Thus, bridge engineers strive to ensure that those bridges function safely and ensure their serviceability. A great amount of experience is required to ensure that a bridge is functioning well and that it is not hazardous to the public. Inspection of any structure is performed to determine the amount of maintenance needed if at all required. This is necessary in order to predict the costs associated with repairs and allocate funds. These two are an important input in the bridge management system. The outputs of the inspection activities are recorded in a database where it could be acquired later if needed. Most of the Departments of Transportation (DOT) have bridge management systems where this information is entered for all the bridges in the DOT’s jurisdiction. The bridge management software gives rankings for bridges conditions and prioritizes the work on bridges according to their conditions.

It was just after the 1967 collapse of the silver bridge in Point pleasant that the Federal Highway Administration (FHWA) decided to draw guidelines for the bridge inspection process. Those guidelines require that all bridges on public roadways should be inspected on a periodic basis at least once biannually. FHWA keeps a record of the inspection results in the national bridge inventory database which also includes information about size, design, construction of new bridges.
Since that day and for more than thirty years now, most of the highway agencies rely on visual inspection as the number one method of inspection. However, newer techniques have emerged. A set of other nondestructive methods were introduced. More of the highway departments are adopting those methods for inspection purposes. Chapter three will discuss several of the methods used for evaluation of concrete bridge decks.

2.1 Types of Inspection

There are many types of bridge inspection; the type used in an inspection activity depends on the level of detail and the extent of access to different structural elements. The degree of testing will vary according to which of the inspection types are used. The following sections will discuss the different inspection categories.

2.1.1 Initial Inspection

This type is also called the inventory inspection. This is the first bridge inspection after a bridge becomes part of the national bridge inventory. It also applies to those bridges where a change in the configuration occurred, such as widening, lightning, or supplemental bents or shoring. This type of inspection is a fully documented one performed by certified inspection personnel and is always accompanied by an analytical analysis of the bridge load capacity.

The purpose of the initial or inventory inspection lies in two points.

1. Provides all the structural and appraisal data required by the federal highway administration (FWHA), along with all the other relevant information, not required by FHWA but would be useful for the owner of the bridge.
2. Determines the baseline of the structural conditions and list all existing problems and gives an idea about locations which may be potential problems. This type of inspection gives the information about members which will need more attention and possibly more inspection.

2.1.2 Routine Inspection

This is a regularly scheduled type of inspection. It consists of a sufficient number of observations enabling the bridge engineer to determine the functional and physical condition of the bridge. It is also important to identify developing problems or changes that were not present during the inventory or initial inspection.

This routine inspection must satisfy the requirements of the National Bridge Inspection Standards with respect to inspection frequency. These inspections are usually conducted on decks, ground or water levels, walkways, and special equipment if present.

The outcome of the routine inspection should be documented, accompanied by photographs and a report recommending maintenance and for use in the scheduling of In-Depth inspections.

2.1.3 Damage Inspection

This is an unscheduled inspection to assess structural damage caused by environmental factors. The reason should be enough to determine the need for lane closures or emergency load restrictions. The amount of effort exercised in this type of inspection depends mainly on the type and the extent of the damage. Inspectors should
check for damaged components. Proper documentation is also needed in this type of inspection \(^1,^4\).

### 2.1.4 In-Depth Inspection

The In-Depth inspection method is used when routine inspection is unable to identify problems not easily determined due to different reasons. This is a close up inspection of members not very accessible. In this case skilled workers and inspectors are required to perform the task. Nondestructive tests may be utilized for this purpose in addition to physical and chemical tests.

This type of test may be a complement of a routine test although it is not as frequent. It might also be a follow up to a damage inspection activity \(^1,^4\).

### 2.1.5 Interim Inspection

Interim inspection is a special inspection scheduled when the decision is made by the bridge owner. This is used to monitor a suspected or a known flaw that appeared in a bridge. This type of inspection needs well trained personnel to perform the inspection and they should be familiar with the bridge condition and the type of deficiency. Special instructions should be provided for inspectors on the task that they are performing \(^1,^4\).

### 2.2 What should be Inspected

It is normal to divide the bridge into a number of main components for the purpose of inspecting a bridge deck. Components that should be inspected include the superstructure, the substructure, decks, approaches, signs, water ways, box culverts as bridges, corrugated metal structures, and encroachments.
The main goal of inspection as mentioned earlier is to determine the degree of repair needed and to determine whether some other testing is required. The discussion here will focus on concrete bridge decks. Thus, details about inspection of the other components are out of the scope of this thesis.

2.2.1 Bridge Decks

Bridge decks are considered the most important part of bridges to be inspected as they are the part that is designed to carry the loads and traffic. In this research the emphasis is on concrete bridge decks and the problems that we should look for and deterioration indications in the decks.

Concrete bridge decks should be checked for a variety of flaws. The most common of those include cracking, leaching, scaling, spalling, corrosion of reinforcement, poor quality concrete, deflections and vibration, rutting and wear that may result in reduced skid resistance, accidental damage, surface damage, Efflorescence, and delaminations are common problems that will be discussed in the following sections.

2.2.1.1 Cracking

Bridge decks should be checked for cracks that form due to tensile forces caused by shrinkage, or those that are due to temperature changes or bending. The existence of such cracks does not always mean that the structure is in critical conditions. Causes of cracks vary and can be determined by its location, pattern and size. Some of those cracks are due to thermal cycles such as freezing and thawing. Temperature changes
during solidification causes cracks and the placement of new concrete over old concrete may cause cracks as the new concrete shrinks during curing\(^5\).

Other cracks may be due to loading and corrosion of reinforcement, sulphates or aggregate reaction and chemical attacks. Basic horizontal or vertical patterns with a little branching mean that the crack is caused by chemical attacks that cause the concrete to disintegrate and collapse.

The characteristics of the surface cracks caused by alkali silica reactions (ASR) may not be easily distinguished from other cracks, in this case some testing might be important, the width of the crack plays a big role in the determination of the cause of the crack and it is up to the bridge engineer to decide to perform more tests or to consider the cracks as superficial caused by shrinkage or temperature changes\(^3,4,6,7\).

Generally due to collisions or physical damage most concrete deterioration begins with the appearance of cracks. If the bridge deck is treated against chemical attacks and chemical agents and cracks appeared, they are usually considered harmless. They do not propagate and become spalls as long as they are not subjected to higher stresses from loads or settlements\(^8\).

Cracks open and close as vehicles travel on the deck. The flexural cracks usually do not cause corrosion of the reinforcement unless they are 0.5mm in width. However, crack width in current designs is limited to 0.25 mm for concrete in moderate climates and .1mm for concrete exposed to see water\(^8,9\).
2.2.1.2. Spalling

Spalls form due to insufficient consolidation during construction and the formation of inner cracks, it also occurs due to internal pressure due to freezing and thawing where some of the bridge deck's concrete falls away leaving behind a little hole or discontinuity. This is usually the case if corrosion exists where the volume of the corroded steel becomes much higher than the original steel creating internal stresses which cause distress to the concrete\(^3\).

Although spalling may not be viewed as a major problem, especially regarding the load carrying capacity, it might be a reason for traffic accidents when cars are traveling at high speeds underneath the deck\(^3\).

2.2.1.3 Corrosion of Reinforcement

Corrosion of reinforcement can be a major reason for concrete flaws. Corrosion can cause delaminations, spalling and cracks.

Corrosion of the reinforcing bars in concrete constitutes the single most expensive cause of local failure. Corrosion of steel is an electrochemical phenomenon which occurs in the presence of oxygen and moisture. The occurrence of corrosion has two steps. First, with no chloride present where the high alkaline environment of the concrete prevents further corrosion by formation of what is called passivation layer. This is a thin, dense layer of oxide which remains until chloride is present. This is considered step two. Chloride attacks the passive layer in the form of de-icing salts causing a half cell to form. Some Nondestructive Techniques can be utilized for the detection of the corrosion state\(^3\).
2.2.1.4 Leaching

This occurs due to dissolving water constitutes such as calcium hydroxide, at crack locations. This may cause loss of alkalinity is concrete that leads to corrosion of the embedded steel.

2.2.1.5. Poor Quality Concrete and Honey Combing

Errors during the construction phase such as adding too much water to the concrete mix, insufficient consolidation, or improper curing, can cause distresses and problems in concrete. Concrete, depending on the quality of construction, may sometimes be porous, exhibit honeycombing, or have incorrect steel cover. Honeycombing is easy to spot and it is recognized by exposed coarse aggregate on the surface without any mortar covering or surrounding the aggregate particles. The honeycombing may extend deep into the concrete. Honeycombing can be caused by a poorly graded concrete mix, by too large of a coarse aggregate or by insufficient vibration at the time of placement. Honeycombing will result in further deterioration of the concrete due to freeze-thaw, because moisture can easily work its way into the honeycombed areas. Severe honeycombing should be repaired to prevent further deterioration of the concrete surface.

As for the concrete cover and porosity, these are hard to spot and tests need to be performed randomly to check if such problems are present.

2.2.1.6 Scaling

Scaling is considered a simple form of disintegration which is defined as the deterioration of concrete into smaller parts and individual aggregates.
Disintegration and scaling may be a result of several factors including freezing and thawing, chemical attacks due to the reaction between acid and the hydrated cement, and poor construction practices of concrete. This is a durability problem that can be solved by following strict construction practices and mix designs. With proper amounts of air, water, aggregate sizes, and cement, concrete will be more resistant to such problems.

Proper drainage can drastically decrease the amount of freezing and thawing. When the concrete is exposed to freezing temperatures, water inside the pores freezes and expands, repeated cycles of freezing and thawing may cause surface scaling and may lead to disintegration. Air entrainment is important to reduce the amount of water inside the pores and by that decrease the chance of freezing of water inside the pores.

2.2.1.7 Surface Damage

Overlays such as asphalt, can suffer damages because of overheating by the sun causing it to flow and rut. It can also be pulled off by ice beneath the surface where water from rain leaks and freezes, or due to insufficient bond between the overlay and the bridge deck. This is to be noted and repaired.

2.2.1.8 Efflorescence

This appears as a white band on the concrete surface very close to the hairline of thin cracks. It is formed by water seeping through the pores and cracks into the concrete. As the deck heats and the water starts evaporating, this water leaves behind some minerals that have been leached from the concrete. This may not be a structural problem but it may be an indication of potential problems like concrete deterioration is
likely to happen. Also this water seeping through the concrete makes the concrete more vulnerable because it encourages freezing and thawing which leads to a variety of problems that have been already mentioned\textsuperscript{3,9}.  

2.2.1.9 Delaminations

Bridge decks consist of a concrete slab that is in some cases covered by asphalt coating. Concrete slabs are generally 25 cm thick with two mats of reinforcing steel, an upper and a lower steel mat. As mentioned, corrosion of the reinforcement steel is one of the most serious causes of deterioration of concrete decks. Concrete delamination is a result of corrosion of the reinforcing steel caused by moisture and dissolved deicing salts entering into the concrete to the depth of reinforcing steel and creating an environment where corrosion can occur, causing laminar separation or a fracture plane to develop. As steel corrodes because of the excessive use of deicing agents as an example, it expands causing the concrete to form cracks or fracture planes at or just above the level of reinforcement. Delaminations can create potholes and can affect the integrity of the bridge deck if they grow large in size. It is one of the deterioration forms that impair the load carrying capacity of the bridge. Thus, they are very important to understand and a big concern from the bridges engineer’s standpoint\textsuperscript{2,11,12}.  

There are two situations where delamination of concrete can occur. These are dependant on the presence of moisture and the concrete cover in the delaminated concrete. The first case is when chloride diffuses through the concrete bridge deck in a rate proportionate to the permeability level of the concrete. This chloride will reach the reinforcing steel eventually causing high stresses to arise as the steel rebars corrode\textsuperscript{13}. However, this case is slow and might take up to several years, producing a concrete
with very high chloride content. The second case is when chloride and water diffuse through cracks in the concrete surface. The most critical ones are those occurring due to settlements since those cracks are usually parallel and located right above the steel mesh. This can occur in sound concrete with low permeability and the concrete may contain little amounts of chloride and moisture (dry concrete). Therefore, the reason for delamination could be the high amount of moisture and chloride content which will accelerate corrosion in permeable concrete, or cracks permitting chloride and moisture to diffuse through the deck in impermeable concrete.

2.2.1.10 Deflection and Vibration

Checking if the bridge vibrates while a vehicle passes on it and the deflections that occur for the deck is very important to monitor.

2.2.1.11 Accidental Damage

Accidents on bridges can cause different problems that should be inspected and monitored. Vehicles hitting different parts of the bridge like over height vehicles hitting the bottom side of the deck may cause removal of concrete and spalling. In the same manner vehicles traveling over the bridge deck may cause surface damage due to skidding, braking and overturning. Damages like this should be monitored and rehabilitated.

2.3 Methods of Inspection

Methods of inspection are the techniques used to evaluate the various elements of the bridge. Those methods could be categorized as destructive and non destructive. This thesis will focus on the nondestructive methods and techniques (NDT) for the
evaluation and assessment of concrete bridge decks without overlays. Non destructive methods will be discussed in chapter three. Advantages and limitations of destructive methods will be discussed briefly in the following sections.

2.4. Nondestructive vs Destructive Testing

Destructive testing is defined as “the form of mechanical test (primarily destructive) of materials whereby certain specific characteristics of the material can be evaluated quantitatively.” The information acquired through destructive testing is quite precise but in most cases only applies to the specimen tested. And since the tested specimen is destroyed or altered during the test, it can not be used for other tests.

Destructive testing can be very useful and gives information about the design specifications and service life. The following attributes could be checked using destructive testing:

- Ultimate tensile strength.
- Yield point.
- Ductility.
- Elongation characteristics.
- Fatigue life.
- Corrosion resistance.
- Toughness.
- Hardness. (resistance to plastic deformation)
- Impact resistance.

Destructive testing is said to be accurate and precise, although the test specimen is assumed to be representative of the whole material, it is not always so reliable to
assume that the rest of the material will have the same exact results as the test specimen and that it has exactly the same characteristics. Destructive testing has many of advantages such as the reliable data collected from the test specimen. Data gathered from destructive testing are quantitative and could be used for setting design specifications, and as mentioned the useful life of the structure can be predicted using destructive testing. However, there are some drawbacks which made the use of non destructive testing a more appealing and important positive step in the testing of concrete. For example, the data collected from destructive testing is applicable to the test specimen only, the specimens can not be used for more than one test, and the cost for destructive testing is high considering the expensive field and laboratory equipment. All this along with the many benefits and few limitations of nondestructive testing made it an extremely valuable and useful tool, especially if used jointly with destructive testing.

2.5 Summary

The Chapter talked about the inspection practices and categories that are followed for the inspection of bridges based on the degree of details required. Four types of inspection are required by the National Bridge Inspection Standards. The types of Common Bridge decks problems are also discussed in this chapter. Table 2.1 summarizes several of the bride decks flaws. A comparison between Destructive and Nondestructive Techniques was also shown to establish the need for both types.
<table>
<thead>
<tr>
<th>Defect</th>
<th>Definition</th>
<th>Cause</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spalling</td>
<td>Concrete falls away leaving a little hole that defines the fracture surface.</td>
<td>Internal pressure due to freezing and thawing, insufficient consolidation during construction and the formation of inner cracks which are later transformed to spalls.</td>
</tr>
<tr>
<td>Corrosion of Reinforcement</td>
<td>The weakening of some metals such as steel due to exposure to corrosive environment where it becomes brittle and goes back to its ore state.</td>
<td>Presence of a conductive solution, corrosion agent, and a corrosion cell.</td>
</tr>
<tr>
<td>Leaching</td>
<td>The drainage or removal of soluble or constitutes in porous materials by water seeping action.</td>
<td>Occurs due to dissolving water constitutes like calcium hydroxide at crack locations.</td>
</tr>
<tr>
<td>Scaling</td>
<td>Deterioration of concrete into smaller parts and individual aggregates.</td>
<td>Scaling may be a result of freezing and thawing, and chemical attacks.</td>
</tr>
<tr>
<td>Cracking</td>
<td>A breakage in the concrete causing a discontinuity without causing a complete separation of the structure.</td>
<td>Cracks form due to tensile forces caused by shrinkage, temperature changes, bending, loading, corrosion of reinforcement, sulphates, and chemical attacks.</td>
</tr>
<tr>
<td>Honeycombing</td>
<td>The presence of exposed coarse aggregate with not enough concrete paste covering the aggregates causing the presence of small holes.</td>
<td>Poorly graded concrete mix, the use of large coarse aggregates, and insufficient vibration at the time of placement.</td>
</tr>
<tr>
<td>Delaminations</td>
<td>Cracks or fracture planes at or just above the level of reinforcement that grow big and can affect the integrity of the structure.</td>
<td>Corrosion of steel reinforcement, high amount of moisture and chloride content, and the presence of cracks in concrete surface.</td>
</tr>
</tbody>
</table>
CHAPTER THREE
NONDESTRUCTIVE TESTING

3.1 Introduction

This Chapter talks about the definition of nondestructive testing (NDT) and discusses the background and history of the nondestructive methods employed for the inspection of concrete bridge decks. The advantages and disadvantages of each of the methods will be discussed. Towards the end of the chapter three methods will be selected for further study and evaluation. The selection will be based upon applicability, advantages, and versatility of the methods.

3.2 Definition of Nondestructive Testing

Nondestructive testing is the form of testing or inspection that can verify the integrity of a structure or any of its components, without compromising its ability to perform in service (2). This definition includes the fact that there will be no change or alteration to the object in any way. This is done in order to determine the presence or absence of any kind of discontinuity or flaw that may have an effect on the usefulness or serviceability of the structure. These tests may be also used to measure other values and characteristics such as size, dimensions, and configuration $^{15}$.

Nondestructive tests have to be non-damaging and non-invasive. These techniques are emerging and becoming a very important means by which bridge engineers evaluate and diagnose the quality and the condition of reinforced concrete structures such as concrete bridge decks $^5$.

Although non destructive tests are a very useful tool to evaluate the state of bridge decks and many other elements, it can not ensure that the part under testing will
not fail or malfunction. This is because every non destructive test has limitations. Codes and standards should be followed and checked if the extent of a discontinuity, if present, is acceptable or should be considered as a major defect that has to be dealt with. Non destructive testing also requires a very high level of proficiency for the bridge engineer or the person who is performing the testing.

3.3 Background and History of Nondestructive Testing

Non destructive testing has been known since the very early ages. A specific date to which the use of non destructive testing began is not known. For example, methods to check soundness of steel and strength of swords were known since ancient times. At those times it was not referred to as non destructive techniques. Nonetheless, those method were actually non destructive in nature. The human body was a tool of non destructive testing using the human eye or senses to check imperfections in different objects, heat sensing to check heat differences, and touch to check for flaws. This is what evolved to be known as visual inspection.

The roots of non destructive testing began in the 1920's with some awareness and knowledge in magnetic testing and radiographic techniques such as X-rays. Other methods used at that time were very close in concept to current methods, of those are methods very close in nature to penetrant dyes and eddy current. From those primary attempts, new and current methods known today as nondestructive evolved.

The development of Nondestructive evaluation techniques (NDE), used alternately with NDT through out this thesis, started in the late 1950’s and continued in developing through the years of computer technology and internet. The ability to store data and restore it at anytime made NDE techniques a very useful tool. Nevertheless,
with all these improvements in NDE, it is still considered to be in its early stages as there will always be room for more developments in different methods. This will be the case until the smallest flaws in materials and objects can be detected and as long as there are new challenges that this science will have to conquer\textsuperscript{15}.

3.4 Advantages and Limitations of NDT

NDT is seen to be a tool to determine the maintenance needs and condition of the structure under study. The advantages of NDT qualify it to be one of the most important approaches to be used for inspection.

Some of the advantages of non-destructive testing include

- Determining the amount of damage in the structure that has to be repaired.
- Helping determine the cause of damage on which the rehabilitation decisions will be based.
- Monitoring deterioration of structures with time.
- Controlling of quality during construction to prevent future repair and maintenance.
- Testing is always done while object is still in service.
- Portability of the methods where they can be taken to the object to test.
- Cost efficiency of the methods when compared to destructive testing.

Although the advantages of NDT are very significant, some limitations also exist. Those should be overcome through more research and validation of the test methods. Some of those disadvantages are

- Failure to provide quantitative data.
- Lack of availability of defined criteria for tests use.
• High level of proficiency for most methods is required.
• Some methods are still expensive.
• Some methods are still labor-intensive, intrusive to traffic and unreliable.

The use of non-destructive testing is vital. The case of bridge decks is no exception as most of the time those bridges can not be shut down to traffic or damaged while inspection is done. The determination of the bridge deck’s condition is important and the use of non-destructive testing can help in generating data for bridge management systems. Compared to destructive testing, data collected by the use of NDT give more comprehensive results, it is generally less expensive, and can give the bridge engineer information about different types of flaws that might cause failures.

3.5 Nondestructive Testing for Concrete Bridge Decks

The deck is that part of the superstructure that spans longitudinally between supports. Bridge decks consist of a concrete slab sometimes covered by an asphalt coating. The concrete slabs generally vary in thickness depending upon the form of bridge deck that supports it.

When it comes to bridge decks, bridge engineers are interested in non-destructive techniques that help them to determine maintenance needs and to detect flaws within the bridge deck. These techniques should be fast, economic, and reliable. Concrete decks are very susceptible to different forms of deterioration, ranging from cracking and ending with large holes in the deck. Problems in concrete bridge decks are summarized in chapter two.

Some difficulties may be encountered when using NDT in bridge decks. This may occur because of several factors.
• Concrete bridge decks are not always accessible.

• The asphalt overlay when present varies in thickness which makes it harder for different NDT methods to detect flaws underneath the layer.

• The size of the defects in the deck might be very small which is hard to detect using some of the techniques available \(^{12}\).

Several nondestructive techniques have proved the ability to detect discontinuities and flaws for the purpose of bridge deck evaluation, and knowing the circumstances under which the bridge deck operates and the difficulties that come along with the use of those methods, The following discussion will address these methods comparing their advantages and disadvantages in terms of applicability to bridge decks and how well they perform. Towards the end of this chapter, three methods will be chosen for further discussion. These methods were potentially the most applicable and will later be validated.

3.5.1 Visual Inspection

The visual inspection method is the predominant nondestructive method used for bridge inspection. It is the first and most widely used NDT. Visual inspection involves inspecting for surface conditions, defects, roughness and dimensional changes \(^{18}\).

Visual inspection must be done by a trained inspector where the various components of the bridge are examined at close range to evaluate their condition and rank them according to the ranking system required by the national bridge inspection standards (NBIS). According to the NBIS format, bridges and its components are given condition ratings ranging from zero to nine, zero representing failure condition beyond corrective action and Nine representing excellent condition \(^{4,11}\). The benefit of such a
ranking system is to provide the ability to measure the bridge performance on a national level so as to determine the future funding needs and to better distribute funds between states. These condition ratings are somehow subjective of the condition rating given by the inspector.

Visual Inspection could be appropriate for many situations. However, due to the subjective nature of this method which depends on the assessment of the inspector, rankings of similar component can vary from one inspector to another, or from one state to another depending on the guidelines followed in each state as a general guideline or criteria are not yet established.

The applications of visual inspection spread and vary according to what is being inspected. For bridges, visual inspection can be used to check the metrology of the component’s geometry and the surface roughness of the deck. It is also used for the detection of cracks, and can give us information about the condition of the decks surface. However, visual inspection can not give us any information about the internal condition of the deck especially delaminations and corroding steel which can not be seen or detected using visual inspection alone.

Many tools are used to facilitate the use of visual inspection and help the operator make a better judgment. Some of those tools include:

- Lighting. It must be at appropriate level and of the correct type.
- Magnifiers. These range from low power magnifiers to high power microscopes.
- Borescopes. These are devices used to permit inspection of the internal parts of pipes and such components.
• Television and video cameras, video recorders, and visual images. These facilitate the interpretation of results and ensuring that the results are accurate by being able to revise the test at any time.

• Surface replication. Using strippable films which permit the remote inspection of different surfaces.

• Access equipment. To reach hidden and elevated areas.

Generally, visual inspection has advantages and limitations, some of the advantages are:

• Excellent assessment of the inspected object based on knowledge and experience.

• Excellent capability to analyze complex scenes run into during the course of the inspection job.

Some of the limitations are:

• The inspection results are subjective and depend on the assessment of the inspector.

• Inspection speed is very slow which costs working hours for highly paid inspectors.

• Doesn’t give a quantitative result, the outcome of visual inspection is qualitative.

• Vulnerable to inspection errors due to inspectors’ fatigue because it depends on the inspectors’ eyesight, environmental conditions such as rain or fog, and the repetitive nature of the inspection process.

• Liabilities due to hazardous environment.
3.5.2 Liquid Penetrant Testing

Apart from visual inspection, penetrant testing is one of the oldest and most widely used methods in detection of surface flaws in concrete bridge decks. This method is applied to any non porous material in order to collect information about surface cracks and irregularities.

A dye is applied to the surface and allowed to dwell for a period of time, the discontinuity because of capillary action, absorbs the dye. The excess dye is then removed and a developer, which is a white powder, is used to remove the excess dye and to provide a contrasting background.

The advantage of this method is that it shows surface cracks. However, its application is very time consuming. In addition, it does not work for rough or poor surfaces and causes a lot of fatigue to the inspector.

3.5.3 Chain Drag Method

Chain drag is considered to be one of the traditional methods for the detection of flaws inside bridge decks especially delaminations. The method involves dragging a chain across the structure and listening to audible sounds. Chain drag, along with hammer tap, are categorized as sounding (acoustic) methods. The resonance of concrete is noticed to decide whether the concrete is sound or defected. A distinctive hollow sound will be heard if the chains are dragged over a delaminated area. Delaminated areas produce a dull sound compared to that acquired from sound concrete.

The chain drag equipment may take several forms from a single long chain to a chain consisted of several smaller length chain segments attached together and to a piece of tubing. The chain drag is dragged in a certain way across the deck’s surface.
depending on the setting of the apparatus according to ASTM D4580-86. After the
deck is divided into orthogonal grids, the operator listens to the reflected sounds from
the concrete. The results are then registered relative to the deck’s grid pattern.

While this method is considered good for detecting mild to severe delaminations
and voids it still has several disadvantages such as:

• It is considered a very tedious job, because the operator works in a crouched
  position.
• Very time consuming as it covers small areas of the deck.
• Has limited inspection speed.
• Difficult to implement in noisy environments.
• Depends heavily on the subjective interpretation and skill of the inspector.
• Depends on size of delaminated areas.
• Lower capabilities in the existence of overlays.

Some of these disadvantages had been overcome by the use of automated chain
drag systems that record the sound and then the signals are processed using a mini
computer that has the ability to distinguish between delamination sounds and external
noise.

3.5.4 Half Cell Potential

One method that is widely used for the detection of corrosion in reinforcing steel
is the half cell potential. It is also used to measure corrosion rate in concrete slabs. To
measure the half cell potential, an electrical connection is made to the steel
reinforcement in the part that is inspected. This is then connected to a high impedance
digital millivoltmeter that is often backed up by a data collection device. The other end
of the voltmeter is connected to a copper/copper sulfate half cell which has a porous connection at the end that is in contact with the concrete. The half cell test registers the corrosion potential at the steel nearest to the point of contact and the results are plotted on a contour map that can be further processed using 3D mapping techniques. Figure 3.1 shows the half cell potential device.

Although this method is widely used and thought to be accurate, some studies show that this method is not so reliable. According to ASTM 876-80 corrosion can only be detected with 95% certainty at potentials more negative than -350mV. If the potentials read are more negative than -200 mV it shows a 10% certainty that corrosion is present.

Half cell potential might also be used for the detection of delamination in bridge decks. However, the reliability of delamination detection is not good enough for the method to be adopted as a delamination detection nondestructive method. Some factors mitigate half cell potential to detect corrosion and thus delaminations in concrete bridge decks. Those factors may include:

- Concrete Cover Depth: with the increase of concrete cover the detection of small corroded areas become harder
- Concrete Resistivity
- High resistive surface layers: if the surface is highly resistive, the measured potentials become more positive and the corroding areas become more difficult to detect.
- Polarization effects
Figure 3.1: Half Cell Potential Testing Device

The advantages of this method are that it gives us a good idea about the presence of corrosion, even with the presence of overlays. Nonetheless, it does not tell us a lot about the corrosion rate. The method also has some drawbacks which can be summarized as follows.

- Interpretation of the results is difficult.
- Deck has to be totally dry when test is performed.
- Steel has to be continuous between points of testing.
- The method is good with upper mat steel, but tends to be inaccurate for lower mat steel.
- Requires lane closures.
3.5.5 Acoustic Emission (AE)

Acoustic emission is the method in which elastic waves (stress waves) in the range of ultrasound with frequencies between 20 KHz and 1 MHz are utilized. This energy is released from flaws within the material when the structure is loaded beyond its service load. This is known as the Kaiser effect, stating that AE signals in unloaded specimens do not occur until previous stress levels are exceeded during reloading. The energy then propagates through the solid to the surface where it is then registered using a transducer. The transducer transforms the mechanical signal into an electrical signal. This signal provides information about the transmitting source within the material as its location and size are extracted.

Figure 3.2 shows the acoustic emission principle and how the transducer works to receive stress waves from cracks and other dynamic sources within the material. Acoustic emission can be used for several of the bridge components including the deck.
Acoustic emission (AE) has a wide variety of uses. It is used for crack detection, corrosion detection and delamination detection. It is considered to be a useful method for investigation of the internal state of materials such as concrete. The method is able to detect active movements of defects and provide real time response using embedded sensors. Drawbacks of Acoustic Emission include the fact that it can only qualitatively estimate the damage inside the material and the service life of a component. AE can not give us quantitative results unless combined with other techniques. The high attenuation nature of concrete results in a very week signal that can not be detected, especially in the case of the presence of overlays. Furthermore, acoustic emission is not a very reliable method since it depends on both eth energy and dynamic of the crack since the transducer might not be sensitive enough to detect low energy transmissions. Additionally, AE is often a costly method.

3.5.6 Ultrasonic Pulse Velocity / Echo

Ultrasonic pulse velocity (UPV) and echo techniques depend on the use of ultrasonic waves, where the wave velocity depends on the properties of the propagation material. These waves are generated by exciting a piezoelectric crystal with high voltage. The ultrasonic pulse velocity equipment consists of a transmitting receiver which has the crystal and a receiving transducer, and a computer data acquisition system where different filters may be applied. In the case of pulse echo the transmitter and the receiver are one.

The waves generated by the ultrasonic pulse velocity device propagate through the tested material causing some deformations. As the wave hits an interface it is
reflected, transmitted, or diffracted. The amount of transmitted and reflected energy depends on the impedance difference of the material, and the flaw, or interface where the wave was reflected. UPV can be used to determine the mechanical properties of concrete by measuring the velocities of waves. The ultrasonic waves are divided into three basic types; longitudinal (compressional) also called p-waves, shear, also called s-waves, and surface waves. The P-waves and S-waves are the ones used to determine the properties of a concrete slab, as those types of waves reflect or refract. Their velocities give information about different characteristics such as young’s, shear, and bulk modular values; as well as the Poisson’s ratio or the specimen where the wave path is defined. The semi direct method is less accurate due to the uncertainty of the true contact point. The indirect method is less accurate because both transducers are in contact with the specimen in the same face. The inaccuracy becomes proportionally greater for shorter transmission path than for longer ones. The UPV instrument measures the time for the pulse to travel between the transmitter and receiver transducer, those measurements can be made in three different modes: Direct, semi direct, and indirect or surface transmission. The direct method is considered to be the most accurate, where the two transducers are on opposite faces of the tested element.

UPV has a lot of advantages. It is used for detection of voids, cracks, delaminations, as well as concrete homogeneity, deterioration and strength. It has an easy test procedure at a relatively low cost. Draw backs of the technique are characterized by difficult interpretation of the results that are usually presented in a frequency domain. This requires a very experienced interpreter. UPV is considered to be a very good method for steel detection and inspection, but might not be as reliable in
concrete structures due to the high heterogeneity of concrete. The attenuation of pulse echo methods is considered to be a problem because of the use of high frequency waves of values higher than 100 KHz. This method also doesn't define the shape of the defect.

### 3.5.7 Impact Echo (IE)

Impact Echo is considered a relatively new method for evaluation of concrete structures. It is based on the use of sound waves generated by elastic impact. Waves are introduced to the surface of the concrete element (i.e. concrete bridge deck) using an impactor, which can be of different sizes ranging from 3-15mm depending on the type and depth of flaw.

The impact echo device consists of a receiver which is a transducer, a data acquisition system where the reflections of waves are recorded to a computer based program, and a set of impactors as spherical steel balls of different sizes as mentioned.

As sound waves are generated, they propagate through the material and are reflected by flaws and interfaces of different acoustic properties. The impact made by tapping the steel sphere on the concrete surface creates displacements in the concrete. These are caused by the travel of three types of waves, P-waves, S-waves, and R-waves inside the concrete component. The transducer receives the reflected waves and registers them as a waveform. This is later transformed into a frequency domain to extract information about different responses.

Impact Echo advantages include the ability to detect delaminations, cracks, voids, steel, and layer thicknesses. It can perform well in the presence of layers. Some of the disadvantages include the result interpretation difficulty.
This method was selected for further testing and validation in this thesis. Chapter five will discuss impact echo in details.

3.5.8 Ground Penetrating Radar (GPR)

Ground Penetrating Radar is a method that employs the use of radar (Radio Detection and Ranging) and radio waves that are emitted from a source to detect an object and determine the size, direction, distance and properties of the object after the object re-emits some of the energy that impinges on it. GPR uses electromagnetic waves to probe lossy dielectric materials to detect structures and changes in materials properties.\(^{38}\)

The concept of GPR relies on the travel of an electromagnetic wave or radio wave within the frequency range of 10 MHz to 2000 MHz\(^{5,39}\) through a dielectric material. This wave is reflected or scattered by objects of different Dielectric properties in its path and returned to a receiving antenna where the pulse is recorded.

GPR equipment comprises of the three main parts which are an interchangeable antenna that works as a transmitter/receiver. This may be two separate antennas in some systems, a control unit that trigger the antenna’s pulses, and a data acquisition system for recording the information in image format and later transferring it to a computer.

Ground penetrating radar has now been used for detection of subsurface anomalies in bridge decks for a couple of decades and the study in this area is still underway. It is considered to be one of the more promising methods for subsurface mapping in any low conductive material such as concrete.
Ground Penetrating Radar will be discussed in more detail in chapter 6. The capabilities of GPR mentioned in literature as well as the potentials of the method to improve made it a good subject of validation and study in this thesis.

3.5.9 Infrared Thermography (IR)

Infrared Thermography is considered one of the easy methods to apply on concrete structures. It is a better method than the traditional visual inspection and acoustic methods such as Chain Drag and Hammer tap. This superiority comes from the fact that it is less sensitive to external factors, with faster results and higher accuracy than sounding techniques.

Infrared thermography depends on the concept of detecting thermal differences between sound and defected concrete. This is accomplished by registering the temperature readings on the surface of the concrete bridge deck or pavement. During daytime, areas with discontinuities heat at a higher rate than sound concrete and cool faster during night time. This approach helps detect concrete spalls, cracks, voids, and delaminations by detecting the thermal differences. Infrared tests can be done passively or actively. Passive infrared depends on the sun to heat the bridge deck while active infrared depends on artificial heat to heat the element under study.

IR It is capable of scanning larger areas in shorter time when compared to other traditional methods. Some of the literature has shown that this method has been able to detect cracks, voids, delaminations and debonding between layers. On the other hand, some research showed limited ability to detect above mentioned defects. Due to the controversy associated with this method this thesis will attempt to validate its
applicability to inspect concrete bridge decks flaws. Chapter seven will deal with IR Thermography greater depth.

3.6 Selected for Further Investigation

Three methods have been selected to be validated. This is largely a result of literature reviews which found them to be the most applicable, most effective and in some cases the most controversial methods for the evaluation of bridge decks. Maser et al\textsuperscript{16} and Halabe et al\textsuperscript{40} state that the published conclusions have been drawn from a limited number of bridge deck studies\textsuperscript{16}. The following chapters of this thesis will deal with the background of each of those methods, describing how they work, the physical bases, and the data processing and interpretation techniques of the results. A study of the results at the end of this thesis will validate the abilities of each of the methods to detect several flaws in concrete bridge decks without overlays. That GPR is the most promising techniques for bridge deck assessment. This method along with Thermography and Impact echo which we found to be interesting methods are going to be further studied and evaluated through some experimental tests. The tests will be performed in controlled environment.

3.7 Summary

Chapter Three talked about several of the Nondestructive tests used for the evaluation of concrete bridge decks. The criteria to choose each of the methods and the types of defects that each method can detect were summarized. Towards the end of the chapter, three methods were selected for further investigation and validation.
Table 3.1 summarizes the methods used for the inspection of concrete bridge decks. The table focuses on the criteria of usage, advantages, and limitations of each of the methods. This table might be considered a guide for the selection of the Nondestructive technique depending on the application.
Table 3.1 Nondestructive Methods Used with Concrete Bridge Decks

<table>
<thead>
<tr>
<th>Method</th>
<th>Uses</th>
<th>Advantages</th>
<th>Limitations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Visual Inspection</td>
<td>Cracks, metrology of components geometry, surface roughness.</td>
<td>Accessibility, oldest known technique, well established</td>
<td>Subjective, time consuming, qualitative results.</td>
</tr>
<tr>
<td>Liquid Penetrant Dye Chain Drag Method</td>
<td>Surface flaws, detection of irregularities Flaw detection inside decks, delaminations</td>
<td>Simple, portable, good for delaminations</td>
<td>Surface preparation, exhausting for inspector, time consuming.</td>
</tr>
<tr>
<td>Half Cell Potential</td>
<td>Detect corrosion state in concrete reinforcement, corrosion rate.</td>
<td>Simple, portable, good for corrosion</td>
<td>Time consuming, tedious, subjective, not good with overlays</td>
</tr>
<tr>
<td>Acoustic Emission</td>
<td>Cracks, delaminations, corrosion.</td>
<td>Real time response, no lane closures</td>
<td>Deck needs preparation, time consuming, not good for delaminations, lane closure, Not very accurate.</td>
</tr>
<tr>
<td>Ultrasonic Pulse Velocity</td>
<td>Homogeneity of concrete. Cracks. Voids. strength</td>
<td>Portable, easy test procedure at relatively low cost, relatively easy to interpret</td>
<td>Qualitative results only, not good with overlays, interpretation, costly, not reliable.</td>
</tr>
<tr>
<td>Ground Penetrating Radar (GPR)</td>
<td>Concrete mapping, mining, geotechnical, road and bridge, forensics, detection of voids, honeycombing, delaminations, moisture content.</td>
<td>Versatility, portability, effectiveness, low cost, good with overlays, minimum traffic control, prediction of repair quantities in roads</td>
<td>Interpretation, complexity of results, interpretation of results requires destructive testing.</td>
</tr>
<tr>
<td>Impact Echo (IE)</td>
<td>Detection of voids, cracks, delaminations, unconsolidated concrete, and debonding, determining thicknesses</td>
<td>Requires one surface of the testes material to be exposed, independent of the geometry of the structure, less susceptible to steel reinforcement, high accuracy</td>
<td>Size of detected flaws is highly dependent on the impact duration, less reliable in the presence of asphalt overlays, interpretation of the results are difficult</td>
</tr>
<tr>
<td>Thermography</td>
<td>Detection of thermal differences, delaminations, cracks, voids</td>
<td>Portable, simple, easy interpretation, minimum traffic interference</td>
<td>No information about depth of defects, dependant on environmental conditions</td>
</tr>
</tbody>
</table>
CHAPTER FOUR
EXPERIMENTAL DESIGN AND TESTING PROGRAM

4.1 Introduction

The common bridge decks problems discussed earlier in this thesis can affect the serviceability of bridges. Thus, the methods selected in chapter three will be validated using an experimental program designed specifically for this purpose. GPR, Impact Echo, and Thermography will be used for the detection of the most common bridge decks flaws. Surface cracks, voids, and delaminations will be tested using the nondestructive methods.

This chapter presents an experimental program designed to establish how Impact Echo, GPR, and thermography can be effectively used to detect voids, cracks, and delaminations. It will also help validate the methods and assist in assessment of bridge decks. The chapter covers the objectives of the experimental study, the design of the test specimens, and the procurement of the testing equipment.

4.2 Equipment Procurement

The systems that are used in this thesis have been chosen to be the latest, state-of-the-art equipment. Different suppliers and vendors were contacted to get the most advanced technology that can help validate the usefulness of the different methods. Appendix A summarizes the suppliers that were contacted for equipment procurement and gives more details about the procurement procedures for each of the three equipment pieces. Section 4.3.2 will discuss the selected devices.
4.3 Experimental Program

During the experimental design phase, different variables were identified. The specimens were designed to study the effects of three of the most commonly present defects in concrete bridge decks on responses obtained from the selected nondestructive techniques. Specimen’s sizes, depths and concrete mixes were carefully selected to mimic the case of bridge decks. Flaws were artificially simulated and introduced to the specimens. Locations, depth, and sizes of the three types of defects were registered. Tests were performed at or over the locations of defects.

4.3.1 Objective of the Experimental Study

The objective of the experimental part in this study was to determine the ability of GPR, Impact Echo, and Thermography to detect flaws in concrete bridge decks. In the real situation, it is also very important to extract as much information about the defects as possible. Three types of simulated flaws of known dimensions, locations, thicknesses, and geometry were introduced to the specimens. The goal was to run each of the tests over the flaws and check the detection ability and how much information each of the methods can provide about the flaw. GPR, Impact Echo, and Thermography as shown in chapter three, were the most promising nondestructive techniques. A comparison between the different methods will be drawn, recommendations will be made on which of the methods would be better for the detection of which defect. And the abilities of the method to detect flaws at different depths and sizes will be validated.
4.3.2 Testing Equipment

The testing equipment, as mentioned before, was selected to be state of the art equipment suitable for the application on concrete bridge decks. A survey was done to find the most suitable, up-to-date specifications in the market. The specifications of the selected Impact Echo, GPR, and Thermography devices are discussed in Tables 4.1, 4.2 and 4.3 respectively.

The selected Impact Echo System characteristics are provided below. Table 4.1 summarizes the specifications of the Impact Echo device

<table>
<thead>
<tr>
<th>Component</th>
<th>Description</th>
<th>Use</th>
</tr>
</thead>
<tbody>
<tr>
<td>Analog/Digital Data Acquisition System</td>
<td>14 bit resolution, 2 MHz maximum Sampling Speed</td>
<td>Fourteen-bit analog/digital converter digitizes analog voltage signal from transducer at rates up to two mega sample</td>
</tr>
<tr>
<td>One hand held pistol grip transducer</td>
<td>Pistol grip with protective disks</td>
<td>Measures displacements and transfers data to acquisition system</td>
</tr>
<tr>
<td>BNC Cables</td>
<td>Two BNC cables to connect Date acquisition and transducer</td>
<td>Connection between data acquisition and transducer</td>
</tr>
<tr>
<td>Impactors</td>
<td>A set of 10 steel sphere balls used as impactors</td>
<td>Provide impact on the concrete component surface</td>
</tr>
<tr>
<td>Computer Software</td>
<td>Impact Echo Software for data analysis</td>
<td>Data analysis software</td>
</tr>
</tbody>
</table>

The Selected GPR device had the following specifications shown in Table 4.2 below. the system comprises of an antenna, a data acquisition system, survey cart, and the post processing software called RADAN.
Table 4.2: Ground Penetrating Radar Device Specifications

<table>
<thead>
<tr>
<th>Component</th>
<th>Description</th>
<th>Use</th>
</tr>
</thead>
<tbody>
<tr>
<td>Antenna</td>
<td>High Resolution 1.5 GHz for Shallow Applications</td>
<td>High Resolution Antenna for Concrete Slabs and Bridge Decks</td>
</tr>
<tr>
<td>Data Storage</td>
<td>512 Mb Flash Memory Card</td>
<td>Data Storage</td>
</tr>
<tr>
<td>Display Modes</td>
<td>Line Scan, O-Scope</td>
<td>For Data Display</td>
</tr>
<tr>
<td>Processor</td>
<td>32-Bit Intel Strong Arm @206 MHZ</td>
<td>The Data Acquisition Processor</td>
</tr>
<tr>
<td>Channels</td>
<td>1 Channel</td>
<td>Number of Antennas That Can be Operated</td>
</tr>
<tr>
<td>Scan Rate</td>
<td>User Selectable</td>
<td>The Number of Scans per Unit</td>
</tr>
<tr>
<td>Number Of Samples Per Scan</td>
<td>User Selectable</td>
<td>Number of Samples (Data Points) in each Scan</td>
</tr>
<tr>
<td>Filters</td>
<td>Vertical Low and High Pass IIR and FIR Filters, Horizontal Filters, Stacking and Background Removal</td>
<td>For Data Display, Processing and Interpretation Assessment</td>
</tr>
<tr>
<td>Survey Cart</td>
<td>3 Wheel Survey Cart or Hand Held Survey Cart</td>
<td>for Data Collection, Distance Measurements</td>
</tr>
</tbody>
</table>

As for the infrared camera characteristics, Table 4.3 provides some of the Infrared camera specifications. The camera comes with a visible digital camera, an LCD screen display, and an integrated IPAQ PDA. The camera can function with the IPOD and without depending on the application.
Several concrete test specimens were fabricated for the study of the three tests. These specimens accurately simulated the internal bridge flaws that were of interest in this thesis. The Specimens included in this study contained simulated features that were indicative of delaminations, voids, and surface cracks. The fabrication of the specimens, simulation of the defects, and the method by which these specimens were selected will be discussed in this section.

Three concrete specimens were fabricated. The dimensions of the specimens was set to be 4ft * 4ft, this was assumed to be a representative dimension as it gives room for testing. The specimens width to thickness ratio was large enough (larger than
5) to avoid interference caused by wave reflections from the side boundaries of the plate.

The depth of the specimens (i.e. slab thickness) varied between slabs in order study
the effect of depth on the ability of the three methods to detect different flaws. Except
for the surface cracks, the increase of the thickness of the slabs meant an increase in the
depth of the other defects as they were placed on the steel layer were they form in real
bridge decks.

The project comprised of the fabrication of six slabs. Three of which contained
the simulated defects. The other three were sound concrete specimens identical in size
and composition to the defected ones. All the slabs contained one layer of steel. The
steel was number 5 steel in two directions spaced at 6 inches. The slabs had different
depths to imitate the typical concrete bridge deck thicknesses. Defective and non-
defective Slabs with thicknesses of four, six, and eight inches were fabricated This was
utilized to get the responses from the baseline specimens and comparing them to the
defected specimens.

The concrete used in the mix was in agreement with the Michigan Department
of transportation (MDOT) classification for bridge decks. Concrete used in the
specimens was according to the Michigan Bridge Design Manual used for bridge decks.
It was grade S2 with 21 Mpa compressive strength. Steel should be grade 60 with \( f_y \) of
400 Mpa.

Figure 4.1 and 4.2 show the formwork and steel reinforcing for one of the slabs.
Figure 4.1: Form Work for the Test Specimens

Figure 4.2: Reinforcing Steel for the Test Specimen
4.3.3.1 Crack Simulation

To simulate cracks in the specimen, Plexiglas of different thicknesses and longitudinal dimensions was used. Plexiglas is a light transparent weather resistant thermoplastic with no electrical conductivity. Some of the methods used to introduce cracks in past studies included using blades to crack the specimen, indulging a metal sheet in the concrete and removing it before setting of concrete, using different diameter straws in different locations and orientations to simulate longitudinal and transverse cracks, others suggested the use of an edge notch and then propagating it in the specimen using mechanical jacks or by filling small diameter plastic pipes with expansive cement mortar and placing them in the concrete. The method used in this experiment as mentioned was the introduction of different sizes and lengths of Plexiglas pieces.

4.3.3.2 Simulation of Delaminations

Styrofoam was used in this study to simulate delaminations. It has proved to be suitable for this purpose in many older studies. Styrofoam was placed in delamination locations and the concrete was cast. Acetone was used on a later stage to melt the Styrofoam by injection through little tubes.

In the past, several methods were used to simulate delaminations in concrete specimens. Methods such as casting on an inflated air bag, embedding ice sheets, using polythene, inserting a low melting wax sheet and then heating to create a void, embedding thin sheets of plastic into the concrete mix, and inducing corrosion cracking by applying an electrical potential. Some of those methods were successful such as induced corrosion using corrosion accelerators, some others proved failure
like using ice sheets since they melt during concrete chemical reactions because of the high heat that is produced.

4.3.3.3 Voids Simulation

The simulation of voids in this thesis was done using PVC pipes that were placed in the form before concrete was cast. PVC pipes of different lengths, diameters, and depths were introduced to the concrete specimen. Methods used in other studies involved mixing cement and sand and placing it in the concrete slab. This creates a mortar chunk inside the concrete slab that is full with little voids. This mimics the case of honeycombing.

4.3.4 Specimen Defects Layout

The three slabs were given identification names for easier reference. The first defected slab with thickness of 4 inches was given the ID S4, the control block for this slab was given the ID S4C, the second defected slab with a thickness of 6 inches was given the ID S6 and its correspondent control block was given identified as S6C. The third defected slab with a thickness of 8 inches was given the ID S8, and its correspondent control block was names S8C.

Specimens S6 and S8 contained six delamination locations while specimen S4 contained seven. Those delaminations were connected to the concrete surface through little plastic pipes in order to inject them with Acetone to dissolve them. Specimens S4, S6, and S8 contained three voids and seven cracks. Some of those were surface cracks and others were vertical cracks deeper inside the specimens. As mentioned earlier, Specimens S4C, S6C, and S8C contained no flaws at all but were identical to S4, S6,
Tables 4.4 through 4.10 show the defects schedule showing sizes, depths, and locations of defects.

The specimens were tested in two stages. The first involved testing the specimens in the presence of the Styrofoam blocks. The second stage involved the testing of the delamination location after melting the Styrofoam blocks by Acetone injection through the plastic pipes that were attached to the Styrofoam blocks and extended above the concrete surface for this purpose. The test was done in two stages to check if the response will differ if the delaminations were filled with air as opposed to them being Styrofoam filled. Delamination number seven in S4C was left with no pipe connection to determine if the presence of the pipes had an effect on the obtained results.

Figure 4.3 shows a plan view for S4. Figure 4.4 shows a plan view for both S6 and S8. Figure 4.5, 4.6 and 4.7 show the sound S4C, S6C, and S8C inches slabs respectively. Figure 4.8 shows one of the specimens ready to be cast. Finally, figure 4.9 shows the specimens during concrete casting.

4.4 Summary

Chapter four talked about the experimental design phase of this study. Summarized the specifications of each of the devices, and discussed the simulation of the common defects selected to be studied with the three nondestructive methods.
Figure 4.3: Plan View of S4
Figure 4.4: Typical Plan View for S6 and S8

Figure 4.5: Specimen S4C (Four Inch Sound)
Figure 4.6: Specimen S6C (Six Inch Sound)

Figure 4.7: Specimen S8C (Eight Inch Sound)
Table 4.4: Delamination Schedule for S4 (Four Inch Specimen)

<table>
<thead>
<tr>
<th>Flaw Type/Name</th>
<th>Length (Inch)</th>
<th>Width (Inch)</th>
<th>Thickness (Inch)</th>
<th>X-Displacement (Inch)</th>
<th>Y-Displacement (Inch)</th>
<th>Depth from Surface (Inch)</th>
</tr>
</thead>
<tbody>
<tr>
<td>D1</td>
<td>2</td>
<td>3</td>
<td>2</td>
<td>33.5</td>
<td>26</td>
<td>1</td>
</tr>
<tr>
<td>D2</td>
<td>2</td>
<td>1.5</td>
<td>1</td>
<td>26</td>
<td>30.5</td>
<td>0.75</td>
</tr>
<tr>
<td>D3</td>
<td>4</td>
<td>4</td>
<td>.5</td>
<td>13</td>
<td>24.5</td>
<td>0.75</td>
</tr>
<tr>
<td>D4</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
<td>23</td>
<td>18</td>
<td>1.25</td>
</tr>
<tr>
<td>D5</td>
<td>4</td>
<td>3</td>
<td>2</td>
<td>34.5</td>
<td>10</td>
<td>1.25</td>
</tr>
<tr>
<td>D6</td>
<td>3</td>
<td>3</td>
<td>1.5</td>
<td>31.2</td>
<td>10.2</td>
<td>1.5</td>
</tr>
<tr>
<td>D7</td>
<td>3</td>
<td>3</td>
<td>0.5</td>
<td>24.5</td>
<td>39</td>
<td>1.5</td>
</tr>
</tbody>
</table>
Table 4.5: Void Schedule for S4 (Four Inch Specimen)

<table>
<thead>
<tr>
<th>Flaw Type/Name</th>
<th>Diameter (Inch)</th>
<th>Length (Inch)</th>
<th>Y-Displacement (Inch)</th>
<th>Depth from Surface (Inch)</th>
</tr>
</thead>
<tbody>
<tr>
<td>V1</td>
<td>0.5</td>
<td>12</td>
<td>39.25</td>
<td>1</td>
</tr>
<tr>
<td>V2</td>
<td>0.25</td>
<td>16</td>
<td>7.5</td>
<td>0.5</td>
</tr>
<tr>
<td>V3</td>
<td>1.0</td>
<td>12</td>
<td>16</td>
<td>1.25</td>
</tr>
</tbody>
</table>
Table 4.6: Delamination Schedule for S6 (Six Inch Specimen)

<table>
<thead>
<tr>
<th>Flaw Type/Name</th>
<th>Length (Inch)</th>
<th>Width (Inch)</th>
<th>Thickness (Inch)</th>
<th>X-Displacement (Inch)</th>
<th>Y-Displacement (Inch)</th>
<th>Depth from Surface (Inch)</th>
</tr>
</thead>
<tbody>
<tr>
<td>D1</td>
<td>2</td>
<td>3</td>
<td>1.5</td>
<td>2</td>
<td>30.0</td>
<td>26</td>
</tr>
<tr>
<td>D2</td>
<td>2</td>
<td>1.5</td>
<td>1</td>
<td>23</td>
<td>32.5</td>
<td>3</td>
</tr>
<tr>
<td>D3</td>
<td>4</td>
<td>4</td>
<td>0.5</td>
<td>9</td>
<td>25.5</td>
<td>2</td>
</tr>
<tr>
<td>D4</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
<td>22</td>
<td>19</td>
<td>2</td>
</tr>
<tr>
<td>D5</td>
<td>4</td>
<td>3</td>
<td>2</td>
<td>34.5</td>
<td>8.5</td>
<td>1</td>
</tr>
<tr>
<td>D6</td>
<td>3</td>
<td>3</td>
<td>1.5</td>
<td>30.25</td>
<td>9</td>
<td>1.5</td>
</tr>
<tr>
<td>D7</td>
<td>3</td>
<td>3</td>
<td>0.5</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Flaw Type/Name</td>
<td>Diameter (Inch)</td>
<td>Length (Inch)</td>
<td>Y-Displacement (Inch)</td>
<td>Depth from Surface (Inch)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>---------------</td>
<td>----------------</td>
<td>---------------</td>
<td>-----------------------</td>
<td>--------------------------</td>
<td></td>
<td></td>
</tr>
<tr>
<td>V1</td>
<td>0.5</td>
<td>12</td>
<td>37</td>
<td>1.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>V2</td>
<td>0.25</td>
<td>16</td>
<td>7</td>
<td>4.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>V3</td>
<td>1.0</td>
<td>12</td>
<td>16</td>
<td>0.5</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### Table 4.8: Delamination Schedule for S8 (Eight Inches Specimen)

<table>
<thead>
<tr>
<th>Flaw Type/Name</th>
<th>Length (Inch)</th>
<th>Width (Inch)</th>
<th>Thickness (Inch)</th>
<th>X-Displacement (Inch)</th>
<th>Y-Displacement (Inch)</th>
<th>Depth from Surface (Inch)</th>
</tr>
</thead>
<tbody>
<tr>
<td>D1</td>
<td>2</td>
<td>3</td>
<td>2</td>
<td>34.0</td>
<td>25.5</td>
<td>2.75</td>
</tr>
<tr>
<td>D2</td>
<td>2</td>
<td>1.5</td>
<td>1</td>
<td>23.5</td>
<td>30.5</td>
<td>4</td>
</tr>
<tr>
<td>D3</td>
<td>4</td>
<td>4</td>
<td>0.5</td>
<td>10.5</td>
<td>25</td>
<td>4</td>
</tr>
<tr>
<td>D4</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
<td>23.5</td>
<td>20.5</td>
<td>3.5</td>
</tr>
<tr>
<td>D5</td>
<td>4</td>
<td>3</td>
<td>2</td>
<td>36</td>
<td>9</td>
<td>2.25</td>
</tr>
<tr>
<td>D6</td>
<td>3</td>
<td>3</td>
<td>1.5</td>
<td>31</td>
<td>9.25</td>
<td>3</td>
</tr>
<tr>
<td>D7</td>
<td>3</td>
<td>3</td>
<td>0.5</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>
Table 4.9: Void Schedule for S8 (Eight Inches Specimen)

<table>
<thead>
<tr>
<th>Flaw Type/Name</th>
<th>Diameter (Inch)</th>
<th>Length (Inch)</th>
<th>Y-Displacement (Inch)</th>
<th>Depth from Surface (Inch)</th>
</tr>
</thead>
<tbody>
<tr>
<td>V1</td>
<td>0.5</td>
<td>12</td>
<td>37.5</td>
<td>3</td>
</tr>
<tr>
<td>V2</td>
<td>0.25</td>
<td>16</td>
<td>7</td>
<td>3.5</td>
</tr>
<tr>
<td>V3</td>
<td>1.0</td>
<td>12</td>
<td>16.5</td>
<td>2.5</td>
</tr>
</tbody>
</table>
Table 4.10: Crack Schedule for S4, S6, and S8

<table>
<thead>
<tr>
<th>Crack</th>
<th>Thickness</th>
<th>Length (Inch)</th>
<th>Depth in Concrete</th>
<th>X-displacement Inch</th>
<th>Y-displacement</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crack One</td>
<td>1 mm</td>
<td>1</td>
<td>1 inch</td>
<td>13.8</td>
<td>13.4</td>
</tr>
<tr>
<td>C1 @ 45 angle</td>
<td>1 mm</td>
<td>3</td>
<td>1 inch</td>
<td>30.5</td>
<td>35</td>
</tr>
<tr>
<td>C11</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Crack two</td>
<td>2 mm</td>
<td>1</td>
<td>2 inches</td>
<td>3.5</td>
<td>14.3</td>
</tr>
<tr>
<td>C2</td>
<td>2 mm</td>
<td>2.5</td>
<td>10.2</td>
<td></td>
<td>4</td>
</tr>
<tr>
<td>C22</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Crack three</td>
<td>3 mm</td>
<td>2</td>
<td>0.5 inches</td>
<td>20.8</td>
<td>22.6</td>
</tr>
<tr>
<td>C3</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Crack Four</td>
<td>3 mm</td>
<td>2</td>
<td>1 inch from surface</td>
<td>1.1</td>
<td>31.7</td>
</tr>
<tr>
<td>C4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Crack Five</td>
<td>1 mm</td>
<td>2.5</td>
<td>2 inches from surface</td>
<td>32.2</td>
<td>39.6</td>
</tr>
<tr>
<td>C5 @ 315 angle</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Figure 4.8: Specimen S4 Ready for Casting

Figure 4.9: Casting of Concrete Specimens
CHAPTER FIVE
IMPACT ECHO METHOD (IE)

This Chapter introduces the Impact Echo method (IE). IE is one of the very useful methods for the assessment of concrete and masonry structures. The chapter discusses the background of the method, the physical basis on which IE depends, talks about the advantages and disadvantages of using IE as a nondestructive method. Towards the end of the chapter, results of testing IE on the designed laboratory slabs will be presented and conclusions will be made on the usefulness of using IE for detection of concrete bridge decks common problems.

5.1 Background

Impact echo is a method of nondestructive evaluation of concrete structures. It is based on the use of sound waves generated by elastic impact. Waves are introduced to the surface of the concrete element (i.e. concrete bridge deck) using an impactor, which can be of different sizes ranging from 3-15mm depending on the flaw to be detected. The technique involves the use of waves with frequencies of 50 kHz to 300 kHz. It is applicable to structural concrete with a maximum aggregate size of 16 mm and thickness of up to 500 mm. As the sound waves are generated, they propagate through the probed material and are reflected by flaws and interfaces. The impact echo device consists of a broad band displacement transducer (receiver) consisting of a small, conically shaped piezoelectric element connected to a brass cylinder, a data acquisition, and a set of spherical steel balls as impactors. A thin sheet of lead approximately 4mm thick and is used between the conical element and the concrete.
surface to complete the circuit and provide coupling with the tested surface. It was found that a thicker lead sheet provides better coupling in the case of rough concrete \textsuperscript{13}, \textsuperscript{46}. In some of the systems, the impactors and transducer are housed in a hand held unit where the distance between the impactor and the receiver is approximately 30 mm \textsuperscript{41}. As the waves are reflected they cause surface displacements which are recorded by the transducer resulting in a displacement versus time plot, this is then transformed to the frequency domain using the Fast Fourier Transformation (FFT) mathematical concept. From those two plots and by noticing the patterns in the wave form and frequency domains information can be extracted about the condition of the material tested whether flaws and discontinuities are present. Figure 5.1 illustrates the concept of Impact Echo.

When properly used, Impact Echo has shown great success in determining the location and extent of several flaws in concrete such as delaminations, voids, honeycombing, and debonding along with other types of defects \textsuperscript{25}.

The method is considered one of the new methods for evaluation of concrete structures. Early works using this method started in 1983 at the National Institute of Standards and Technology and had evolved since, using computer simulation, finite element models, stress waves generated by elastic impact, the use of displacement transducers, and the final breakthrough by the use of frequency analysis \textsuperscript{36}. 
The method depends on the propagation of elastic waves within a solid. A wave is called elastic when it causes deformation within the elastic range of the material where it is traveling. That is, when a wave is applied with small magnitudes and for very short periods or it changes rapidly with time. These waves generate deformations that are within the elastic range of the material. Two types of elastic waves can propagate through solids. These are the dilational wave which is also called compression or primary waves (P-waves) and the distortional waves, also called shear waves (S-waves). A third type of waves propagates along the surface and plays a vital role in the interpretation of the results in the wave form and frequency domain is called the Rayleigh wave (R-wave).

The transient waves from impact echo are generated by mechanical impact. Small steel spheres are tapped against the surface. The sizes of the sphere balls vary between 3mm and 15 mm depending on the type of defect. Some of the important characteristics of the impact that determine the waves ability to propagate and detect
flaws in concrete is the impact time $t_c$, the diameter of the sphere $D$, and the kinetic energy produced by the impact.

The contact time is calculated using equation (5.1). Contact time depends on the sphere’s diameter and the condition of the concrete surface, an impact on a smooth hard surface has a shorter time than that of a rough concrete surface. The height from which the sphere was dropped has a minimum role in determining the contact time.\textsuperscript{47}

$$T_c = 0.0043 D \quad (5.1)$$

Where

$T_c =$ the contact time of the impact.

$D =$ diameter of the sphere used in impact.

The relationship between maximum frequencies and the sphere diameter is given by the equation (5.2) below.

$$F_{\text{max}} = \frac{291}{D} \quad (5.2)$$

Where

$F_{\text{max}} =$ the maximum useful frequency of the impact.

As the diameter of the sphere increases, it produces a lower range of maximum frequencies and has a higher contact time and amplitude. This is because it depends on the force of impact which is proportional to the size of the sphere.\textsuperscript{36} The contact time determines the frequency content and this is related as mentioned to the diameter of the sphere.\textsuperscript{41}

The propagation of the different types of waves characterizes each of the waves. P-waves travel in the direction of propagation and produce compression or tensile stresses and have the highest speed of all waves. In addition, the displacements caused
by the propagation of P-waves are at their maximum directly under the point of impact. It is larger than that of the displacement of the S-wave. R-waves travel radially away from the point of impact along the surface and have the lowest speed. On the other hand, S-waves travel in a perpendicular manner to the direction of propagation producing shear stress and have a speed that is in between the P-wave and the R-wave.

The speeds of these waves depend on a variety of factors such as the elasticity modulus, mass density and the Poisson’s ratio. Equations (5.3) and (5.4) are the equations that govern the speed of the waves and are listed below:

\[ C_p = \sqrt{\frac{E(1-v)}{\rho(1+v)(1-v)}} \]  \hspace{1cm} (5.3)

And \[ C_s = \sqrt{\frac{E(1-v)}{2\rho(1+v)}} \]  \hspace{1cm} (5.4)

Where

- \( C_p \) = P-wave speed
- \( C_s \) = S-wave speed
- \( E \) = modulus of elasticity
- \( V \) = Poisson’s ratio
- \( \rho \) = Mass density

The wave speeds of the P-wave, R-wave and S-wave follow the general equation of wave speed. The speed of the P-wave within the concrete varies from 3000 to 5500 m/s depending on the characteristics of the concrete. Equation (5.5) is the general equation for wave speed and is given below:

\[ C = f \lambda \]  \hspace{1cm} (5.5)

Where

- \( C \) = Wave speed
\[ f = \text{Frequency} \]
\[ \lambda = \text{Wave length} \]

The relation between the wavelength and the usefulness of the wave in detecting discontinuities is very important. Wavelength plays a big role in detecting the depth and dimensions of the discontinuity. In this context, stress waves with a wavelength of \( \lambda \) will be reflected from flaws having dimensions equal or greater to \( \lambda \). \(^{36}\)

As a wave hits a different interface or flaw its behavior changes. This change is due to a difference in acoustic properties between the two different materials or between the material and the discontinuity. The property that determines the behavior of the acoustic wave is the acoustic impedance \((Z)\), which is defined as "the opposition to the flow of sound through a surface; acoustic resistance is the real component of acoustic impedance and acoustic reactance is the imaginary component." \(^{48}\) When the wave hits an interface it can be reflected or refracted. The type of interface determines the way the waves behave in whether they reflect or refract. For example, at a solid to air interface most of the wave is reflected, in solid to solid interfaces waves experience refraction and reflection. The amplitudes of the refracted and reflected waves are given by equations (5.6) and (5.7).

\[
A_{\text{Reflected}} = A_i \frac{Z_2-Z_1}{Z_2+Z_1} \quad (5.6) \\
A_{\text{refracted}} = A_i \frac{2Z_2}{Z_2+Z_1} \quad (5.7)
\]

Where

\( A_i = \) the amplitude of the particle motion is the incident wave.

\( Z_1 = \) the acoustic impedance of the region where the wave is approaching the interface.

\( Z_2 = \) the acoustic impedance of the region beyond the interface.
The relationship between $Z_1$ and $Z_2$ defines the way the wave behaves as it hits an interface. The effect of the relative difference between $Z_1$ and $Z_2$ is explained in the following paragraph.

If $Z_2 \ll Z_1$ then $A_{\text{reflected}}$ approaches $-A_i$ and $A_{\text{refracted}}$ approaches zero. This means that the amplitude of the reflected wave approaches that of the incident wave the change in the sign indicates a phase change. This is the case between concrete and air (Crack, Delamination) the impedance of concrete exceeds that of air by $10^7$. On the other hand, if $Z_2 >> Z_1$, $A_{\text{reflected}}$ approaches $A_i$ and $A_{\text{refracted}}$ approaches $2A_i$. The amplitude of the refracted wave is equal to the amplitude of the incident wave, and the refracted wave amplitude is twice as much as the incident wave. This case exists when the boundary is concrete to steel where the impedance of steel is larger than that of concrete by 5-6 times. If $Z_2 = Z_1$, $A_{\text{reflected}}$ is zero and $A_{\text{refracted}}$ is 1. In this case all the wave energy is transmitted in the solid. This is the case in concrete on concrete patches. Reflection will only occur if there is a significant flaw $^36$.

Diffraction of P-waves occurs when the wave hits the edge of a crack. Waves travel outward in a cylindrical manner, and are important to calculate the depth of cracks.

As the P-wave travels within the tested material, it continuously loses some of its energy due to attenuation. Attenuation is due to absorption and scattering which is the result of internal friction that produces heat. Attenuation of the wave energy is one of the biggest problems in nondestructive tests as the waves lose their energy while propagating and can not reach deeper depths.
5.1.2 The Waveform

The results of the impact echo test are recorded as a waveform, which is a voltage–time signal. The receiving transducer reads the displacements caused by the impact of the small spheres and registers them against an analog voltage that is proportional to the displacement recorded. The waveform contains beneficial information about the frequencies of the reflected and refracted P-waves between the surface of impact and the different interfaces and flaws where the waves change behavior.

It is usually difficult to interpret the results using the waveform alone, but it is still vital to understand the nature of the waveform reflections to know if the test went fine. Figure 5.2 shows a typical waveform on a concrete slab.

In the case of bridge decks, the waveform is defined as the multiple reflections between the surface and the other interfaces or flaws. The P-waves provides information about the reflection times inside the tested structures while the R-waves travels along the surface. The reflections of R-waves from the edges of the deck are not recorded in the test due to the relatively large lateral dimensions to the thickness of the
structure in the small time of the test. The R-wave's arrival at the transducer causes a large downward displacement (negative voltage). Although these waves do not provide information about the flaws inside the concrete it does however, give information about the impact and the frequency content of the resulting stress wave.

The Multiple reflections of the P-waves within the structure provide the most important information in the impact echo test. The basic equation in impact echo in the case of bridge decks and concrete plates is given in equation (5.8) below.

\[ F = \beta C_p/2T \]

Where

\( F \) = Frequency of the P-wave

\( \beta \) = Shape factor for plates and is equal to 0.96

\( C_p \) = Wave speed

\( T \) = Thickness

Equation (5.8) is only valid for reflections from interfaces where the material at the interface is acoustically less stiff than the concrete. In other words if \( Z_1 > Z_2 \). For example, the concrete/air, concrete/water, concrete/soil interface follow equation 5.8 where the acoustic impedance of concrete defined by equation (5.9) is greater.

\[ \text{Acoustic impedance} = (\text{Density} \times \text{P-wave speed}) \]

When the underlying material is acoustically stiffer, it has higher acoustic impedance such that \( Z_1 < Z_2 \), for example, a concrete/steel interface. Equation (5.8) is no longer valid and is replaced by equation (5.10)

\[ F_{st} = \zeta C_p/4T_{st} \]

The responses obtained from steel will be discussed in details in section 5.9.
5.1.3 The Frequency Domain

As mentioned before, the interpretation of the Impact Echo test results using only the waveform is sometimes very complex. The determination of the arrival times and the important frequencies from which the results can be extracted is very difficult. A transformation is done from the waveform to the frequency domain where the important frequencies appear as distinct peaks. The transformation depends on the principle that any time dependant function can be represented as a sum of sine curves of different amplitudes using the Fourier equation.

5.1.4 Advantages and Disadvantages of the Method

IE is used for the detection of different flows and discontinuities in different materials especially concrete. The attention here will be given to concrete bridge decks. This method can be used for the detection of voids, cracks, delaminations, unconsolidated concrete, and debonding between interfaces, and used for the determination of thicknesses among many other benefits. IE test requires only one surface of the testes material to be exposed and is independent on the geometry of the structure unlike Ultrasonic testing. The IE test is considered reliably accurate with an error of about 2-3 %. In addition, the method provides information about the depth and extent of flaws detected, and the test is considered less time consuming and cost effective. And unlike GPR, the method is less susceptible to metal reinforcement.

IE has some drawbacks as well. The size of detected flaws is highly dependent on the impact duration (i.e. the size of the sphere used). It is also dependant on the wavelength which is in turn directly related to the knowledge of the lateral dimension of the flaw. The knowledge of the P-wave speed inside the tested material is hard to
acquire. IE is unable to detect grouted areas in concrete and the results tend to become less reliable in the presence of asphalt overlays. Aggregate size in the concrete mix may affect the accuracy of depth calculations as bigger aggregates cause more attenuation of the signal. The pulse propagates in all directions instead of a focused beam such as the case in ultrasonic, this means more difficulty understanding the reflected pulse in showing the different boundaries, and finally the interpretation of the results are difficult and need a lot of experience. Nonetheless, these shortcomings do not make this method inapplicable and its advantages surpass its drawbacks, which makes this method one of the best methods to use for bridge decks.

5.1.5 Measuring the P-wave Speed in Concrete

The P-wave speed can be determined using equation (5.8) in the form \( C_p = \frac{2f_t T}{0.96} \) where \( f_t \) is the thickness frequency. Uncertainties in the measurement of wave speeds using this method depend on the accuracy with which the thickness of the bridge deck or the specimen under consideration is known. The speed can also be determined from cores taken from the concrete specimen or cylinders during casting.

5.1.6 Impact Echo for Detection of Voids and Cracks

A crack or a void forms a concrete to air interface within a tested material, the difference in the acoustic impedance between air and concrete causes the incident wave to undergo reflection since it follows the case when \( Z_1 > Z_2 \). The thickness of the flaw doesn’t play a role in the reflection mechanism as the wave reflects from the first interface and does not travel through to detect its thickness. The response obtained from
a void or crack filled with water is very similar to that of an air filled void as the impedance difference is high in both cases\textsuperscript{36}.

As mentioned, the response from impact echo depends on the lateral dimension of the flaw. The lateral dimension of the flaw has to be at least one fourth of its depth in order for IE to be able to detect it\textsuperscript{41}. If the flaw is very deep and small it would be unlikely for impact echo to detect it.

The impact duration for the case of voids and cracks plays a big part in acquiring good results. It determines the frequency content of the waves\textsuperscript{13, 46}. Higher frequency contents mean shorter wave length appropriate for detection of smaller objects and flaws. The requirements for detection of voids and cracks also include wavelength which has to be less than the lateral dimension of the flaw, and less the twice the depth of the crack or void. These requirements determine the frequency using equation (5.5) which leads to the determination of the impactor size by determining the impact duration using equation (5.11) below.

\[ t_c = \frac{1.25}{f_{max}} \]  \hspace{1cm} (5.11)

Where

- \( t_c \) = impact duration

- \( f_{max} \) = maximum frequency

In practice, since the depth of the voids and the lateral dimensions are not known, different sizes of impactors are used to determine whether the area that is under study has a defect. The inspectors' experience plays a big role in the selection of the impactors' size. An approximate relationship for choosing the impactor size depending on the dimensions of the flaw is given in the Table 5.1 below (from IE book).
Table 5.1: Approximate Relationships between Sphere Diameter, Contact Time, Depth and Size of Flaws

<table>
<thead>
<tr>
<th>Sphere Diameter (mm)</th>
<th>Contact Time (µs)</th>
<th>Max. Useful Frequency (KHz)</th>
<th>Minimum Depth of flaw that can be detected (mm)</th>
<th>Minimum Size ( l ) of flaw that can be detected (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>22</td>
<td>57</td>
<td>35</td>
<td>70</td>
</tr>
<tr>
<td>6.5</td>
<td>29</td>
<td>43</td>
<td>46</td>
<td>91</td>
</tr>
<tr>
<td>8</td>
<td>35</td>
<td>36</td>
<td>56</td>
<td>112</td>
</tr>
<tr>
<td>9.5</td>
<td>42</td>
<td>30</td>
<td>67</td>
<td>133</td>
</tr>
<tr>
<td>11</td>
<td>48</td>
<td>26</td>
<td>77</td>
<td>154</td>
</tr>
<tr>
<td>12.5</td>
<td>55</td>
<td>23</td>
<td>88</td>
<td>175</td>
</tr>
</tbody>
</table>

For the case of cracks detection, the optimum test configuration was found when the receiver and the impactor are placed on different side of the crack with a relatively small spacing \(^{49}\). The P-wave generated by the impact is diffracted from the edges and bottom of the crack. Thus, to be able to determine the depth of a crack, the distance from the location of the impactor to the closest edge of the crack must be greater than the cracks depth. Same applies to the distance from the receiver to the crack. This is because the first reflection will occur due to the wave hitting the edge of the crack. The calculated depth will represent the distance to the edge of the crack rather than its depth. It was found that the size of impactor or the impact duration do not play a role in calculating the crack’s depth. The time-of-flight technique is used to detect the arrival times of the \( P_dP \)-waves, these are the spherical waves that are diffracted when the incident wave strikes the cracks tip. The depth of the crack \( D \) can be determined using equation (5.12) and (5.13) \(^{49}\).
\[ D = \sqrt{\left\{(C_p \Delta t) + H_1^2 - H_2^2\right\}/2 C_p \Delta t}^2 - H_1^2 \]  

(5.12)

Where

\( C_p \) = P-wave speed in the structure

\( H_1 \) = distance between the impact point and the crack

\( H_2 \) = distance between crack and transducer

When \( H_1 \) is equal to \( H_2 \)

\[ D = \sqrt{\left\{(C_p \Delta t)/2\right\}^2 - H^2} \]  

(5.13)

Figure 5.3 shows an illustration of the Time-of-Flight technique for determining the depth of surface cracks.

Figure 5.3: Time-of-Flight Technique Schematic Configuration

Inclined cracks are characterized by their angle of inclination (\( \beta \)) measured between the center line of and the crack’s tip, as shown in figure 5.4. Crack’s depth is measured as the perpendicular distance from the surface to the crack’s tip. Equations 5.10 and 5.11 are used to determine the depth. In most of the cases, the calculated depth will be larger than the real depth but with a slight difference that increases as the angle of inclination increases\(^{49,51,52}\).
5.1.7 Impact Echo for Detection of Delaminations

The same concepts of the detection of cracks and voids are applicable in the case of delaminations. The deck is usually divided into grids of 1m by 1m (3 ft * 3 ft) or less. In the case of presence of very shallow delaminations, a very small impactor is used. Shallow delaminations result in two modes, vibration of the thin layer above the delamination and the P-wave reflection. These two cause low and high frequency consecutively but the determination of the delaminations’ depth will be hard to acquire.

The number of peaks resulting in the spectra also depends on whether the location of the impactor is in the middle of the delamination or towards the edge. In the ideal case, the spectrum obtained from an impact echo test over a delamination is expected to have two dominant frequency components. The first is a peak corresponding to the multiple reflections of the P-wave between the top surface and the delamination, and the second is a peak or possibly multiple peaks correspondent to the flexural modes of vibration of the delaminated concrete. These peaks are called the thickness frequency peak and the flexural frequency peaks consecutively. Flexural
frequency peaks usually have longer periods compared to those of the thickness vibrations. As mentioned earlier, the stress pulse produced by impact must be in the range of the thickness frequency so as to be able to detect a delamination.

5.1.8 Impact Echo and the Response from Steel Reinforcement

In the event of the presence of reinforcement in concrete, the frequency obtained is usually a little higher than that obtained by mathematical calculations using equation 5.10 without the value $\zeta$. The presence of steel bars inside the concrete specimen results in a set of closely spaced multiple peaks.

The frequency spectra obtained from the impact echo test over steel rebars is dependant on the bar diameter, and depth of the rebars and thus the ratio of the bar diameter to its depth $D/t$ is of importance in studying the response of steel reinforced concrete. It was found that the ratio $D/t$ has to be at least 0.3 in order for the impact echo device to be able to detect a response from steel. If the ratio was less than 0.3 the peaks produced by reflections from the rebar cannot be clearly identified in the response. Figure 5.5 shows a typical section of a concrete slab with rebars showing the depth and diameter of the rebars.

The responses from steel rebars in the spectra are obtained by using small diameter, small contact time impactors. The impact has to have a contact time $t_c$ of less than 30 micro seconds in order for impact echo to be able to detect the rebar. The typical response of the test will contain several peaks centered at $f_{\text{rebar}}$ if the test was done right on top of the rebars.
As the ratio \(\frac{D}{t}\) is of high relevance to the response from steel bars, studies showed that the amount with which the response is different than the calculated frequency is a direct function of the diameter to depth ratio. This observation led to a new empirical relation showing how to calculate the value of \(\zeta\) in equation (5.14) for bars with \(0.3 < \frac{D}{t} < 1\).

\[
\zeta = -0.6 \frac{D}{t} + 1.5
\]  

(5.14)

Using this equation along with equation 5.10 can help the operator determine the concrete cover or diameter of the steel rebars if either of them is known.

As mentioned, the impact duration is a very important variable upon which the detection of steel in concrete specimens using IE depends. The impact duration has to be usually less than 30 microseconds. If longer durations are used, the result in a reduction or elimination of the peaks corresponding to reflections from steel reinforcing bars because the stress waves generated do not contain sufficient energy to be reflected.
This in some cases could be beneficial if the response of steel rebars is of no significance to the test results but it will also reduce the size of flaws detected to dimensions larger than the rebar diameter.

5.2 Testing and Results

Testing of the concrete slabs was done using impact echo test in two stages, the first with the Styrofoam blocks present and the second after dissolving the Styrofoam blocks. The results will be discussed to confirm impact echo’s ability to detect the presented flaws in the specimens.

5.2.1 P-wave Speed Measurement

The first step in performing the impact echo test was to determine the P-wave speed within the concrete. A small concrete specimen with known dimensions and thickness 15* 17*3.5 inches (381*432*89 mm) was prepared during casting of the specimens for the purpose of measuring the P-wave speed in concrete. After performing the P-Wave speed measurements on this specimen, it was found that it was not practical to get a good reading from this specimen. The reason behind it was that the specimen was very small and the reflections from the sides would arrive to the receiver before the reflection from the full thickness is received. This causes the resonance of the slab boundaries to be setup at an earlier stage and cause the complication of the spectra. The wave speed had to be determined using another approach. Literature suggested that wave speed can be determined using cores or cylinders. Another approach that was used in this test was measuring the thickness frequency of a concrete slab containing a steel layer. Steel can not be detected if an impactor of a contact time of more than 30
Microseconds was used. This corresponds to any impactor 8 mm or bigger. Figure 5.6 shows the little slab and the cylinder used to calculate P-wave speed.

![Figure 5.6: The Slab and Cylinder Used to Determine C_p](image)

The results obtained from the test done on the cylinder 6 inches in diameter and 12 inches long are presented in figure 5.7 below.

The P-wave speed in this case is calculated using equation (5.15) below.

\[ C_{\text{core}} = 2 \times f_1 \times L \]  

(5.15)

Where

\[ C_{\text{core}} = \text{The P-wave speed inside the cylinder or core.} \]

\[ f_1 = \text{fundamental longitudinal frequency} \]

\[ l = \text{length of the cylinder} \]
From the equation above, the value of \( C_p \) was calculated to be 4390 m/s. This is only an approximation of the wave speed.

The determination of the P-wave speed was also validated by testing three specimens of known thicknesses containing steel, figures 5.8 and 5.9 show sample results obtained from testing for \( C_p \) on the 6 and 8 inch thick specimens respectively. The response obtained in these figures is similar to what would have been obtained in the case of the absence of steel reinforcement as for solid plates. This was acquired by using an impactor of 10 mm diameter.
These responses had a single peak in the spectra corresponding to the thickness frequency from which the P-wave speed was calculated to be 4470 m/s.

The Impact Echo test was performed on the locations of delaminations, voids and close to cracks to determine how the response differs from other locations of sound concrete. It was apparent that impact Echo was successful in determining the locations of delaminations and voids. The responses obtained from cracks were complicated.

The following paragraphs will discuss the responses obtained from each of the defected specimens when the test was performed on the locations of the flaws before the Styrofoam was melted. The presence of Styrofoam had no effect on the obtained results.
The results of the Impact Echo test for the defected four inch specimen (S4) are presented in this section. All the flaws in this slab had depths less than 4 inches which makes them within the shallow delamination category. This means that the responses expected from these flaws are going to be different than the solid response of the slab, but, the information obtained from these tests will not be enough to extract information about the depths of the flaws. The spectra will show peaks correspondent to the flexural vibration modes of the concrete above each of the flaws. These frequencies will usually be less than the thickness frequency and much less than the flaw frequency. The following paragraphs explain the responses of each of the flaws inside the four inch specimen.

DELAMINATIONS

Figure 5.10 shows the response obtained from a test done at the center of delamination one with dimensions 2*3*1.5 inch (50*76*38 mm) and at a depth of 1 inch (25.4 mm). For this test a 10 mm (0.4 inch) impactor was used
Figure 5.10: Impact Echo Response above Delamination 1 in S4

Observing the response, one major peak appears in the spectra, the delamination’s lateral dimension was large compared to its depth. Thus, the response of a solid plate was expected. The calculated $f_{\text{delamination}} = 75.9$ kHz. The thickness frequency $f_{\text{thickness}} = 24.8$ kHz. Both values were calculated using equation 5.8. The obtained response has a frequency of 18.6 kHz which corresponds to downshifted thickness frequency for the concrete above the delamination. When a case like this exists the depth of the delamination can not be calculated but the presence of such a response gives an indication that there is a flaw present since the thickness frequency was not achieved. In this case the impactor was too large that no response was obtained from the steel bars.

Figure 5.11 shows the impact echo result done on delamination two $2*1.5*1$ inch ($50*38*25.4$ mm) at depth of 0.75 inch (19 mm). The response in this case will be the same as that obtained for delamination one as shown.
The impact echo test couldn’t achieve \( f_{\text{thickness}} \) because of the presence of a shallow delamination. Since the delamination is shallow, the response can not be used to calculate the depth of the delamination.

Figure 5.12 through 5.16 show responses obtained from delaminations D3, D4, D5, D6, D7 respectively. As all these flaws are less than 4 inches deep, they are all considered shallow and the responses expected from all of them would be a single or multiple peaks corresponding to the flexural modes of vibrations of the concrete plate above the flaw. These peaks will be less than \( f_{\text{thickness}} \) and can not be used to calculate the thicknesses. In all the cases, the response of the steel inside the concrete block was negligible as the flaws had big lateral extent compared to their very shallow depth.
Figure 5.12: Impact Echo Response above Delamination 3 in S4

The response from figure 5.13 above delamination 3 shows the frequency of the flexural vibrations of the concrete layer above the delamination. The value of \( f_{\text{Flex}} \) shows that a shallow flaw is present but information about the delamination can not be obtained.

Figure 5.13: Impact Echo Response above Delamination 4 in S4

The response obtained from figure 5.14 shows a downshifted thickness frequency value at 20.4 kHz and a lower frequency high amplitude peak corresponding to \( f_{\text{Flex}} \). The
impact was towards the edge of the delamination and done with an 8 mm impactor. The reflection from steel could not be detected with contact time provided by this impact.

Figure 5.14: Impact Echo Response above Delamination 5 in S4

The response from figure 5.15 is the same as that for delamination 3 in figure 5.13 the flexural frequency shows in the figure.

The response obtained in figure 5.16 above delamination 6 and that obtained in figure 5.17 above delamination 7 show responses from flexural vibrations. These two cases are identical to that of figure 5.13 and 5.15. The frequencies obtained are those caused by the flexural vibration of the concrete above the delaminations.

VOIDS

Figures 5.17, 5.18, and 5.19 show responses obtained for voids V1, V2, and V3 respectively. The responses obtained from these tests show a downshift in the thickness frequency caused by the presence of the voids. More information about the voids couldn’t be collected as they were very shallow. The downshift on the other hand shows that the waves took longer time to reach the full thickness indicating presence of flaws.
Figure 5.15: Impact Echo Response above Delamination 6 in S4

Figure 5.16: Impact Echo Response above Delamination 7 in S4
Figure 5.17: Impact Echo Response above Void 1 in S4

Figure 5.18: Impact Echo Response above Void 2 in S4
The responses obtained from the tests done near cracks as per Table 4.1 obtained a different response when compared with tests done over sound concrete. The test was done with the transducer on one side of the crack and the impact on the other placing them almost perpendicular to the crack direction. We were not able to calculate the depth of cracks because a second transducer was not available for this study, more information about the method of determining the surface crack depth is available in Sansalone 1998.

Figure 5.20 shows the response from crack C1. The reflection obtained from this test shows several low frequency peaks. These correspond to the peaks obtained from the reflections of the P-wave with the crack and a higher frequency low amplitude peak close but a little less than the thickness frequency due to the reflection of the P-wave from the bottom of the slab. The smaller thickness frequency value shows that the crack
was detected and caused a longer travel time for the P-wave to reach the thickness of the plate.

Figure 5.21, 5.22, 5.23, and 5.24 show the response from crack C11, C2, C22, and C5 respectively, these responses are identical to that of figure 5.19. Responses from crack C3 and crack C4 could not be obtained because the concrete rough and it was very difficult to establish full contact between the transducer and the concrete surface.

Figure 5.20: Impact Echo Response above Crack C1 in S4
| Thickness, mm | 50 |
| Wave Speed, m/s | 4469 |
| Thickness | 24.8 |

Figure 5.21: Impact Echo Response above Crack C11 in S4

| Thickness, mm | 50 |
| Wave Speed, m/s | 4469 |
| Thickness | 24.8 |

Figure 5.22: Impact Echo response above Crack C2 in S4
5.2.3 The Six Inch Specimen

IE tests were performed on the sound six inch (S6C) specimen several times and the response obtained was presented in figure 5.8. Tests were also performed for the defected specimen (S6) which was 6 inches deep. The following paragraphs discuss the results obtained from those tests.
DELAMINATIONS

The results obtained after the test was done above delamination number one which is 2 * 3 * 2 inches (50*76*50) and a depth of 1 inch (25.4 mm) is shown in figure 5.25 below. The high peak in the spectra is correspondent to the downshifted thickness frequency as it is very shallow.

Figure 5.25: Impact Echo Response above Delamination 1 in S6

The response of the delamination was supposed to be that of a solid plate as the lateral dimension of the flaw is more than 1.5 of the depth which is 1 inch in this case (25.4 mm). However, in this case the shallow delamination needed very high energy content in the impact to be detected. The frequency response from the delamination was calculated from equation (5.8) to be 84.5 KHz. No impactor can achieve this very high frequency so the delamination could not be detected. The size of the impactor as mentioned earlier in this chapter depends on the wavelength $\lambda$ which in turn depends on the lateral dimension (l) and the depth of the flaw (D) such that $l > \lambda > 2d$. For this particular case, the impact used had to have a wave length less than 3 inches and less
than 2 times the depth of 1 inch. To achieve both conditions λ had to be less than 2 inches. Using equation 5.5 the value of F was calculated, and then substituting that into equation 5.11, contact time \((T_c)\) was calculated and from equation 5.1 the sphere diameter \((D)\) is calculated. Sphere diameter in this case was 3 mm. The expected response was that the delamination is very shallow and that the delamination frequency calculated would not be achieved. Rather, a single peak of a lower value would show the presence of the delamination but without giving us the ability to calculate the depth of that delamination.

Results of the test above delamination number two with dimensions \((2*1.5*1)\) and at a depth of 3 inches is shown in figure 5.26 below. Response obtained shows a single peak at a value lower than that of the solid thickness response at 14.6 kHz and a little peak showing at around 28.3 kHz. The calculated \(f_{del}\) was 28.1 kHz which is very close to the value obtained from the IE test and corresponds to a depth of 79 mm (3.1 inches) as shown in the rectangle in the right side of figure 5.10. The real depth of the
delamination was 76 mm (3 inches). The downshift in the thickness frequency from 16 kHz to 14.6 kHz is due to the longer path the P-wave took to travel around the delamination.

Figure 5.27 presents results obtained from the test done above delamination three. Delamination three was 4*4*0.5 (101*101*13 mm) and located at depth of 2 inches (50 mm). The depth is within the shallow range so the reflection would look like that obtained from delamination one. A distinct peak at 8.8 kHz is less than that of both the delamination and thickness frequency. It is caused by the flexural vibration of the thin concrete layer above the delamination. The calculated delamination frequency using equation (5.8) was 42.2 kHz with a wave speed of 4469 m/s. As was the case of delamination 1, the lateral dimension of the flaw was more than 1.5 the depth. The wavelength used in this test had to be less than 4 inches which corresponds to a 7mm impactor. Using this information an \( f_{\text{flex}} \) value of 8.8 kHz was obtained which is less than that of \( f_{\text{thickness}} \) value of 15.6 kHz which shows that a flaw is present but without
offering the ability to calculate the correct depth of the flaw because it needs a very small impactor to detect it. The figure also shows a comparison between the results obtained using a 12 mm impactor shown in the solid black line and a 6 mm impactor in the red dotted line. The difference is that the 6 mm impactor accentuated some of the low frequency responses but didn’t have enough energy to detect the very high frequency delamination response. The use of a lower contact time impactor would be useful in a case like this. The fact that the specimen’s surfaces were very rough made it very hard to obtain low contact times to detect such defects.

Figure 5.28 present the results obtained from a test above delamination 4 which was 1.5*1.5*1.5 (38*38*38 mm) and at a depth of 2 inches (50 mm). The response expected in this case is the same as that obtained for delamination 2 as the lateral dimension is not more than 1.5 the depth, so the reflection will not be like that of a solid plate. The reflection will have two distinct peaks if the impact was done towards the center of the delamination. If otherwise more than two peaks will be present in the waveform and a frequency very close to the thickness frequency will also appear in the results. In the case of a solid plate response as that obtained for delamination one and three, if the test was done closer to the edge of the flaw and not close to the center the response will be quite different and two or more peaks will be present as the waves will be reflected of the edge of the flaw and continue traveling to the other side of the specimen. So a thickness frequency and a flaw frequency would be present in this case.
Figure 5.28: Impact Echo Response from Delamination 4 in S6

The response obtained here was a single peak with a frequency less that that of \( f_{\text{thickness}} \). This is because the impact did not have enough energy to detect the delamination with \( f_{\text{delamination}} = 42.2 \text{ kHz} \). The downshift in the thickness frequency from 16 kHz to 13.7 kHz gives an indication of the presence of the flaw. A smaller impactor was needed with a lower contact time. The high roughness of the surface prevented us from obtaining such a low contact time.

Figure 5.29 present the results obtained from a test done above the center of delamination number five. The results look identical to those of delamination one and three where a solid plate response was expected for a shallow delamination. The lateral dimension was larger than 1.5 the depth, on the other hand, the delamination is less than 4 inches deep which makes it a shallow delamination. The expected response is a single peak in the spectra correspondent to the flexural modes of vibration.
The frequency value obtained in this test was 8.8 kHz. The frequency value calculated for the delamination using equation (5.8) was 84.5 kHz. The single peak is the $F_{\text{flex}}$ as anticipated which is much less than that of $F_{\text{delamination}}$. This result, and since it is less than $F_{\text{thickness}}$ of 16 kHz it gives the indication of the presence of a flaw but doesn’t give information about its depth. No impactor would have been able to give information about such a shallow delamination.

The test done on delamination number 6 is presented in the following figure. The response in figure 5.30 will be comparable to that of 5.29 and all the similar cases as the response is of a solid plate since the lateral dimension is more than 1.5 the depth. The delamination is $(3*3*1.5)$ at a depth of 1.5 inches. This means that the calculated $F_{\text{delamination}} = 56.3$ kHz. The response obtained from the test shows a single peak at 9.8 kHz. This is correspondent to the flexural vibration as mentioned earlier and is less than $F_{\text{Thickness}}$ and $F_{\text{delamination}}$ which prompts the inspector to think of the presence of a shallow defect. The impactor used in this test was the 10 mm impactor shown as the
solid black line in figure 5.30. The results were compared with a smaller impactor of lower contact time (6 mm impactor). The results show that the 6 mm impactor accentuates some details but still does not have the energy required to detect the shallow delamination as the energy dies near 50 kHz as shown in figure 5.30.

**VOIDS**

Figure 5.31 shows the results obtained from a test done above void number one, the test was set up such that the impactor and the receiver are right on top of the void and in the longitudinal direction. The response shows a single peak at a value lower than that of the thickness frequency. This prompts the inspector to think of the presence of the flaw. A response near the $f_{\text{void}}$ was not obtained because the 10 mm impactor had no sufficient energy in the range of the expected frequency from the flaw. The usage of a smaller impactor was not applicable because smaller contact time was hard to achieve as the surface above the void was rough.
Figure 5.31: Impact Echo Result for Void 1 in S6

Figure 5.32 shows the response from a test done above void V2 at a depth of 4.5 inches (116 mm). The peak at 15.6 kHz is correspondent to the full thickness of the plate and is one digital point less than the 16 kHz thickness frequency, the peak at 18.3 is correspondent to the reflection from the top of the void.

Figure 5.32: Impact Echo Result for Void 2 in S6
Figure 5.33 shows the response obtained above void number three which is a shallow flaw. Thus, the response will contain an $f_{\text{flex}}$ value. Rather in this case since the void is large in lateral dimension the response only showed a downshifted thickness frequency.

The downshifted value of the thickness frequency show that there is a flaw present, but the flaw can not be detected with impact echo as it need much higher frequency content than any of the impactors can provide. Thus, the P-wave took longer time to travel around the flaw and reach the full thickness. But generally the impact echo method showed that it is not able to give valuable information about very shallow defects.

**CRACKS**

The responses from the cracks were also obtained and compared to the solid response, the responses show several low frequency peaks along with a thickness
frequency peak for the plate. These peaks are due to the reflections of the P-wave between the tip of the crack and the transducer as explained in the 4 inch specimen case. Figure 5.34 through 5.39 shows the reflections obtained when the test was performed at the locations of cracks C1, C11, C2, C22, C3, and C5. A response from C4 was not possible to obtain because the surface around the crack was rough that a full contact between the transducer and the concrete couldn’t be established.

Figure 5.34: Impact Echo Result for Crack C1 in S6

Figure 5.35: Impact Echo Result for Crack C11 in S6
Figure 5.36: Impact Echo Result for Crack C2 in S6

Figure 5.37: Impact Echo Result for Crack C22 in S6
5.2.4 The Eight Inch Specimen

The defected eight inch specimen (S8) was tested in the same manner as the other two slabs before and after the Styrofoam was melted. The results shown here are for tests before melting the Styrofoam. The results shown is the following figures show...
the responses obtained from tests performed above the different flaws. The response obtained from the test above slab S8C with no defects is shown in figure 5.9.

DELAMINATION

Figure 5.40 shows the response above delamination D1. The response expected from this flaw is two peaks, one corresponding to the reflection of the P-wave from the delamination and the other as a response from the total thickness.

![Image](image_url)

Figure 5.40: Impact Echo Response from Delamination 1 in S8

The response shows a peak at 9.8 kHz and another at 30.3 kHz. $f_{thickness}$ was calculated to be 11.7 kHz. The drop in the frequency from 11.7 kHz to 9.8 kHz is due to the presence of a flaw. The peak at 30.3 corresponds to a reflection at a depth of 74 mm as shown in the rectangle in the right of the figure. The delamination was located at 71 mm from the surface and had a calculated $f_{delamination}$ of 30.7 kHz.
Figure 5.41 shows the response obtained from a test done at the center of delamination 2. The response expected in this case will be very similar to that of delamination one above.

The obtained response included a frequency that is lower than the thickness frequency at 10.7 kHz due to the presence of a flaw. The frequency showing at 21.5 kHz corresponds to a depth of 104mm (4.1 inch). The delamination was at a depth of 102 mm (4.1 inch) which is in agreement with the result obtained here. The frequency response of the delamination appears low because the impactor used was large. Switching to a smaller Impactor would have helped in obtaining a more apparent response from the delamination.

The response obtained above delamination number 3 is discussed in figure 5.42. The expected response is very much the same as that of D1 and D2. Most the delaminations and flaws in the eight inches slab are more than 4 inches deep and can be considered as normal (not shallow) flaws.
The response from delamination number four is shown in figure 5.43 and is expected to have the same characteristics as those mentioned earlier.

The figure shows a comparison between responses obtained using a 6 mm impactor which contained the required energy to detect the flaw, and another response using a 10 mm impactor which didn’t have enough energy and couldn’t show any signs
of peaks at 24.4 kHz which is the frequency corresponding to the delamination. The maximum frequency that can be picked up by the 10 mm impactor was 23 kHz. This is less than the frequency response from the delamination and thus couldn’t detect it as shown.

Figure 5.44 shows the response from delamination five that is 2.25 Inch (57 mm) deep. $f_{\text{Delamination}}$ calculated using equation 5.8 was about 37.5 kHz. The response expected is that of a shallow delamination. The first peak at 7.8 kHz is due to flexural vibrations. The figure also compares the responses obtained using a 10 mm in the solid line and 6 mm in dotted red line impactors. The larger impactor doesn’t have enough energy to detect the delamination. When a smaller one was used, a little peak is apparent at 37.1 kHz. This peak is correspondent to a depth of 60 mm. This is comparable to the 57 mm depth of the delamination. The response is week and the peak has low amplitude the contact time of the steel sphere is high due to roughness of the surface.
Figure 5.45 shows the response acquired from a test above delamination 6. The response is similar to those of D1 and D2. A frequency less than the thickness frequency and another close to that of the delamination will be present. A comparison between the 10 mm impactor in the dotted line and the higher energy 6 mm impactors is presented again. Two peaks are labeled. The little labeled peak corresponds to the depth of the delamination. The 6 mm impactor contained energy in the range required to detect the delamination while the 10 mm had no sufficient energy to detect the delamination. The downshift in the thickness frequency alone in the case of the 10 mm impactor tells us that a flaw is present.

**VOIDS**

The response in figure 5.46 represents a test done above void 1 in the eight inch specimen.
The response in figure 5.46 is a comparison between the solid response presented with the dotted line, and the response above void number one. The test was set up such that the impactor and the transducer were right on top of the longitudinal direction of the void as the void is very long in one lateral dimension. The response obtained had a low frequency, high amplitude peak corresponding to the thickness of the slab but will a little downshift from the solid response as shown. This is due to the presence of the void. The three lower peaks correspond to multiple reflections between the void and the transducer. These are centered on a value of 28.3 kHz. The calculated $F_{\text{void}}$ using equation 5.8 is 28.1 kHz while the peak in the figure corresponds to a depth of 79 mm (3.1 inches). This is close to the depth of the void at 77 mm (3 inches). The 18.6 kHz is neglected in this case as it represents almost twice the thickness frequency for this case.

Figure 5.47 shows the comparison between the responses obtained over void number two. This case is identical to the preceding for void number one.
The responses show the thickness frequency, the frequencies centered around 24.4 kHz. This is close to the calculated $f_{\text{void}2}$ of 24.1 kHz. The thickness corresponding to the 24.4 kHz value is 90 mm (3.54 inches) while the void was actually at a depth of 88 mm (3.46 mm).

The response obtained from a test performed above void number 3 is presented in figure 5.48. The results shown in this figure are similar to those obtained from the previous two voids. The comparison shows the solid plate response as opposed to this of the void.

The frequency obtained is lower than that of a solid plate due to the presence of the flaw. The three peaks correspond to the modes of reflection by the void and are centered at 33.2 kHz. This corresponds to a depth of 67 mm (2.6 inches). The real depth of the void was 64 mm (2.5 inches).
Figure 5.48: Impact Echo Response from Void 3 in S8

CRACKS

Figure 6.49 shows the response obtained when the test was done close to crack two C2, the response shows several low frequency peaks along with a thickness frequency peak for the plate. These peaks are due to the reflections of the P-wave between the tip of the crack and the transducer as explained in the 4 inch specimen case. Figures 5.50, 5.51, and 5.52 show the reflections from C22, C3, and C5 respectively. A response from C1 and C11 were not obtained in this test.
Figure 5.49: Impact Echo Response from Crack 2 in S8

Figure 5.50: Impact Echo Response from Crack 22 in S8
Figure 5.51: Impact Echo Response from Crack 3 in S8

Figure 5.52: Impact Echo Response from Crack 5 in S8

5.3 Tests after Dissolving the Styrofoam Blocks

Tests performed at the centers of the delaminations in the defected six inch specimen (S6) after the Styrofoam blocks were dissolved showed no difference in the obtained response when compared to tests done on the present of Styrofoam blocks.
Figures 5.53, 5.54, and 5.55 show the responses obtained from tests performed on the centers of delaminations D3, D5, and D6 respectively. These results are identical to those performed on the same delaminations in the presence of Styrofoam shown in figures 5.27, 5.29, and 5.30.

5.4 Discussion of the Results

The results obtained from the impact echo test on the defected specimens showed the ability of impact echo to detect flaws in concrete. This ability depended on several variables. The depth of a flaw, its lateral extent, impactor's size, and contact time played a vital role. Relationships between the size of impactor, contact time, size of the flaw and the depth were discussed. It was also validated that the location of the impact is important to understand the obtained response. Response from steel reinforcement was also discussed. Steel layer detection needs special techniques and the use of special impactor sizes. The results obtained from this study are summarized in Table 5.2.
Figure 5.53: Impact Echo Response from Delamination D3 with no Styrofoam in S6

Figure 5.54: Impact Echo Response from Delamination D5 with no Styrofoam in S6
Figure 5.55: Impact Echo Response from Delamination D6 with no Styrofoam in S6
### Table 5.2: Summary of the Results Obtained from Impact Echo Testing

<table>
<thead>
<tr>
<th>SLAB</th>
<th>DELAMINATIONS</th>
<th>VOIDS</th>
<th>CRACKS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>D1 D2 D3 D4 D5 D6 D7</td>
<td>V1 V2 V3</td>
<td>C1 C11 C2 C22 C3 C4 C5</td>
</tr>
<tr>
<td>FOUR INCHES DEFECTED</td>
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<td>Y Y Y Y</td>
<td>YN YN YN YN NA NA YN</td>
</tr>
<tr>
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<td>Y YD Y Y</td>
<td>YN YN YN YN YN NA NA YN</td>
</tr>
<tr>
<td>EIGHTH INCHES DEFECTED</td>
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<td>YD YD YD</td>
<td>NA NA YN YN YN YN YN YN</td>
</tr>
</tbody>
</table>

**Legend:**
- **Y** = flaw was detected without the ability to calculate its depth
- **YD** = the defect was detected and its Depth was Calculated
- **YN** = Flaw was detected but no attempt to calculate its depth was made.
- **NA** = test was not done to detect this flaw
CHAPTER SIX

GROUND PENETRATING RADAR (GPR)

Chapter six of this thesis talks about the Ground Penetrating Radar method (GPR). This versatile method is used in various applications. The use of GPR in the area of Civil Engineering will be discussed in this chapter. The focus will be on concrete bridge decks. The GPR test will be conducted on the designed laboratory specimens to verify the capabilities of the method. The results will be introduced and conclusions will be made about the practicality of GPR use for concrete bridge decks problems.

6.1 Background

Ground Penetrating Radar is commonly abbreviated as GPR. It is a high resolution electromagnetic technique designed to map the subsurface of the earth\textsuperscript{45} by employing the use of radar (Radio Detection and Ranging) and radio waves emitted from a source to detect an object and determine the size, direction, distance and properties of the object after it re-emits some of the energy that impinges on it\textsuperscript{39}. GPR uses electromagnetic waves to probe lossy dielectric materials to detect changes in materials properties\textsuperscript{38}. GPR depends on the principle of scattering of electromagnetic waves to locate buried objects in any non metallic material by relating the delay time and the amplitude of the return signal to objects\textsuperscript{6,39,53}.

Penetration of electromagnetic waves in lossy dielectric materials occurs when the GPR machine sends signals inside the tested material. This signal is then returned to the receiver and information about the changes in wave characteristics is observed.
Penetration depth is an issue in GPR surveys because of the attenuation that takes place due to absorption and scattering.

The GPR method is used in environmental engineering, archaeology, and a variety of other applications. Although it was originally used for geophysical surveys such as subgrade investigations, it proved to be very effective for the assessment of integrity and condition of concrete structures and in civil engineering applications in general. Recently, GPR has evolved as one of the very reliable methods for civil engineering applications especially in concrete bridge decks.

The concept of GPR relies on the travel of an electromagnetic wave in the frequency range of 10 MHz to 2000 MHz\(^5\)\(^{39}\) through a dielectric material. This wave is reflected or scattered by objects in its path and returned to a receiving antenna where the pulse is recorded. The returned signal is easy to recognize as it is of the same characteristics of the emitted signal \(^{38}\). By knowing the electrical and electromagnetic characteristics of the media where the pulse traveled and knowing the travel time, information about the defects present in the concrete from which the pulse was scattered can be extorted \(^{38,47}\).

The basic theory of GPR was developed by James Maxwell in the 1860's, the idea went to a halt until the early 1920's when the British physicist Edward Appleton estimated the height of a layer in the upper atmosphere using long radio waves. In 1935 Robert Watt from Britain developed the first practical radar system. The biggest GPR related development occurred in 1976 when a variety of systems were developed and GPR was commercially available. Since that day the evolvement of GPR has been
unprecedented and various technologies have surfaced in the area of GPR testing, processing and interpretation of results.

The GPR equipment comprises of three main parts. The first part is an interchangeable antenna that works as a transmitter/receiver. The antenna is the part of the GPR system that works on converting the electrical signal to electromagnetic signal while transmitting and from electromagnetic signal to an electrical signal while receiving. The selection of the antenna depends on the frequency required which is related to the depth and resolution of the required subsurface information. When applying GPR techniques some optimization must be made by selecting the most appropriate antenna frequency, polarization, sampling rate and some other important factors. The second part is the data acquisition system where the information is recorded in image format and later transferred to a computer as raw data is normally presented in wiggle format where each return signal is displayed adjacent to the previous signal. The third part is the control unit that triggers the antenna’s pulse and transfers the data from the antenna to the data acquisition system.

Two types of GPR systems are available depending on the type of application and antenna used. Ground coupled antennas are used for deeper penetration when the required data is qualitative more than quantitative and where the speed of data acquisition is not so important. Air coupled horn antennas, on the other hand, are used when driving speed measurements are required and high resolution information is collected. In both cases, the raw data collected from GPR surveys is processed for enhanced view using several processing techniques which will be discussed later in the chapter.
6.1.1 Physical Basis of the Method

The foundation of this method is the electromagnetic theory. This method depends on sending a pulse radar signal into the tested material and detecting the arrival time and magnitude of electromagnetic waves reflected from interfaces and defects with different dielectric properties. The signal is transformed from an electrical signal into an electromagnetic signal using the transmitting antenna that probes the material. Signals travel within the media in certain speeds that change when the signal hits a boundary or an object because of the change in the medium’s properties. When the signals hit a boundary or an object the energy is reflected, refracted, or transmitted. The amount of energy reflected is determined by the wave impedance of the two different media. The strength and direction of the reflected signals help determine the position and size of the reflector. In most of the cases, the change in the direction and magnitude of the waves in due to changes in the electrical properties and this is what dominates the GPR response while electromagnetic properties play a minimal role.

The reflection coefficient determines the fraction of the incident wave that is reflected to the first medium. Transmission coefficient on the other hand, determines the fraction of the wave that passes the interface and travels through into the second medium. Equations (6.1) and (6.2) are used to calculate the reflection coefficient and the transmission coefficient respectively.

\[ R_{12} = \frac{Z_2 - Z_1}{Z_2 + Z_1} \]  \hspace{1cm} (6.1)

\[ T_{12} = \frac{2Z_2}{Z_2 + Z_1} \]  \hspace{1cm} (6.2)

Where

\[ R_{12} = \text{reflection coefficient between medium 1 and 2.} \]

\[ T_{12} = \text{transmission coefficient between medium 1 and 2.} \]
$Z_1 =$ wave impedance of medium 1 (Ohm).
$Z_2 =$ wave impedance of medium 2 (Ohm)

While wave impedance for metals is zero, equation (6.3) is used to determine the wave impedance value for nonmetallic materials.

$$Z = Z_0 / (\varepsilon_r)^{1/2} \quad (6.3)$$

Where

$Z_0 =$ wave impedance for free space = 376.1 ohm
$\varepsilon_r =$ relative dielectric constant.

Electromagnetic waves are reflected from steel rebars because the wave impedance of rebars in zero. The amplitude of the reflection from the rebars is computed using equation (6.4) considering each rebar as an infinite cylinder.

$$SAF = (R / R + d)^{1/2} \quad (6.4)$$

Where

$SAF =$ “scattering attenuation function”

$R =$ radius of the rebar
$d =$ distance from the rebar to the antenna (rebar cover)

The material’s dielectric constant is one of the important properties in GPR studies. It is defined as the amount of electrostatic energy stored per unit volume for a unit potential gradient. It shows the material ability to be polarized and therefore its ability of storing a charge when an electric field is applied. The dielectric constant can be affected by many factors depending on the temperature, moisture content, salt content, and the frequency of the pulse transmitted into the material under investigation, and the material’s composition (cement matrix, aggregates, air, water, chlorides, etc).
The relative dielectric constant is the ratio of the material’s dielectric constant to the
dielectric constant of free space. Relative dielectric is a dimensionless quantity defined
by equation (6.5)

\[ \varepsilon_r = \varepsilon / \varepsilon_0 \]  \hspace{1cm} (6.5)

Where

\( \varepsilon_r \) = relative dielectric constant
\( \varepsilon \) = dielectric constant of the material
\( \varepsilon_0 \) = dielectric constant of air which is 8.85 * 10^{-12} (Farad/ meter).

The velocity of the traveling electromagnetic pulse within the medium is given
by equation (6.6) \(^{43,58}\)

\[ V = C / \varepsilon_r^{\frac{1}{2}} \] \hspace{1cm} (6.6)

Where

\( V \) = velocity in the medium (m/s)
\( C \) = velocity in free space (3*10^8 m/s)

If the permittivity (dielectric constant) and the travel time of the signal inside the
material are known, the speed of the electromagnetic pulse can be calculated.
Consequently, the depth of reflecting targets can be calculated. The velocity with which
the pulse signal travels through the probed material is a function of the relative
permittivity (\( \varepsilon_r \)). The relationships between velocity and dielectric constant and depth
will be defined later in the chapter.

Conductivity is another important factor in GPR surveys because it controls
attenuation. Conductivity is defined as the inverse of resistivity and is a measure of the
ability to conduct electrical current. Higher conductivity is usually related to high moisture or water content. As conductivity values increase the penetration ability of GPR decreases because of the high attenuation associated with high moisture contents. 

Attenuation is highly related to conductivity.

Attenuation in GPR signals is related to several factors comparable to the fashion by which sound waves dissipate in the impact echo method (chapter five). As GPR signals are transmitted through the material, they encounter different electrical and electromagnetic properties at interfaces and flaws. While GPR signals are traveling through the material they are constantly losing some of their energy. The magnitude of energy lost is difficult to quantify and a lot of research has been done in this area. The process of quantifying the attenuation caused by scattering is out of the scope of this study. An approximate expression for calculating the attenuation in a specific material is given by equation (6.7)

\[ A_m = \left[ \frac{\sigma_m^2}{4 \epsilon_r m} \right]^{1/2} \times 377 \quad \text{for} \quad \sigma_m / \omega \epsilon_r m << 1 \]  

(6.7)

Where

- \( A_m \) = attenuation in medium M (m\(^{-1}\))
- \( \sigma_m \) = conductivity in medium M (ohm/m)
- \( \epsilon_r m \) = relative dielectric constant of medium M
- \( \omega \) = wave frequency (Hz)

Another method to quantify attenuation was presented in Clemena 1991. He calculated the attenuation of the wave’s energy as it propagates through the concrete medium using equation (6.8) below.

\[ A = 12.863 \times 10^{-8} f \epsilon_r^{1/2} (1 + \tan^2 \delta)^{1/2} \]  

(6.8)
Where

\[ A = \text{the wave attenuation is dB/m} \]

\[ f = \text{the wave frequency in Hz} \]

\[ \tan \delta = \text{The loss or dissipation tangent and is related to the conductivity } \sigma \text{ by equation (6.9)} \]

\[ \tan \delta = 1.8 \times 10^{10} \left[ \frac{\sigma}{fC_r} \right] \]  

(6.9)

6.1.2 Dielectric Properties of Concrete

Concrete dielectric properties vary greatly due to variations in the concrete mixes and the proportions of ingredients, chloride content, moisture content, and degree of deterioration.

Soutsos et al (2001) 46 and Cheng et al (1995) 41 studied the effects of different factors on the dielectric properties of concrete being a very complex material using the coaxial transmission line device, the parallel plate capacitor, and the TEM horn antenna depending on the frequency range required. Their findings showed that the dielectric constant of concrete is frequency dependant in the low radio frequency range and this dependency decreases with the increase of frequency. In addition, the dielectric constant increases with the increase of the w/c ratio or air content and that cement and aggregate types have big influence on the value of the dielectric constant.

6.1.3 GPR System Used in Study

The selected system was a one channel antenna system used for bridge deck evaluation. It is a monostatic system employing a single antenna for both transmission and reception of radar waves. The antenna is a high frequency antenna with a center frequency of 1500 MHz appropriate for concrete bridge deck applications. These
antennas provide both sufficient penetration up to 18 inches and the best available resolution compared to the lower frequency antennas appropriate for higher depths with lower resolutions. The system included the SIR 3000 data acquisition system, survey cart with encoder wheel, and the RADAN data processing software for post processing. System characteristics were introduced in Chapter Four of this thesis.

6.1.4 Advantages and Disadvantages of GPR

GPR is one of the evolving methods for bridge deck evaluation and in civil engineering applications in general. GPR proved to be one of the most versatile methods for different applications ranging from mining and forensic applications to geotechnical and road and bridge surveys.

Ground penetrating radar has been used for detection of subsurface anomalies in bridge decks for a couple of decades now and the study in this area is still underway. References pointed to the fact that GPR has had some success in detecting voids and delaminations\textsuperscript{58}. The fact that GPR is not so sensitive to ambient temperature and the big area that the method covers when testing is done makes it favorable to other well established and well known methods\textsuperscript{16,58}

Ground penetrating radar's advantages according to \textsuperscript{5, 11, 12, 16, 25, 26, 27, 39, 47, 53, 59, 60, 61} include the ability to detect structural composition, detection of objects in ground such as pipes and cables, locating and depth calculations of steel reinforcement, localization of damaged areas and deteriorated areas in bridge decks, the ability to determine thicknesses, the ability to determine honeycombing locations, the ability to detect voids and delaminations, determination of moisture content, the ability to detect
metallic objects with very high accuracy, and the ability to assess concrete strength and maturity from the data acquired.

The best characteristic of the GPR method could be its ability to map roads and bridges in the presence of overlays while minimally affecting traffic\textsuperscript{16}. GPR is very useful for assessment of piers, abutments, and as built conditions of existing structures. GPR also demonstrated great ability in determining deterioration and repair quantities in bridge decks with better accuracy compared to other non-destructive methods \textsuperscript{62}. In short, GPR has showed to be a very promising method for different applications.

GPR has some limitation as well. Results interpretation and understanding of the signal can get complicated and is very slow\textsuperscript{16}. This is because there is a limitation in the use of mathematical models that will relate the registered radar data to the state of the structure \textsuperscript{40}. In many cases, the interpretation requires information obtained by destructive methods such as coring or drilling \textsuperscript{63}.

6.1.5 Radar for Detecting Voids and Delaminations.

The ability of GPR to detect voids and delaminations in concrete bridge decks has been a topic of study for several years. The need for nondestructive methods that can give information about voids inside concrete is increasing as these voids can cause structural failures.

Studies have showed that GPR is capable of detecting voids and cavities under concrete sidewalks, runways, and approach slabs. Consequently there is a trend towards the use of GPR in a wider range for the detection of voids within concrete pavements and bridge decks.
Clemena et al \(^{64}\) discussed the use of Radar in the detection of voids in jointed reinforced concrete pavements. Their results indicated that GPR was an effective method in detecting voids deeper than 1/8 inch but was not as effective in determining the presence of shallower voids \(^{64}\). Another study by Clemena et al stated that different GPR reflections were registered in the presence of delaminated concrete \(^{27}\). According to Buyukozturk and Rhim \(^{65}\) the detection of anomalies such as voids and delaminations inside the concrete decks depends on a variety of factors; the antenna center frequency, frequency bandwidth, size of flaw, polarization, measurement distance and angle, and geometric and material properties of the target \(^{65}\).

As the electromagnetic wave propagates through the concrete medium, a portion of the incident wave is reflected to the transducer at the air/concrete boundary. This is the first boundary between two highly contrasted dielectric media. The remaining energy travels inside the concrete until it strikes another boundary of different dielectric constant. In this case, the concrete steel boundary where another portion of the energy is then reflected to the transducer as it is moving. The rest of the energy travels until it hits another boundary and so forth until the incident wave reaches the bottom of the concrete slab where some of the remaining energy is reflected to the transducer. In the case of voids or delaminations presence, the same concept applies except that the electromagnetic energy is also reflected at the boundaries between concrete/air and then air/concrete created by the presence of the flaw. If the delamination or void is deeper inside the slab the amount of energy that reaches the flaw is lessened because of attenuation and scattering, thus, the use of post processing techniques is rather
beneficial to amplify the returned signal as to be able to detect such low amplitude responses.

Maser mentioned that the ability of GPR to detect delaminations is very limited although there are claims that GPR can actually detect voids and delaminations. He elaborates on the fact that the GPR wave length is much too large to resolve a small delamination crack. Chen et al study showed that it is only hard to detect any changes in the GPR response from concrete specimens when the delaminations are simulated by Plexiglas pieces but are obvious in the GPR response if filled with saline water. Another study by Halabe et al states that the detection of delamination cracks has been achieved in field concrete decks, the study showed that it is possible to detect 0.12" crack especially when filled with water. Maser et al state that there is an indication of the presence of delaminations as the returned signal is different of that in the case of sound concrete. The same study claims that radar's response to a thin (3mm) air gap simulating a small delamination was slightly different compared to that of sound concrete, the presence of moisture added to the attenuation of the signal and made the difference in the signal more noticeable. The study suggests that the direct detection of delaminations is not possible. On the other hand, the detection of moisture and chloride associated with deterioration is possible. Experiments have shown that radar imaging can be effectively used to detect thin delaminations embedded in concrete. Halabe and Bandarkar concluded that voids and delaminations can be detected using GPR but detection becomes difficult in the presence of asphalt overlays especially as the asphalt layer increases in thickness or the defects are small or located deeper inside the deck. From all the above we see that the detection of voids and delaminations has had
its successes and failures were some were able to detect it while others found that GPR is incapable of detecting small subsurface anomalies.

6.1.6 GPR and the Response from Steel Reinforcement

Steel reinforcement bars are the most encountered targets in concrete bridge decks. Rebars oriented perpendicular to survey lines produce very strong responses as the antenna passes over them. This is because the wave impedance for metal targets is equal to zero and most of the energy that falls on it is reflected. The amplitude of the reflection increases as the rebar's size increases and decreases as the rebar's depth decreases. Sometimes the rebar's reflection is not so important in the survey. In this case, surveys should be taken in the transverse direction parallel to the upper steel. This way the reflections will be subtle and other targets will be visible.68

6.2 Processing of the Ground Penetrating Radar Data

Although unprocessed GPR data can produce images of the subsurface of the area being scanned, sometimes this data is hard to understand, read, and interpret reliably.69 Usually raw data contains noise. Thus, several processing techniques have been developed and applied to GPR data in order to modify and enhance the image to assist in the interpretation and understanding of the data in a more efficient manner. Processing of GPR data involves several steps, some of which are discussed below. Although processing techniques can be very helpful, care should always be taken while applying these techniques because despite all the benefits that we get from them they could mask reflections from real features of interest.70
6.2.1 Filtering and Removal of Low Frequency Components

This step is usually referred at as de-wowing of the data. Wow is the name given to the short range GPR signals which possesses low frequency components which cause distortions along individual traces. Low frequency components are associated with the inductive phenomena or are related to the dynamic range limitations of the instrument.

There are several types of filters used in GPR and those include the IIR, FIR, and the F-K filters which correspond to infinite impulse response filters, finite impulse filters, and 2-D spatial filters respectively.

The IIR filters are considered to be the oldest of the filters and where present even before the use of computers. IIR filters include the horizontal and vertical low and high pass filters which will work on removing or significantly reducing noise and antenna ringing caused by the range being set close the maximum limits of the antenna.

The FIR filters are different than the IIR filters in that they are symmetrical and have linear phase characteristics which will result in an output that is not time or position shifted. This kind of filter also has horizontal and vertical filters that will reduce or eliminate noise and ringing and remove surface reflection and background reflections which will look like horizontal bands in the data.

Surface reflection is the high amplitude produced immediately below the GPR antenna at the interface between the air and concrete or asphalt in case of overlays and is caused by the contrast in the dielectric properties of the two media. This usually produces direct and air waves that might mask or reduce the important reflections corresponding to objects within the tested material. The removal of these direct waves is
done by computing the arrival times and wavelengths of these waves then subtracting the theoretical wavelength from the actual wave in each trace of the GPR data. An illustration of the direct and air waves is presented in figure 6.1

Figure 6.1: The Effect of Direct and Air Waves

The F-K or spatial 2D filter is a filter that works on attenuating the noise and is achieved by inverting the Fast Fourier transformation functions (FFT) which produce data with less noise. This filter is better than the successive use of horizontal and vertical filters in that it distinguishes between noise and frequencies. It is also used to isolate sloping features with known frequencies. F-K filters are also used to enhance target reflections in the GPR data and can be applied before or after applying gain which we will discussed in the following section. F-k filters are a good tool to remove the point-source reflections hyperbolas that may work on reducing the responses from deeper features.
6.2.2 Time Zero Adjustments

It is necessary sometimes to adjust the vertical scale of the image obtained from GPR surveys to obtain a more accurate depth calculation. This is done so the image starts at the beginning of the surface.

6.2.3 Amplitude Adjustments of the Data

As the GPR transmitted pulse travels inside the tested material it loses some of its energy and attenuation of the GPR traces occur. This attenuation can be compensated for by applying gain. Gain is usually used to accentuate small amplitudes or to improve visibility of low amplitude features. Low amplitude regions of the data make them difficult to interpret so the need for gain adjustments comes into play. The use of filters while removing the noise and background reflections also result in reducing the signal amplitudes for some important features. Using gain allows compensating for the amplitude loss. Gain can be manually or automatically applied but in all circumstances it should be applied with care.

6.2.4 Deconvolution of the Data

Deconvolution is used to remove the multiple ringing and resolve closely spaced layers and make them more visible. Ringing happens when the signal keeps on bouncing back and forth between the antenna and an object, causing the masking or obstruction of responses from deeper features or objects. Deconvolution is not an easy or straight forward step and doesn’t provide a lot of benefit to the GPR data if not performed right.
Deconvolution is considered difficult to apply because it may alter wave characteristics if it is applied before gain. In addition, deconvolution artifacts may mask weaker reflections in time gain if not applied before deconvolution. In addition, after deconvolution is applied it is hard to guess how the data looked like originally since deconvolution may alter the data into a state that is not real. Deconvolution has proved so far to be an ineffective processing technique.

6.2.5 Migration

Radar radiates the energy in different beam widths depending on the dielectric of the material for the detection of deep objects. Reflections from objects appear as hyperbolic reflections caused by the direction of movement of the antenna along the scan line as shown in figure 6.2. The reflections from shallow objects may cause the obscuration of deeper objects. In the same manner, reflections of steep dipping surfaces also mask other reflections. Migration moves the dipping reflectors to their real position and collapses the hyperbolic reflections. Migration is considered to be very useful and reconstructs the radar image in a form which is a better presentation of the subsurface heterogeneities. The problem with migration is that it requires understanding of the velocity within the tested material, but migration is an interactive process where one can adjust the velocity to optimize the migrated results. Figure 6.3 shows a typical raw GPR data collected from a bridge deck showing the hyperbolic reflections.
Figure 6.2: Antenna Movement along a Scan Line Producing Hyperbolic Reflections

Figure 6.3: Hyperbolic Reflections from Top Steel Layer of a Bridge Deck

Figure 6.4 shows the difference between raw and migrated data at different velocities and how that affects the collapse of the hyperbolic reflections from a typical bridge deck.
Figure 6.4: Effect of Velocity on Migration
(A) Raw Data, (B) Migrated Data at a Velocity of 5 in/ns, (C) Migrated Data at a Velocity of 4 in/ns, (D) Migrated Data at a Velocity of 6 in/ns

6.2.6 Event Picking

This useful technique allows picking of desired events from the data sets that are thought to be important by the processor. Thus, this is a subjective technique and depends on what the processor sees as important.

6.2.7 Static Correction of the GPR Data

Static corrections compensate for the elevation variation in the data sets and the leveling effects of the GPR. It is also used for the removal of the high frequency noise in the horizontal direction. Static corrections correct horizontally without affecting the vertical scale.
6.2.8 Velocity Analysis

Velocity analysis involves determining the velocity of the GPR waves within the tested materials. The values of velocity are determined by figuring the relative dielectric constant using equation 6.6. The depth of the reflections is then calculated using equation (6.10) below, for that reason the knowledge of the velocity of each medium is an important step to determine the depths of features of interest.

\[ D_r = \frac{V T_r}{2} \]  

Where

- \( V \) = the velocity in the medium
- \( T_r \) = Two way travel time to the reflector
- \( D_r \) = depth to reflector

One of the methods for velocity calculations is the one performed on site where the travel time is directly measured to objects of known depth. It should be noted in this case that the velocities acquired are only correlated with the condition of the material at the time of the test because the difference in material properties result in differences in the velocity.

6.3 Results and Discussion

The GPR test was performed over the specimens prepared for this study. Scans were conducted in both directions in order to determine the direction of the upper steel bars. It is very important to know which direction the upper steel is running in order to determine which direction the scan lines will follow. GPR signals depend on the polarization which has a great effect on the response obtained as mentioned in section 6.8. The test is usually done perpendicular to the target unless the response of that target
is not of significance and is to be minimized. For example, if we are looking to see the steel reinforcement the test is performed perpendicular to the rebars direction. If we need to see targets beneath the rebars, tests parallel to the upper rebar should be done. The ability to see beneath the rebars is still dependant on the rebar spacing among other factors. For non metal object detection it is assumed best to run the GPR test parallel to the direction of the flaw (i.e. PVC pipe).

Figures 6.5 shows images of the GPR data collected once perpendicular and once parallel to the upper steel layer. The two parts of the image are taken from a test done on the six inch defected specimen S6. The images help in the determination of the upper steel direction.

The ability of GPR to detect different types of defects is validated in this section. Tests were done on the locations of the different simulated flaws. The objective was to check the precision with which GPR can detect the flaws locations, and depth.

Figure 6.5: GPR Data Collected Perpendicular (left) and Parallel (right) to Upper Steel
6.3.1 Four Inch Specimen

Scans were performed over the four inch defected and sound specimens S4 and S4C. A scan was taken over S4C in order to get an observation from the sound concrete specimen S4C, the response from the sound concrete specimen was used for comparison with the scans taken over the defected four inch specimen S4. Figure 6.6 shows a scan taken over S4C in the direction perpendicular to upper steel.

![GPR Scan Taken Perpendicular to Upper Steel in S4C](image)

Figure 6.6: GPR Scan Taken Perpendicular to Upper Steel in S4C

VOIDS

The first scan taken over the defected four inch specimen S4 was taken perpendicular to steel and passed over V1, V2, C2, and C4 (refer to figure 4.3) the response obtained from this pass is shown in figure 6.7 below. No responses were detected from voids V1 or V2 or any of the vertical cracks C2 and C4.
A second pass was done adjacent to the first pass scanning over Voids V1 and V2 and crack C22. Results from this scan are shown in figure 6.8. Again no response was obtained from voids V1, V2 or from crack C22.

The image shows that GPR couldn’t detect the PVC pipes when the scan was taken in the perpendicular direction to steel. The processed image shows steel at a depth of 1.3 inches (33 mm). Since the depth of the steel is known, it will be used to calibrate the depth calculations for other targets using the velocity analysis equations discussed in section 6.9.8.

The scan done over V1, V2, and C2 was not able to detect any of the defects. The inability to detect the PVC voids was due to the fact that they were very shallow. V1 is 1 inch (25.4 mm) deep while V2 is 0.5 inches (13 mm) deep and is 0.25 inch (7 mm) in diameter.

Figure 6.9 shows the response obtained from a scan done perpendicular to the steel direction and the PVC pipe. The scan was done over V2 and passed right by Delamination D5. The PVC pipe void could not be detected in this pass either. This shows that the shallow depth (0.5 inch) makes it hard for GPR to detect defects, especially when the defect is thin and the steel cover is very small.

A similar test was done over V1 in the perpendicular direction. The same response was obtained and the image looked identical to that in figure 6.9.

A scan passing over V3, C1, and C2 taken in the direction parallel to steel and PVC pipe is shown in figure 6.10. The figure shows that only the 1.0 Inch diameter pipe was detected by GPR with 78% accuracy. The calculated depth from the scan was 1 inch (25.4 mm). The real depth was 1.25 inches (32 mm). This is because PVC pipes
are better seen when tested along their long direction. The two Cracks could not be seen by GPR rays.

Figure 6.7: Scan Taken Over V1 and V2, C2, and C4 in S4

Figure 6.8: Scan Taken Over V1, V2, and C22 in S4
Figure 6.9: Scan Taken over V2 in the Parallel to Steel Direction

Figure 6.10: GPR Response over V3 in S4
DELAMINATIONS

The GPR Antenna was run over the delamination in S4. A pass was made over delamination D1 in the direction parallel to steel in order to get a more subtle response from the steel. Figure 6.11 shows the response obtained from delamination D1. The response shows a disruption in the direct coupling. Usually direct coupling masks the responses below up to 1.5 inches\textsuperscript{68}. The presence of the disruption shows that there is a flaw but no information could be extorted about the flaw. The dimensions of delamination D1 is 1.5*1.5 inch (38 * 38 mm) and is 1 inch (25.4 mm) deep. This means that it lies inside the blind zone of the direct coupling.

A scan over delamination D2 was made. D2 dimensions are 2*1.5 inches (50*28 mm) and at a depth of 0.75 inch (19 mm). The depth is very shallow and the response is masked by the direct coupling and airwaves. Figure 6.12 shows the image obtained from that scan.

Same test was done above delamination D3. This scan passed over D1 as well and was done in the direction parallel to the upper steel direction. The image in figure 6.13 shows responses from D1 and D3 in the form of disruption in the black portion of the direct coupling because the flaws were very shallow.

Similarly, tests were done over delamination D4. The dimensions of D4 are 1.5 *1.5 inches (38 mm) and it has a depth of 1.25 inches (32 mm). Figure 6.14 shows the response obtained from the scan done over D4. A very subtle response was seen.
The GPR antenna was run over delamination D6. GPR was able to detect D6 as shown in figure 6.15 below. The response from delamination D6 which is 3*3 inches (76 *76 mm) with a depth of 1.5 inches (38 mm) shows a non uniform reflection at a depth of 1.5 inch which is the real depth of delamination D6. Delamination D6 was calculated with 100% accuracy.

Figure 6.16 shows the response obtained from delamination D7. The response obtained is non uniform at the location of delamination D7. The calculated depth was 1.4 inches (36 mm) when the real depth was 1.5 inches (38 mm). Delamination D7 was calculated with 95% accuracy.

None of the vertical surface Cracks were detected using the GPR scans. All the Cracks were not visible to GPR scans. GPR is not a method for detection of surface cracks or thin vertical discontinuities.
Figure 6.12: GPR Image over Delamination D2 in S4 Parallel to Upper Steel

Figure 6.13: GPR Image over D1 and D3 in slab S4 Parallel to Upper Steel
Only delaminations D6, D7 and Void V3 were detected in the four inch defected specimen using GPR. This is because those defects had large lateral dimensions when compared to the other defects that could not be detected. The fact that the steel layer was very close to the surface made its reflection very high and masked reflections from small targets. The lateral dimension of targets plays a role in the amplitude strength obtained.

6.3.2 Six Inch Specimen

Similar to the four inch specimen, the GPR machine was run over locations of flaws in the six inch specimen. The following paragraphs discuss the responses obtained from each of the scans.
Figure 6.15: GPR Response over D6 in S4

Figure 6.16: GPR Response over D7 in S4

Figure 6.17 shows a scan taken over the sound six inch slab S6C in the directions parallel and perpendicular to upper steel. The steel cover was 2.7 inches (69 mm). Since the steel depth is known, it will be used for depth calibration. The velocity of the wave inside the medium will be calculated using equation (6.6). When depth
calculations of other features are needed, equation (6.10) is used since the medium velocity is known.

Figure 6.17: GPR Response from S6C Parallel (Left) and Perpendicular (Right) to Upper Steel in S6C

The data in figure 6.17 can be used to calculate the rebars diameter. If bars intersect and are in touch, we can usually determine the rebar diameter. Figure 6.17 shows the two lines at spacing equal to the rebar diameter. In this case the rebar spacing reading from GPR was 0.68 inch (17.2 mm). The real rebar diameter is 0.63 inch (16 mm).

VOIDS

The first scan was done over V3 in the direction parallel to the upper steel in order to reduce the effect of steel reflections. The results of this scan are shown in figure 6.18. Results show that GPR was able to detect V3 which had a 1.0 inch (25.4 mm) diameter and was 0.5 inch (13 mm) deep. The calculated depth from the GPR data
was 0.9 inch (23 mm). The flaw was very shallow and resulted in a very low accuracy in the depth calculation. Void V3 was calculated at a depth 76% lower than its real depth.

The depth calculation was done using equation (6.10). The first step was to perform the time-zero adjustment for the data. Step two involved calibration of the depth in the data using targets of known depth, steel rebars were used as the target in this case. Steel had a concrete cover of 2.7 inches (69 mm). Using equation (6.10) the velocity inside concrete was calculated to be 86.8 mm/ns (3.42 inch/ns). For targets with unknown depth, equation (6.10) is used again to calculate the depth since the velocity of the medium is known. Using this approach the depth of the PVC pipe of V3 was calculated above.

The second scan was taken to cover voids V1 and V2. It was also taken in the direction parallel to the upper steel to minimize the reflections from steel. The scan was perpendicular to the direction of both V1 and V2. The response obtained from this scan is shown in figure 6.19.

GPR was not able to detect V2 when tested in the six inch specimen. Void V2 is 0.25 inch (7 mm) in diameter and 4.5 inches (114.3 mm) deep located under the steel layer. The reflection from steel masked the appearance of V2. In addition, the response from V2 would be very subtle because it is very thin. V1 was visible to GPR waves as shown in figure 6.19. The calculated depth for void V1 was 1.54 inches (39 mm) while the real depth was 1.50 inches (38.1 mm). There was only a 2% difference in depth calculation.
Figure 6.18: GPR Response Taken above V3 in S6

Figure 6.19: GPR Response above V1 and V2 in S6
DELAMINATIONS

A scan was taken on top of delamination D5 in the direction parallel to steel. Figure 6.20 shows the response from this scan. The scan also passed over V3 discussed earlier.

![Figure 6.20: GPR Data Taken above D5 and V3 in S6](image)

The response obtained shows a strong response of both D5 and V3. The calculated depth of delamination D5 was 1.4 inch (35 mm) while the real depth was 1.5 inch (38 mm). There was a 7% difference between the real and calculated depth on this case.

Another scan was taken adjacent to the aforementioned scan. This scan passed over delaminations D6 and D1. Figure 6.21 shows the raw data file after gain addition.
The results in the raw data file show a strong response coming from delamination D6 which is 3*3 inch (76 * 76 mm) at a depth of 1.5 inch (38 mm). When this is compared to that of delamination D1 2*3 inches (50*76 mm) at a depth of 1.0 inch (25.4 mm), the difference in amplitude brightness is obvious. This shows that if the depth is about the same then the bigger the flaw is, the higher the amplitude reflection. After processing (Image not shown here), the calculated depths for D6 was 1.4 inch (36 mm) and 0.96 inch (24.5 mm) for D1. The real depths were 1.5 inch (38 mm) and 1 inch (25.4 mm) respectively. D6 was calculated with 95% accuracy while 96% accuracy was obtained for D1.

Another scan was taken in the perpendicular direction to validate the responses when the antenna is polarized perpendicular to the upper steel direction. Figures 6.22a and 6.22b show two scans passing over D6 and D1 in the direction perpendicular to steel. The responses obtained show that GPR could detect these flaws in both directions.
and in both cases the reflections obtained were strong and visible. Figure 6.22 (a) shows D1 and D3 while figure 6.22 (b) shows D5 along with D6. These although laterally close, appear as two separate targets.

![GPR Scan Images](image)

Figure 6.22: GPR Scan Perpendicular to Upper steel Direction over D3 and D1 in S6, and a Scan Perpendicular to Upper Steel over D5 and D6 in S6

A scan was taken to pass over delaminations D4 and D2 in the direction parallel to steel. Figure 6.23 shows the response obtained from this scan. No responses were observed as GPR antenna passed over these two flaws. D4 is 1.5*1.5 (38*38 mm) inches and 2 inches (50 mm) deep and D2 is 2*1.5 (50*38 mm) and 3 inches (76 mm) deep. To validate this finding, another scan perpendicular to steel was taken over delamination D2 shown in figure 6.24. This verifies the first finding that D2 and D4 are invisible to GPR. This might be because there depth is high compared to their lateral dimensions. The lines in the images are locations of markers taken during the test to specify locations of flaws.
As was the case for the defected four inch specimen, the vertical and surface cracks introduced to the specimen using Plexiglas were not visible to GPR waves. None of the vertical or surface cracks were detected by the use of GPR.

Summary

GPR was able to detect all horizontal flaws except D2, D4 and Void V2. Void V2 was very small for GPR waves to detect since its diameter was 0.25 inch (7 mm). D2 and D4 were not detected because their lateral dimensions were small. D2 and D4 had small dimensions and were relatively deep, this lead to very small response and low strength amplitude that could not be detected.
6.3.3 Eight Inch Specimen

GPR test were also performed on the defected and sound eight inch specimen (S8) and (S8C). S8 differs from S4 and S6; most of the flaws are relatively deep. Figure 6.25a and 6.25b show responses obtained from test taken over S8C. Similar to what was done for the defected four and six inch specimens. Scans were taken over flaws in defected slab S8. The results obtained from these tests will be introduced in the following paragraphs.

The GPR antenna was run over the sound eight inch specimen S8 as done in the previous two samples. The response obtained from the scan taken in the direction parallel to the upper steel bars figure 6.25 below.
VOIDS

The image obtained in figure 6.26 shows that GPR was only able to detect void V1 which is 0.5 inch (13 mm) diameter and 3 inches deep in the concrete. Void V2 was not detected as was the case in the defected four and six inches specimens. The calculated depth of V1 was 2.95 inches (75 mm) while the real depth of V1 was 3 inch (76.2 mm). V1 was calculated with 98 % accuracy.

A scan was made to pass over void V3 alone. Void V3 was air filled. The pass was made parallel to upper steel and perpendicular to the void’s orientation. This void was 1.0 inch (25.4 mm) in diameter and 2.5 inches (63.5 mm) deep. The response obtained from this pass is shown in figure 6.27.

The image in figure 6.27 shows a strong response obtained from void V3. The response shows a phase shift as the dielectric of air is smaller than of concrete. When
this response is compared to that of V1 it shows that the hyperbola had wider tails. This is because this void is bigger in diameter. The amplitude response in this case was higher than that of V1 because V3 is shallower and larger in size. The calculated depth of V3 in this case was 2.6 inches (66 mm) while the real depth was 2.5 inches (63 mm). The accuracy of depth calculation was 95% in this case.

![GPR Scan Taken over Voids V1 and V2 in S8 Parallel to Upper Steel](image)

Figure 6.26: GPR Scan Taken over Voids V1 and V2 in S8 Parallel to Upper Steel

**DELAMINATIONS**

Figure 6.28 shows a scan taken parallel to upper steel over delaminations D1, D5 and partially over void V3 shown and discussed earlier in figure 6.24
Reflections from D1 and D5 appear visible in the data after processing. D1 is 2*3 (50*76 mm) inches and 2.75 inches (70 mm) deep while D5 is 4*4 (101*101 mm) inches and 2.25 inches (57 mm) deep. The calculated depths for D1 and D5 were 2.68
inches (68 mm) with 97% accuracy in depth calculation and 2.3 inches (58 mm) with 98% depth calculation accuracy. The response obtained from D5 shows a phase shift in the wave. This might be caused by the waves reflecting from an air interface formed while casting on top of the Styrofoam blocks.

Figure 6.29 shows the response obtained from a test taken over delaminations D2 and D4 in the direction parallel to upper steel in order to minimize the reflections from steel that might mask the weak responses. The obtained image shows that GPR was able to detect D2 and D4 in S8 while it could not detect it in S6. The response was very subtle, this is because there lateral dimensions are small when compared to there depth.

Figure 6.29: GPR Scan Taken over D2, D4 in S8 Parallel to Upper Steel

The image obtained shows a very weak response from both D2 and D4. This is, as mentioned, mainly due to their small lateral dimensions and high depth in the concrete when compared to other responses from bigger or shallower defects. D2 is
(2*1.5) inches and is 4 inches (102 mm) deep. Delamination D4 is 1.5*1.5 inches (38*38 mm) and is 3.5 inches (89 mm) deep. Calculated depths for both delaminations were 3.8 inches (97 mm) and 3.3 inches (83 mm), respectively. Both delaminations had 95% depth calculation accuracy.

Figure 6.30 shows a scan taken over delamination D3 in the direction parallel to upper steel. The results obtained from this scan shows delamination D3 visible but with no uniform shape. The hyperbola is not present because voids and delaminations give non uniform responses. Delamination D3 has big lateral dimensions. D3 dimensions are 4*4 inches (102 *102 mm) and 4 inches (102 mm) deep. The calculated depth of D3 using equation 6.10 since the velocity is knows was 3.9 inch (99 mm). Depth of delamination D3 was calculated with 97% accuracy.

Figure 6.30: GPR Scan Taken over D3 in S8 Parallel to Upper Steel

Figure 6.31 shows a response obtained from a scan taken in the direction perpendicular to steel. This scan shows two delaminations closely spaced.
Delaminations D5 and D6 appear visible in the GPR scan as two separate objects. The lateral distance between D5 and D6 is 5 inches. If the lateral distance was small enough to be smaller than the wavelength of the 1.5 GHz antenna those two flaws would have appeared as one. Delamination D5 was discussed in figure 6.25. Delamination D6 has a 3*3 inch dimensions (76 *76 mm) and is 3 inch (76mm) deep. The calculated depth of delamination D6 using the GPR data was 2.85 inch (73 mm). Delamination D6 was calculated with 96% accuracy.

Figure 6.31: GPR Scan Taken over D5 and D6 in S8 Perpendicular to Upper Steel

The results obtained from all scans taken over the surface cracks and the vertical cracks were in accord with what was found when testing the defected 4 inch and defected 6 inch specimens. GPR was unable to detect thin vertical flaws. GPR was unable to detect any of the vertical cracks in this specimen as well.

Summary
The GPR method was able to detect all the flaws in the defected eight inch specimen S8 except void V2. Void V2 is very thin in diameter and GPR can not detect anything less than 0.25 inches. All the other defects were visible and depth calculations were accurate. This proves that GPR is a very practical method for detection of voids and delaminations in concrete structures. As for Cracks, GPR was unable to detect any of the vertical or surface cracks.

6.4 Tests after Dissolving Styrofoam Blocks

Another set of testing was done on the delaminations of the six inches defected specimen (S6) after the Styrofoam blocks were dissolved by injection. Acetone was injected into the Styrofoam blocks through the attached plastic pipes. The responses obtained in this case looked identical to that when the Styrofoam was present. GPR was able to detect the same flaws it detected when Styrofoam was present. Delaminations D2 and D4 and void V2 were invisible to GPR again due to reasons explained earlier. Delaminations D1, D3, D5, D6 and voids V1, V3 provided the same response.

The response obtained from delaminations D1 and D6 is shown in figure 6.32. This response is identical to that in figure 6.24 and the calculated depths are exactly the same.
Figure 6.32: GPR Scan Taken Over D1 and D6 in S6 with no Styrofoam

Figure 6.33 shows the response obtained from a test done over delamination D3 after the Styrofoam was dissolved. The response in this test looked identical to that shown in image 6.22 for delamination D3.

Figure 6.33: GPR Test over D3 with no Styrofoam in S6
A scan performed over D5 is shown in figure 6.34. The response is identical to that of figure 6.28 and the calculated depth of Delamination D5 was also the same.

Summary

The results Obtained from tests performed after the Styrofoam blocks were dissolved gave identical responses to when the test was done with the Styrofoam present. The same delaminations where detected in both cases and depth calculations were the same in both cases.

Figure 6.34: GPR Test over D5 with no Styrofoam in S6

6.5 Discussion of the Results

The results obtained from the tests done on the defected concrete specimens illustrate the ability of GPR to detect voids and delaminations with high accuracy. Table 6.1 summarizes the findings of the performed tests.
<table>
<thead>
<tr>
<th>SLAB</th>
<th>DELAMINATIONS</th>
<th>VOIDS</th>
<th>CRACKS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>D1</td>
<td>D2</td>
<td>D3</td>
</tr>
<tr>
<td>FOUR INCHES DEFECTED</td>
<td>Y</td>
<td>N</td>
<td>Y</td>
</tr>
<tr>
<td>Depth calculation</td>
<td>NC</td>
<td>NC</td>
<td>NC</td>
</tr>
<tr>
<td>Accuracy</td>
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<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>SIX INCHES DEFECTED</td>
<td>Y</td>
<td>ND</td>
<td>Y</td>
</tr>
<tr>
<td>Depth calculation</td>
<td>96%</td>
<td>N/A</td>
<td>95%</td>
</tr>
<tr>
<td>Accuracy</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>EIGHT INCHES DEFECTED</td>
<td>Y</td>
<td>Y</td>
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<tr>
<td>Accuracy</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>

Legend:
Y = detected
ND = not detected
NC = not calculated
N/A = not applicable
CHAPTER SEVEN

INFRARED THERMOGRAPHY (IR)

Chapter seven discusses the Infrared Thermography method (IR). The chapter starts with a background survey about the physical basis of the method. The advantages and disadvantages of the method are pointed in this chapter. The chapter also explains the conditions under which IR works and explains the factors affecting the testing. Results of IR tests performed to assess its capabilities on the fabricated slabs will be presented in this chapter.

7.1 Background

Thermography is the third of the nondestructive techniques that will be studied in this thesis. The interest in using Thermography for bridge decks analysis started in the seventies and research has been ongoing since 44. Infrared Thermography is common in both military and civilian applications. It is considered to be a promising method in detecting delaminations and other potential bridge decks discontinuities and has been standardized by ASTM D 4788 28,44.

Infrared thermography (IR) is a better method than the traditional visual inspection and acoustic methods such as Chain Drag and Hammer tap. This superiority comes from the fact that it is less affected by external factors and less subjective compared to sounding and visual methods. Results are obtained faster and with higher accuracy28.

Infrared Thermography can be done anytime of the day as long as there is heat transfer between the tested element and the atmosphere28, 73. The concept behind
Infrared Thermography test is that areas with discontinuities heat at a higher rate than sound concrete during daytime and cool faster during night time. This is utilized in detecting concrete spalls, cracks, voids, and delaminations. Infrared tests can be done passively or actively. Passive infrared uses solar energy to heat the bridge deck while active infrared uses artificial heating to heat the element under study.

### 7.1.1 Physical Basis of Infrared Thermography

Infrared thermography depends on the concept of detecting the thermal differences between sound and defected concrete. This is done by registering the temperature readings on the surface of the concrete bridge deck or pavement. As mentioned earlier, the bridge deck heats during the day, the areas which contain flaws heat at a faster rate than sound concrete, this results in higher temperature readings from the concrete right above the flaws. The reason behind that is the disruption in the heat transfer paths within the concrete deck. At night, those areas cool at a faster rate than the rest of the bridge causing a lower temperature reading.

Readings from Infrared Thermography obtained during the day are considered more reliable.

The Infrared test can be done anytime of the day as long as heat transfer is taking place between the concrete and the surroundings. Heat transfer occurs as long as the atmosphere is cooler or hotter than the concrete bridge deck and it occurs in one of three ways; conduction, convection, or radiation.

Thermal energy travels from the surface to the interior and vice versa in the concrete deck by means of convection and conduction. Convection is defined as the transfer of heat between the surface and the surroundings. Conduction, on the other hand is the flow of heat within the material from a hot to a cool region. Radiation is
defined as the movement of heat by the effect of electromagnetic waves \(^{16, 44}\). Infrared thermography detects the radiation that occurs between the concrete and the surroundings.

The difference in the temperature reading between sound and defected concrete can reach up to \(5^\circ C (8^\circ F)\). The larger the size of the discontinuity the higher the temperature difference will be. This is because of the break in the heat transfer process that the deck will suffer.

Several factors affect Infrared Thermography testing. Environmental factors play the biggest role in IR tests. Cloud cover, wind speed, and moisture content are factors that might affect IR Thermography tests. It is suggested that wind speed does not to exceed 15 miles per hour when the test is done. This is due to the fact that wind accelerates attenuation of radiated or reflected heat from the deck \(^{24}\). Moisture at the surface or water seeping inside the concrete bridge into the discontinuity also affect the reliability of IR Thermography readings \(^{16, 42, 44, 73}\). Studies have shown that infrared Thermography is not dependant on temperature. It is assumed that IR testing only works when there is 70 % or more sunshine\(^{22}\).

Literature suggests that there is no fixed time to perform the test. Studies suggest the best results are obtained midday when the deck is absorbing as much sunshine as possible \(^{75}\). Manning et al (1980) \(^{24}\) suggests that the longer the elements are exposed to the sun results in a higher temperature difference between sound and defected portions. The study suggests that a temperature difference will be available anytime of the day or night except around sunrise and sunset. This is due to the fact that the delaminated areas have higher temperature during the day, but as the deck starts to cool around sunset,
delaminated areas cool faster and reach the same temperature as the surrounding concrete. At this time, no difference between the delaminated area's temperature and the sound area of the deck will be detected.\textsuperscript{24}

Emissivity is defined as the ability of concrete to radiate energy and it has values from 0 to one, a value of one denotes an object that is totally emissive and a value of zero denotes an object that is totally reflective. It is higher for rougher areas and darker bodies. Thus, the parameter is used to quantify the absorptive or reflective properties of a material. Emissivity is one of the important factors to take into consideration when performing the IR test because wrong emissivity inputs may lead to misinterpretation of the results.\textsuperscript{13}

The behavior of the emitted radiation in IR thermography is described by the Stephan-Boltzman law as shown in equation (7.1)\textsuperscript{42}.

\[
Q_e = \sigma \epsilon \; T^4
\]

Where

\( Q_e \) = total radiant emission of the surface.

\( \sigma \) = Stephan Boltzman constant \( 5.673 \times 10^{12} \) watts/cm\(^2\) k\(^4\)

\( \epsilon \) = emissivity factor which is less than one (one is for a black body)

emissivity for concrete in 0.92 @ 20\(^\circ\) C

\( T \) = absolute temperature of the object (K)

The conductive heat flow is determined by the temperature at the top deck and the bottom surface of the deck and the deck thickness, and is given by equation (7.2)

\[
Q_c = (k/d) \; (T_4-T_3)
\]

Where
Q_e = conductive heat flow through a material. (Watts/m²)

K = thermal conductivity of the material (Watts/m°C)

D = thickness of the layer (m)

T_+ = temperature of the hotter side. (K)

T_− = temperature of the cooler side. (K)

The combined heat transfer is given by equation (7.3)

\[ Q = h^* (T_s - T_a) \]  

(7.3)

Where

Q = combined heat flow (watts/m²)

H= the heat transfer coefficient

T_s= surface temperature

T_a = ambient temperature

The rate of heat flow into and out of the deck must be equal to the sum of the net radiant flow and the convective flow. Any imbalance must be absorbed by the decks surface. This is detected by the IR camera.

7.1.2 The Thermal Camera Used in this Study

The IR camera used in this study was procured from Electrophysics. It is a high resolution IR camera with a 12 bit digitization, has a thermal sensitivity of 0.08 C, and is sensitive to medium radiation (8-12 micron). For more camera specifications refer to chapter four.
7.1.3 Advantages and Limitations of Infrared Thermography

IR Thermography is capable of scanning larger areas in shorter time when compared to other traditional methods. The method is capable of detecting cracks, voids, delaminations, debonding between layers, and horizontal extent of defects when present. Infrared Thermography works well with overlays but the results are less reliable as the thickness of the overlay increases. One of the great advantages of IR Thermography is the minimum disruption to traffic and minimum required lane closure. IR results are simple to interpret, less subjective, and can be obtained in real time with no need for post processing.

IR Thermography has some disadvantages. IR Thermography can only be used during some months of the year because it is dependant on environmental factors. IR Thermography can not be used to detect the depth of defects. Furthermore, it can not be used in preparing repair documents. Finally, the method is not so reliable in detecting water-filled voids as it is in the case of air-filled voids.

7.1.4 Infrared Thermography for Bridge Decks and Pavements

Infrared thermography proved to be a very good method for detecting anomalies in concrete bridge decks and pavements. Several reports have shown success of the method. Zachar 1992 stated that “it was found that the IR Thermography method found essentially all (97 percent) of the delaminated areas on a bridge deck at a cost competitive with less accurate methods”. Uomoto stated that Thermography was able to detect an “A” shaped anomaly inside a concrete specimen when the passive method was used after the specimen was sun heated during daytime. Khan stated that Thermography still needs a lot of improvements to be a reliable method for...
delamination detection in concrete bridge decks. Testing of IR Thermography showed that its ability to detect Styrofoam simulated delaminations is not validated and not enough data is available. On the other hand, it failed to detect delaminations simulated by the use of polyethylene sheets \(^{28}\).

In the case of pavements, IR is widely used to detect problems in airport pavements. Debonded areas will have a different thermal signature than that of a bridge deck delamination. Debonded areas are non uniform in shape and the temperature distribution when tested by IR is non uniform, while it is usually uniform for a delamination \(^{22}\). However, thermograph was able to detect debonding between the asphalt and concrete layers \(^{28}\).

7.2 Experimental Testing and Results

Testing of the concrete blocks was done on different days and at different times of the day in an attempt to cover the variations of temperature and time of day on the obtained results. It has been stated in literature that results from Infrared are most accurate during the hours of heating lying between 11:00 am and 4:00 pm. The results obtained from tests done on the concrete slabs verified this fact. The test was done in two stages, the first involved testing the specimens with the Styrofoam blocks still present while the second was performed after the Styrofoam blocks were dissolved using acetone.

IR testing on the concrete blocks showed that surrounding temperature is not a very big factor in the obtained result as long as there is enough sunshine. All tests were done on sunny days and in temperatures that ranged between 68° F to 78° F. The idea of doing the test with and without the Styrofoam blocks was to validate whether
infrared can detect Styrofoam simulated defects for research purposes. Previous studies indicated that Styrofoam blocks could be detected using infrared while some others suggested that Styrofoam simulated flaws couldn't be detected using the IR Thermography technique.

The tests were done on four different days and at different times. It was concluded that it is really difficult to detect delaminations and voids if they were Styrofoam simulated using IR Thermography. Delaminations and voids were only detected after the specimens were exposed to sun heat for several days when there was no cloud cover. It was also noticed that roughness of the concrete surface plays a role in making interpretation of the results more difficult. The detection of the Plexiglas cracks was not possible. In addition, it was concluded that the depth of the delamination or void is an important factor that affects IR Thermography ability to detect a flaw.

The first step in applying IR Thermography tests was to determine the emissivity value of the concrete specimens. This was done using a calibration procedure. A material of known emissivity was used as a guide to determine the values of emissivity for the specimens. Electric tape with an Emissivity value of 0.95 was attached to the surface of the concrete specimen and left for a period of time until it acquired the temperature of the concrete. The value of the emissivity of Electric tape was entered to the IR camera and a temperature reading for the electric tape was taken. The next step was to measure the temperature of the concrete using the IR camera and changing the emissivity value until the temperatures matched. Using this procedure the value of emissivity for the concrete was found to be 0.93.
Delaminations and voids could only be detected after a long period of heating and only in the four inch and six inch specimens (S4 and S6). None of the delaminations could be detected in the 8 inch specimens (S8) because their depth increased compared to the 4 and 6 inch specimens.

Figure 7.1 through 7.10 show some examples of the infrared images obtained from tests done on the concrete blocks. The images will discuss some of the results. For full results Tables 7.1, 7.2 and 7.3 at the end of the chapter will provide a comprehensive guide. The images shown here will illustrate some of the findings.

Figure 7.1: The Four Inch Non-defected Specimen S4C

Figure 7.1 shows the four inches sound concrete specimen (S4C) taken at noon time after two days of exposure to the sun. The temperature reading at the concretes surface was 92°F and no variations in the temperature were present. The sides of the specimen appear a little hotter because they tend to heat and cool faster than the other parts as they are more exposed to the sun and to ambient temperature.

Figure 7.2 shows a plan view of the 4 inch defected specimen (S4) tested on 9/02/04 at noon after the specimen has been exposed to the sun for three days. The
figure shows a different temperature reading at the location of the delaminations. Delamination D3 had the largest response, all the other delaminations were detected and temperature differences between flaw locations and sound concrete were observed. Responses from voids are not showing in this image. Nonetheless, voids were detected when the test was done in the maximum heating period. Plexiglas cracks were not detected.

![Image of Four Inch Defected Specimen](image)

**Figure 7.2: Four Inch Defected Specimen**

Crosshairs in the image are used to compare temperature readings between different points in the image, in this particular case it was used to determine the difference in temperature reading between sound concrete and a delamination location. The difference was found to be $5^\circ$ F. The blue (light color) region in the image can be interpreted as areas with lower temperatures due to shadow or difference in concrete roughness which causes a different thermal response.
Figure 7.3 and 7.4 show the six and eight inch sound concrete slabs respectively. It is observed that there is hardly any temperature difference on the surface in figure 7.3 for the six inch sound concrete specimen. The temperature reading on the surface of the six inch defected specimen (S6C) was consistent. The reading was about 100°F and did not change along the surface. Figure 7.4 shows that there is no difference in the temperature reading on the surface of the sound eight inch specimen (S8C) indicating the absence of any kind of flaw within the specimen.

Figure 7.3: The Six Inches Sound Concrete Specimen S6C

Figure 7.5 shows the defected 6 inch slab (S6) tested on 9/02/04 at noon. Results of other test are summarized in Table 7.2. The arrows in the image show locations of detected delaminations and voids in the slabs. Temperature differences in this case were found to be less than those in the four inch specimen because the depth of delaminations and voids is larger.
For example, delamination D3 was 3°F higher in temperature than sound concrete in this case. The reflections from locations of voids V1 and V3 show a higher temperature response. No response was detected for void V2 because it is located very deep inside the specimen at a depth of 4.5 inches (115 mm). Plexiglas surface or deep cracks showed no different response.
Figure 7.6 shows the 8 inch defected specimen tested on 9/02/04 at noon. The response obtained from an Infrared Thermography test performed on this specimen does not show any responses from the delaminations or the voids. The reason behind that is that the depth of the defects in the specimen is very high. This makes the concrete above the defect thick enough that there will be very minimal or no thermal difference. The image shows higher temperature at the edges of the specimen while cooler temperatures in the middle with no hotter spots where the delaminations are present. This is due to the fact that the thermal transfer occurs from the outside inwards and that explains why the middle region of the concrete block is a little cooler with a temperature of about 84°F while its about 93°F at the edges.

Figure 7.6: Eight Inch Defected Slab
As mentioned earlier, Plexiglas simulated cracks could not be detected using the Infrared thermography camera. This might be due to the fact that Plexiglas has the same thermal properties as the concrete and heated or cooled as fast as concrete.

Figures 7.7 and 7.8 show location of a Plexiglas crack (inside the rectangle in the image), a higher temperature (red or darker color) is a response in figure 7.7 is obtained because the inspector placed his finger on the crack to show the exact location of the Plexiglas. The color contrast is due to the difference in temperature between the inspector’s finger and the concrete. In figure 7.8, the inspector removed his finger. No response was detected. The location of the Plexiglas showed a response identical to that of the surrounding concrete. Similarly, Plexiglas Cracks plunged inside the concrete slabs at variable depth were not detected either using IR Thermography.

Figure 7.7: Plexiglas Crack Location
Table 7.1 shows the results of ten tests done on different days and times for the four inches specimen. Tables 7.2 and 7.3 show results for the six and eight inches specimens respectively.

The results from Table 7.1 illustrate the ability of Infrared to detect delaminations and voids at periods of maximum heating between 11:00 am and 3:00 pm. Most of the tests performed during the day were able to detect the flaws. Tests done at night, as mentioned, failed to detect the flaws because of dissipation issues.

The results shown in Table 7.2 show that infrared was less capable of detecting the same flaws detected in the four inch specimen. This is due to the fact that the defects had higher depth in this case. The table shows that IR had some success in detecting some of the defects, but the responses were subtle most of the time and not as apparent as in the four inch specimen.

The strength of the responses is related to the depths and sizes of the flaws. For example, no response whatsoever was detected from flaw V2 as it is 0.25 inch in
diameter and 4.5 inches below the surface, while in the 4 inch specimen this defect could be detected when it was only 0.5 inches below the concrete surface.

Table 7.1 Infrared Results for the Four Inches specimen

<table>
<thead>
<tr>
<th>Date</th>
<th>Time</th>
<th>D1</th>
<th>D2</th>
<th>D3</th>
<th>D4</th>
<th>D5</th>
<th>D6</th>
<th>V1</th>
<th>V2</th>
<th>V3</th>
<th>Cracks</th>
</tr>
</thead>
<tbody>
<tr>
<td>8/22/04</td>
<td>12:30 pm</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
<td>Subtle</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>No</td>
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<tr>
<td></td>
<td>3:30 pm</td>
<td>No</td>
<td>No</td>
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<td>No</td>
</tr>
<tr>
<td>8/24/04</td>
<td>1:00 pm</td>
<td>Yes</td>
<td>No</td>
<td>Subtle</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>No</td>
</tr>
<tr>
<td></td>
<td>11:00 am</td>
<td>Yes</td>
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<td>Yes</td>
<td>Subtle</td>
<td>Yes</td>
<td>Yes</td>
<td>No</td>
<td>No</td>
<td>No</td>
<td>No</td>
</tr>
<tr>
<td></td>
<td>2:00 pm</td>
<td>Yes</td>
<td>No</td>
<td>Yes</td>
<td>Subtle</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
<td>Yes</td>
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<tr>
<td>9/01/04</td>
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<td>No</td>
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<td>No</td>
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<tr>
<td></td>
<td>5:00 pm</td>
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<td>Sub.</td>
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<td>Sub.</td>
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<tr>
<td></td>
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<td>No</td>
<td>No</td>
<td>No</td>
<td>Sub.</td>
<td>No</td>
<td>No</td>
<td>No</td>
<td>No</td>
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</tr>
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<td>Depth (Inch)</td>
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<td>.75</td>
<td>1.25</td>
<td>1.25</td>
<td>1.5</td>
<td>1</td>
<td>5</td>
<td>1.25</td>
<td>variable</td>
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</table>

Exposure time is another factor that determines whether the defects could be detected, results obtained for the 6 inch specimen on 9/02/04 show that all delaminations could be detected while those same ones could not be seen on 08/22/04. The reason lies in the fact that the specimen was left for three days in the sun to heat before the test was done on 09/02/04, while it was done a few hours after the specimen was placed in the sun on 08/22/04. All the tests done at night without sunlight didn’t show any positive outcome and the results could not be considered reliable even in the cases were there was subtle response detected. This is because heat dissipation occurs
faster and that the specimens containing the Styrofoam blocks need a lot of heating to be detected. The subtle responses were detected from air filled flaws.

Table 7.2: Infrared Results for the Six Inches Specimen S6

<table>
<thead>
<tr>
<th>Date</th>
<th>Time</th>
<th>D1</th>
<th>D2</th>
<th>D3</th>
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<th>D5</th>
<th>D6</th>
<th>V1</th>
<th>V2</th>
<th>V3</th>
<th>Cracks</th>
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</thead>
<tbody>
<tr>
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<td>No</td>
<td>No</td>
<td>No</td>
<td>No</td>
<td>Subtle</td>
<td>No</td>
<td>Subtle</td>
<td>No</td>
<td></td>
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<td></td>
<td>pm</td>
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<tr>
<td>3:30</td>
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<td>Subtle</td>
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<td>Subtle</td>
<td>No</td>
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</table>

Depth (Inch) | 1 | 3 | 2 | 2 | 1 | 1.5 | 1.5 | 4.5 | 0.5 | Variable
Results from Table 7.3 shows the inability of Infrared to detect any of the defects even at times of maximum heating, this is due to the high depth of the Styrofoam blocks within the specimens and the low ability of infrared to detect the Styrofoam interface.

In all tests, the dimensions of flaws could not be detected precisely. IR Thermography is not capable of providing information about the extent and dimension of flaws. Depth calculation using IR Thermography is also not possible.

<table>
<thead>
<tr>
<th>Date</th>
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<th>D1</th>
<th>D2</th>
<th>D3</th>
<th>D4</th>
<th>D5</th>
<th>D6</th>
<th>V1</th>
<th>V2</th>
<th>V3</th>
<th>Cracks</th>
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<td>No</td>
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<td>No</td>
<td>Subtle</td>
<td>No</td>
</tr>
<tr>
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<td>No</td>
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<td>No</td>
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</tr>
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<td>No</td>
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<tr>
<td></td>
<td>5:00 pm</td>
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</tr>
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<td>1100 pm</td>
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<td>No</td>
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<td>3.5</td>
<td>2.5</td>
<td>variable</td>
</tr>
</tbody>
</table>
7.3 Test after Dissolving the Styrofoam Blocks

Tests were performed after Styrofoam blocks were dissolved using Acetone. Figure 7.9 shows a sample result obtained from tests performed on delamination locations in the defective six inch specimen S6. The tests were done after the specimen was exposed to the sun for two days.

![Figure 7.9: Six Inch Defective Slab with No Styrofoam Showing Delaminations](image)

The response obtained from this test indicates that air filled voids or delaminations are more visible for the Infrared camera when compared to Styrofoam filled delaminations. IR Thermography was able to detect all delaminations in this case as was the case with the Styrofoam present. Temperature differences between sound and defective concrete areas reached up to 7°F. The image in figure 7.9 shows that the geometry or size of the defect is not possible to determine as the responses are non uniform in shape.
7.4 Discussion of the Results

IR Thermography is a fast, easy to apply method for detection of flaws in concrete bridge decks. The method depends on the detection of temperature differences between sound concrete and deteriorated concrete. The tests performed in this chapter validated these facts. On the other hand, the ability of the method to detect defects is dependant on environmental factors. In addition, geometry of the defects, depth, and size affect the ability of the method to detect defects. This method is applicable were detailed information about the defects are not required since the method can not give information about depths, sizes, or geometry of defects.
8.1 Summary

The transportation infrastructure is deteriorating and in need of new techniques for inspection and maintenance. Out of the total number of bridges in the United States, 28.5% are functionally or structurally deficient. Problems in concrete bridge decks are considered to have major contributions in bridge deterioration. A bridge deck is a major component of any bridge and concrete bridge decks are facing several types of problems. The commonly used inspection techniques for the determination of bridge decks conditions are mostly time consuming, labor intensive, destructive in nature, and intrusive to traffic.

Nondestructive techniques emerged as new means of bridge inspection. Several nondestructive techniques have proved to be invaluable for the case of bridge decks. However, each of the techniques has some draw backs. Nondestructive techniques are becoming favorable to destructive testing as they are faster, less intrusive to traffic and give more qualitative results and their cost is usually less than destructive tests.

This thesis focused on the nondestructive techniques used for concrete bridge decks. A comprehensive study of those techniques was presented and the three most appropriate methods for concrete bridge decks were selected for further study. Ground Penetrating Radar (GPR), Impact Echo (IE), and Infrared Thermography (IR) proved to be good methods for detection of different types of flaws in concrete bridge decks.

An experimental program was developed. Three slabs flawless slabs and three defected slabs were cast. The defected concrete slabs contained simulated flaws found in real bridge decks. For this study, voids, delaminations, surface cracks, and vertical
cracks were introduced. The tests were performed over the fabricated specimens using the three nondestructive methods. Conclusions for each of the methods are presented.

8.2 Conclusions

The Study reached the following conclusions

8.2.1 Impact Echo Tests (IE)

- Impact Echo proved to be a very good technique for flaw detection in concrete.
- Impact echo was successful in detecting all flaws present in the fabricated slabs.
- The ability of impact echo to determine the depths of flaws depend on several factors, including the impact contact time, size of impactors, and real depth of flaw.
- Impact echo is successful in detecting steel layers and steel cover.
- Detection of steel layers is dependant on the contact time of the impactor and the energy content in the impact. In addition, steel cover and diameter of steel rebar play a role in the detection capability.
- Impact echo was successful in detecting vertical and surface cracks.
- The ability to detect the depth of surface cracks is dependent on the presence of a second transducer and can not be done using one transducer.
- Flaws less than 2.5 inches (65 mm) deep are harder to detect than deeper flaws using Impact Echo. However, information about the presence of the flow can be obtained.
- The response from a solid concrete plate will have one dominant peak in the spectrum. This peak is called the frequency peak.
• If the lateral dimension of the flaw is larger than 1.5 times its depth, the response obtained will be identical of that of a solid plate. If otherwise, the response will have one or more peaks in the spectra.

• Responses obtained from defects less than 2.5 inches (65 mm) deep will contain one or more dominant frequency peaks at values less than the thickness frequency. These correspond to either a flexural peak caused by the vibration of the think concrete layer above the flaw or by a shifted thickness frequency caused by the delay in the P-wave as it travels around the flaw.

• Concrete surface roughness had a great influence on contact time, for example, the surface roughness was very high in the eight inch specimen close to delamination D4. This flaw could not be detected because a small contact time could not be established.

• The selection of the impactor depends on how much information is known about the structure and the experience of the inspector.

• The wavelength (size of impactor) is related to the lateral extent and depth of flaw. The wave length has to be less than the lateral dimension and less than two times the depth. If information like this is available, the selection of the impactor becomes easier.

• Responses obtained from IE test in the presence of Styrofoam as a filler to delaminations was the same as that acquired when Styrofoam was dissolved and air was the filler.
8.2.2 Ground Penetrating Radar (GPR) Tests

- GPR is a newly emerging technique for subsurface evaluation.
- Antenna used in GPR surveys governs the resolution of GPR data.
- GPR was not able to detect vertical and surface cracks.
- GPR is an excellent method for steel detection.
- GPR is a useful method for detecting PVC pipes.
- Antenna polarization plays a role in the GPR capability to detect flaws.
- Scans taken parallel to upper steel layer was found useful for delamination and PVC pipes detection as the amplitude reflections from steel are less.
- GPR was able to detect delaminations and voids more than one inch (25.2 mm) deep with great precision.
- Depth calculation depends on the knowledge of wave velocity inside concrete. This is in turn dependant on the dielectric constant which varies from batch to batch.
- Raw data collected from GPR studies usually needs processing.
- Concrete cover plays a big role in the results obtained from GPR surveys. It was found that the higher the concrete cover the more accurate the GPR data are.
- The ability to calculate depths of reflectors using GPR technique depends on the depth of the reflector and depth of the steel layer if present.
- The resolution of the GPR image decreases as the targets become deeper. Reflections from targets at deeper depth are more subtle than the same targets at a shallower depth.
• Targets appear as hyperbolic reflections in the GPR data. However, delaminations sometimes appear as non uniform shapes.

• The reflection amplitude from a target depends on the size and depth of the target. Closer and larger targets have higher amplitudes.

• Larger targets appear larger in GPR data with larger tails to its hyperbolas.

• In some cases, steel diameter can be calculated using the GPR technique.

• The accuracy of calculating real depths of targets using GPR increases as the target is within four inches (102 mm) from the surface.

• Target depth calculation was >95% accurate for delaminations and voids when steel layer is more than 1.25 inches deep.

• The ability of GPR to detect closely spaced targets depends on the spatial resolution of the antenna and the antenna’s wavelength. The spacing between the targets has to be at least equal to the antenna wavelength in order to be seen as two separate targets. If targets are closer than antenna wave length, targets will appear as one.

• Flaw has to be at least 0.5 inch (13 mm) in order for GPR to be able to detect it. GPR was not able to detect flaws 0.25 inch (7 mm) thick.

• Targets located under the steel layer are difficult to detect.

• GPR tests performed on air filled voids gave the same results as when Styrofoam was present as filler in the voids.

8.2.3 Infrared Thermography (IR)

• Infrared Thermography is very dependant on environmental factors.
Temperature plays a minimum role in enhancing the ability of IR to detect flaws. However, presence on sunshine and absence of high speed winds assist the testing.

Test can be done anytime of the day but best results for IR testing are obtained during periods of maximum heating between 11:00 am and 4:00 pm.

Tests done during night time are less reliable as heat dissipation occurs fast.

Test is seasonal, best results are obtained during summer months.

IR test could detect delaminations and void when they were very shallow.

The detection of flaws in concrete bridge decks depends on the depth and size of the flaw. Bigger and shallower flaws are easier to detect using Infrared cameras.

The tested bridge should be left to heat for several days before test is done in order to get the optimum result from the test. This was adopted for the specimens in this thesis and the best results were obtained when specimens were left to heat for three days.

Roughness of concrete surface affects the ability to interpret the results.

IR is able to detect flaws up to three inches (76 mm) deep. Especially when the flaw is relatively large in lateral dimension.

IR does not provide information about the geometry or depth of the detected flaw.
8.2.4 General Discussion

The conclusions from the three methods show that GPR, IE, and IR are promising methods for detection of concrete bridge deck defects. The question of what method to select for a specific job depends on the degree of details required and the types of flaws under study. Infrared Thermography is a fast method with real time results possible; however, Infrared Thermography is a surface method. Its ability to detect deep flaws is controlled by several factors and is not so reliable because it depends on the environment. GPR, on the other hand, is a more sophisticated method used for subsurface scanning. It is also less time consuming and requires minimum lane closures. GPR possesses high capabilities in detection of different flaws but is dependant on the antenna type for resolution and minimum detection depth. Impact Echo (IE) is tedious since it requires a lot of testing points and is time consuming, but it gives results that are as reliable as much as GPR results. The only drawback of IE is that it is time consuming and requires experience while doing the test and while interpreting the results.

8.3 Contributions

This thesis has made the following contributions

- A comprehensive study of the common defects in concrete bridge decks.
- A comprehensive study of the nondestructive evaluation techniques used for concrete bridge decks.
- Validated the capabilities of Ground Penetrating Radar, Impact Echo, and Infrared Thermography in the detection of several types of defects.
• Provided a comparison of the capabilities of the three methods showing the strength and weaknesses of each of the methods.
• Provided a guideline for the selection of IE, GPR, or IR depending on the application.
• Demonstrated the abilities to detect several defects using designed lab specimens.

8.4 Future Research

The following are future research areas that can support and enhance this study

• Testing GPR, Impact Echo and Infrared Thermography in real life bridges for validation of obtained results on real bridges.
• Developing a database system for the responses obtained from different flaws in order to facilitate the interpretation of GPR and Impact Echo (IE) data.
• Developing mathematical models that can convert the responses obtained from Impact Echo into images for easier interpretation and depth calculations.
• Developing a national database for dielectric properties of materials in order to facilitate depth calculations when GPR is used.
• Developing mathematical models that can provide information about depth of flaws from Infrared Thermography images by sensing color intensities related to temperature differences.
• Validation of the effect of steel reinforcement on the Impact Echo response.
REFERENCES


23. ASTM D 4580-86 “D4580-03 Standard Practice for Measuring Delaminations in Concrete Bridge Decks by Sounding”


50. ASTM C 1383-98a “Standard Test Method for Measuring the P-Wave Speed and the Thickness of Concrete Plates Using the Impact Echo Method”


68. GSSI Handbook for Radar Inspection of Concrete.


71. GSSI RADAN Manual( Unpublished Data)


APPENDIX

Nondestructive Testing Suppliers for Impact Echo, Ground Penetrating Radar (GPR), and Infrared Thermography
Impact Echo Equipment:

Several manufacturers of the impact echo systems are available in the United States of America. Some of those where contacted and the following paragraphs present the systems that they provide.

Impact Echo Instruments LLC: Located in Ithaca, New Y. Impact echo instruments are a manufacturer and provider. The system they provide had been specifically designed for testing of concrete bridge decks for locating flaws in concrete structures. The system was developed by the people who first researched the usability of Impact Echo and developed the method through years of study. This device was chosen for the purpose of this thesis. Details and characteristics of the system used in this thesis were presented in chapter four.

Physical Acoustic Corporation: located in Princeton, New Jersey. They provide systems that could be used in the detection of flaws in bridge decks and masonry.

NDT James: located in Chicago, Illinois. They provide a portable Impact Echo device for the detection of flaws.

Qualitest: located in Plantation, FL. Provides and manufactures revolutionary portable impact echo testing devices. Figure 4.3 shows the Qualitest Impact Echo System.
Ground Penetrating Radar (GPR):

A great number of companies in the states provide GPR service and/or manufacture GPR equipment as GPR is evolving as a comprehensive method for detection of underground flaws. Some of these companies are listed below.

Geophysical Survey System: located in North Salem, New Hampshire. It is a manufacturer and a service provider for GPR systems. The company sells complete GPR systems for different applications and provides software to analyze and process data collected using their air-coupled or ground coupled antennas. A ground coupled system manufactured by GSSI was chosen for this thesis. The acquisition system is a SIR 3000 with a 1.5 GHz antenna. Details about the system used were presented in chapter four.

Mala Geoscience USA, Inc: located in Charleston, South Carolina. They manufacture and provide GPR services. The systems they provided are configured for detection of cavities, buried utilities, profiling of soils, and other geotechnical applications.

Penetradar Corporation: located in Niagara Falls, New York. Penetradar manufactures a variety of GPR systems and provides inspection services. They offer software for data analysis and processing along with portable GPR systems.
Sensor & Software Inc located in Mississauga, Ontario, Canada. Sensor and software is a manufacturer and service provider for GPR services. Different configurations of the GPR systems are available through this company.

**Infrared Thermography (IR):**

A big number of vendors and manufacturers provide thermograph equipment for the purpose of thermo analysis and temperature studies. Some of those where contacted for the purpose of this study to obtain an infrared camera to study the ability of the technique to detect flaws in bridge decks. The following are some of the providers.

**Electrophysics:** located in West Fairfield, New Jersey. Electrophysics develops advanced near infrared, night vision and thermal imaging systems for use in different areas. The EZ Therm Portable Thermography System was selected for the use in this thesis. Details about the device selected along with its specifications were provided in chapter four.

**IRCON Inc.:** located in Niles, Illinois. IRCON is a manufacturer, sales and service provider for thermal imaging and infrared cameras along with other services. It also provides image analysis software to help interpret and read images.

**Land Instruments International:** located in Newton, Pennsylvania. Land Instruments International is one of the world's leading specialists in the design and manufacture of systems and instruments for industrial infrared temperature measurements.
Infrared Solutions Inc.: located in Plymouth, Minnesota. The company works on developing and selling infrared sensing and imaging devices. A variety of products are available through this company for different purposes.

Ashtead technology: located in different locations in the United States. This company sells, leases infrared cameras and temperatures measurement solutions along with a variety of non destructive testing equipment.