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Non-destructive condition assessment of concrete slabs with artificial defects using wireless Impact Echo

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ABSTRACT

This thesis presents the development and validation of a new wireless Impact Echo (IE) system for condition assessment of reinforced concrete slabs. The new IE prototype was compared with other commercially available non-destructive testing (NDT) devices used for similar purposes, namely Ground-Penetrating Radar (GPR) and Ultrasonic Pulse Echo (UPE). Monitoring and structural inspections are critical to effective management of civil infrastructure and NDTs can enhance the quality of condition assessments by providing objective visualizations of the interior of a structural element.

The IE method, first developed in the 1980s, has seen few advancements in the last 20 years. The method has been standardized and used on site, but the underlying technology has become outdated. The data obtained from the transducer is difficult to interpret and requires a computer to post-process it before being usable, thus limiting the direct feedback of the method when conducting tests on-site. Because of those limitations and the test being relatively more time consuming than other alternatives, the method is lacking in usability. A new prototype IE device was designed and built by the project industry partner, FPrimeC Solutions. The methodology followed the traditional approach, but it was designed to work with today's technology. The device is operated wirelessly via a Bluetooth connection, uses smaller-sized electronic components, and connects with a user-friendly interface on a small tablet to set-up the tests and compute the results immediately. The first part of the project focused on product development by testing iterations of the prototype and providing user feedback to improve the device and accompanying software.

The second part of the project aimed to validate the new technology using a set of three large reinforced concrete slabs containing artificial defects. The studied points of interest were sound concrete, effect of boundaries and steel reinforcements, vertical cracks, presence of a hollow conduit, artificial voids and delamination. The IE results were also compared with those from commercial GPR and UPE devices. GPR was found to be the quickest method by far, although the results gathered seemed to be limited by the presence of steel reinforcement and also failed to locate certain defects. UPE was a bit slower than GPR, but was generally able to locate more accurately the artificial flaws created in the test specimens. The results showed poor definition of the flaws making it difficult sometimes to properly locate them. The UPE results also seemed to be negatively affected by the presence of reinforcement which were causing frequent abnormal values. Lastly, the IE method was used. This method was greatly improved during the first phase, but it is still a time-consuming method. The value of the data, however, has great potential when compared to the other options. It accurately located most of the flaws generated and was practically unaffected by the presence of steel reinforcing bars. Also, with further analysis of the data, it was possible to determine the depth of some of the flaws accurately.

Due to the time-consuming testing phase and the longer analysis of the data required to obtain the higher quality of results, this study suggests that IE is not likely to be the best choice for a general inspection of a large area (depending on the nature of the information needed). Rather, it is suggested to first conduct a general review of the structure using a quicker method like GPR to locate the problematic areas. After that, refining the grid at key locations to test with IE should provide the best quality of data in a reasonable amount of time.

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1 INTRODUCTION

1.1 BACKGROUND

Civil engineering is one of the oldest fields of work in human history and could be described as the design, construction, and maintenance of public infrastructure. It is a very broad field as it is the source of almost everything that we see and need in our societies, from buildings and foundations, to bridges, dams, sewage systems and treatment, city streets, highways, and much more. There is always a place for civil engineering and a need for improvement in the field. While most of the academic attention is focused toward finding new ways to construct, design, or repair infrastructures, relatively fewer projects investigate ways to evaluate existing infrastructures. A large part of the engineer's role is to be able to assess the condition of a structure and be able to accurately predict its limits. That is true for both old and new structures. It is not always the best option to simply remove the old infrastructure and build a new one. It might be much simpler that way, but it represents a large waste if the service life of the structure could have been extended through higher quality inspections and proper maintenance strategies. Frequent inspections and maintenance allow to reduce the overall cost of infrastructure significantly; more importantly, it ensures that the structure can still fulfill its role without risk of failing and endangering lives.

Inspection methods have been largely unchanged for a very long time. For reinforced concrete, the traditional ways of conducting simple visual inspections assisted by destructive methods such as coring or drilling to visually see inside the material are still in use. Those methods are simple and often directly provide the answers needed, but they also have limits and negative impacts. Visual inspections, while being a strong method if done systematically, are still subjective to the inspector's judgement and limited by the access to a structure. For destructive testing, as the name implies, it requires to remove a portion of a structure to either see underneath or test the material properties of the sample in a laboratory. If kept to a minimum, the removal of a material sample generally does not affect the structural integrity of the structure significantly, but it does limit the amount of tests that are acceptable and limits the locations that can be tested. The method is also limited by only investigating a precise and discrete location, and can be costly.

To solve those issues, many developments on Non-Destructive Testing (NDT) have been proposed in recent years. The goal of these methods is to provide tools that can provide immediate, repeatable, and reliable results without affecting the structure. Those methods are designed to utilize indirect

measurements to assess the condition of the structure, such as the use of waves. Generating a wave (e.g. acoustic, electromagnetic, or stress wave) that will interact with objects or flaws present in the material and studying their effect is a popular approach to the problem, but other approaches are available depending on the problem being investigated.

For this project, the primary focus is on the Impact Echo (IE) method, a stress-wave-based non-destructive method. The method was invented in the mid 1980's by the U.S. National Bureau of Standards and further developed at Cornell University from 1987 to 1997. During that time, many papers on the development and reliability of the method were published, mostly by Mary J. Sansalone, the principal inventor of the method (Sansalone and Streett 1996). The method was proven effective and then standardized for use in ASTM C1383-15. However, since then, in the last 20 years, very few papers were published on the subject and little or no advancements have been made on the method. Its use has been relatively popular for the condition assessment of concrete slabs, but due to its limitations is beginning to be superseded by newer methods.

The premise of the IE technique is to locate air pockets inside the reinforced concrete element. The impact-generated waves travel through the solid material and get reflected at the concrete-air interfaces due to a sudden change in the acoustic impedance between the material and the air. Those waves return to the surface and create a resonance vibration in the concrete section which is captured by a transducer. The sensor tracks the perpendicular undulations over time which can then be transformed to a frequency domain and used for the analysis. The analysis allows the user to estimate the depth at which the wave was reflected, which is most likely the full depth of the concrete element in the case of sound concrete. However, if an air-containing flaw is present, it will reflect the wave in various ways depending on its nature and shape resulting in a wrong prediction of the depth. Those results stand out allowing the user to locate the flaws in a concrete element once compared to other points.

The strengths of the IE method include its accuracy and the detailed information it can provide on the tested element, but it lacks in usability. The main disadvantages are that the method is time consuming and the results are difficult to interpret. The data interpretation requires a decent computer to transform the data into a usable format and then a strong knowledge of the physics behind the method to achieve the right conclusions. For this reason, the inspectors using the device need to have good training and some experience to use it properly. The tests also take longer time than other similar methods as the sensor used is required to be set up on a clean surface and maintain a good contact. This part is very sensitive as a small angle of the sensor or dirty surface can cause bad readings.

For these reasons, alternative NDT methods are often preferred to Impact Echo. Utilizing advances in technology in the recent years, the usability of the method has been improved through this research. The research partner, FPrimeC Solutions, used the results of this study and the feedback provided from the testing to design a new and upgraded device which improves most of the known issues tied to the method's usability. The new version is much smaller, Bluetooth-enabled, and handheld, making it much easier to use than conventional devices. A user-friendly software was also developed to use on a tablet that provides immediate access to the results on the site without requiring the user to go through a complex data analysis.

1.2 OBJECTIVE AND SCOPE

The purpose of this project was to test and improve the Impact Echo device and software prototypes in collaboration with an industry partner. The new device built for this project aimed to improve the analysis phase by making it simpler and quicker while keeping the strong accuracy and precision the method is known for. The first objective of this project was to help with the development of the prototype by sharing the collected data and providing feedback to the developers concerning the issues encountered and suggesting improvements. The first phase of the project focused on simpler tests conducted on sound concrete sections since their goal was to understand the significant usability issues with the prototype and the software. This iterative approach led to a functioning device that was capable of performing the more complex tests planned for this project in the second phase.

Subsequently, the second objective was to validate the method's ability at finding defects. The working prototype was validated according to a review of the main uses of the IE technique to understand the current strengths and weaknesses of the method and confirm that the smaller wireless prototype did not cause unexpected results.

Furthermore, the third objective was to compare the results with alternative NDT that are commonly used for similar purposes. NDT in general are based on indirect measurements, making them imperfect by nature. None of the current methods are able to fully assess a structure with perfect accuracy. For this reason, it is not expected for the method to perfectly find all of the artificially created flaws in each of the test specimens. This makes it difficult to objectively evaluate the quality of the analysis using the new IE system. The head-to-head comparison was intended to provide the data required to properly describe the relative strengths and weaknesses of the IE method. The primary NDT methods considered in this project were Impact Echo (IE), Ground-Penetrating Radar (GPR), and Ultrasonic Pulse Echo (UPE). Both GPR and

UPE are methods that are commonly used for reinforced concrete slab elements in the field when investigating physical deterioration.

The scope of the project comprised of the design and fabrication of three reinforced concrete slabs containing several points of interests such as artificial flaws or slab characteristics that were expected to affect the results such as the presence of steel reinforcement and boundaries. For the artificial flaws, the selected types included vertical cracks, voids, and delamination. Those types of internal defects occur frequently and contain air; hence, locating and measuring these defects represents some of the main uses of an IE device. The other points of interest that were considered are changes in slab thickness, boundary effects (both at the sides of the specimen and at a discontinuity at the bottom of the slab), the presence of a conduit buried in the material, and the effect of steel reinforcement. This testing program does not cover all of the possible problems the device could have or all of its potential uses, but is indicative of the strengths and weaknesses of the method for various applications.

When comparing the various NDT methods (objective 3), the first characteristic looked at during the testing phasing was the speed and ease of use. While it is essential for the equipment to be reliable and accurate, it might not be the most desirable option if it takes considerably longer to test and analyze than the other options available. The second and main characteristic assessed during the analysis was the accuracy of the methods. The tests specimens were constructed such that the actual location and size of each defect was known; therefore, the new IE system could be evaluated according to its ability to accurately detect each feature. Lastly, the precision of each device was another point of interest that appears in the results as the degree of definition of the located flaws.

1.3 THESIS OUTLINE

The thesis follows a conventional outline, starting with this introductory chapter. The following chapter, Chapter 2, presents a literature review on the relevant subjects used in the experiments and analysis. The purpose of that review is mostly to describe and define the necessary theory needed to understand the content of this project. It will quickly cover the basics about relevant reinforced concrete deterioration mechanisms and the defects they cause. After that, the literature review tries to illustrate the state of the current knowledge on the methods used in this analysis and more precisely the Impact Echo method, which is the main subject of this project.

Chapter 3 describes the experimental work done for this project in detail by showing the plans, tools used, and the methodology. The design and fabrication of the three reinforced concrete slabs used as test

specimens in this study are discussed. Each slab is considered individually, describing their key points of interest and their purpose in the analysis. This allows to understand why each of the slabs were relevant. The chapter also accurately describes the characteristics and locations of the artificial defects made. Lastly, the chapter covers the testing methodology for each of the methods used during the experiment and briefly mentions the developments that were made on the prototype based on the work of this project.

Chapter 4 presents the results and the analysis done on the data collected for each method. This serves as a basis for a comparison between the methods. The results and their implications are discussed in detail. Finally, in Chapter 5, the conclusions drawn from this study are summarized, and recommendations are provided for future work.

2 LITERATURE REVIEW

2.1 REINFORCED CONCRETE

Reinforced concrete has been the most common type of material used for large structures for a long time. In general, it is a strong and durable building material that is also relatively economical. Its fresh state properties allow it to take on any form; this versatility has contributed to its popularity around the world. Recent advances in concrete technology and improved understanding of the internal hydration reactions and microstructure have allowed engineers to drastically improve its performance according to the intended applications (ACI 201.2R-16).

Plain concrete (i.e. unreinforced) is a material with a relatively high compressive strength and a low tensile strength. It is almost never used on its own for structural purposes, since the formation of a crack could result in almost immediate collapse. For this reason, reinforced concrete was developed. It solves the problem by embedding another material within the concrete to handle the tensile forces. Nowadays, most design procedures are based on the use of steel reinforcing bars to take all the tensile forces present in the structure (ACI 201.2R-16).

Reinforced concrete, like every type of material, has limited durability in certain environmental conditions. Improvements in design and quality control may have a considerable effect on its service life, but long-term deterioration continues to be an important consideration. From Canada's Core Public Infrastructure Survey (CCPI, 2016), the average estimated service life is 68 years for highway bridges while being only 48 years for local road bridges. In Canada, 40% of the bridges were built between 1940 and 1969. Those old bridges with more than 45 years of service life are either requiring regular maintenance or are likely to need intervention in the near future as they are reaching their predicted service life. The survey classified the condition of slightly more than 10% of the bridges as poor to very poor, where poor is described as "the asset is approaching the end of service life; the condition is below standard, and a large portion of the system exhibits significant deterioration." The aging infrastructure nearing the end of their service life requires a higher frequency and quality of inspections to remain safe until their replacement. It shows that there is a need for better instrumentation and methodologies to provide rapid inspections with good accuracy.

The causes of deterioration in reinforced concrete structures are so varied that it is virtually impossible to fully protect the structure against all of them. Damage can occur from physical sources such as excessive

loading of the structure or repeated loading (i.e. fatigue). Deterioration can also be caused by chemical reactions that can occur inside the element like corrosion and alkali-aggregate reactivity. Also, there are environmental conditions that may contribute to accelerated deterioration; in Canada and other northern countries, the harsh climate during winter can cause water to freeze and expand within concrete pores leading to cracking and scaling. Finally, defects may also occur due to human error. During construction, errors may result in consolidation problems, excessive bleeding or shrinkage, and other deficiencies that can increase risk of deterioration.

The common ways to handle those problems include a strict quality control process during the construction, frequent visual inspections, and maintenance when required. Many methods can be used to fulfill those purposes. It is possible to use destructive methods to measure the characteristics and thickness of the concrete (i.e. core extraction). This provides a direct measurement of the characteristics such as the composition and strength but require samples to be sent to a laboratory to perform the testing. Furthermore, information may only be collected for samples extracted at discrete locations. Visual inspections are relatively cheap and simple, but are also highly dependent on the skills of the inspector and limited to surfaces that can be visually assessed. Non-destructive testing (NDT) is now becoming increasingly popular as a more advanced way to perform quality control and inspections. NDT methods enable enhanced identification and quantification of defects within an element, and may be used even if there are no visible clues of the presence of defects from the surface. Many types of NDT exist for different applications, but for the purpose of this study, the primary focus will be on a wave-based technique called Impact-Echo.

2.1.1 Composition

Concrete is a heterogeneous man-made rock comprised of smaller aggregates bonded together with a cement matrix. The most common type of cement used is Portland cement, but other types exist. It comes in a powder-like state which is mostly calcium silicate, before being hydrated. Once it is hydrated, a chemical reaction occurs which causes the concrete to progressively gain strength and harden. The cement hydrates grow and spread until they come in contact with other cement particles or a surface. The reaction is slow enough to allow time to manipulate the composite material before it hardens, known as the setting time. The mixture is in a semi-solid state which allows it to take on any form and gives it a lot of versatility (CAC, 2016).

The quantities of each component are calculated through a mix design process to achieve desired properties. What used to be a simple process has evolved into a rather complex science that has seen

considerable improvement in recent years in terms of customizability. The mix design process allows engineers to design the characteristics of the material to meet various requirements, being its strength, durability, or workability. Initially, mix designs focused mostly on the optimization of the water to cement ratio, which has a large impact on the characteristics of the concrete since a smaller ratio resulted in the highest strength, but lower workability. However, in the more recent years, researchers started to test new components to add to the mixture and further change its characteristics.

Nowadays admixtures are commonly used to optimize the concrete behaviour. Accelerators are used to speed up the hardening of the concrete and retarders are used to slow the process. Air-entraining agents can be used to reduce the damage caused by freeze-thaw cycles. There are also corrosion inhibitors, bonding agents, and admixtures to reduce the permeability of the concrete. These provide ways to adjust the properties of the concrete and overcome many limitations (CAC, 2016).

2.2 CAUSES OF DETERIORATION

Reinforced concrete has seen a lot of improvement through the years and even though the deterioration processes are better understood and mitigation measures are implemented, it still occurs for many types of structures. There are many reasons based on the location and use of the material that cause it to deteriorate at variable rates. Most types of deterioration are influenced by the presence of moisture or chemicals that can enter the concrete. The effects of deterioration are often manifested as cracks that may (or may not) be visible on the concrete surface which further increase the speed of the deterioration processes. For this reason, it is important to identify the underlying cause(s) of deterioration and repair the concrete before the damage becomes critical. Several of the main causes of deterioration and defects will be described in this section.

2.2.1 Construction Errors

Many deterioration processes can be traced to errors or poor methodology used during the construction phase of a project. Good quality concrete has the possibility to be a very strong and durable material but can also be prone to a wide range of problems if not managed properly. Improper curing protocols, early removal of shoring, misplacement of reinforcement, and poor concrete consolidation are just a few examples of errors that can occur during construction. These types of errors are usually accounted for in design through reduction factors that are applied to the nominal resistance of structural members.

The projects are designed and reviewed by qualified engineers, but the construction itself is usually performed by a contractor of varying competency. Construction workers may not always be familiar with

the science of concrete or may try to take shortcuts to complete the project faster. Adding water to the concrete mix to make it more workable is a common mistake made by those who are not aware of how the chemical reaction of cement works. Concrete strength is based mostly on its water to cement (w/c) ratio, therefore adding more water reduces the strength of the material considerably, while also increasing porosity and susceptibility to moisture ingress. It also affects the concrete characteristics that were used for the calculations. Adding water can increase the temperature differential which can potentially cause cracking. Even if more cement is added to keep the w/c constant it will result in more shrinkage than predicted which also causes cracking (ACI 224.1R-07).

Another mistake that often occurs is a poor consolidation of the concrete (Figure 2.1). Projects often progress quickly, and the right amount of vibration can be difficult to judge. A poorly consolidated concrete will be weak, non-uniform, porous, and have a poor bond to the reinforcement.

Figure 2.1 Honeycombing and voids caused by poor consolidation (ACI 309R-05)

Voids caused by improper consolidation represent a weakness that will increase the rate of deterioration of the structure. The weak material will have less capacity than expected and will crack at lower loads while the high porosity will allow water as well as chemicals like deicing salts to enter the concrete easily, resulting in faster corrosion of the steel reinforcing bars. Other types of errors can occur, such as not following the plans properly or even simple mistakes in measurements. All of these errors will affect the loading capacity and the deterioration processes of the structure (ACI 224.1R-07).

Because there are many errors that can occur during the construction phase, projects are required to have frequent inspections and may even have a qualified inspector on site following the project during the entire construction phase. The inspections are also usually performed by a different party to ensure a good work ethic. Inspections are required to ensure the construction methodology is followed and the material produced is of good quality. Quality control has become a large aspect of project management

and quick and reliable methods to test the materials are required to make the whole process faster, easier, and accurate.

2.2.2 Excessive Loading

Excessive loading is one of the potential causes of damage in structures. Concrete structures are designed to remain elastic under service loads, with limited cracking. In Canada, concrete structures are designed according to the National Building Code of Canada (2020) and CSA A23.3 (2019) standard (or CSA S6 (2019) for bridges) that require the designer to consider various load cases and apply load and resistance factors to satisfy ultimate and serviceability limit states. These factors are calibrated to maintain a consistent reliability, ensuring a low probability of failure, and account for the possibility of overloading. The variability of the material quality is also accounted for with resistance factors that are specific to the material used in the design. Since these design standards are updated periodically, older structures do not necessarily meet the stringent requirements of modern codes. Many reinforced concrete structures are getting old and are starting to deteriorate while the usage continues to be increasingly demanding. The equipment in buildings and the trucks using the bridges are heavier than they were when some structures were designed. Older structures often require a reevaluation of their strength to ensure they can handle the current loads.

Reinforced concrete is designed in such a way as to prevent a catastrophic failure even if the limit states are reached. This is partially due to the ductility of the reinforcing steel which permits plastic deformation of structural members, providing visual warning prior to collapse and redistributing loads to other parts of the structure. Deformations are expected to occur without failure, but will result in micro and macro cracks in the structure in the transverse or longitudinal direction. These cracks represent a reduction in structural integrity, and can also lead to other types of deterioration afterward as they provide access for moisture, chlorides, and other chemicals to reach the core of the concrete.

2.2.3 Fatigue

Fatigue is the process of deterioration caused by repeated loadings. Loading and unloading a structural element repeatedly can often cause more damage than having a heavier load permanently on the structure. Cyclic loads result in an accumulation of microcracks in the concrete that can lead to a gradual reduction in strength and stiffness. Fatigue damage does not require a large force to deteriorate the structure. The simple fact that a structure deflects and regains its original shape repeatedly will cause problems related to fatigue. Under fatigue loading, a micro crack will be initiated at a stress riser such as

a discontinuity or a change in the section geometry (ACI 215R-74, 1997). The steel crossing the crack will take on the cyclic load and suffer the repeated deformations.

Fatigue must be considered when designing structures that have unavoidable cyclic loadings. Bridges are particularly affected by fatigue due to the large live loads and number of vehicles crossing on a daily basis. The damage caused by fatigue may not necessarily appear in regions with high average stresses due to tension, compression, or shear.

2.2.4 Reinforcement Corrosion

Corrosion is a very common process that affects steel reinforcement, especially in Canada. In reinforced concrete the steel reinforcement is embedded in concrete which protects it with its high alkalinity. The pH of the concrete naturally forms a passive protective oxide film on the steel that protects it from corrosion. That film can be damaged by carbonation or by the presence of chlorides that are present in deicing salt or sea water. Once the film is destroyed, an electric cell is formed along the bars and the corrosion begins. The electrochemical process will create rust on the surface of the steel which will gradually expand, damaging the concrete.

Figure 2.2 demonstrates the corrosion process of steel. The dissolution of the iron takes place at the anode, while moist concrete acts as an electrolyte. The ferrous ions then combine with hydroxyl ions, oxygen and water to form various corrosion products (CAC, 2016), (ACI 201.2R-16).

Figure 2.2 Corrosion Process (CAC, 2016)

The rate of corrosion may be affected by multiple factors. The main one is the presence of water, which plays an important role in the reaction. Hence, limiting the availability of moisture that comes in contact with the reinforcement is a good way to prevent the corrosion of the steel. The second main component of the reaction is oxygen. Preventing the access to oxygen would slow the corrosion process but would not prevent it. Usually the reinforcements are covered by a layer of concrete to limit the access to water

and air. However, concrete is a porous material, and cracks or spalling allow moisture to easily penetrate to the level of the reinforcement. Maintaining a high pH in the concrete also causes a protective film to form on the steel. Due to its composition, concrete initially has a high pH (about 14) that gradually decreases over time due to carbonation. Other factors that affect the process are the presence of chloride ions which act a catalyst for the reaction, the temperature, the electrical resistivity of the concrete, and the permeability of the concrete (CAC, 2016).

Figure 2.3 Corrosion damage example (FprimeC Solution, 2016)

Figure 2.3 demonstrates the typical deterioration caused by corrosion in reinforced concrete. The deterioration to the structure is caused by the rust produced from the reaction. The rust occupies a volume 6 to 10 times larger than the original steel (CAC, 2016). The expansion of the reinforcement inside the concrete element causes tension from within the element. As result, the concrete cover will crack, spall, or delaminate leaving the reinforcement exposed to the weather and chemicals (Figure 2.3). Corrosion also affects structural performance as it reduces the section size and the strength of the reinforcing steel, leading to decreased load capacity and greater susceptibility to other types of deterioration such as fatigue.

2.2.5 Alkali-Aggregate Reactivity

Alkali-Aggregate Reactivity (AAR) is a reaction between some constituents of the aggregates and the sodium and potassium alkali hydroxide in the concrete. The reaction causes a gel that expands when it comes in contact with water which can damage the concrete if it is significant enough. There are two forms of the reaction, the alkali-silica (ASR) and the alkali-carbonate (ACR). ASR is the most common

because reactive silica is a more common constituent of the aggregates used for concrete. Because of that, the problem can be mostly avoided by using aggregates that do not contain reactive silica (CAC, 2016) (ACI 201.2R-16).

Figure 2.4 Photograph of a parapet wall showing typical map-cracking at the surface (ACI221.1R-98)

The damage caused by AAR can often be identified by a network of small cracks, damaged joints, or displacement of parts of the structure (Figure 2.4). The reaction is a slow process that has low probability of causing the failure of a structure. However, the damage it causes can lead to serviceability issues or increase the speed at which the structure is affected by other types of deterioration by cracking the concrete cover and exposing the core (CAC, 2016).

2.2.6 Freezing and Thawing

One of the most damaging environmental factors for reinforced concrete in Canada is the effects of repeated freezing and thawing cycles when there is presence of water in the concrete pores. Water accumulates inside the pores of the cement paste and the aggregates and expands by approximately 9% when it freezes. The problem is exacerbated when deicing salt is used as it increases the ions concentration in the pore solution and impacts the freezing behaviour. When the water freezes, the hydraulic pressure rises and applies a large force on the concrete from within its structure. The resulting tension forces lead to cracking, scaling, and crumbling of the concrete. A proper curing and maturity are necessary to provide the element with enough strength to withstand the freezing cycles. At its early age, concrete is much more susceptible to freezing and thawing damage. For this reason, concrete castings are usually planned so that the material has sufficient maturity and strength when the temperature drops below 0°C (CAC, 2016).

Air-entrainment was developed to prevent issues associated with freezing porewater. Having air bubbles evenly distributed inside the concrete allows the water to expand and dissipate the water pressure within

the concrete element. A good quality of cement and a low cement-to-water ratio is also necessary to reduce the freeze and thawing problem. From a design point of view, the proper way to handle the problem is to limit the freezable water that can access the material (i.e. the degree of saturation). Keeping a low water to cement ratio will allow to reduce the initial amount of water trapped inside the material that is not required for hydration. This can be pushed further by adding admixtures to the mix which allow to reduce the amount of water even more. Those issues are also affected by the characteristics of the aggregates, mainly the coarse aggregates. Some types are more resistant to freezing damage while porous aggregates may be more prone to damage (ACI 201.2R-16).

2.3 INSPECTION OF REINFORCED CONCRETE STRUCTURES

Inspections are required to improve the longevity of a structure as well as to ensure that the risk of failure is low. The Ontario Structural Inspection Manual (OSIM, 2008) describes the goal of the inspection as “to ensure, within an economic framework, an acceptable standard for structures in terms of public safety, comfort and convenience.” Structures are designed with an expected service life in mind which may be reduced or extended depending on the conditions they are in and the quality and frequency of the maintenance. Reinforced concrete structures represent a very large cost, therefore, it is important to estimate the severity of the degradation of a structure and its economic implications. Certain components of a structure have a larger role in the stability of the whole system. It is essential to locate the critical components and properly inspect them as they can lead to a failure which represents a large cost as well as a danger for human lives. Based on the extent of degradation, the knowledge of the strength of the material and which components are critical, a decision can be made to determine if remedial actions are required. For most types of degradation, if detected early, a small maintenance or repair of the structure can stop or slow down the deterioration. Early identification of deterioration can prevent costly repairs as larger repairs or replacement can get very expensive. For this reason, it is economically beneficial to inspect a structure frequently. In Ontario, for example, the provincial government requires that most public structures such as bridges must be properly inspected at least once every two years [OSIM, 2008]. That represents the minimal inspection that must be conducted, although structures may be inspected more frequently if signs of deterioration have been observed or if it is a new type of structure. Usually in those cases, the engineer in charge would judge the situation and demand an increase in the inspection frequency. The inspections allow to increase the life span of structures by identifying the needs of the structure for maintenance and rehabilitation. They also provide the owner with information required to develop a management plan for the structure (OSIM, 2008).

Many inspection methods exist to either locate signs of degradation or to ensure the quality of the materials. For quality control of reinforced concrete during construction, cylinders are casted as samples and tested for their strength and durability. Similarly, cores can be extracted from an existing structure and tested to measure their properties over time (ACI311.4R-05). After the structure is built, assessments are mostly performed through visual inspections, which are relatively inexpensive, or by using various types of sensors that monitor certain characteristics of the components.

Following a standard method such as the one proposed by the “Guide for conducting a visual inspection of concrete in service” (ACI201.1R-08), most types of deterioration can be found once they have progressed to a certain extent. The detailed inspection is conducted and documented by a qualified inspector with a good understanding of the structural component. Prior to the inspection, the overall integrity of the structure must be assessed to identify the areas requiring more attention and areas requiring more specialized tools to inspect. The structure must also be investigated under loads for abnormal deflection or noises. After that, the inspection must be done systematically following a list to ensure every component has been inspected. The inspector must take notes on any material defects, performance issues, maintenance needed, and estimate the time frame in which the work must be completed. The inspection must be well documented using pictures and quantitative measurements, when possible, to easily follow the progression of the deterioration over time (OSIM, 2008).

Visual inspections offer a good review of the structure and are sufficient most of the time if done properly for structures not showing signs of deterioration. However, it comes with a severe disadvantage which is that the method is subjective and cannot quantify the damage on the structure (especially internally). The results and interpretation depend on the judgement and experience of the inspector performing the visual inspection.

Visual inspections are often considered sufficient for structures in good condition as long as no visible signs of deterioration appear. Some deterioration processes are quickly visible from the surface, but it is not always the case. Defects such as corrosion originate from the reinforcing steel and need to expand sufficiently to crack the concrete and become visible from outside. This type of internal deterioration requires more advanced techniques to be found early such as the use of NDT. However, in many cases, internal investigations are not conducted if the deterioration has not progressed enough to be demonstrating visible signs on the surface.

Since many types of defects are internal or in inaccessible locations, a defect may not be identified by the inspector. This has led to many failures in the past like the failure of the De la Concorde Overpass in September 2006 which resulted in five deaths (Figure 2.5). The designer used a type of half-joint that can deteriorate easily while being impossible to fully inspect and repair without removing the precast span. Those types of joint designs are not used anymore, but at the time it did follow all the design code recommendations (MTQ, 2017).

Figure 2.5 De La Concorde Overpass Collapse (Montreal Gazette, 2007)

The commission in charge of studying this failure found that the lack of accountability related to the quality control of the work and the material was a large cause of weakness of the structure. Another cause would be the lack of proper evaluation of the structure. The structure was impossible to fully inspect due to its design and during the few instances when maintenance work was performed, the authority in charge of the bridge did not use the occasion to conduct a detailed inspection. Many structures have design or construction issues that can lead to faster deterioration than expected. A good inspection is required to ensure the structure is not deteriorating faster than expected at a critical location (MTQ, 2017).

Nowadays inspections of the structure must be considered during the design process to make sure the critical components are accessible. New techniques were also developed to help with the inspections and quality control. Non-destructive techniques are gaining popularity as an additional tool for inspections as they present many advantages compared to conventional methods.

2.3.1 Destructive and Non-destructive Testing

The most definitive way to test a material or structure is to use destructive tests. Usually, destructive tests require the extraction of core samples from the structure. The cores can be taken for many reasons

depending on the needs of the situation. Sometimes it will be taken to a private laboratory to test the sample's durability and describe its characteristics using measurements. Other times, it is simply to investigate the interior of a structural element for deterioration that may not be visible from the surface. Those tests require to remove or break a portion of the structure to investigate it.

Extracting samples for destructive testing may be problematic in terms of structural integrity, cost, inability to repeat measurements, and challenges associated with obtaining representative samples for an entire structure; therefore many types of non-destructive test (NDT) methods have been developed through the years. They all have their purpose with certain advantages and disadvantages. Baldev, et al. (2007) provide a good comparison of the characteristics of destructive and NDT methods shown in Table 2.1.

Table 2.1 Advantages and limitations of destructive and non-destructive techniques (Baldev, et al., 2007)

	Destructive tests	Non-destructive tests
Advantages	<ul style="list-style-type: none"> ▪ The measurements are direct and reliable. ▪ The measurements are usually quantitative. ▪ The correlation between the test measurements and material properties are direct. 	<ul style="list-style-type: none"> ▪ The tests are made directly on the object. A 100% test of the object is possible. In-service testing is possible. ▪ In many cases, more than one NDT technique can be applied to the same object, allowing for testing of many or all material properties of interest. ▪ The same non-destructive test can be carried out several times on the same object. ▪ Most of the tests are rapid and require little preparation.
Limitations	<ul style="list-style-type: none"> ▪ The tests are not conducted directly on the entire object, thus the correlation between the object sample and the object must be proven. ▪ A single test may yield limited information. ▪ In-service testing is not possible. ▪ Destructive tests usually take longer time than NDT. ▪ It may be impossible to measure over time. ▪ Test preparations is expensive. 	<ul style="list-style-type: none"> ▪ The measurements of NDT are indirect. Thus, the reliability of the test must be verified. ▪ The measurements are usually qualitative, but can in many cases be done quantitatively- ▪ NDT often require skilled personnel, as results may be hard to interoperate.

Table 2.1 demonstrates that there is a need for improvement of the non-destructive techniques (NDTs) as they can be difficult to use effectively, and the results are not always reliable. Since it is impossible to see through a solid element, researchers have come up with alternative ways to get the desired information using indirect processes. Various NDTs use different approaches to solve the problem such as the use of acoustic, electromagnetic, electrochemical, or thermal principles, which are all indirect

methods. The measured properties correlate to mechanical or geometric properties of the element being inspected once the data has been mathematically transformed or interpolated in order to calculate the desired information. Improper use or interpretation of NDT results to find defects or measure components can lead to bad conclusions due to their indirect nature which is why a strong training is recommended in most cases. Every type of NDT uses different approaches and have their strength and weaknesses. For this reason, they usually excel at finding certain types of defects. When used for the purpose they were designed for, they provide information that would be difficult or impossible to get using more traditional approach.

Destructive techniques usually require taking a sample and testing it in various ways in a lab to find its characteristics. It can take a long time to get the results and the number of samples is limited. Both destructive and non-destructive methods have their advantages and limitations, but while destructive techniques cannot be made faster, NDT continue to be developed and improved constantly.

2.3.2 Non-destructive Techniques

There are many NDTs that have been developed in recent years. These tools were designed to fill various types of needs from the detection of physical or chemical deteriorations or to precisely measure components or characteristics of the various elements. There are currently no techniques that can solve all of those problems at the same time. For reinforced concrete testing, an interesting method is Impact Echo (IE). IE is a technique that is only able to determine the location and dimensions of physical deterioration containing air using stress waves and sensors. In the detection of physical deteriorations, many other NDT exist and can be compared to IE to better understand each of their advantages and disadvantages. Ground Penetrating Radar (GPR), Ultrasonic Pulse Velocity (UPV), Ultrasonic pulse echo (UPE) and Schmidt hammer are other non-destructive techniques that can be used to obtain information on geometric features and defects in reinforced concrete to compare with Impact Echo technology.

2.3.3 Ground Penetrating Radar (GPR)

GPR is a very quick method that is based around the use of electromagnetic waves to detect objects or defects in the tested element. Its main use is that it can draw contour maps of the interior of the tested element at various depths and is particularly useful for detecting the location of steel reinforcement. GPR is able to detect objects embedded in the element easily which can be very useful for a large variety of applications. In civil engineering, the main use is to find solid objects inside concrete and to some degree it can be used to locate various types of defects. The electromagnetic waves are generated by an antenna installed on the device. The antenna can be set at different frequencies depending on the precision

required for the analysis. The waves travel through the element and either get reflected on solid objects such as steel reinforcing bars and come back to the device or they continue diminishing and eventually disappear. Since all materials have different electrical properties, the electromagnetic wave gets highly affected by the interface of two materials. The wave either loses some velocity due to the change or will be completely reflected due to the large difference between the two materials. That is the case for steel reinforcement which are much more conductive than concrete. Figure 2.6 demonstrates the principle. (SHRP2, 2013)

Figure 2.6 GPR Principle (SHRP2, 2013)

In Figure 2.6, the device is moving from the left to the right side while sending electromagnetic waves through the material. The bottom section shows the result from the test. As observed in the figure, at the location of a reinforcing bar the wave is dispersed causing a parabolic shadow. From scanning a single line on an element, GPR can see both the top and bottom row of steel reinforcement with reasonable accuracy. Considering a bridge deck for example, GPR scanning can be repeated for many straight lines in two orthogonal directions to fill a grid. Once data from both directions are gathered, a contour map can be created which allows to decently describe the interior of the element (SHRP2, 2013).

2.3.4 Ultrasonic Pulse Velocity (UPV)

Ultrasonic pulse velocity method (UPV) is the simplest form of stress waves-based NDT. It is a well-recognized tool used for non-destructive assessment of structures. Even though it has its limitations as a qualitative method, it still provides an effective assessment of the conditions of the tested element. The

pulse or stress wave's velocity (V_p) is directly related to Young's modulus (E), density (ρ) and Poisson's ratio (ν) (Equation 2.1):

$$V_p = \sqrt{\frac{E(1-\nu)}{\rho(1+\nu)(1-2\nu)}} \quad (2.1)$$

Figure 2.7 Schematic of typical relationship between pulse velocity and compressive strength (ACI 228.1R-19)

Based on that relation, the concrete compressive strength can be estimated based on the wave velocity (Figure 2.7). However, it remains a very approximate method that can differentiate between areas of sound and damaged concrete, but should not be used as an accurate method to quantify the extent of deterioration.

The method uses two transducers, one that sends a pulse and a second one that captures it (Figure 2.8). By knowing accurately the distance (L) between the transducers and measuring the time it takes for the wave to travel (T), the wave velocity can be calculated (V), as given by Equation 2.2:

$$V = L/T \quad (2.2)$$

Figure 2.8 Schematic of Pulse Velocity Apparatus (ASTM C 597, 2016)

The simplicity of the method allows even inexperienced inspectors to understand and perform the test easily. Interfaces between different materials or the presence of a solid defect will cause the wave to experience a change in velocity. If the velocity changes, the time changes as well. However, not all types of deterioration cause a direct change in the velocity of the wave as some types will simply not let the pulse pass or will reflect it back. Even in that case, the obstruction will require the wave to go around using the shortest path to reach the other side and therefore increase the travel distance. Increasing the travel distance with a constant velocity will also result in a change in the travel time (Figure 2.9).

Figure 2.9 Wave going under a crack (ASTM C 597, 2016)

Both situations result in a reduced calculated velocity using Equation 2.2. There are some limitations to the method. For example, cracks which are parallel to the direction of the wave are difficult to see using UPV because they do not interfere with the wave propagation (ASTM C 597, 2016).

The test is usually performed with the sensors placed on opposite sides of the tested element as seen in Figure 2.9. If one surface is inaccessible, placing the sensors side by side on the same surface is also an option but induces some errors in the readings. Furthermore, the test procedure has been standardized as “Standard Test Method for Pulse Velocity through Concrete” (ASTM C 597, 2016).

The calculated velocity is compared to a theoretical base to determine the quality of the concrete. For a measured velocity of 4500 m/s or more, the tested concrete is considered in excellent condition. From there, it can only decrease due to obstacles in the wave's way or changes in the quality of the material. This method is qualitative as it only provides an idea of the quality of the concrete. Defects such as delamination, voids or cracks reduce the traveling velocity of the wave and require more time to reach the second transducer (Table 2.2).

Table 2.2 Relation between pulse velocity and quality of concrete (FPrimeC Solutions, 2017)

PULSE VELOCITY	CONCRETE QUALITY
>4.0 km/s	Very good to excellent
3.5 – 4.0 km/s	Good to very good
3.0 – 3.5 km/s	Satisfactory
<3.0 km/s	Poor

The limitations of the method are mostly due to its qualitative nature. It cannot precisely find the location of a defect, but by doing a full assessment of a structural element, the presence of defects will demonstrate a significant variation in the pulse velocity. Another limitation would be a high number of reinforcing bars. The pulse travels much faster in steel than it does in concrete (FPrimeC Solutions, 2017). This can induce some errors in heavily reinforced elements as the presence of steel will compensate to some extent for reductions in velocity caused by defects. When interpreting the results, the final calculated velocity might indicate a good quality concrete even in the presence of defects.

2.3.5 Ultrasonic Pulse Echo (UPE)

UPE is a method based around sound waves. Its main use is to detect objects, interfaces and other anomalies inside an element. It was not primarily designed for civil engineering applications, but its use as an NDT became possible only recently. The reason is that sound waves disperse easily and do not travel uniformly in concrete due to the heterogeneous nature of the material. In addition, the original transducers used for this method required a strong connection to the surface using grease or wax. The result was high dispersion results for an inconvenient hard-to-use method. To solve the issue, different types of transducers were tested with a center frequency between 50 and 200 kHz. Center frequency refers to the middle point between the cutoff frequencies for a transducer and essentially represents the frequency for which the transducer performs the best. The new-generation transducers could be dry coupled, removing the need to grease it for every reading. This effectively solved both issues associated

with the use of UPE for concrete structures and made it a viable option to use when investigating physical deterioration (SHRP2, 2013).

The physical principle used by this method is to measure the time it takes for an acoustic wave to travel through a material and get reflected at an interface of the tested element to come back to the device. Other materials, air, or a severe change of density of concrete all represent a change of the acoustic impedance. That means that a sound concrete element will have a much higher wave velocity than an element in poor condition containing significant deterioration (SHRP2, 2013).

2.3.6 Schmidt hammer

The Schmidt hammer, also called rebound hammer, is an easy-to-use method that allows the user to estimate the compressive strength of the concrete due to the correlation with its surface hardness. This method is mostly relevant as an additional tool for inspections. Most NDT are calibrated according to the material's compressive strength which is not always known. This is exactly what this tool provides. It measures a single rebound number "R" that can be used with empirical equations to estimate the compressive strength of concrete. The test is quick and allows the user to generate a map of the quality and uniformity of the material.

The methodology of this method is presented in Figure 2.10 below. For the first step, the device is applied perpendicular to the surface as shown in Figure 2.10a. It is pushed against the surface until the plunger is completely inside the device (Figure 2.10b). At that moment, the latch unlocks releasing the hammer pushed by the compression spring (Figure 2.10c). Once the hammer hits the surface, it rebounds up to a certain height (Figure 2.10d). That is the rebound number "R".

Figure 2.10 Schematic illustrating operation of rebound hammer (ACI 228.1R-19)

The process is designed to measure the kinetic energy of the hammer. The potential energy from the spring is transformed into pure kinetic energy once the latch is released. The hammer will hit the surface and lose some energy there. It goes back up until the kinetic energy reaches 0. The absorbed energy is related to both the compressive strength and the stiffness of the concrete. A low-strength and low-stiffness material would result in more energy absorbed during the impact. This also implies that concrete of similar strength, but different stiffness would result in a different rebound number (ACI 228.1R-19).

There are a few limitations for the method. One of them is the presence of a strong aggregate in the immediate vicinity of the test hammer. If the test is conducted right above a high strength aggregate or a piece of steel reinforcement close to the surface, the rebound number would be unusually high. The opposite is also true. If the plunger is above a large void or a soft aggregate, it would result in a lower number (ACI 228.1R-19).

2.4 IMPACT-ECHO TESTING

For this project, impact-echo technology was studied. Impact Echo is a non-destructive method based on stress waves that is primarily used to detect defects such as cracks, voids, delamination and honeycombing in concrete or masonry elements as described by Sansalone and Streett (1996). It can be used for a large selection of concrete elements and some other types of materials. The technology uses

stress waves; an impact caused by a hammer or a steel ball is performed on a surface inducing stress waves that propagate through the concrete.

The wave is intercepted, deviated, or reflected by discontinuities in the material or by reaching the other end of the tested element. The waves are captured using a transducer which is installed close to the stress source to register. The reflections of the wave between the surfaces and defects cause a resonance state which lead to a large number of waves travelling back and forth between the source of impact and closest layer of air. The transducer receives a lot of information back from the stress wave, but a frequency analysis is used as a means to understand the information coming from the test. The transducer receives a time-domain signal that can be changed via a Fourier transformation to a frequency-domain function. Sansalone and Streett (1996) created a simplified flow chart to demonstrate the process as shown in Figure 2.11.

Figure 2.11 Simplified diagram of impact-echo method (Sansalone & Streett, 1996)

The frequency spectrum shows the characteristic frequencies at which the test element vibrates. It can be used to identify the depth at which the waves were reflected. The waves are reflected when they reach air which is mostly found in defects (e.g. voids or delamination) or at the other side of the structural element. For example, in a sound concrete slab, the wave would travel through the material and be fully reflected when reaching the bottom. Because the wave is fully reflected, it would appear in the graph as a single peak of high amplitude (Figure 2.12).

Figure 2.12 Fully reflected stress wave on sound concrete

The peak is caused by the resonance condition and using the graph and knowledge of the speed at which the wave travels in the material, one can determine the depth at which the wave was reflected. The typical velocities for stress waves in sound concrete are approximately 4000 m/s. It is however, affected by the quality and presence of defects. A very good quality concrete can be as high as 4500 m/s while a poor quality one can drop to around 2000-3000 m/s (ASTM C 597). In sound concrete, the depth calculated from the frequency analysis would be equal to the real depth of the element. In case of a defect, it would result in a different measurement being either higher or lower than the expected value (Azari and Nazarian 2015).

It is important to note that this method also has limitations and it is not suitable for every situation. Impact-echo generally performs well for its intended purpose of detecting physical damage that has already occurred and contains air, but it does not detect every type of defect. For example, chemical reactions such as AAR or corrosion cannot directly be identified using the impact-echo technique until it results in damage, such as cracking or delamination. It should therefore be used in conjunction with other non-destructive techniques to fill in where the technique is weaker. Another limitation is that a surface of the tested element must be physically accessible to install and perform the testing.

Taking measurements and transforming the data to a frequency domain is rather straightforward, but the difficulty of the method comes with the interpretation of the data. In sound concrete, the task is simple as it shows a single peak on the frequency graph. With that information, it is possible to find an expected wave frequency as well as the important frequency range to avoid observations indicating false results (Mi, et al. 2005).

This is not always the case as the technique can be affected by many things such as complex boundary conditions or interference by defects or other internal elements such as reinforcement or conduits. The

boundary conditions may cause interferences in the readings because the waves emitted can be reflected in unpredictable ways (Figure 2.13). It may result in a single peak similar to sound concrete but out of the expected range, or result in a completely unusable frequency spectrum without a clear peak. It is better to locate and avoid the problematic boundary.

Figure 2.13 IE testing near a boundary

If a boundary is unavoidable, the interpretation of the results becomes even more complex. For example, the interface of a concrete deck on steel girder represents a complex boundary that can affect the result. The authors (Mi, et al. 2005) have found a method to find preliminary information for the case of a composite bridge deck. Through a finite element model, the author found the priori information required to reliably test the structure using Impact Echo. The method involves three sets of tests. The first test is to find the horizontal response of the vertical impact. The second is to find the vertical response of the vertical impact, and lastly the horizontal response for the horizontal impact. Once those details are known, a reliable prediction of the frequency can be obtained, and the complex boundary can be analyzed. Lastly, interference from other types of materials is also a possibility. Reinforced concrete is a heterogeneous material, therefore the stress wave does not travel evenly through it. This may result in various degrees of noise in the results as it causes a refraction of the waves.

2.4.1 Stress waves

This section describes the composition and role of the stress wave in the impact-echo technique. As mentioned earlier, impact-echo uses a hammer or stress ball to create a stress wave that propagates through the concrete element. The wave can be described with three components: dilatational wave (P), distortional wave (S), and Rayleigh wave (R)(Carino, 2015).

When a stress wave is produced through an impact, a spherical wave propagates through the material and on the surface. The R wave is the surface wave component where most of the energy from the impact is dissipated as a high amplitude surface wave. It can be compared to the visible surface waves created

when an object is thrown into water. In the Impact Echo readings, it appears as a single high amplitude wave as seen in Figure 2.14.

Figure 2.14 Perpendicular Wave Components (Carino, 2015)

The P wave and S waves are two components of the same spherical wave that propagates from the impact. The P wave propagates in a direction perpendicular to the surface and is the fastest of the three. The wave can be related to a normal stress. The P wave causes the particles to vibrate in the same direction as the wave causing a linear propagation of the wave starting from the impact location and travelling outward. It is visible in Figure 2.14 since the sensor used for Impact Echo testing captures displacements perpendicular to the surface. The S wave can be associated to a shear stress. It causes the particles to vibrate in a perpendicular direction to the original direction of the wave which makes this wave complex to analyze. It is not visible in Figure 2.14 since it would require three dimensional displacements to be captured. Table 2.3 describes how the wave speed can be calculated and how much impact energy is utilized by each wave component (Carino, 2015).

Table 2.3 Three types of waves created by point impact on a solid (Carino, 2015)

Wave type	Particle motion	Wave speed*	Relative wave speed for $\nu=0.2$	Energy content, % [5]
P-wave	Parallel to propagation direction	$C_p = \sqrt{\frac{E(1-\nu)}{\rho(1+\nu)(1-2\nu)}}$	1	7
S-wave	Perpendicular to propagation direction	$C_s = \sqrt{\frac{E}{\rho 2(1+\nu)}}$	0.61	36
R-wave	Retrograde elliptical	$C_r = C_s \frac{0.87+1.12\nu}{1+\nu}$	0.56	67

* E = Young's modulus of elasticity; ν = Poisson's ratio; ρ = density.

In the case of Impact Echo, the P-wave is the component that is used in the analysis. Since it causes the wave to travel in a linear manner, it is easier to understand and describe the behaviour of the wave under

the surface. The wave also travels much faster than the others which makes it easier to identify on a frequency spectrum (Carino, 2015).

2.4.2 Reflection

Reflection of the P-wave is based on the difference of acoustic impedance between materials as well as the angle of approach. The coefficient of reflection R can be calculated using Equation 2.3:

$$R = \frac{Z_1 - Z_2}{Z_1 + Z_2} \quad (2.3)$$

Where Z is the acoustic impedance of a material. Table 2.4 shows the value of acoustic impedance for the P-wave in common materials. The second column of the table shows the reflection coefficient R when the P-wave travels through concrete and encounters that material (Carino, 2015).

Table 2.4 Acoustic impedance for various materials and reflection coefficient at interface for P-wave in concrete (Carino, 2015)

Material	Specific acoustic impedance, kg/(m ² s)	Reflection coefficient at interface
Air	412	-1.00
Water	1.48 x 10 ⁶	-0.65 to -0.75
Soil	0.3 to 4 x 10 ⁶	-0.3 to -0.9
Concrete	7 to 10 x 10 ⁶	
Steel	47 x 10 ⁶	0.65 to 0.75

As seen in Table 2.4, the wave experiences a complete reflection when it encounters air. That is the main reason why it is useful to use stress waves to see inside a concrete element. This makes Impact Echo very strong at finding defects containing air such as delamination, cracks and voids. It may also be observed from Table 2.4 that if defects are filled with water instead of air, the effectiveness of Impact Echo is reduced. The last interesting thing to note is the sign of the reflection coefficient; when a wave travels through concrete and encounter airs, the initial acoustic impedance is so much larger that the wave is fully reflected with an opposite sign. The original compressive wave becomes a tensile wave when reaching the surface, pulling the surface downward. In the opposite case, when the wave encounters steel, the second material has a larger impedance than concrete resulting in a positive coefficient. In that case, the initial wave remains compressive when encountering a steel layer meaning that it does not cause a reflection. Encountering steel should simply lead to a minor change of the wave velocity (Carino, 2015).

2.4.3 Refraction

The presence of other materials such as reinforcing steel, having a different acoustic impedance than concrete, can lead to a refraction of the stress wave and a change in the frequency spectrum resulting in multiple peaks or various degrees of noise. In theory, when a wave is created it will travel from the impact point in a perfect circular shape through the concrete. In reality, imperfections in the concrete cause refractions of the wave such that the circular shape is modified based on changes of material or due to the shape of the structure. Essentially as the wave propagates, points on the circle will either lose velocity or deviate from their original linear direction causing the original perfect circular propagation to become deformed in an unpredictable way. Very thick concrete can also lead to similar types of noise problems since the wave has to travel a longer distance. Longer distances mean it encounters more material particles that can potentially cause refraction.

However, this usually does not impede the use of Impact Echo since their effect is relatively small and the wave will eventually reach a defect or the other end of the tested element (Azari and Nazarian 2015). A bad quality concrete or a high amount of reinforcement would create more noise and make it more difficult to interpret the results, but the technique would generally still be usable and reliable.

2.4.4 Waveform spectrum to frequency spectrum

The raw data generated by an impact-echo sensor is the voltage measured by the accelerometer as a function of time following the initial impact. This data is usually transformed using a fast Fourier transform from a time-domain signal to a frequency-domain function to create an amplitude-frequency spectrum of the return signal which is then used for further analysis. In sound concrete, the thickness mode of vibration governs and a single peak in the graph shows that the highest magnitude occurs at a certain frequency called the thickness frequency (f). In this case, the amplitude value of the wave is mostly irrelevant, it is the frequency that is useful for the analysis. For this reason, researchers often normalize the amplitude to compare or compile curves having different peak amplitudes.

If the compression wave velocity (V_p) is known or estimated, the frequency can be easily transformed into a thickness measurement (h) using Equation 2.4:

$$h = \frac{\beta(V_p)}{2f} \quad (2.4)$$

Where β relates the plate wave speed to the P-wave speed. It can be taken as approximately equal to 0.96 for reinforced concrete structures based on experiments (Carino, 2015). It is also the suggested value provided in the ASTM C1383-15 standard for Impact Echo.

The numerator term βV_p is often described as the plate velocity (C_p) in literature.

V_p can be estimated using the physical and mechanical properties of the concrete, namely the elastic modulus (E), the Poisson's ratio (ν) and the density of the material (ρ) (Azari and Nazarian 2015) according to Equation 2.5:

$$V_p = \sqrt{\frac{E(1-\nu)}{\rho(1+\nu)(1-2\nu)}} \quad (2.5)$$

Equations 2.4 and 2.5 are dependent on the characteristics of the concrete as the speed at which the waves travel is affected by the density of the material. It is therefore difficult to obtain an accurate reading without knowing the precise density of the concrete. As an alternative approach, Equation 2.4 can easily be calibrated for a specific concrete element as long as there is a sound section with known thickness so that the reference wave velocity, V_p , may be determined.

2.4.5 Vibration modes

There is more than one mode of vibration for a reinforced concrete plate. The more useful mode is the thickness vibration mode that allows to easily calculate the plate's depth. In some cases, such as thin sections, presence of shallow delamination, or close to a boundary, the thickness vibration mode does not govern. In those cases, the shallower depth may tend to cause a flexural mode of vibration instead (Figure 2.15) while the boundaries can cause the wave to reflect in multiple vibration modes collectively named cross-sectional modes (Carino, 2015). In those cases, Equation 2.4 is not valid to determine the exact locations of defects or the thickness.

Figure 2.15 Thickness and Flexural Vibration Modes (Carino, 2015)

The thickness mode of vibrations shows a perfect symmetry of the top and bottom of the test element. This mode of vibration allows to locate the thickness using Equation 2.4. The flexural mode shows an anti-symmetrical shape where the top and bottom surfaces follows the same pattern. This will result in a lower frequency peak.

2.4.6 Impactor

Understanding the different components of the stress wave is important when determining the optimal distance between the impact location and the transducer receiving the results. The impact should be produced at a distance from the transducer between 20% to 50% of the depth of the shallowest interface. If the impact is made too far from the receiver, the receiver will receive reflections from the horizontal component, S-wave, of the impact wave in addition to the vertical one. This will lead to errors when interpreting the results (Carino, 2015).

Furthermore, the selection of a suitable impactor is also important. Carino, 2015, suggests that to ensure the thickness mode frequency is significant in an Impact Echo test, the maximum usable frequency must be greater than the thickness frequency. The maximum usable frequency is determined by the contact time of the impactor. The contact times for impactors of different sizes have been estimated in literature and are provided in Table 2.5. However, it is important to note that in real applications, the contact time of the ball is expected to be slightly longer as it is subjective to the operator and affected by inelastic behaviour during the impact.

Table 2.5 Approximate relationships between ball diameter and minimum measurable depth for $C_p=4000$ m/s (Carino, 2015)

Ball Diameter, mm	Approximate Contact Time, μs	Maximum Useable Frequency, kHz	Minimum Measurable Depth, mm
5	22	47	43
6	26	39	52
7	30	33	60
8	34	29	69
9	39	26	77
10	43	23	86
12	52	19	103
15	65	16	129
20	86	12	172

Table 2.5 provides an estimate of the contact time for different sizes of impactors as well as their maximum usable frequency. The last column represents the resulting maximum thickness for a plate wave speed (C_p) of 4000 m/s. For example, if a reflection is expected at a depth of 120 mm within the element, the optimal hammer size to excite a thickness vibration mode would be one that has a maximum usable frequency greater than the thickness frequency. In this case, 120 mm depth with a plate velocity of 4000 m/s results in an expected frequency of 16.7 kHz. This means that the optimal ball size would be 12 mm or less (Carino, 2015).

Using smaller impactors does induce some problems. The issue with short contact times is that the depth of penetration of the wave is limited, the results often have multiple peaks making the interpretations of the data problematic, and the R-wave effect becomes much more significant than the P-wave. The suggested methodology is to use the largest impactor based on the thickness of the element, and then try smaller ones to try locating potential reflections at lower depth. The R-wave can be removed with post-processing of the data as it always is the first displacement the transducer receives as shown in Figure 2.14 .

2.4.7 Recent studies

Impact Echo is a method that was mostly developed between 1987 and 1997. The work done by Mary J. Sansalonne was the foundation of the method and is still the suggested approach. The work on IE method was standardized in the ASTM C1383-15. Following the standard, research on Impact Echo became less common. In recent years the research has mostly focused on the limitations of the method and the difficulty associated with the interpretation of the results.

The limitations of the methods come from the usability. The method was proven accurate multiple times in the first development, but it remained difficult to use with a lot of inconvenience. One issue is the surface contact. The method analyzes the perpendicular vibrations occurring in the tested element. For this reason, it requires near perfect perpendicular placement of the accelerometer on the surface and a good contact. This can become difficult to achieve in some cases where the surface is not smooth. Shin, Popovic and Oh (2012) used a different approach with an air-coupled sensor. The microphone used increased the testing speed by removing possible issues tied to the surface contact. The test was done on concrete slabs with a variety of delamination defects. The conclusion was that the use of air-coupled sensors is faster at the cost of sensitivity. It was accurate at finding shallow large flaws but was unable to locate deep or smaller area delamination (Shin, Popovics and Oh 2012).

Another issue with IE is the difficulty of the interpretation. A strong knowledge of the physics involved and the possible outcome to expect are required to properly understand the results. Because of that, the method is limited to physical defects such as voids or delamination in plate-like concrete elements. Doing a full condition assessment with identification of defect types and severity presents many challenges. For this reason, researchers in recent years worked to find alternatives to the Fast-Fourier Transform in order to improve the data analysis and the interpretation of the results. One approach, tested by Zhang, Yan and Cui (2016) is to use machine learning technologies to handle the problem. This could lead to more accuracy of the results and a larger number of situations where IE can be used. The specimen tested was a large block of concrete with large defects installed. The surface was also perfectly polished to ensure a good contact and optimize the quality of the reading. The tests done by the authors were positive on the approach, but it still needs further testing to see the application on other types of defects and geometries (Zhang, Yan and Cui 2016).

2.5 DETECTING PHYSICAL DETERIORATION

This section covers the main types of physical deterioration that can be detected and characterized using Impact Echo technology. The main issues related to them will be described to better understand the importance of finding them.

2.5.1 Vertical Cracks Perpendicular to the Surface

Vertical cracks are a frequent type of deterioration that occur in most concrete structures during their service life. Many strategies are used to avoid or control them (e.g. making saw cuts in the concrete at strategic locations to force the crack to occur there). The main problems caused by cracks are a reduction of the effective cross-section that resists the loads, a reduction in stiffness, and facilitating moisture ingress to the core of the concrete element. In some cases, this can lead to significant durability issues. Countries like Canada with a cold winter must use deicing salt for safety purposes. Those chemicals can penetrate the concrete, cause damage to the concrete, and accelerate corrosion of the reinforcement. Water entering the material is also an issue when it freezes and expands. In Ontario, the severity of a crack is typically assessed based on its width. The four degrees of severity for cracks are hairline (0.1 mm), narrow (0.1-0.3 mm), medium (0.3-1.0 mm) and wide (>1.0 mm) (OSIM, 2008).

Lin & Su (1996) proposed a method to use Impact Echo to estimate the depth of a crack in reinforced concrete structures. The method was later used again by Sun and Huand (2018) to achieve similar results. For this method, the Impact Echo device must be equipped with two sensors that will be carefully installed

on each side of the measured crack. A stress wave will then be generated by an impactor or steel ball. The shortest path the stress wave generated at the point of impact can follow to reach the other side is passing through the tip of the crack (Figure 2.16). Based on that reasoning, Equation 2.6 can be used to calculate crack depth:

$$D = \sqrt{\left[\frac{(C_P \times \Delta t)^2 + H_1^2 - H_2^2}{2 \times C_P \times \Delta t} \right]} - H_1^2 \quad (2.6) \text{ (Lin and Su 1997)}$$

Where: H_1 is the distance between the impact and the crack

H_2 is the distance between the second sensor and the crack

C_P is the travel velocity of the P-wave

Δt is the travel time between the impact and the sensor 2

Figure 2.16 Schematic of testing configuration for estimating the depth of the crack (Sun, et al. 2018)

The typical Impact Echo result in this case would appear similar to that shown in Figure 2.17 (Sun, et al. 2018). This method utilizes the untransformed readings of the sensor in the time domain. The first and second curves represent the first and second sensors respectively.

Figure 2.17 Typical recorded waveforms showing P-wave arrivals (Sun, et al. 2018)

Based on the results shown in Figure 2.17, Δt consist of two portions, an original time where the wave travels to the first sensor as well as the difference between the start of the reading of the first sensor and the second.

$$\Delta t = (t_1 - t_0) + (t_2 - t_1) \quad (2.7) \text{ (Lin and Su 1997)}$$

The first part from t_0 to t_1 can be described by the distance the wave has to travel divided by its velocity.

$$t_1 - t_0 = \frac{H_3}{c_p} \quad (2.8) \text{ (Lin and Su 1997)}$$

Combining Equation 2.7 into Equation 2.6 results in Equation 2.8:

$$\Delta t = \frac{H_3}{c_p} + (t_2 - t_1) \quad (2.9) \text{ (Lin and Su 1997)}$$

The last unknown required is the travel velocity of the P-wave. To calculate this value, a sound section of concrete is required. Placing the two sensors side by side at a known distance from each other and creating an impact in line with them allows for a simple calculation of the velocity using Equation 2.9. Figure 2.18 demonstrates the method (Sun, et al. 2018)

$$c_p = \frac{L}{t'_2 - t'_1} \quad (2.10) \text{ (Lin and Su 1997)}$$

Figure 2.18 Placement sketch (Lin & Su, 1997)

2.5.2 Delamination and Voids

Delamination was one of the primary targets for which Impact Echo was first developed as seen in early work on the method by M.J. Sansalone (1996). It takes the form of a layer of air inside the concrete and is essentially a horizontal crack parallel to the concrete surface (Figure 2.19). Delamination is often not observable visibly from the surface until the cover concrete completely detaches from the substrate.

Figure 2.19 Example of Delamination

Delamination often initiates from the corrosion of the steel reinforcement. Placing the reinforcing bars side by side can create a plane of weakness when corrosion-induced expansion occurs, allowing cracks to propagate between the bars. The other common cause of delamination is the debonding of two layers of concrete placed at different times, such as when a repair is performed on an older structure. Delamination can often be located using chain dragging or hammer sounding due to the hollow sound that is created when the concrete is struck with a metallic object (OSIM, 2008). Figure 2.20 shows a concrete core that was taken through a delamination layer.

Figure 2.20 Concrete core containing delamination (SHRP2, 2013)

A delamination may be either deep or shallow (Figure 2.21). In the presence of deep delamination, the peak frequency will tend to shift toward higher values than that corresponding to the thickness of the tested element due to partial wave reflection. However, in the case of shallow delamination, the peak frequency will usually be lower than that corresponding to the full thickness, meaning that a large portion of the energy from the wave is reflected by the defect. Shin, Popovics and Oh (2012) described that for the case of a sound concrete specimen or the presence of deep delamination, the studied element follows a “thick plate” case of vibration and Equation 2.4 can be used. However, the situation is different if there is a near surface defect. A “thin plate” type of vibration would be dominant which has a flexural mode of vibration and cannot be accurately described by Equation 2.4 due to their lower frequency. What they found is that the “thin plate” case generates flexural mode frequencies which are affected by depth, area, and defect boundary conditions. It can be assumed the “thin plate” case is applicable when the lateral length of a delamination to thickness ratio exceeds 8.

The two Fourier spectra in Figure 2.21 show the result for a deep and a shallow delamination of the same dimensions (20 cm by 20 cm).

Figure 2.21 Shallow (a) and deep (b) delamination (Shin, Popovics, & Oh, 2012)

The results found in the study demonstrate that in most cases for shallow defects, the presence of delamination is observed as a dominant frequency component that is lower than the frequency representing the full thickness of the slab. For the case of deep delamination however, it is the opposite and the graph will show a frequency higher than the thickness of the slab. The slab being 250 mm thick, the frequency can be calculated as 7.81 kHz using Equation 2.4 with a P-wave velocity of 4100 m/s and a β of 0.96 (Shin, Popovics and Oh 2012).

Even though there are multiple different cases that affect the frequency analysis, all vibration cases are affected by the presence of a defect containing air and can be characterized based on the change of the measured frequency (Shin, Popovics and Oh 2012).

The typical form the frequency spectrum can take when there is the presence of delamination were described in the (SHRP2, 2013) and shown in Figure 2.22.

Figure 2.22 Grade of various degree of deck delamination (SHRP2, 2013)

In Figure 2.22, the first test (Figure 2.22a) shows the case of sound concrete. It has a single peak caused by thickness vibration that can easily be used to determine the thickness. The second test (Figure 2.22b) shows a discontinuous layer of delamination. In that case, some of waves can go through the defect while some are reflected. It results in two peaks representing the two slabs thickness the device is measuring which are the full thickness and the thickness above the delamination. The third test (Figure 2.22c) represents a delamination which is harder to identify and characterize due to the presence of multiple peaks. The flaw causes strong reflections of the wave which result in flexural vibration peaks at lower frequencies. Then the thickness vibration from the bottom is slightly lower than expected due to loss of slab stiffness and diffraction of the waves. Finally, a third peak is caused by thickness vibration from the portion of the slab over the delamination. The last two peaks are useful to determine the depth of the flaw. The last one (Figure 2.22d) represents a shallow delamination. As mentioned before based on (Shin, Popovics and Oh 2012), shallow delamination creates a “thin plate” type of vibration which presents a large peak at a lower frequency. Consequently, if Equation 2.4 is used to characterize the defect, it would result in a thickness much larger than the full thickness of the studied element.

Another type of defect that is similar to delamination are voids. For concrete, they may generally be detected in a similar manner as delamination with slightly different degree of reflection based on their size. Even though the causes of the defects are different, they are physically similar as they both consist

of layers of air inside the concrete element which will reflect the stress wave in the same way. The principal difference comes from their shape. Delamination usually occurs over a thin horizontal plane while voids are more three-dimensional. Their size may provides an easier reflection of the wave but can also make them harder to locate on the grid. Impact Echo takes a reading directly under the sensor, therefore if the grid is not dense enough, it is easier to miss a void with small planar dimension but large thickness. Generally, large voids would cause a stronger reflection of the wave causing mainly flexural vibration while the smaller ones would be the opposite. The small surface to reflect the wave would make it difficult to locate and would likely only cause a small variation in the thickness frequency.

2.6 SUMMARY

In this chapter, the literature was reviewed to better understand the various deterioration mechanisms and their resulting defects as well as relevant non-destructive techniques used to detect them. Reinforced concrete structures deteriorate over time; physical signs of deterioration typically include cracks, voids, and delamination. Those defects can lead to various costly problems and potentially dangerous accidents. Regular inspections of the structures by a qualified inspector are necessary to ensure a strong structural assessment is done. Multiple methodologies were developed over the years to improve the assessment. Many of them are designed around the use of non-destructive techniques (NDT) intended to locate the defects within the material through indirect measurements using different forms of waves that interact with internal irregularities within the concrete member.

Several non-destructive techniques were presented and described in this chapter. Specifically, Ground Penetrating Radar, Ultrasonic Pulse Echo, Ultrasonic pulse velocity, Schmidt hammer and Impact Echo methods were explored in order to better understand their ability to locate and describe the various types of physical deterioration mechanisms often occurring in reinforced concrete. The Impact Echo method was created and validated about 30 years ago and further development and testing occurred for the next decade. However, after 2000, very limited research has been conducted on this method and few technological advancements have been achieved. Its strength resides in its ability to locate air-containing defects and objects buried inside a reinforced concrete element. This makes it a good and reliable method for non-destructive testing if done and analyzed properly. On the other hand, the main disadvantages associated with the Impact Echo method relate to the usability of the method. Often, when poor results are obtained they can be attributed to either operator error or complex boundary conditions. The advanced method requires the user to have good understanding of the principles of the method like the

stress waves and their interactions with internal flaws (Carino, 2015). Furthermore, the analysis is performed using a frequency spectrum which is not directly available from the raw data. It requires the data to go through mathematical transformation from a time-based raw data to a frequency spectrum. Once that is achieved, the data interpretation can be done.

Since conventional Impact Echo systems require a qualified user and cumbersome equipment, the method did not get much attention compared to other non-destructive methods that fulfilled similar purposes such as Ultrasonic Pulse Echo and Ground Penetrating Radar. Those two are nowadays the go-to commercially available non-destructive devices used to investigate physical defects. For Impact Echo to become relevant again, usability problems should be addressed through integration with an easy-to-use interface, immediate results, and mapping. The method needs to be simplified for the user in order to limit the possible challenges with data interpretation. That may be possible with the advances in technology that occurred in the recent years such as the development of handheld tablets that are easy to use and transport while still having strong computing power and wireless capabilities.

This research aims to fill the current gap through the development and validation of a new-generation wireless Impact Echo device and software application designed for a handheld tablet intended to provide an intuitive user experience and accurate results. The results are compared to other commercial NDT systems to identify the strengths and weaknesses of the new technology, and recommendations are presented based on the findings.

3 EXPERIMENTAL PROGRAM

This chapter will present the methodology and experimental tests that were performed for this research project. To recall, the primary goal of this project was to develop a new generation wireless Impact Echo (IE) system and validate its performance with respect to detection of different types of defects in reinforced concrete slabs. The second goal was to compare its performance to other common NDT available to assess its suitability for various applications. The experimental program for this research project included three main phases: 1) development of a new wireless device and software for a handheld tablet; 2) validation of the accuracy and precision of the new Impact Echo device; and 3) comparison of the results with other selected NDT devices to compare their effectiveness at finding different types of defects.

Phase 1 followed an iterative process to refine and improve the functional design and ease-of-use of the device and software. The hardware prototypes and software programs were built by the project industry partner (FPrimeC Solutions), while the validation and comparison of the measurements were performed at the University of Ottawa. During the course of the project, multiple iterations of the wireless Impact Echo device and software were developed based on the experience in the lab. The development process will be explained later in this chapter. But for the purpose of this project, the results presented in Chapter 4 were all obtained with the most recent iteration of the device for continuity in the results.

To be able to draw clear conclusions for Phase 2 and 3, three concrete slabs were fabricated containing multiple artificial defects of known geometry. The test specimens were square slabs with dimensions of 2 m by 2 m by 0.3 m depth. The three types of artificial defects selected for this project were intended to simulate vertical cracks (perpendicular to the concrete surface), internal voids, and delamination (cracks parallel to the concrete surface). A change of thickness (i.e. geometric discontinuity) was also designed to be investigated and a hollow conduit was installed internally along the length of one of the slabs. The attenuating effect of steel reinforcement was also investigated. The selected parameters are representative of most of the typical uses of Impact Echo in the field.

The specimens were cast using a very typical concrete mix provided by a ready-mix supplier. The concrete mix was specified to reach 25 MPa at 28 days using 20 mm coarse aggregates, a water/cement ratio of 0.63, and a slump of 80 +/-30 mm. The detailed mix design was not provided by the supplier and is not considered to be significant for the outcomes of this project. Table 3.1 below summarizes the concrete mix used.

Table 3.1 Concrete mix for test specimens

Design strength	25 MPa
Aggregate	20 mm
water/cement ratio	0.63
Slump	80 +/- 30 mm

The actual average compressive strength tested at an age of 28 days was 29.6 MPa based on three cylinders crushed at the lab on that day. The tested cylinders were made and tested following the Canadian standard (CSA A23.2, 2019). The cylinders were 100 mm diameter by 200 mm height and filled in three equal layers. Each layer was compacted with 20 rod compactions as required by the procedure.

This chapter will first describe the test specimens that were fabricated for the purpose of this project as well as the types of artificial defects that were selected and how they were reproduced for the experiment. The second part of the chapter concerns the methodology of the experiment and details of the IE technology, including a description of how the various tests were conducted.

3.1 SLAB 1

3.1.1 Overview

The first slab, named Slab 1, was designed in order to test the functionality of the device at measuring slab thicknesses. This slab was also intended for calibration of the wave velocity, V_p , and therefore did not contain any internal defects. A geometric discontinuity was introduced in the form of a step-change in thickness from 200 mm to 300 mm as shown in Figure 3.1. One half of the slab contained reinforcing steel while the other half was unreinforced to evaluate the effect of reinforcing steel on the device readings. Figure 3.1 below represents the plan view of the first test specimen of this project.

Figure 3.1 Slab 1 Plan: Thickness Measurement

3.1.2 Grid pattern

For the purpose of this project, a large grid was drawn on the slabs (Figure 3.2). The grid size was selected to be uniform and sufficiently dense to locate the smaller objects buried in the concrete and to capture the influence of different boundary conditions. The high density is not necessarily required for Slab 1, since its design is relatively simple compared to the other two, but to keep the experiment uniform, a 20 by 20 grid was drawn with a line spacing of 100 mm in both directions.

Figure 3.2 Slab 1 Casting Result and Grid

The first line of the grid was located 50 mm from the edges of the slabs and comprises 19 squares in the horizontal direction by 19 squares in the vertical direction for a total of 400 intersection points to use as test locations for each slab. A simple coordinate system was developed, as shown in Figure 3.3, which will be used to refer to the points of interests in the slabs throughout the following chapters. The vertical columns are labelled A through T, and the horizontal rows are numbered 1 through 20.

Figure 3.3 Grid View of Slab 1

3.1.3 Geometric Discontinuity

A geometric discontinuity was introduced as a sudden change of thickness from 200 mm to 300 mm between lines J and K (Figure 3.3). The presence of this change creates a boundary at the bottom of the slab. Like the boundaries at the edges of the slab (vertical lines A and T, and horizontal lines 1 and 20), this discontinuity in the slab geometry is expected to affect the readings in unpredictable ways due to partial reflections and scattering of the stress wave propagating through the slab. It was expected that the IE readings taken from the measurement points near these boundaries may suffer from a loss of both accuracy and precision, although the extent of this effect is not clear from available literature.

3.1.4 Reinforcing Steel

As mentioned earlier, only a portion of the slab is reinforced corresponding to the region from lines 10 to 20 in Figure 3.3. The reason for this design is to investigate the effect of the reinforcing steel on the readings of the devices tested. Steel has different properties than concrete and therefore can affect how waves travel through the material.

For stress-wave based methods, the wave velocity can be estimated using the Young's modulus, density, and Poisson's ratio of the material (Equation 2.1). Steel is a much stiffer and more uniform material than concrete. The nominal characteristics for steel reinforcement could be taken as a Young's modulus (E) of 200 000 MPa, density of 7850 kg/m³, and Poisson's ratio (ν) of 0.28, which result in a predicted wave velocity of 5707 m/s. That velocity can be compared to the 3378 m/s for concrete based on the same approach with Young's modulus of 24 650 MPa (30 MPa concrete), density of 2400 kg/m³, and Poisson's ratio (ν) of 0.20 (Carino, 2015). This suggests that the wave velocity through steel is about 69% faster than concrete. Hence, it is possible that steel may have an impact on the results despite their small area. Having both reinforced and unreinforced sections of the slab was intended to enable a better understanding of the effect of steel bars on the capability of the equipment to estimate section thickness and detect defects in real field applications. This is the case for UPV, UPE, and Impact Echo since they are designed around the same wave mechanics. GPR, however, uses electromagnetic waves that propagate through the material and get affected by changes in the electric impedance. For this reason, it is expected that the presence of reinforcing steel will have a major impact on the GPR readings since the steel is much more conductive than concrete. The reinforcement closest to the device should clearly appear on the results.

The steel reinforcement's spacing and size was selected to achieve a reinforcement ratio $\rho_s = 0.3\%$ for both the top and bottom grids (Figure 3.4) and a concrete cover of 40 mm. This value was selected in order to achieve a steel ratio that is significant enough to potentially have an effect on readings while still

being a reasonable representation of common structural members such as walls and slabs. The Canadian standard CSA A23.3 (2019) states in Cl.7.8.1 that the minimum steel reinforcement ratio for a slab is 0.2%. However, the slab reinforcement was not designed for a specific service load in this case because the slabs were not intended to be actual load-carrying structural elements. Therefore 0.3% was selected as a reasonable reinforcement ratio that would be encountered in real applications.

Figure 3.4 Slab 1 Form and Reinforcement

Figure 3.4 shows the formwork of the Slab 01 before it was casted. It can be observed that some reinforcing bars were extended into the non-reinforced section along the slab perimeter. The reason for that is strictly a safety issue. The slabs were to be lifted using a crane in the laboratory to move them and dispose of them after completion of the testing program. Since concrete isn't very strong in tension, steel bars were added at the bottom of the section in regions away from the points of interest to prevent it from falling apart during lifting and transportation. Figure 3.5 shows a simplified plan for the location and size of the extra reinforcing bars used for this purpose. The bars have various sizes because they were picked from left-over pieces from other projects. They were installed at 50 mm from the bottom of the formwork to add some flexural strength to the slab and ensure it would not break into pieces if moved.

Figure 3.5 Slab 1 extra reinforcements

3.1.5 Thickness Measurement and Wave Velocity

The thickness measurement is one of the most important functions of the IE device. Since the slab geometry was known, the thickness measurements could be used to verify that the equipment functioned properly and to calibrate the results according to the specific properties of the concrete for later calculations. As seen in Figure 3.1, half of the slab included reinforcement steel while, in the other direction, half of the slab experienced a 100 mm depth difference. This resulted in four zones with either 200 mm or 300 mm thickness and with or without steel reinforcement as shown in Figure 3.6.

Figure 3.6 Slab 1 Grid View four Zones

The first thing that was tested on the slab was the ability of the device to determine the thickness of the specimen. The IE technique works by sending a stress wave through a material and using the frequency of reflected waves returning from the opposite boundary to determine the thickness of the element. The first goal for Slab 1 was to test the functionality of the device with simple thickness measurements and geometric discontinuities. The second purpose of Slab 1 was to obtain measurements from sound concrete sections without defects that could be used to calibrate Equation 3.1, which relates the peak frequency measured by the device (f) to a thickness value (h):

$$h = \frac{\beta(V_p)}{2f} \quad (3.1)$$

Where β , a shape factor is approximately equal to 0.96 for reinforced concrete based on standard ASTM C1383-15, and V_p is the wave velocity for a given concrete material. The wave velocity can be approximated using the concrete material properties including elastic modulus (E), Poisson's ratio (ν), and the density of the material (ρ) as given by Equation 3.2[3]:

$$V_p = \sqrt{\frac{E(1-\nu)}{\rho(1+\nu)(1-2\nu)}} \quad (3.2)$$

Since Equation 3.2 requires prior knowledge of the properties of the concrete tested, it would require taking samples of the specimen to properly characterize it through destructive testing in order to interpret the data found by IE measurements. This approach generally offers a good approximation of the velocity but is conducive only to cases where accurate measurements of these properties are available. For this project, it would result in a velocity of approximately 3 378 m/s based on the following simplifying assumptions:

$$E = 4500\sqrt{f'c} \quad (3.3) \text{ (CSA A23.3, 2019)}$$

$$\nu = 0.2 \quad (3.4) \text{ (Carino, 2015)}$$

$$\rho = 2400\text{kg/m}^3 \quad (3.5) \text{ (based on average for the type of material)}$$

In real field applications, concrete cylinders might not be available and extracting concrete cores negates some of the advantages of using non-destructive testing. Additionally, many industrial labs are not equipped to accurately measure the Young's modulus or Poisson's ratio of concrete cylinders. Therefore, an alternative is to calibrate Equation 3.1 using a sound concrete section of the tested element with known thickness. In Equation 3.1, the numerator is made of two variables that directly relate to the

properties of the concrete. Since the thickness is known in the case of Slab 1 and the concrete in this slab does not have any defects, the results from IE readings in that section can be used to calibrate the numerator of the equation. Since all three slabs were casted at the same moment using the same ready-mix concrete, it is appropriate to assume they will share the same material properties and calibrating the equation using Slab 01 results should be sufficiently accurate for all the slabs. This approach is a more reliable method to estimate the stress wave velocity since it is based on the properties of the actual test specimen rather than approximate simplifying assumptions.

3.2 SLAB 2

3.2.1 Overview

Slab 02 had a constant depth of 300 mm and contained two artificial vertical cracks of different depths, an empty post-tensioning conduit, and two artificial voids (Figure 3.7). For the same reasons as Slab 1, half of the slab was reinforced in order to investigate the potential effects of the steel bars on the various NDT measurements. The reinforcement ratio, bar size, and spacing were identical to Slab 1. Some steel reinforcing bars were also extended into the unreinforced portion of the slab between the points of interest to provide some safety to the slab when it needed to be moved.

Figure 3.7 Slab 2 Plan: Cracks, Conduit and Voids

The positioning of the defects in the formwork prior to casting is presented in Figure 3.8 (note that the orientation of the figure is rotated 90° with respect to Figure 3.7). The post-tensioning conduit had an outer diameter of 50 mm and was hollow. The duct was made of a steel coil encased in plastic to be slightly flexible. It was installed at the mid-depth of the slab along the centerline between Lines J and K (Figure 3.9). The voids were made using 150 mm wide cubes of foam and installed at the mid-depth of the slab near measurement points Q4 and Q17. The voids are one of the reasons that a high-density grid was required as their area is relatively small compared to other artificial defects tested. The high density allows at least 4 measurement points to be located directly above the defects and several others might be close enough to be affected by it. After casting and curing the concrete specimens, the artificial cracks were simulated by cutting the slab with a wet concrete saw, perpendicularly to the surface. Crack A had a depth of 100 mm and crack B had a depth of 200 mm, separated by a distance of 300 mm; these crack depths

correspond to $\frac{1}{3}$ and $\frac{2}{3}$ of the slab thickness, respectively. The cracks were created between lines C & D and lines F & G, as shown in Figure 3.9.

Figure 3.8 Slab 2 Form: Crack, Conduit and Voids

Figure 3.9 Slab 2 Grid View

For the reinforcements, half of the slab (from lines 10 to 20) was reinforced with 15M bars at 220 mm spacing representing a 0.3% steel ratio for both top and bottom layer. For the same safety reason as for the other slabs, extra steel reinforcing bars were added to ensure the slab would not break into pieces when lifted. The extra steel bars were placed near the edges at 50 mm from the bottom of the slab and

two bars were installed within the central portion of the slab far from the defects to ensure they would not affect the important readings. Figure 3.10 shows their approximate placement and size.

Figure 3.10 Slab 2 Extra Reinforcements

3.2.2 Vertical Cracks

Cracks are the most frequent signs of deterioration present in reinforced concrete. Reinforced concrete is designed in a way that it is expected to crack under service loads because concrete is weak in tension. Reinforcing bars are proportioned to transfer tension across the crack and maintain integrity of the structure. However even if structural cracks are permitted by design codes, not all cracks are acceptable as some large ones could provide an easy entry point for ingress of moisture and chemicals to reach the concrete core and the reinforcing steel. Some types of structures in Canada, such as bridge decks, which are regularly exposed to chlorides due to use of deicing salts, are susceptible to deterioration by steel reinforcement corrosion. In other cases, cracks may present evidence of other causes of damage. When inspecting structures, it is often useful to measure the depth of cracks to determine if they reach past the protection cover and represent a risk of further deterioration or loss of structural integrity.

Figure 3.11 Crack cutting

Two artificial cracks were cut on the slab for purposes of testing. One of the uses of the IE technique is to estimate the depth of a crack, therefore an accurate known depth was required in order to properly evaluate the capability of the device on that matter. It is very difficult to create a natural crack of known length and depth; therefore, the cracks were simulated by cutting the concrete using a saw. This procedure allowed the artificial cracks to reach a specified constant depth and make the measurement of said crack reliable.

The artificial cracks were professionally cut using a large circular saw as seen in Figure 3.11. The measured depth was drawn on the blade for the operator to follow. Two separate saw-cuts were made, one with a depth of 100 mm and the other with a depth of 200 mm. In Figure 3.11 it can also be observed that the

cut did not extend the full length of the slab; this was simply intended as an extra precaution to keep the slab safe and prevent additional cracking during lifting. The slabs were not designed to resist loads, especially in the unreinforced regions. Therefore, it was important to ensure that the slabs did not collapse during transportation and disposal at the end of the project. The ends of the cuts follow a circular shape made by the 300 mm blade.

It is worth noting, however, that the artificial cracks were not perfectly representative of natural cracks because they had a large constant width of about 5 mm over their full depth and were perfectly perpendicular to the slab surface. A natural crack will typically propagate around aggregates creating rough crack surfaces with significant variation in width and depth. IE waves that are generated by the impactors are, in theory, fully reflected when reaching air boundaries (such as the artificial cuts), whereas natural cracks with points of partial contact would likely result in partial reflection. Nevertheless, the result is expected to be a reasonably close approximation of a real application for the purposes of product development, and it is recommended that the performance of the device in the field or using naturally-cracked members should be investigated in the future.

3.2.3 Conduit

Another potential use of IE testing is to locate conduits or other embedded objects within the concrete section. In Slab 2, an empty post-tensioning conduit of 2 inch (50.8 mm) diameter was placed at a depth of 150 mm as seen in Figure 3.12. The conduit inner diameter is made from steel with 3 mm rectangular grooves to increase the bonding when post tensioning (Figure 3.12). The outer diameter is made from a thin layer of rubber to protect the steel from outside interactions and moisture ingress. The conduit was installed at the mid-depth of the slab and held in place using several small wires attached to a straight piece of wood as seen in Figure 3.8.

Figure 3.12 Picture of the conduit post casting

Conduits are quite frequently used in concrete structures for various reasons including non-structural uses such as utility pipes or for structural purposes such as post-tensioning. Prestressing concrete has gained a lot of popularity in recent years to increase the efficiency of concrete structures. By reducing the tension stresses in the concrete significantly, it allows the designer to build much longer structural elements. This is particularly useful for structures such as bridges to allow longer spans or parking garages that require large open areas with less columns. It also reduces the cracking of the concrete which is important for structures exposed to deicing chemicals. Those types of structures are almost exclusively built using prestressed concrete nowadays.

Post-tensioning a concrete element requires a conduit to be installed prior to casting where the post-tensioning strands will be installed. Once the strands are installed and tensioned, grout is pumped into the conduit to create a strong bond between the tendons and the concrete. It is assumed that once the grout reaches the other side of the conduit, the conduit is full. This might not actually be the case as air pockets could still be present in the conduit. In this case, non-destructive techniques are required if one intends to identify that possible issue because coring through the tendon is not an option. Pockets of air inside the conduit could potentially allow moisture to easily reach the tendons and induce corrosion.

Other types of conduits such as electricity or gas could also be hidden inside an element for various reasons and would most likely contain air or gas. If work is required on the element, those types of conduits represent a severe hazard if broken, therefore being able to locate them is essential. For the purpose of this project, the conduit was left empty to represent that presence of either gas or air pocket.

3.2.4 Artificial Voids

Voids are another frequent defect that can occur in concrete for various reasons, including poor consolidation of the concrete or other construction errors. It is impossible to create natural voids of specific known dimensions. The low density of foam allows the stress waves to be reflected in a similar manner as the presence of air. Therefore, to create voids artificially, foam was used. A cube was made from multiple layers of 12 mm thick Styrofoam. The foam sheets were cut into multiple square sections and assembled into a cubic shape. The final dimensions of the artificial voids were a cubic shape with a 150 mm width, length and depth as shown in Figure 3.13.

Figure 3.13 Slab 2 Artificial Voids

Two artificial voids were installed in Slab 2 prior to casting and were positioned at 300 mm from the edges of the formwork to the edge of the artificial defect to make sure that the slab boundaries would not influence the readings. One void was placed in the steel-reinforced portion of the slab, and the other was placed in the unreinforced region. The cubes were fixed in place using a small steel frame underneath to secure the depth as well as many small wires to prevent it from moving sideways during casting.

3.3 SLAB 3

3.3.1 Overview

The third slab contained multiple delamination defects of different sizes and depths. Six artificial delamination defects of size 300 mm by 300 mm were simulated using 7 mm thick pieces of foam installed at different depths with and without the presence of reinforcement (Figure 3.14). The ability of the IE device to find delaminated regions of different sizes was tested as well using three additional rectangular defects (#7, 8, 9 in Figure 3.14).

Figure 3.14 Slab 3 Plan: Delamination

3.3.2 Delamination

Delamination is the principal defect that the IE technique is expected to excel at finding. Because it is a layer of air within the concrete, it generates a nice reflection of the wave at the level of the delamination as well as at the bottom of the element. Theoretically, this results in two clear peaks on the graph generated by the IE software in the frequency domain. Creating natural delamination zones in the slab with known dimensions is not practical. To make an artificial delamination layer, thin foam layers of 7 mm thickness were cut and installed within the forms before the concrete was casted. The dimensions and depth of the artificial delamination were varied to provide different results once tested as seen in Figure 3.15. Accurate measurements of the locations of those defects were taken prior to the casting.

Figure 3.15 Slab 3 Form

As seen in Figure 3.15, the sizes of the squares were set as 300 mm by 300 mm for 6 out of 9 specimens. For those 6, only the depth at which they were placed varied by 50 mm intervals (100, 150 and 200 mm) in order to better understand the effect of defect depth on the measurement results. As discussed in Chapter 2, different modes of vibration may be excited for shallow versus deep delamination regions. The other three artificial defects on the right side of the slab (#7, 8, 9) were cut to different sizes which were 50 mm by 300 mm, 150 mm by 300 mm, and 400 mm by 400 mm. All three were placed at a depth of 150 mm corresponding to the mid-depth of the slab. Those were placed to study the effect of varying the size of the defect on IE measurements. As seen in Figure 3.15, the placement of the foam was difficult to achieve with a high level of precision. The actual as-built placements and depth of each piece of foam were carefully measured and are shown in Figure 3.16.

It is important to note that during the casting of Slab 3, some of the thin pieces of foam installed were destroyed by the weight and flow of the concrete. Therefore, the delamination #3, 5, 6, and 7 were broken during fabrication; most of the foam pieces were retrieved and removed, while some other small pieces were lost in the concrete in the vicinity of their original intended location. The casting was stopped temporarily, and thicker pieces of foam (15 mm) of the same size were inserted to replace the defect # 3, 5, and 6 that were damaged. The replacement foam had very similar dimensions to the original pieces, and were placed as closely as possible to the original location, but their final depth could not be controlled as precisely as desired.

Delamination size and depth

- 1- 300mm by 300mm at 180mm from bottom
- 2- 300mm by 300mm at 155mm from bottom
- 3- 300mm by 300mm at 105mm from bottom
- 4- 300mm by 300mm at 190mm from bottom
- 5- 300mm by 300mm at 150mm from bottom
- 6- 300mm by 300mm at 110mm from bottom
- 7- 50mm by 300mm at 165mm from bottom
- 8- 150mm by 300mm at 155mm from bottom
- 9- 365mm by 365mm at 155mm from bottom

Figure 3.16 Slab 3 Grid View

3.4 TEST METHODOLOGY

3.4.1 Impact Echo

This section covers the instrumentation and methodology required to effectively use the IE prototype as a non-destructive testing device. As seen in Figure 3.17 below, the instrumentation required for conventional IE testing is relatively simple.

Figure 3.17 Conventional Impact Echo Device (Schubert 2008)

Figure 3.17 shows two components required for the method, which are a sensor and a computer. The sensor, an accelerometer, measures the vibration caused by an impact on the surface. It captures the change as a function of time and transfers it to the computer. It represents a large amount of data as the precision of the method depends on the rate at which the measurements are done. The second component is a computer because this method collects and transforms a large amount of data to be usable. The transformation of the time-based data to a frequency spectrum via a Fast Fourier Transform is a requirement of the method to understand the result. Strong computing power is required to handle the high amount of data and allow instant feedback to the user concerning the tested point.

In addition to the device itself, an impactor is required to generate the wave. Earlier research by Sansalonne and Carino (1986) used a steel ball connected to a steel rod as a hammer and it is still the standard method used by engineers. Using a mechanical hammer instead of an electronic impactor allows to generate waves of higher energy which are essential for good results in IE testing. The spherical shape of the hammer is to ensure a very low contact time with the surface which increases the method's reliability. The ball can have various sizes, but the smallest size is generally recommended to limit the impact time. The sizes used generally range from 5 mm to 30 mm. The differences between them is the contact time. The smaller the steel ball diameter, the smaller is the contact time. A short contact time makes it easier to obtain thickness vibration mode for the detections of flaws at shallower depths.

3.4.2 Prototypes

For this project, a new generation wireless IE prototype was developed and validated together with the project industry partner. The product evolution included multiple iterations of the device and associated software during the testing phase. The hardware and software were designed by the project partner

based on the testing, analysis, and feedback provided by the testing conducted at the University of Ottawa. The first iteration of the device is presented in Figure 3.18.

Figure 3.18 Impact Echo Prototype 1.0

The first prototype included the essential components (i.e. a high-precision sensor and recording device) without any utility features. The accelerometer measures the change in voltage and transfers it via Bluetooth to the tablet which handles the data. The electronic components were simply glued to a plastic box and attached to a battery. Apart from problems with the unreliable battery and loose sensor connections, there were multiple issues with this prototype. First, the transducer needs to be perpendicular to the surface, stable and connected to the surface using a greasy substance like Vaseline. A temporary foam support was built to hold the transducer in place and provide stability. The procedure required the user to hold the transducer while setting up the software, pushing a trigger on the box, and striking the concrete with the impactor. This version required too many actions to be done while keeping the transducer in place properly. Using it for multiple points was tedious and inefficient.

The second issue with the first iteration is that this version had sensitive wires exposed which broke multiple times by simply moving the sensor around. The main purpose of this iteration was to test if the results were reasonable and locate the possible problems. Following preliminary testing and detailed

recommendations provided to the product team, a second iteration of the device was developed as shown in Figure 3.19.

Figure 3.19 Impact Echo Prototype 2.0

The second prototype was a hand-held device with a 3D-printed housing that allowed testing to proceed much more efficiently. The housing was comfortable to use, provided the required stability to the transducer as well as the protection to avoid damage. This device still required the user to push a button on the device to start the readings and there was no battery compartment in the housing. This meant that the device needed to be connected to a laptop to provide power from a low voltage source. The push-button was also problematic as it broke quickly. The electronics inside the device were reused from the previous iteration which lead to issues with the wires that were damaged previously during testing. After multiple tests and failures, a third prototype was made using the same housing. This time, the push button was bypassed to remove the potential failure and improve the efficiency of the testing. The wires were also replaced with new ones and securely connected and protected inside the housing.

In parallel to the testing of the device hardware, the IE software was also being developed. The software was modified from another product application and had to go through multiple updates to be usable for IE measurements. Many features were added, removed, or changed throughout this project based on the test results and recommendations provided, such as the addition of a grid feature (to store and present the location of the readings for each measurement point) and the automatic data analysis (i.e. transforming the time-domain data to the frequency domain using a Fast Fourier Transformation algorithm). Several technical issues also had to be resolved that were associated with the use of Bluetooth

for transferring the data wirelessly from the accelerometer to the iPad. Most of these issues are outside the scope of this thesis and are not discussed in detail here; however, the product user interface will be briefly presented as a useful contribution of this project.

A tablet was used to receive the data, perform the fast Fourier transformation, and plot the results. It also provides the interface for managing the data and selecting the locations of the test points. The software is a user-friendly touchscreen-based software, currently built for iOS only. It allows the user to define the project and set-up grids to use for the testing.

For this test, the impactor used was 20 mm. The reason is that it was the most suitable size available for a 200 mm depth slab based on Table 2.5. The slabs are expected to be of good quality due to being made in a controlled environment which usually leads to a good wave velocity of close to 4000 m/s. In this case a 20 mm impactor would be good for reflection surfaces beyond 172 mm thickness. This should be good for both the thin region and the thicker one. The suggested methodology is to use the largest impactor meeting the thickness requirement and reduce the size for more precision when an irregularity is located. The smaller impactor gives more definition for reflections at lower depth, but it also generates less energy for the wave and induces more noise to the results. For consistency, the 20 mm impactor was used for the primary test.

Figure 3.20 shows the base interface of the software that allows the user to create multiple projects and set up individual grids within the project (Figure 3.21). On Figure 3.20a, the main features are the option to create a project and create a test. The project contains the general information like the name, location, description and date. Within one project, the user can generate multiple tests containing the information about the tested grid such as a brief description of the purpose, the number of squares forming the grid, and the spacing. The same page can also be used to export the data in mass in CVS format for further analysis of the data using external sources. There is also an option to create contour maps to interpret the results, but it is not entirely completed yet. The software experienced many iterations throughout the project, and is still a beta version; therefore there are a few remaining bugs, unfinished features, and in some cases, useless information displayed (e.g. the length of 16 m displayed in Figure 3.20). The contour map is also an unfinished feature that will surely be useful once it is fully functional.

Figure 3.20 Software Project Creation Screen

Figure 3.21 shows the grid interface of the software. To move to the test screen, the test location needs to be selected individually by tapping on the touch screen. For this project, a 20x20 grid was used as seen below. The grid has to be created and defined via the “New Test” seen in Figure 3.20. The software allows to generate any size of rectangular grid with some limitation. Currently, it is difficult to use a grid larger than 20x20 like the one used for this project and there is no possibility to update or change the information selected for the grid. In other words, once the test is created, it cannot be changed. This causes some limitations if a mistake or unpredicted outcome occurs, as there is no way to modify the grid. To change something, a new test must be created, and the measurements are restarted. In occurrence for this project, after most of the tests were done, it was realised that the input height and width of the grid were wrong. The grid below displays 2m by 2m, but that is for the size of the slab; the grid is actually 1.9 by 1.9 m to accommodate a 50 mm spacing next to the edges since testing there is impossible. However, since it is a small display error, the tests were not redone.

Figure 3.21 Software Grid

Figure 3.22 shows the testing interface of the software. To get to that interface, a point has to be selected by tapping on it. A prompt will ask if the user wants to add the grid point and presents the interface shown below.

Figure 3.22 Software Testing Interface

The top graph is the raw reading that the device's sensor receives from the test. It is simply a graph showing the displacement versus the time. That graph cannot be directly interpreted. In a previous version of the software, the user was required to upload the raw data into another program to transform it to a frequency spectrum before analysis. Now, this step is completed automatically without the need for post-processing. The second graph at the bottom of Figure 3.22 shows the frequency spectrum which is required for analysis. As described in Chapter 2, the IE technique is used to measure the thickness of a concrete element based on the peak frequency received after an impact. In this example, the peak frequency was at 6436 Hz. The program can then be used to calculate a thickness for that point using the wave velocity and shape factor. The other features present on the interface, such as the 5-10-15-20 representing the ball size used and the Accept/Discard buttons, are not needed and will probably be removed eventually.

Performing the test using the new IE device is relatively simple (Figure 3.23). The idea is to use the impactor to create a stress wave that travels through the element and is reflected at the concrete-air interfaces. Every time the wave bounces on the surface, it causes a very small displacement. Those displacements are captured by the high precision transducer and sent to the tablet through a wireless Bluetooth connection. The captured data has a displacement associated with the time of the

measurement. Using a software, the time-based data is transformed into a frequency spectrum using a standard fast Fourier transformation.

The Fourier transform is a mathematical function that allows the user to decompose a wave or signal into base frequencies. It is used to go from time-based data like that obtained from IE readings to a frequency spectrum that is required to interpret the data.

That frequency spectrum is then used for the condition assessment of the tested element. Using Equation 3.1, the frequency spectrum can be transformed once again into a depth spectrum. Doing that for many points allows the user to build either a precise tomography of the tested specimen or a contour map demonstrating the depth of the concrete-air interfaces at tested points on the specimen.

Figure 3.23 Impact Echo Testing Flowchart

This procedure requires the user to have a certain experience with the device or at least a strong understanding of the physics the method is using. One of the requirements to make the interpretation of the results possible is to have an expected idea of the results. A good quality concrete would likely have mostly points resulting in a single thickness vibration peak typical to sound concrete. By knowing the wave velocity and the thickness of the tested element, the user is able to predict the expected frequency where the thickness peak should appear. Results that differ significantly from the expected frequency are likely caused by the presence of a defect under that location. Redoing the test allows the user to confirm the measurement value. This makes the method more difficult to use in cases where information is limited. For instance, cases where the thickness of the element is unknown or varies per location would be difficult to test using impact echo. In those cases, using another type of NDT beforehand as a preliminary test would be suggested.

3.4.3 Ground Penetrating Radar

The results obtained from the new IE device were compared to those from other commercial NDT systems. GPR is a simple tool used for non-destructive testing. The testing process is very quick and could be done in only a few minutes for the three slabs tested. Rather than taking measurements at discrete points like the other methods discussed, this technique allows the user to guide the sensor along a line which speeds up the process considerably. GPR uses electromagnetic waves that are emitted from the device and travel perpendicularly to the surface in a cone shape. Since they are electromagnetic waves, they are affected by the change of electric properties. Significant changes in the wave behaviour usually occur from either the presence of steel, which is much more conductive than concrete, or the presence of an air pocket such as delamination or voids.

The GPR device used in this project is shown in Figure 3.24 below. This is a small model of commercially available Ground-Penetrating Radar device. It comes with the utilities expected from a tool frequently used on sites. A compact computer allows the user to simply set-up the test to do and complete the analysis on site. The scanning tool is light and hand-held for efficient work. It is also equipped with small wheels underneath making it very easy and fast to push it along the grid lines as seen in the Figure 3.24. Other models of GPR are available for larger surfaces, but this one suited the small slabs tested perfectly.

Figure 3.24 GPR Testing

Prior to testing, the equipment must be set-up and calibrated using the properties of the concrete and the dimensions of the tested area. A grid is also necessary to test straight lines and allow to make a decent contour map afterward. For the testing itself, the procedure is simple as seen in the following flowchart (Figure 3.25).

Figure 3.25 Ground-Penetrating Radar Testing Flowchart

3.4.4 Ultrasonic Pulse Echo

The procedure for testing with UPE is very similar to the one for the IE technique since they are based on similar concepts. Both are based on the propagation of waves that are reflected at concrete-air interfaces and come back to the device. To perform a full assessment, it is required to proceed one measurement point at a time which is time consuming. It has an advantage over IE for the ease of use as it does not require a greasy material to ensure a good contact of the transducers.

The UPE device shown in the Figure 3.26 is a commercially available device. As seen in the figure, it is a light and compact model for easy use and is composed of two components. The testing equipment is comprised of the transducers on the left and an attached computer on the right. The testing equipment is designed to be air-coupled to remove the requirement for a good contact between the transducers and the surface by using a greasy material. It is made so that pushing the device toward the surface creates an air-tight zone to prevent external effects. The other component, the computer, is used to set-up the test and handle the data. It allows the user to define the materials and test properties to use and organize the tests. Once the tests are done, it can be used for simple analysis of the data such as generating contour maps.

Figure 3.26 UPE Testing

Before doing the actual test with this device, the user needs to calibrate it. Using known properties of the tested element and possibly an area of sound concrete, the wave velocity must be calculated. The method calculates the depth of the first reflecting interface by comparing the wave velocity to the time between

the wave generation and the moment it is captured again by the transducer. A good estimation of the wave velocity is required to have good results. Once the calibration is done and the test is set-up in the computer, the procedure itself is very simple (Figure 3.27).

Figure 3.27 Ultrasonic Pulse Echo Testing Flowchart

3.4.5 Ultrasonic Pulse Velocity and Schmidt Hammer

It was originally intended to conduct additional comparisons with UPV and Schmidt hammer tests to complement the testing program presented. Unfortunately, these tests were not possible due to restrictions associated with the COVID-19 pandemic that occurred in 2020. At the time of the testing, the laboratory was closed for 3 months, and after re-opening access was limited. Details on these methods, including their advantages and limitations, are presented in Chapter 2.

4 RESULTS AND DISCUSSION

This chapter will present and compare the various results obtained from the testing of three selected non-destructive techniques (NDT). The selected methods are the Ground-Penetrating Radar (GPR), Ultrasonic Pulse Echo (UPE), and Impact Echo (IE) which is the main method studied in this project. (Initial plans to include two additional NDT methods described in the earlier chapters were not completed due to timing and lab access restrictions caused by the COVID-19 pandemic.) For the experiment, three reinforced concrete slabs were fabricated to serve as the test subjects, as described in Chapter 3. The slabs contained well-defined artificial defects and characteristics to identify using the NDT. The results presented in this chapter will be structured around the three slabs with their distinct features; the measurements obtained for each slab will be presented individually, along with analysis and explanation of the findings. Based on the results obtained with each method, a comparison of their advantages and limitations will be discussed.

4.1 SLAB 1 THICKNESS MEASUREMENT

Slab 1, designed as a control sample, is the simplest of the three test specimens as it only had a change of thickness (i.e. geometric discontinuity) along its centerline and was partially reinforced with steel bars. Figure 4.1 shows a view of the 100 mm x 100 mm grid pattern on Slab 1 and corresponding labelling scheme to better understand the results that will be discussed later. The grid pattern includes 20 rows (numbered 1 to 20) and 20 columns (labelled A through T) for a total of 400 measurement points. The intersections of the rows and columns are used to identify each of the measurement points; for example, point J9 refers to the intersection of column J with row 9.

Figure 4.1 Slab 1 Grid View

Even though the design of Slab 1 is very simple, it still fulfills several important objectives. The main purpose of this slab was to provide large areas of sound concrete with different thicknesses. This is very useful to calibrate the equipment and the equations used. The second purpose of this slab was to better understand the effect of reinforcing steel on the measurements obtained by various NDT methods without additional interference from the presence of defects. This was achieved by dividing the slab into two parts, with and without the presence of internal steel (divided along row 10 in Figure 4.1). The third effect investigated is the potential for measurement errors induced by section boundaries, including edges and sudden changes in geometry. A sudden 100 mm step-change in thickness occurs between columns J and K, creating four zones as shown in Figure 4.1. The other boundaries are found around the perimeter of the slab, creating the potential for a “wall effect”. These will be discussed in further detail in the following sections.

4.1.1 Slab 1 GPR

The first method tested was GPR. To recall, the testing device generates electromagnetic waves that travel through the material and are affected by any objects with an electric impedance different from the concrete. Because of its nature, the method is strong at finding reinforcing steel since it is a highly conductive material compared to concrete resulting in a significant effect on the electromagnetic waves. Modern GPR technology allows the user to visualize the interior of reinforced concrete elements at any desired depth. The commercial system generated a map similar to Figure 4.2 for every depth at a 0.025 m interval. Due to the large number of maps generated, only the most relevant ones are presented as they demonstrated more distinct results. The selected depths are 0.100 m, 0.200 m, and 0.300 m respectively, which are the locations directly below the top layer of reinforcement, the full depth of the thinner section (200 mm), and the full depth of the thicker section (300 mm), respectively.

Figure 4.2 GPR Slab 1 Results 0.100 m, 0.200 m and 0.300 m depth

From Figure 4.2, it can be observed that the GPR method was able to easily locate the reinforcing steel inside the concrete element. The colours represent how much the electromagnetic wave was affected at that depth ranging from blue (no reflection) to red (high reflection). The GPR device detected both the top and bottom reinforcement layers that were around 50 mm from the bottom and top surfaces of the concrete. The figure also illustrates the limitations of the method.

As discussed in Chapter 2, the electromagnetic waves travel in a cone shape starting from the GPR device on the top surface of the concrete. When it encounters an obstacle, the wave travels around it forming a “shadow” shaped like a hyperbola; the unobstructed portion of the wave continues to propagate until it is intercepted by another object if there is one. Hence, the top reinforcement layer represents an obstacle in front of the bottom reinforcement layer. This can be understood by comparing the quality of the three images in Figure 4.2. The 0.100 m depth represents the top mat of steel bars, which is the first layer of obstructions the waves intercept and are clearly displayed to a point where the user could realistically estimate the steel ratio and spacing if desired. The image in the middle corresponds to a depth of 0.200 m which is the map that showed clearer signs of reflections coinciding with the depth of the thinner regions. The Zone 1 on the top left (see Figure 4.1) has a bottom reinforcement layer at a depth of 160 mm which is slightly above the depth of the image layer, explaining why the reinforcing bars appear as very wide undefined shapes. The bottom left quadrant, Zone 3, is unreinforced and has a thickness of 200 mm. The colouring in Zone 3 is likely caused by the reflection of the bottom of the slab that is elevated and sitting on wooden blocks. Similarly, on the right side in the last map (0.300 m), the poorly defined colouring corresponds to the full thickness of the slab. Some “shadows” of the bottom reinforcement layer are also visible. Based on the results, this method seems to be able to locate the steel reinforcement accurately for the top layer and the thinner region of the slab, but was unable to provide clear images in the deeper portions of the slab. Furthermore, the method was able to locate the bottom of the slab in the relatively shallow regions (Zone 1 and 3), but struggled to find anything in the deeper section making it an unreliable tool to measure thicknesses.

4.1.2 Slab 1 UPE

The second method tested was UPE. To recall, the method is designed around the use of sound waves to locate defects in concrete. The ultrasonic pulse waves are generated electronically by transducers and sent into the material. The waves bounce at the first interface and get reflected to the device. The travel time of the wave between the moment it is sent to the moment it is received back after being reflected is

measured and used in the analysis. An example of the raw data (signal amplitude vs. time) obtained from this method is presented in Figure 4.3.

Figure 4.3 Slab 1 UPE Signal Curve

By using a sound concrete section of known thickness, it is possible to back-calculate the wave velocity. This allows to calibrate the device to the specific test specimen. In this case for the first slab, the velocity was found to be 2500 m/s. The calculation for this method is relatively simple. The time between the moment the wave is generated to the moment it comes back is measured and multiplied by the velocity to estimate the travel distance. For example, in the signal curve above, the travel time is found to be 238 microseconds:

$$2500 \frac{m}{s} * 238 \times 10^{-6} s = 0.595m \text{ traveled}$$

Since the wave travels to the bottom of the slab and returns, the total distance travelled is twice as much as the actual depth of the reflecting interface. The result is a depth of 296 mm which is only 1.3% lower than the expected value of the slab thickness (300 mm). Once many points are tested, a contour map of the element can be generated to better illustrate the results (Figure 4.4).

Figure 4.4 Slab 1 UPE Contour Map

It is important to note that the contour map generated from the UPE device and software is rotated 90 degrees with respect to the orientation shown in Figure 4.1. Hence, Figure 4.4 shows the slab oriented sideways and the right side of the map is the reinforced section (Zone 1 and 2 in Figure 4.1). For the testing itself, it followed the same grid with the exception of the perimeter lines as they were too close to the edges to use this device. Based on the contour map alone, the device seems to generally provide accurate estimates of the actual slab thickness.

However, a relatively coarse scale is used for the contours provided by the UPE device, allowing for a relatively large tolerance for error as the colour scale changes only every 50 mm which is quite a lot for a 300 mm concrete slab. This could be solved by using third-party software to generate the contour map based on the raw data.

The boundaries at the perimeter of the slab did not appear to have a significant effect on the results, although it should be noted that no measurements were obtained within 150 mm from the slab edges. The geometric discontinuity in the middle of the slab can also be easily located with the contour map.

The possible effect of steel reinforcements can be determined by comparing the unreinforced zones to the reinforced ones. For the zone 4, the unreinforced 300 mm thickness region, 86 % of the points resulted in a measurement within 10 % difference from the expected 300 mm. We can compare this result to the ones from zone 2, the reinforced 300 mm thickness region. In zone 2, there was 62% of the points that were within an error of 10% based on the 81 points tested per zones. This indicates that the steel reinforcement influenced negatively the readings of the device. This can also be visualized using the

following graphs in Figures 4.5 and 4.6. The first graph shows the depth along the vertical lines (based on Figure 4.4) in the unreinforced section while the second graph represents the reinforced section.

Figure 4.5 Slab 1 UPE Depth measured along the vertical lines in the unreinforced zones 3 and 4

Figure 4.6 Slab 1 UPE depth results along vertical lines in the reinforced zones 1 and 2

Figures 4.5 and 4.6 clearly demonstrate the effect of reinforcement on the reliability of the readings. In the first graph, the sound concrete allows the device to measure the depth with very high precision and only a few anomalies. However, in the second graph, the interference of the steel reinforcing bars induced many false readings and reduced the precision significantly.

These graphs also allow to better understand the effect of the geometric discontinuity on the device. In Figure 4.5, it is observed that the device picks up either the 200 mm or 300 mm depth precisely while the second graph has much more inconsistency in the results.

4.1.3 Slab 1 Impact Echo

The third and main method tested for this project is IE, using the newly developed prototype device and software described in Chapter 3. The method is designed around the use of stress waves. The wave generated by an impactor travels through the material and gets reflected when it encounters concrete-air interfaces such as the bottom of the slab or physical defects.

4.1.3.1 Wave Velocity Calibration

Slab 1 is very important for the IE method as it was used to calibrate Equation 3.1 to determine the wave velocity for the concrete used in this study. The equation used for this method is a relation between the wave velocity and the primary frequency at which most of the wave is reflected. To determine the wave velocity, an area of sound concrete is required. A typical result for a sound concrete point is shown in Figure 4.7.

Figure 4.7 Slab 1 Impact Echo Point M7

The point M7 presents an ideal result from the IE technique, with a well-defined peak frequency at 6684 Hz. It is located in Zone 4 far enough from any potential negative interactions caused by an edge boundary or geometric discontinuity. Many points on the slab are similar to this one with slightly different peak locations. For this reason, compiling all of them to calculate the average is preferred.

Figure 4.8 shows a compilation of the results from 46 different measurement points tested in the unreinforced 300 mm sound concrete section (Zone 4) at a distance of at least 150 mm from the edges. The amplitude values were also normalized with respect to the peak amplitude to fit within the same graph and allow for a direct comparison. For this portion of the analysis, obvious outliers in the results were removed to improve the accuracy of the calibration.

Figure 4.8 Impact Echo 300 mm Thickness Unreinforced Zone 4

To facilitate the interpretation of the results obtained, the minimum, maximum, and average values from the 46 measurement points in Zone 4 are summarized in Figure 4.9 below.

Figure 4.9 Impact Echo 300 mm Thickness Average Unreinforced

A data analysis suggested an average peak of 6910 Hz with a standard deviation of 244 Hz. This average was calculated from the peak frequencies of each of the selected curves for this calibration. For a sound concrete section with a depth of 300 mm, this peak frequency can be used to calculate the average wave velocity through the concrete:

$$h = \frac{\beta(V_p)}{2f} \rightarrow 0.3 = \frac{(0.96)(V_p)}{2(6910)} \rightarrow V_p = 4\,319 \text{ m/s}$$

Concrete is a heterogeneous material, and hence the wave velocity may vary even within the same concrete element; nevertheless, the computed wave velocity is assumed to be representative, on average, of the concrete in all three slabs tested in this project. This velocity can be used to translate the peak frequency to a depth calculation. For instance, using the velocity of 4319 m/s, the average estimated depth for Zone 4 is 300 mm; however, as seen in Figures 4.8 and 4.9, the measurements were not uniform and the depth estimates corresponding to the average velocity ranged from 289 mm to 322 mm. Figure 4.10 below shows the distribution of the measured points and the corresponding estimated depth. All of the points were tested, but some were left out for the calibration process due to irregularities in the curve. All of those points were taken in Zone 4, the unreinforced 300 mm depth region of slab 1.

Figure 4.10 Impact Echo Points distribution for calibration on zone 4

Figure 4.10 shows that there is no obvious pattern in the measured points and that the values of those points do not vary greatly. Table 4.1 below shows how the measured values were distributed in the section.

Table 4.1 Impact Echo Calibration Distribution

Estimated depth	Number of points corresponding	Error (%)
289	16	3.67
299	14	0.33
310	12	3.33
322	4	7.33
Excluded	18	

In this case, 299 mm is the closest possible result to the expected value of 300 mm because of the sampling frequency used (as discussed below). Other points selected are 289 mm and 310 mm which have approximately a 10 mm (3.3%) difference from the expected value. Lastly, the 322 mm measurements were only 22 mm (7.3%) different. Those points are fairly close to the expected value with a maximum of 7.3% difference. The points selected for the calibration were carefully selected based on the appearance of their curve. The curves with multiple peaks due to the presence of flexural vibrations or extensive noise were not used for this analysis. In this case, most of the excluded results are relatively close to the edges (150 mm) which is likely one of the causes of the irregularities in the results. The other unused points are grouped in the bottom left side which indicate there is most likely an unpredicted anomaly with the slab at that location affecting the results.

Furthermore, it is important to note the discrete values shown in Figure 4.10. The wireless prototype induces a form of error tied to the sampling speed limitation of the hardware. The device records data at a fixed frequency of once every $1.97 \mu\text{s}$ (i.e. 508 Hz). Because of that fact, the frequency intervals obtained from the fast Fourier transform are also fixed at 247.56 Hz. In other words, the only frequencies that can be recorded by this system are multiples of this value. This is an important limitation of the current hardware, since the equation used to calculate depth is inversely proportional to the frequency. Using the average wave velocity of 4319 m/s, a peak frequency of 6910 Hz corresponding to a thickness of exactly 300 mm is impossible for the device. The closest possible frequencies are 7179 Hz (289 mm), 6932 Hz (299 mm), or 6684 Hz (310 mm). The lower the frequency, the larger is the step between the possible calculated depths. For the current slab with a thickness of 300 mm, the error is relatively small (i.e. 10 mm or 3.3%), but it adds up with other small errors including slight variations in thickness, the non-uniform wave velocity, the presence of unexpected defects, or areas of poor consolidation. Therefore, increasing the sampling speed of the testing device is a potential area of further development to improve the precision of the measurements.

4.1.3.2 Impact Echo Effect of Steel Reinforcement

To better understand the effect of reinforcement on the results, the above analysis was repeated for Zone 2. Figures 4.11 and 4.12 present the collected results from 100 points in the reinforced 300 mm thickness sound concrete region.

Figure 4.11 Impact Echo 300 mm Thickness Reinforced Zone 2

Figure 4.12 Impact Echo 300 mm Thickness Average Reinforced Zone 2

Using the same process as described earlier, the average for the reinforced section is 6614 Hz which results in an average wave velocity of 4134 m/s. The standard deviation for this data analysis is 333 Hz

which is slightly higher than the unreinforced section. This represents an average reduction in wave velocity of 4.28% compared to the unreinforced section, which is an acceptable error. Based on this result, the presence or lack of reinforcement should not generally be a source of problems for IE testing.

4.1.3.3 Impact Echo Section of 200 mm Thickness (Zones 1 and 3)

The thinner section of Slab 1 had a reduced thickness of 200 mm instead of 300 mm. The intent of this section was to verify whether the change in thickness would have any effect on the calibration wave velocity. However, during testing it became evident that the response of the thinner slab section was being influenced by other factors. Specifically, the lower thickness and the shape of the element (i.e. cantilever overhang) caused a low frequency peak induced by flexural vibrations to dominate the response. This mode of vibration does not allow the user to determine the thickness of the element using the Impact Echo equation. It usually appears in the result as a large peak at low frequencies.

The preferred main mode of vibration, is the thickness vibration. This is the desired mode as it can be used with a simple equation to determine the thickness. Luckily, the two modes do not affect each other significantly and can both be present in the same test. Figure 4.12 shows the result for the unreinforced 200 mm thickness section (Zone 3).

Figure 4.13 Unreinforced 200 mm Thickness Zone 3

Figure 4.13 clearly shows the presence of the flexural vibration mode at lower frequencies. A peak around 5000 Hz would represent a thickness of approximately 400 mm which is twice the actual thickness of the slab; hence the first peak should be discarded. On the graph, a second peak is evident on the right side

which is caused by the thickness vibration mode. The average for the second peak occurs at a frequency of 10 220 Hz which corresponds to a thickness of 203 mm using a velocity of 4319 m/s found in the previous calibration. Since each mode can be present, one can simply not consider the first peak for the calibration and take the second peak as the true reading for this section.

If the 10 220Hz frequency is used for calibration for a 0.2 m thickness, the wave velocity could be calculated using the following equation:

$$h = \frac{\beta(V_p)}{2f} \rightarrow 0.2 = \frac{(0.96)(V_p)}{2(10220)} \rightarrow V_p = 4258 \text{ m/s}$$

$$\frac{4319 - 4258}{4319} * 100\% = 1.41 \%$$

The difference between this result and the previous calculation is only 1.41%, which is acceptable and shows that ignoring the flexural vibration peak did not have a considerable effect on the accuracy of the results. The negative effect comes with the difficulty it adds to the data interpretation. The high amplitude flexural vibrations can potentially hide the desired vibration mode making the analysis difficult. It may also mislead the user into wrong conclusions based on the results. This is one of the reasons why the user must be trained properly and have a good knowledge of the possible outcomes of the tests.

Another way to deal with the presence of flexural vibrations is to use a smaller impactor. The size of the impactor affects its contact time with the surface. The lower the contact time, the more likely it is to generate thickness vibration modes for shallow sections. The trade-off of using a smaller impactor is that it often results in more noise. As reported in literature (Carino, 2015), a 20 mm impactor, such as that used in this study, is generally appropriate for measurements at depths of 176 mm or more. Therefore, in theory, the 20 mm impactor used should have produced mostly thickness vibration mode peaks. However, the contact time is also dependent on the user which is probably the reason why the flexural vibration mode peaks are larger; this result may also be partially attributed to the cantilever overhang which has a relatively low overall stiffness. Nevertheless, additional measurements were obtained using a 10 mm impactor at multiple points looking for more precision for the shallow slab section. Figure 4.14 shows the results from 9 measurement points taken within Zone 3 (200 mm unreinforced section far from the edge boundaries and the geometric discontinuity at the middle).

Figure 4.14 Slab 1 Impact Echo 10mm Impactor test in Zone 3

It is evident from Figure 4.14 that the main peak occurs at a frequency of approximately 10 kHz which is in agreement with the previous value. The low frequency peak caused by flexural vibration is still present, but significantly reduced. Hence, it may be concluded that simply ignoring the first peak in the first tests resulted in the correct answer.

4.1.3.4 Impact Echo Effect of Geometric Discontinuity

The geometric discontinuity (i.e. step-change in thickness) represents a boundary at the bottom of the slab that can potentially influence the IE measurements by causing additional reflections of the stress wave. The boundary is found precisely between the lines J and K on the grid for Slab 1 (Figure 4.1), J being on the thinner section. Figure 4.15 (a and b) shows a compilation of the results from both sides of the discontinuity.

Figure 4.15 Impact Echo Geometric Discontinuity

From the data presented in Figure 4.15, the effects of the boundary caused by the geometric discontinuity are observed. For the thinner section, J line (Figure 4.15a), the flexural vibration mode is still present and could be discarded as determined in the previous section. The secondary peak in that case occurs at a frequency of approximately 10 kHz. This result is very similar to the other points taken in the 200 mm thickness section which implies that there is no significant effect from the boundary. In this case the second peak seems to be suppressed by the first one. The reason is that the flexural vibrations often cause a much higher amplitude peak than the thickness vibrations when using a larger impactor. Since the curves

are scaled, the points with high amplitude cause the second peak to either be significantly reduced or completely disappear.

For the thicker side, K line (Figure 4.15b), the boundary has a larger effect. The hypothesis is that the wave reflects off both the bottom surface and the vertical boundary nearby (i.e. wall effect) before coming back to the top surface where the sensor is. This results in data that sometimes presents a slightly shifted peak toward lower frequency and has generally much more noise.

4.1.3.5 Impact Echo Effect of Edge Boundaries

The edge boundaries are found around the perimeter of the slab. The tested points used for this analysis were taken at 50 mm from the sides on all three slabs, which are close enough to be affected by the boundary. Figure 4.16 presents the results compiled from 156 points located on the outside boundaries of the slabs in the 300 mm thick region and at least 100 mm away from any defects.

Figure 4.16 Impact Echo Boundary tests results

Figure 4.16a presents the results from 156 points, which makes it very difficult to analyze directly. Figure 4.16b summarizes the information to show the envelope of results. As seen in the figure, the average peak is clearly defined at 5016 Hz (413 mm) with a standard deviation of 543 Hz. There was 66.0% of the points within a 5% difference from the calculated average which make sense since most points resulted in a clean shifted peak at lower frequency than the expected value. The individual frequency curves for each point had limited noise, but the peak values varied quite a bit between approximately 3kHz to 7 kHz as seen with the high value of the standard deviation. The reason is most likely due to the extra reflection caused by the vertical boundary increasing the travel distance of the waves. Figure 4.17 represents one of the typical raw results from a measurement point near the edge boundary.

Figure 4.17 Slab 1 Impact Echo Point A6 Raw

This result is representative of most points gathered near a boundary. The curve shows a shift toward lower frequency than expected with a peak at approximately 5 kHz instead of 6.4 kHz and very little noise. The lack of noise in the results can probably be attributed to the fact that the boundaries are straight and continuous along the slabs while the concrete itself shows no signs of deterioration in those areas that could affect the results. Furthermore, the higher amplitude of the peak also hides smaller variations that appear as noise.

4.1.3.6 Impact Echo Slab 1 Contour Map

To provide a more direct comparison to the results from the other devices, a contour map of the slab based on the IE measurements is presented in Figure 4.18. The contour map feature was recently added to the IE software based on recommendations from the testing conducted at uOttawa. However, it is currently under development and not yet finalized. Using the software, it was possible to create a contour map, but no customizations were possible with the current update which limit the analysis. For example, the figure below shows the contour map generated for the first slab using the Impact Echo software. For every point, a single peak frequency is selected which translates to a thickness estimation for that point.

Figure 4.18 Impact Echo Software Contour map (units in m)

The map above delineates between the shallow and deeper regions of the slab, as well as the effect of the boundaries and the geometric discontinuity. However, the default scale changes the colour every 50 mm which is quite large and unable to locate small variations of thickness calculations. Furthermore, it does not show a significant contrast in colour between the 200 mm section and the 300 mm thick region and instead shows higher contrast between points already very far from the expected values. For the analysis done on this slab, this contour map would have been sufficient. However, the issues with this contour map will be more apparent and discussed further once applied to the artificial defects in Slab 2 in the next section.

To improve the quality of the information displayed with the contour map, an external software was used based on the same data. The software Surfer (goldensoftware, 2020) allowed for a much more customizable map that is presented in Figure 4.19.

Figure 4.19 Slab 1 Impact Echo Contour Map (mm)

The map in Figure 4.19 was built from thicknesses calculated using the average wave velocity of 4319 m/s calibrated for the unreinforced 300 mm thickness sound section. The thickness represented in this map comes from the thickness peaks of each of the points tested. The map shows that the depth was properly determined with less than 10% error in most cases. Table 4.2 shows the distribution of the points within certain error thresholds for each of the zones excluding the points closest to the boundaries.

Table 4.2 Impact Echo Thickness Measurement Errors

Zones	Expected	<5% (%)	<10% (%)	<15% (%)	<20% (%)
1	200	15.2	57.0	82.3	86.1
2	300	43.8	76.3	78.8	91.3
3	200	81.0	81.0	81.0	81.0
4	300	79.4	88.9	88.9	92.1

This table serves the purpose of showing the accuracy of the method at finding thickness values, which is its primary purpose. The results are good as expected with few exceptions mostly in the thinner region. The reasons for that are mostly due to the slight difference in wave velocity and the presence of flexural vibrations. Furthermore, when looking at the map in figure 4.19, we see that the boundaries seem to affect a wider area as well.

The map allows to easily see that the boundaries are the most problematic points to measure using IE on this slab. The boundaries that were at 50 mm from the perimeter of the slab cause a shift of the peak toward lower frequencies. Since everything was calculated using a fixed velocity, it resulted in the edges showing a larger depth than the actual slab thickness. Similarly, the geometric discontinuity at the middle of the slab often caused odd reflections of the wave indicating a larger depth than the expected value.

4.1.4 Slab 1 Summary

In summary, Slab 1 was tested using the three selected devices. The first device, GPR proved to be very strong at finding reinforcing steel and was able to determine the thickness of the slab to some degree. The second device, UPE, was able to accurately find the thickness of the slab and locate the discontinuity. The simple method also did not have any problem with the boundaries, but it seemed to have some bad readings in the reinforced section. Lastly, the IE device was tested. The first step was to use the data to calculate an accurate value for the wave velocity. After that, the device was able to accurately measure the thickness of the slab. The problems associated with the IE method were the presence of boundaries that reflected the waves in unpredictable manner. This led to a shift of the peak frequency for the middle and edges as well as increased noise for the middle boundary. The contour map based on IE data was much more precise than the one obtained using the commercial UPE system, but it required external software to conduct the analysis. The conclusion based on the first slab results is that IE has more potential than the other methods at finding depth accurately but is currently more time-consuming. For reinforcement, the effect on IE readings is too small to be able to accurately locate them. To accurately detect the location of reinforcing bars, GPR remains the best option.

4.2 SLAB 2 CRACKS, CONDUIT AND VOIDS

The second slab casted had three main features to locate and describe as shown in Figure 4.20. On the left side, there are two vertical cracks. Crack A had a depth of 100 mm and Crack B was 200 mm deep. The second point of interest is the conduit. The post-tensioning conduit used for this test was made of steel covered with a thin layer of rubber to protect it and had an inner diameter of 50 mm. It was installed at a

depth of 150 mm and left empty. The voids represent the last points of interests on the slab. They were made of foam shaped into cubes with a width of 150 mm. Figure 4.20 shows the grid of the slab as well as the location of the defects on the slab. Rows 1 to 10 were unreinforced, while rows 11 to 20 contained two layers of steel reinforcing bars as described in Chapter 3.

Figure 4.20 Slab 2 Grid View

Based on the grid above, the important points to be considered on this slab include the lines next to the vertical cracks which are the C, D, F and G. The second location are lines J and K on both sides of the conduit. Finally, the points near the voids were tested carefully to properly identify the locations of the defects.

4.2.1 Slab 2 GPR

GPR was used on this slab as well to locate the various defects. Figure 4.21 shows the results at depths of 0.100 m and 0.150 m. The GPR system generated a map every 0.025 m throughout the slab thickness, which would be too many to present, therefore maps were selected that demonstrated the strongest effect on the device. Also, the 0.100 m depth is right below the top reinforcement which results in very clear visualization of their location while the 0.150 m depth is in the middle of both voids and the conduit.

Figure 4.21 GPR Slab 2 Results 0.1 m (left) and 0.15 m (right) depth

Similarly to the first slab, the steel reinforcing bars are the most obvious objects located with the device. They had a well-defined shape for the upper reinforcement while the objects beneath it are less clearly defined. At 0.100 m depth, the shadow caused by the voids and the conduit can already be observed. Since the second map is in the middle where all objects are located, it is the clearest image obtained with the device, showing both voids, the conduit, and the reinforcement grid. However, it is important to notice how challenging it is to differentiate the conduit from the reinforcement in the reinforced section. Hence, if the slab was fully reinforced, it would be difficult, if not impossible to locate the conduit properly. For the vertical cracks, unfortunately, there were no signs of their presence using this method.

An interesting advantage of this method is that it allows the user to characterize the embedded object or void. The geometry of the voids can be approximated based on the following images taken at different depths (Figure 4.22).

Figure 4.22 Slab 2 GPR 0.075 m (a), 0.100 m (b), 0.250 m (c), and 0.275 m (d)

These pictures allow to estimate more precisely the location and the size of the voids. The map (a) shows that the void at the bottom right corner of the slab starts to appear at a depth of 0.075 m while the void at the top right corner barely has a sign of its existence. In the map (b), the start of the second void is more clearly seen. The map (c) shows the last depth at which the voids are clearly visible at a depth of 0.250 m. The next map at a depth of 0.275 m (d) shows only vague signs of the voids cast by their shadow. The map also shows that the void at the top right is slightly more visible than the one at the bottom right at the depth of 0.275 m.

To summarize the findings, the bottom right void started appearing prominently on the pictures at a depth of 0.075 m and stopped at a depth of 0.250 m for a total size of 0.175 m. The top right void starts appearing at 0.100 m and stops at 0.250 m for a total size of 0.150 m. Furthermore, the spacing of the gridlines used were 0.100 m which leads to the correct conclusion that the voids are cubic with a thickness between 150 to 175 mm. It can also be determined that they are placed at a depth between 75 mm and 100 mm from the surface and that the top right void is slightly deeper than the one at the bottom right. It is accurate considering the voids were 150 mm thick and placed with a 75 mm cover.

4.2.2 Slab 2 UPE

For the second slab, UPE was able to locate all of the defects to some degree. Figure 4.23 shows the contour map made from the UPE result on Slab 2.

Figure 4.23 Slab 2 UPE Results

In the middle, the interference from the conduit slightly shows even though its location seems to be off. It offered a reflection interface at the middle of the depth at 150 to 200 mm. However, similarly to the other types of defects the locations are imprecise and undefined. There were also many points that resulted in false positive readings mostly located in the top half of the map which represents the reinforced region. This lack of accuracy concerning the location as well as the poor definition of the shapes makes this method unreliable for these types of tasks.

For the vertical cracks, the lack of precise location is not as important since they can usually be located directly by looking at the specimen. The device also allows to measure the depth more precisely using a different approach. By measuring the time it takes for the wave to travel from one side of the crack to the other, the depth of the crack can be estimated. The measured crack depths using the UPE system were 138 mm for the shallow crack and 208 mm for the deeper one. Both measurements were based on two readings each. The actual values are 100 mm and 200 mm which make the errors 38% and 4% for the shallow and deep crack respectively. This indicates the method is able to accurately measure the vertical cracks depth but is still subject to measuring error or irregularities. Further testing should have been done along the crack to confirm the results.

4.2.3 Slab 2 Impact Echo

Finally, the IE prototype device was used on the slab to locate the same points of interest which are the vertical cracks A and B, the empty post-tensioning conduit, and two artificial voids made from foam.

4.2.3.1 Impact Echo Effect of Vertical Cracks

The vertical crack was one of the main concerns for this method since the other devices failed to locate them. Crack A had a depth of 100 mm which is $1/3^{\text{rd}}$ of the full thickness of the slab. Figure 4.24a shows one of the measurement points near Crack A as an example, while Figure 4.24b shows the average results from 30 measurement points along lines C and D which are located 50 mm from either side of the crack.

Figure 4.24 Slab 2 Impact Echo Crack A

The average peak frequency in this case occurs around 6.2 kHz which translates to a thickness of 334 mm. This is 11.3% larger than the expected 300 mm for the full thickness. A peak at 6.2 kHz represents the reflection of the stress wave on the bottom surface of the slab. The crack also affected the quality of the results, as the shallow crack caused some uneven reflections of the waves which mostly resulted in increased noise in the readings.

Another observation from Figure 4.24a is the presence of a secondary peak around 8.2 kHz that appears for most of the measurement points adjacent to the shallow crack. This is indicative of a secondary reflection of the wave caused by another reflecting interface, i.e. the crack in this case. This represents a reflection at a depth of 252 mm using the thickness frequency calculation. This is not the correct depth for the crack, which should be 100 mm, but since the crack is vertical, it is not expected to directly reflect the waves like a horizontal interface does. A few of the measurement points did not present this peak or was insignificant, and therefore this response is difficult to identify without close examination of the results. Also, it is worth noting that in a contour map, which is based on only a single peak measurement, this peak would most likely not appear at all and thus the crack would not be found. In this case, the crack appears on the contour map as a small local increase in slab thickness. The second peak represents a way to determine the presence of a crack but is difficult to locate in case of noise or other anomaly in the curve. It is a sign to expect when testing near a crack but cannot be reliably used to locate cracks. Most of the time, the vertical crack can be located from the surface through simple visual inspection, the interest is mostly in measuring the depth of the crack.

A more precise way to determine the depth of the crack is to use a secondary sensor with the device as described in the previous section. This method is similar to the one used for UPE earlier. Using two sensors, one on each side of the crack, the depth of the crack can be estimated based on the time it takes for the second device to detect the reflected waves. Although it was initially planned to investigate this effect using the IE technique, the current version of the device did not have the capability to add a second sensor yet and the depth of the crack could not be measured. This feature is planned for future iterations of the device and presents a topic for further research.

Lastly, another thing to note from the graph in Figure 4.24 is its general shape. The upper bound envelope covers a larger range of frequencies because the measurements adjacent to the cracks were inconsistent. The peak frequencies had a standard deviation of 850 Hz which is quite large. Sometimes the crack would present the main thickness frequency with the secondary peak as discussed earlier. But other times, it would result in a shifted peak toward 5 kHz similarly to the results from the side boundaries of the slabs.

The results around the cracks were very inconsistent which is why the average line in Figure 4.24b does not have a defined peak. Instead, it has some kind of plateau between 5.2 kHz and 6.4 kHz.

Crack B was analyzed in a similar fashion. Figure 4.25 shows the maximum, minimum, and average from the 30 points taken next to it (lines F and G).

Figure 4.25 Slab 2 Impact Echo Crack B

The second vertical crack tested showed a lot more inconsistency than the first one. The deeper crack induced a lot of noise to the result and often caused odd peaks at lower frequencies. Those lower frequency peaks were not stable as each point had a different value. This response is somewhat similar to that of the edge boundaries discussed previously; the edge boundaries analyzed earlier represented a perfect vertical interface that was reflecting the stress waves in a consistent pattern which resulted in most points having the same shifted peak value. For Crack B, the 200 mm depth represented 2/3 of the total slab thickness. This implies that the reflecting interface was not as perfect as a full boundary and led to greater variation in the measurements, with peak values ranging from 3 kHz to 6 kHz. Also, there were multiple cases where the main peak could not be easily determined as multiple peaks in that range were present. Generally, from the data collected, the deep crack resulted in a shifted peak toward lower frequency with a lot of noise.

4.2.3.2 Impact Echo Effect of Conduit

The hollow conduit is an object that should, in theory, be easy to find using the IE technique. It was also relatively easy to locate using the other devices. Figure 4.26 shows the maximum, minimum and average of the 30 compiled results taken from the lines J and K next to the conduit.

Figure 4.26 Slab 2 Impact Echo Conduit

It is clear from Figure 4.26 that the average peak frequency was shifted toward a lower value compared to the sound concrete section with an average of 5446 Hz. If a velocity of 4319 m/s is used as defined earlier in the chapter, it would result in a thickness estimate of 381 mm at that location. Although the thickness estimate is clearly wrong, this result is expected. There are two possibilities that explain how the conduit influences the apparent peak frequency of the wave.

The first hypothesis is that the wave simply diffracts around the conduit while going back and forth between the transducer and the reflecting interface. The waves travel through the material and encounter the object. A portion of the waves interact with the surface of the conduit while the rest propagate around the conduit. This increases the minimum distance the wave must travel to encounter a good reflecting interface and come back (Carino, 2015).

The second potential explanation for a lower peak frequency is that the average slab stiffness is reduced locally due to the presence of the conduit. The wave velocity is directly related to the characteristics of the concrete based on Equation 2.1 as explained in Chapter 2. If the addition of the conduit in the slab causes a significant loss in stiffness, the value of the wave velocity would have to be reduced accordingly. Not correcting the velocity causes the analysis to overestimate the thickness value of the slab at that location resulting in a significantly larger depth measured.

It is likely that both effects occurred simultaneously and resulted in the peak frequency distribution shown in Figure 4.26. Although the thickness estimates obtained are unreliable, the fact that the results are

significantly different from the sound portions of the slab does allow the user to easily identify the location of the conduit on a contour map (as discussed later).

The IE measurements adjacent to the conduit were repeated using a smaller impactor of 10 mm diameter in order to determine whether it would allow for greater sensitivity to capture the waves reflected from the conduit in order to determine its depth. Using a smaller impactor size reduces the contact time with the surface and significantly increases the precision of the reading for thinner sections or for objects relatively close to the surface. Figure 4.27 shows the results from measurement points taken directly over the conduit using a 10 mm impactor.

Figure 4.27 Slab 2 Impact Echo Conduit Center 10 mm impactor

Comparing Figure 4.27 with Figure 4.26, it is clear that the smaller impactor was able to better capture both the primary and secondary peaks associated with the bottom of the slab and the conduit. The peaks present in Figure 4.27 occur at 5941 Hz and 11 388 Hz, which represent reflecting interfaces at depths of 349 mm and 182 mm which are fairly close to the actual values of 300 mm for the full depth (16% error) and the depth of the conduit at 150 mm center (21% error). Furthermore, if a reduction in average velocity due to the presence of the hollow conduit is assumed, a new velocity can be calculated considering that the first peak at 5941 Hz represents a thickness of 300 mm:

$$Vp = 0.300 * \frac{2 * 5941}{0.96} = 3713 \text{ m/s}$$

$$h = \frac{0.96Vp}{2f} = \frac{0.96(3713)}{2 * 11388} = 156 \text{ mm}$$

This results in a reflecting interface from the conduit at an average depth of 156 mm which is even closer to the actual value with an error of only 4%. Considering the limits of the method and the possibility that the flexible conduit might have moved slightly during casting, 156 mm is considered to be an acceptable approximation.

4.2.3.3 Impact Echo Effect of Artificial Voids

In theory, the artificial voids should be easy to locate using IE, as they are relatively thick defects that should cause a significant reflection of the waves. The difficulty is that they have a relatively small projected size and require a fairly tight grid pattern to locate them. Only a few measurement points in the grid shown in Figure 4.19 were directly above the voids. Figure 4.28 shows the results from these tests.

Figure 4.28 Slab 2 Impact Echo Voids

These results represent the 6 points that successfully found the void. The other points expected to be over the defect resulted in a lot of noise without any clear peaks. The results shown in Figure 4.28 are typical of the response that is expected from a shallow flaw. Over the flaw, the concrete section becomes very thin which makes it more likely to be affected by flexural vibrations as mentioned earlier in the chapter. This results in a large peak at low frequency as seen in the graph above. Furthermore, a secondary peak is observed around 5.6 kHz most likely due to a reflection from the bottom interface. The void itself did not present any visible sign on the graph. The reason is that testing for thin sections requires a smaller impactor than what was used in the primary test.

For this reason, a second series of tests were conducted with a 10 mm impactor. This step made it possible to get slightly more precise points. Figure 4.29 shows the result from those tests for Void A.

Figure 4.29 Slab 2 Impact Echo Void A using 10 mm Impactor

From this figure, it may be observed that most of the main peaks are between 5446 Hz and 6684 Hz. Those values usually represent the reflection for the bottom of the slab. The variation may be attributed to different degrees of the diffraction of the stress wave and loss of stiffness of the slab around the flaw. Furthermore, a few points on the graph showed the presence of a peak around 20 kHz. That peak represents a reflection at a depth of 100 mm which is close to the depth of the top of the void and is most likely a reflection from the top surface of the defect. However, only a single point presented that effect clearly which does not make it a reliable conclusion. The same points also present a peak around 16.5 kHz which translate to a depth of 120 mm where nothing significant is present. Considering the velocity is likely reduced in unpredictable manner, estimating the depth of the void using Impact Echo results seems challenging. The main peak is still dominated by the flexural vibration; therefore using an even smaller impactor could possibly provide more precision. However, the points around the void A were tested a third time using an 8 mm impactor which provided fairly similar results to the 10 mm one without the presence of clear peaks past the total depth thickness range (5.4 kHz-6.4 kHz).

4.2.3.4 Impact Echo Slab 2 Contour Map

Finally, after the tests on the second slab, the results were used to create a contour map. Figure 4.30 shows the map generated by the Impact Echo software while Figure 4.31 was generated using an external program.

Figure 4.30 Slab 2 Impact Echo Contour map 1 (m)

The map above shows the result generated directly from the Impact Echo software. Contrarily to the first slab contour map (Figure 4.17) this one clearly demonstrates the limitations caused by the lack of customization on the map. On the map above, the effect of the deeper crack is observed around 0.4 m from the left side of the slab. A slight effect of the conduit near the middle of the slab is also observed, and the effect of the void in the bottom right is also seen. However, apart from the deep crack, the other defects are not clearly defined and can hardly be located using this map. A second map was generated using a scale that was more sensitive to small changes in thickness measurements to capture small variations caused by shifted frequency peaks while still allowing some tolerance for the estimated thickness values. An approximation of the defect locations were also added over the map to better understand the contour map. The second map achieved the goal of differentiating the areas that are sound concrete from the location of the flaws.

Figure 4.31 Slab 2 Impact Echo Contour Map (mm)

The map was adjusted in order to have the colour scale change quickly, i.e. to increase the sensitivity to small changes in peak frequency. As a result, the locations of each of the defects are visible in Figure 4.31; although the boundaries of the defects are not well-defined and the thickness measurements over the defects are greater than the depth of the slab, locating the abnormalities is an important first step that can be followed by more sensitive measurements using a smaller impactor as discussed in the preceding sections. The effect of the vertical cracks can be seen at 300 mm and 600 mm from the left side of the slab. In the middle, the conduit affected a relatively larger area probably due to its size. On the right side of the slab, it is easy to find the location of both voids with the bottom one being a bit more prominent. This demonstrates the benefit of having a strong tool to make the contour maps for improved interpretation of the results. As with Slab 1, interference from the edge boundaries around the perimeter of the slab is also observed.

The flaws present on this slab were relatively small and demonstrated effects only on a few points located near the defects. The vertical cracks were precisely at 50 mm between two lines of measurements and both sides resulted in similar results. The effects of the defects were largely localized to the closest points near the defects as seen from the relatively small size of the affected areas, but increased noise could be seen in the results for the points 150 mm away from them especially on the left side of the map between the first crack and the side of the slab. Similar results were observed for the conduit, as the points directly adjacent to it at 50 mm showed the effect of the defect, but those at 150 mm generally did not show any signs of the conduit.

One thing to note is that many types of flaws appeared as a secondary peak on the resulting graphs. This means that the frequency representing the defect or the more direct effect of the flaw might not correspond to the main frequency peak selected for this contour map. The contour map only shows one thickness value per point which is a limitation in the analysis. The contour map is a good tool to find the general locations of the flaws, but in-depth analysis of the points through graphs showing the frequency domain data, as presented in the preceding sections, and through the use of smaller impactors when needed, often leads to more precise information on the flaws.

4.2.4 Slab 2 Summary

In summary, the second slab was a challenge for all three methods used, although the results varied for each technique. For the vertical cracks, GPR failed to locate either of them; there were no signs of vertical cracks on the contour maps. For the UPE method, the results were highly variable and the effect of the cracks was uncertain. For the IE prototype device, the built-in contour map generator did not allow the cracks to be clearly visible although some interference was observed. After some further analysis and the use of a third-party application, it was found that the cracks did have a noticeable effect on the results and their locations could be detected using the IE technique.

For this type of analysis, the contour map generated should be able to account for thickness changes much smaller than the original scaling of 50 mm. The reason is that the smaller the defect, the smaller the impact on the readings. In the case of the shallow crack, it was found that the original 50 mm intervals for the colour scale did not allow to locate some of the defects at all while the customized one with a much smaller scale achieved it. The contour map is a tool to understand and present the overall data. The best way to do so is to customize the maps to fit the needs of the current project.

Each device successfully located the hollow conduit. GPR provided the most well-defined image of the conduit in the unreinforced portion of the slab, but did not show a noticeable distinction between it and the regular reinforcement bars in the reinforced portion. Based on the results, it seems that narrow objects such as conduits may be difficult to identify when installed parallel to the reinforcing bars. For UPE, the conduit appeared as an abnormality in measurements indicating the presence of either an object or flaw in the middle of the slab that was poorly defined. Additionally, it approximated the reflecting interface at a depth of 160 mm for multiple points which is close to the expected value as the conduit was installed at 150 mm depth center and had a diameter of 50 mm. For the IE prototype device, the general location of the conduit was found by examination of the contour map due to a shift in the peak frequency, and after conducting secondary tests using a smaller impactor directly over the conduit, the depth was accurately determined.

Finally, for the voids, all three methods were successful at locating the defects. The GPR method showed a strong result and the capability to characterize the geometry of each of the flaws with good accuracy. Since the method allows for the generation of internal maps at different depths, it is possible to locate the flaws and estimate their thickness fairly precisely using this method. For UPE, it was able to locate both voids to some degree although the results were fairly imprecise. Using the IE prototype device, all defects were properly found. The main issue encountered is that it required additional testing and analysis to achieve good results. Furthermore, clear interpretation of the results requires some post-processing to compile the results together and identify trends beyond the simple thickness estimate associated with the frequency with the highest amplitude. The Impact Echo device offers a very strong tool to properly identify cracks, conduits, and voids, but requires the user to have a good understanding of the theory behind the device to interpret the results, and may require secondary testing with different impactor sizes to achieve a clear result.

4.3 SLAB 3 DELAMINATION

The third slab contained artificial delamination layers installed at different depths and of various sizes. Figure 4.32 illustrates the grid used for the measurement locations as well as the points of interest in the slab.

Figure 4.32 Slab 3 Grid View

To recall, the flaws #3, 5, 6 and 7 were damaged or destroyed due to errors during casting. The flaws #3, 5 and 6 were replaced with thicker foam, but the exact locations and depth are unreliable. Otherwise, the flaws are as described in Figure 4.32 and the area around the flaws #1, 2 and 3 is reinforced.

4.3.1 Slab 3 GPR

GPR was the first method used on the slab to determine the location and size of the defects. Figure 4.33 shows some of the results from testing with this device. The device generates many maps which are either repeating the same information or showing no point of interest. For the sake of simplicity, only four maps were selected. For this analysis, the first clear flaw started appearing at a depth of 0.075 m (a). The following maps were simply taken every 50 mm past that point to represent the full depth where it was expected to find flaws, which are 0.125 m (b), 0.175 m (c) and 0.225 m (d). The last portion of the slab from 0.225 m to 0.300 m did not provide any additional information, therefore is not presented.

Figure 4.33 Slab 3 GPR results at 0.075 m (a), 0.125 m (b), 0.175 m (c), and 0.225 m (d)

As observed in Figure 4.33, most of the defects in the slab were detected even though they are poorly defined. The delaminations #1, 2, 4, and 7 (refer to Figure 4.32) were not visible on any of the maps generated. The first reason is that the thickness of the foam used was 7 mm which makes them quite thin and difficult to locate. The second reason why they did not appear is the presence of reinforcement on the left side over the flaw #1, 2, and 3. As discussed earlier in this chapter, the reinforcement often hides what is underneath since they are the first interaction with the electromagnetic waves generated by the device.

For the flaw #2, there is a reoccurring color appearing at the correct locations, but it is difficult to distinguish from the reinforcing bars. In general, reinforced concrete slabs would be fully reinforced, which indicate the method may have difficulty finding this type of defect when it occurs below a reinforcement layer.

Defect #4 was 7 mm thick and also not clearly visible. This delamination did not have reinforcing bars above it and does not have any clear reasons as to why it did not appear on any maps other than its small

thickness. The depth of the flaw was at 110 mm from the surface which implies that the map shown in Figure 4.26b is directly underneath the flaw. There are signs of something in the vicinity where the delamination should be, but it does not look like the defect expected. The foam layer was held in place by some small steel rods which seem to be what the device captures at the location where defect #4 should have appeared. Flaw #7 can be ignored as it was destroyed and removed during the casting due to some errors and was not replaced due to its small size and other constraints during the casting.

Several of the flaws were successfully located; defect #3 at the bottom left side was clearly visible on multiple maps. One explanation for the improved result is that the original 7 mm thick foam layer that was severely damaged during the casting was removed and replaced with a 15 mm thick foam of the same dimensions. The larger thickness causes it to have a stronger effect on the readings making it visible even through the layer of steel reinforcement. Similar results were obtained for defects #5 and 6 which were also replaced during the casting and resulted in a much clearer appearance on the readings.

The other visible defects include defects #8 and 9 on the right side. Flaw #8 was a 150 by 300 mm rectangular foam piece of 7 mm thickness installed at a depth of 145 mm. It is not clearly evident why this one appeared on the maps while flaw #4 which is larger with the same thickness did not. Lastly, flaw #9 also showed relatively clearly on the map. This defect was 7 mm thick but covered a larger area than the other artificial flaws with an area of 365 mm by 365 mm. The flaws #8 and 9 were installed at a depth of 145 mm below the surface and started slightly appearing in the map at a depth of 150 mm and more clearly at a depth of 175 mm presented in Figure 4.33c. The depth of 175 mm is directly underneath the flaws and therefore it is logical that they show a stronger effect. Hence, the GPR method was accurate at finding both the locations and depths for these defects.

4.3.2 Slab 3 UPE

The second NDT used on Slab 3 was the UPE method. Figure 4.34 presents a contour map representing the closest reflecting interface at each measurement point.

Figure 4.34 Slab 3 UPE results

This method correctly found the locations for all of the defects. The measurement points on the grid were spaced closely enough to accurately find the locations for most of the flaws, although their boundaries are not very well-defined. The depths of each delamination layer seem to also be correctly determined through the analysis, but similarly to the second slab the map is not precise enough to fully understand the depths. The 50 mm interval used for each colour in the legend is quite large for this type of analysis. This results in a very qualitative result when looking only at the map. The data extracted from the device allows the user to get more precision on the actual measured depth. The map generated by the commercial system shows the approximate locations of the defects but does not precisely identify their depth. The measurements taken adjacent to the flaws also cause some distortion in the reading which leads to poor definition of the shape of each delamination layer. Overall, the results were satisfactory, but a more accurate analysis based on the raw data using external software may be justified depending on the details of the project and intended application.

4.3.3 Slab 3 Impact Echo

Lastly the IE prototype device was used to locate the defects on the slab. The first full analysis of the slab was conducted with a 20 mm impactor. The results overall are quite good, but it was not possible to locate the depth of the defects precisely. Figure 4.35 shows the contour map made from an external software based on the data collected with the Impact Echo method.

Figure 4.35 Slab 3 Impact Echo results

The figure above shows that the method managed to locate all of the artificial defects which appeared as an increase in the apparent thickness of the slab. The delamination layers caused either a shift in the main frequency peak or an additional low frequency flexural vibration peak. Those abnormal results caused an overestimation of the depth that is observed on the contour map. At the location of the delamination, the map shows thickness estimates over 400 mm which exceeds the actual slab thickness. This may be considered as an indication of the presence of a defect at that point, in this case the delamination. An Impact Echo analysis using a single impactor size is good enough to locate the points presenting a problem and determine the general location of the flaws. However, to estimate the depth, a peak at higher frequency is required. For this reason, a secondary analysis using smaller impactors is required. The smaller impactor has a stronger effect at higher frequency which translates to lower thickness values. This should in theory allow to estimate the depth most of the time. Based on theory, flaws with a small size to depth ratio ($d/T < 1/4$) would not be possible to measure, but the other situations should be (Carino,

2015). In this case the optimal testing method would be to do a primary analysis using an impactor suitable for the full thickness of the tested element. This allows to locate the general areas affected by flaws which can then be retested using a slightly different set up. The secondary analysis should be done with a much smaller impactor to capture possible peaks at higher frequency. This secondary analysis should provide more accuracy on the size and depth of the flaw.

Figure 4.36 presents a compilation of the results using the 20 mm impactor over flaw #9 which had the largest dimensions and was located near the mid-depth of the slab (Figure 4.32). In total, 10 measurement points were located above this defect.

Figure 4.36 Slab 3 Impact Echo Delamination 9 Using 20 mm Impactor

An interesting thing to note about the data is that some of the points had multiple peaks while others had a single one. There are three main clusters of peaks appearing in the result. The first peak occurs around 4.2 kHz and was likely caused by flexural vibrations. This peak was consistent with every point and was often strong enough to hide the thickness frequency. Similar reasoning may be used for the other peaks appearing at 2.7 kHz. It was determined earlier in this chapter that those types of peaks are not useful and should simply be ignored for the analysis.

The interesting results are the thickness peaks that ranged from 5.2 kHz to 6.0 kHz in this case for delamination #9. The reflection of the bottom of the slab should, in theory, occur at a frequency of 6.4 kHz, as discussed for Slab 1. This shift most likely comes from the loss of stiffness of the slab at that point due to the flaw in addition to diffractions of the wave going around the flaw to reach the bottom. The

stress wave does not get affected by the flaw exactly the same way at every location tested resulting in the varying frequencies for the thickness peaks.

So far, the primary analysis using the 20 mm impactor was only good at providing an idea of the general location and size of the flaw. To improve the quality of the data and estimate the depth, a second series of tests were conducted over the delamination #9 using a 10 mm impactor. The primary test was done with a 20 mm impactor which is good up to a maximum frequency of 12 kHz (173 mm or more). Since the defect was installed at 145 mm it was not well-captured, but it was expected to be detected using a 10 mm impactor more suitable for higher frequencies up to 23 kHz (90 mm or more). Figure 4.38 shows the obtained results from the tests using a smaller impactor.

Figure 4.37 Slab 3 Impact Echo Delamination 9 Using 10 mm Impactor

This graph contains many similarities with the first one presented. First, the majority of the points tested have a peak at a frequency of 4.2 kHz which was the main visible peak in the previous graph likely caused by flexural vibrations. There are also other peaks at 2.7 kHz frequently appearing, but both of those peaks were judged unusable for the analysis and will be ignored. After that peak, multiple peaks appear between 5.2 kHz and 6.2 kHz representing the reflections from the bottom of the slab as seen in the previous analysis. The new detail from this secondary analysis is the presence of a new, much smaller peak that occurred at frequencies between 10 kHz and 11 kHz. This peak corresponds to a reflection interface at a depth between 207 mm and 188 mm using a velocity of 4319 m/s which was determined as the velocity in sound concrete. However, since the full thickness peaks were found to be slightly shifted toward lower frequencies, it may be assumed that the average stiffness of the concrete at that location was reduced

due to the presence of the flaw reducing the average wave velocity. Considering that most of the points reflected the bottom interface at a frequency of 5.2 kHz, the average velocity could be recalculated as 3250 m/s:

$$V_p = h * \frac{2f}{0.96} = 0.300 * \frac{2 * 5198}{0.96} = 3250 \text{ m/s}$$

$$h = \frac{0.96(3250)}{2 * 10150} = 0.154 \text{ m}$$

$$h = \frac{0.96(3250)}{2 * 11140} = 0.140 \text{ m}$$

Using that velocity, the third peak would represent a reflecting interface between 140 mm and 154 mm which is accurate since the flaw was installed at 145 mm.

4.3.4 Slab 3 Summary

In summary for the last slab, most methods used were successful at finding the artificial delamination defects to some degree. The first method used, GPR, found most of them, but had difficulties. It accurately found the location, shape, and size of most of the flaws. However, in the reinforced section, only 1 out of 3 defects was located. Earlier in this chapter, it was determined that the reinforcements act as an obstacle when testing at larger depths. The steel bars intercept the electromagnetic waves early and possibly act to hide flaws, objects, or other anomalies that could be underneath. This seems to be the case once again for this type of flaw. The method successfully found the accurate location, size, and depth for two of the defects in the unreinforced section while completely missing another one. The results may be able to accurately describe a flaw, but it is also possible that it completely misses it (especially for thin layers). This is even more problematic for reinforced sections and since the vast majority of real applications of the method will be on reinforced elements, this method is somewhat unreliable.

The second method tested was UPE. This method successfully found the location of all of the flaws. However, accuracy was a problem because even though it located all of the areas affected by the flaws, the shapes of the flaws were distorted. The contour map made from the results showed odd shapes instead of squares and rectangles. This resulted in many false positive readings on the map. Furthermore, the depths of those defects were wrong. The map showed all of the flaws at depths between 150 and 250 mm with the lowest depth at 161 mm (based on the raw data). This is incorrect since most of the artificial delamination layers were installed higher than that. The top ones (#1 and 4) were located at depths of 110 mm and 120 mm, respectively, while the ones on the left side (#8 and 9) were installed at 145 mm. It

is not a large difference in depth, but it demonstrates that the method failed to capture the difference between them.

Lastly, the IE method was used. Similarly to the UPE method, the locations of the defects were found, but the shape of the flaws were not clearly defined. Most of the flaws showed as distorted shapes because sometimes the measurement close to a flaw would be affected while in other cases measurements directly over the flaw were unclear. Even with that issue, the general location of the flaw was still reasonably easy to find. As seen in the analysis for the delamination 9, several unwanted peaks appeared in the results when testing over the flaw and none of them represent the depth of the flaw. To properly estimate the depth, a secondary analysis using a smaller impactor is required. As mentioned in the literature review, the size of the impactor correlates to its impact time. The lower the impact time, the better the result at high frequency range (lower depth). This induces more noise making the interpretation more difficult sometimes. Therefore, it is not suggested to use small impactors everywhere. Using a secondary analysis with the smaller impactor revealed a peak around 11 kHz representing the correct depth of the flaw. In conclusion for this slab, the IE method resulted in the best result of the three methods tested. However, similarly to the other slabs *tested*, the higher accuracy of the test came at the cost of time. The test itself was longer and the analysis afterward also required more work than the other methods.

4.4 COMPARISON

This chapter presented the results from three NDT methods for the condition assessment of reinforced concrete slabs. They were tested for their ability to accurately locate and describe objects, physical flaws or element characteristics often investigated in reinforced concrete elements. The key feature of those points of interest was the presence of air in them because of the primary objective of developing a new wireless IE system in this project. The experiment was therefore designed around the IE technique and its target applications, and hence the results are a bit biased toward it. The goal was to test the device for what it was designed to accomplish and compare it to other tools that can possibly serve the same purpose.

The first method tested was not designed specifically with finding physical defects in mind. The Ground Penetrating Radar is designed around the use of electromagnetic waves to locate abnormalities inside the reinforced concrete element. These abnormalities can include either objects like the conduit or the reinforcing steel, or deterioration mechanisms simulated by the artificial flaws created for this project.

One of the main strengths of this method is the speed of testing. This test is simple, as the device is dragged along the lines of the grid, and it sends/receives the waves instantly. After a few minutes, the test was done, and the results were compiled into contour maps in real time. This is a large advantage for the usability of the device compared to the other two. Furthermore, it allows the user to investigate features at specific depths and locate the reinforcing steel which is impossible using the other devices. For the selected points of interest of this project, this method was good only under certain conditions. The type and size of flaws it can detect is limited. It was consistently observed that only the larger flaws appeared while the smaller ones were unreliable. The device had no problem locating the conduit, the voids and the larger delamination. However, it was unable to find most of the smaller or thinner delamination defects and completely failed to locate the cracks. Using another approach to deal with the cracks, this method can still be considered adequate for most analyses since it finds the more severe defects. This method also provides information not available using other methods. First, as seen earlier, the ability to test at any depth allows the user to estimate the thickness of a defect while the other methods were only able to locate the closest reflecting interface. In addition, this method strongly interacts with the reinforcing steel which is different from the other two. This allows the user to accurately locate reinforcing bars and assess their condition.

The second method used was Ultrasonic Pulse Echo. This system is designed around stress waves and is quite similar to Impact Echo. For this reason, the strength and weaknesses of the method are similar. For the usability, this test required more time to complete than the GPR Analysis, but still much less than Impact Echo. The device was quite easy to set up, it requires simple information to calibrate and get decent results. It also comes with a contour map generator which allows the user to see the overall results in real time. This is essential to have for this type of tool nowadays. However, this map generator had limited options for customization. As seen and discussed earlier, the map from the UPE device often failed to properly locate the flaws simply due to the coarse scaling used. A denser grid is also suggested if more precision is desired. This would result in a longer analysis but would have better definition of the flaws. In a similar manner as Impact Echo, the data generated by the method is point based and can be used for further analysis of the results. It can be used with an alternative software to get a better contour map if desired or compiled and analyzed directly. For the accuracy of the method, it was highly variable. Most of the defects were found, but the area they affected was much larger than the actual flaw most of the time. On the same topic, the presence of reinforcing steel seems to have a negative impact. The results from reinforced sections demonstrated much more false readings than the unreinforced regions. This considerably reduces the reliability of the method since most structures are reinforced. It results in a map

that considerably overestimated the area affected by flaws and makes estimating the actual affected area difficult. Furthermore, this method has a disadvantage toward the amount of information it can get from a single location since UPE can only get a single value for any point tested. The way UPE works is that it finds the location of the first reflecting interface the wave interacts with. This limits greatly the viability of the method with larger concrete elements or elements with more than one layer of delamination. The first method allowed to map any depth desired while the third allows to locate any reflecting interface through stronger analysis; hence, this presents a key weakness of the UPE system.

However, this method was not entirely negative. Measuring the depth of the slab was successful even with the irregularities due to the reinforcing steel; those anomalies could possibly be reduced if the points are retested when abnormal results appear to confirm the measurement. For the second slab, it was able to locate all of the defects on the contour map. The cracks were less defined on the map, but the depth could be estimated using an alternative method. The depth of those cracks was measured using a similar approach as the one suggested for Impact Echo, i.e. installing a transducer on each side of the crack and generating a wave. The wave travels to the other side using the shortest path which allows the user to estimate its depth. For this slab, the method was quite accurate at estimating the depth of the cracks with 138 mm and 208 mm for artificial cracks having actual depths of 100 mm and 200 mm, respectively. Overall, based on the result obtained through this project, UPE is a good tool for a general analysis of a reinforcing concrete element. It allows to locate any type of physical deterioration mechanism or objects buried in the element to some degree. It does seem to overestimate the area affected, but it still gives a general idea of the presence of flaws or not.

The last method tested, and main subject of this project was Impact Echo. The device tested was recently designed and the model used was a prototype. It was mostly completed with the exception of a strong contour map generator and a secondary sensor to use when measuring the depth of the cracks. For the usability of the method, the testing process is tedious and takes a much longer time to complete compared to the other two. Testing with this method is not as straightforward as the other methods. It required the user to have a good knowledge on the method prior to the testing, otherwise the results or the interpretation can be incorrect or misleading. This is because the results are difficult to interpret, especially when multiple peaks appear, the degree of noise is highly variable, and the results are strongly affected by any object, boundaries, change of concrete strength or any other irregularities that can be present in concrete. This makes the method very sensitive to any problems and makes the interpretation of the results difficult. It is often impossible to say with certainty which type of defect is present based

only on the contour map. The map provides an idea of the overall shape of the affected area and allows to estimate the severity of the deterioration, but it does not explicitly indicate a type of defect. Many types of results were explored through this project. Vertical reflecting interfaces such as boundaries or vertical cracks caused a shift in the thickness frequency measured resulting in an overestimation of the results, the deeper the crack, the larger the effect. The horizontal reflecting interfaces were the conduit, voids and delamination which all had similar type of results and difficult to differentiate from one another. The results indicate that the conduit could be identified due to a smaller surface area and the difficulty to capture reflections from the object itself, the voids had strong reflections causing purely flexural vibrations, and the delamination had larger surface area and weaker reflections. That would be true for this project, but in another case, a larger conduit, multiple smaller voids or severe delamination would likely have different characteristics. They would still act as horizontal reflecting interfaces, but not necessarily in exactly the same way as they did for this project.

The methodology is also a potential source of wrong results. It requires a clean surface and a greasy substance to improve contact, and the device should be nearly perfectly perpendicular to the surface. Even with those conditions satisfied, all tested points still require multiple attempts to get decent results making it essential to have an idea of the expected result when testing. The repeatability of the method is also affected because there are many things that can influence the readings. The sensitivity of the method and the overall procedure cause the speed of the analysis to be much slower than the other two methods. The last testing session with the final prototype had an average speed of 2.5 points per minute. It seems good, but considering there were 1200 points on the 3 grids, it is still much longer than the other methods.

Furthermore, the results from this method were less affected by the presence of steel reinforcements. For GPR, in some cases the reinforcement would hide the presence of the points of interest. For UPE, they would cause the method to have many false readings. For Impact Echo, it only caused a slight loss of wave velocity that would most likely be avoided completely when calibrating the equipment over an undamaged area of reinforced concrete.

For the data analysis, the Impact Echo results had more depth to them than the other two. The contour maps were built around the main reflecting interface as a single point. However, there is more to the data than the contour map. Using more in-depth analysis of the data and a secondary testing phase using a different impactor, it was possible to determine more accurately the locations of the points of interest and potentially their depth as well. This makes the Impact Echo analysis more tedious, but also much more

rewarding. This method has the potential to achieve much better results than the other options tested if time isn't an important factor. Another solution would be to use a quick method for the general assessment and the IE method to refine the results where flaws are found using various impactors and a denser grid. This would allow the user to get the good details from IE while limiting the time spent testing sound concrete.

Table 4.3 shows a summary of the strengths and weaknesses of the various methods with respect to the various points of interest for this project.

Table 4.3 Summary of the Results

Point of interest	GPR	UPE	IE
Thickness measurement	Possible but unreliable (depending on obstructions)	Accurate measurements	Accurate measurements
Effect of Boundaries	No interaction	Negative influence	Severe negative influence
Effect of Steel Reinforcements	Easy to locate but may also hide other points of interest	Negative influence	Negligible influence
Vertical Cracks	No interaction	Difficult to locate and depth can be measured	Possible to locate but cannot measure the depth with single sensor
Presence of Conduit	Possible to locate and hard to differentiate with reinforcements	Possible to locate and shape was badly defined	Easy to locate and possible to estimate the depth
Artificial Voids	Easy to locate and possible to estimate the dimensions	Easy to locate	Easy to locate
Artificial Delamination	Possible to locate more severe flaws but missed the smaller ones	Easy to locate but depth was inaccurate	Easy to locate and depth can be estimated

5 CONCLUSIONS

5.1 CONCLUSIONS

In conclusion, the new Impact Echo device was tested on the three reinforced concrete slabs with great success. The primary objective of contributing to the development of an improved IE device and software through extensive testing, providing feedback on performance, and frequent interactions with the developers was mostly completed. The device evolved from an initial prototype that was unreliable and difficult to set up to a functional and user-friendly tool for non-destructive condition assessments of concrete structures. Most of the requested features were added in some form by the developers. The device is now handheld and sturdy enough to resist extensive testing periods while the sensor now has a strong support to hold it in place perpendicularly. There are still some minor improvements being finalized for the software like upgrading the contour map generator and removing small bugs, but it is working well. For the hardware, the addition of a secondary sensor to allow for estimating crack depths was planned, but was not yet added by the end of this project.

The comparison of Impact Echo to other NDT systems was also a success. The first characteristic, the speed of the test and ease of use, was simple to compare while doing the tests. The quickest method was GPR, followed closely by UPE, which both took 1 to 2 hours to test and analyze the data. For Impact Echo however, it took much longer to take the measurements and perform the analysis that goes with it. The main reasons are that the tests are done one point at a time and that each measurement requires multiple steps. The final testing speed after several months of experience was about 2.5 points per minute, which represents 8 hours of work to test the 1200 points present on the test specimens. In addition, some post-processing of the data was required to gather a comprehensive understanding of the readings with multiple peaks.

The second characteristic looked at was the accuracy of the methods. All methods proved to be fairly accurate considering NDTs in general are always inducing some errors because of their indirect approach. The GPR test sometimes resulted in very good definition of the flaws and objects that were shallow and relatively large, yet it also completely missed others. The results from the test also seemed to have problems caused by the presence of reinforcing steel. Because of that, the results were not completely accurate. One advantage this method has over the others is that it is not limited to discrete measurement points, but rather takes continuous measurements as it is moved along linear segments. UPE and Impact

Echo are both point-based and can only test directly beneath the equipment at discrete locations. This is the cause of the higher definition of the flaw shapes using the GPR method. The second method, UPE, was able to locate all of the points of interest, but struggled to define them properly. The shapes were difficult to identify and the flaws seemed to be much larger and displaced. Lastly, Impact Echo was quite accurate for a first analysis considering the depth of the slab. However, the depth of the flaws themselves could not possibly be found until a secondary analysis was done using a smaller impactor. Their general location and shape were defined by the points causing abnormal thickness measurements. The definition and locations of the flaws was better than UPE, but the methods suffer the same problem concerning their point-based approach. The results and analysis are therefore heavily dependent on the spacing of the grid used.

The precision of the GPR and UPE systems could not be judged as the tests were not repeated multiple times. For Impact Echo, the precision and repeatability of the prototype were relatively poor because the device is highly sensitive. The results were easily affected by the uneven surface or the presence of dirt, and readings can be compromised if the sensor is not stable or perfectly perpendicular to the surface. Even if everything is done correctly, the waves could be distorted by nearby boundaries. These potential discrepancies are the major reason why it takes much longer to test using Impact Echo. High noise or unexpected results are a frequent occurrence which cause every point to be tested multiple times to confirm or discard the first readings.

In conclusion, based on the results of this study, Impact Echo is not recommended for general use over a large area. The higher accuracy of the method and the ability to review the data further through secondary analysis is interesting and useful, but the time and effort required to obtain high quality results may not be practical or cost-effective for most applications. The first method used, GPR, was able to locate the more severe flaws within a fraction of the time. To achieve a similar testing speed, the number of points would have to be reduced significantly. This would result in a much larger spacing in the grid and since the method is designed as a point-by-point method, the quality of the generated contour map and the ability of the device to locate smaller flaws would be greatly reduced. For these reasons, Impact Echo is primarily recommended as a great tool for small areas requiring more information. The suggested testing plan would be to use a quicker method such as GPR to generate an overall map of the area and follow up with Impact Echo to refine the results in select areas where flaws were detected.

5.2 FUTURE WORK

There are many things that could have been added to this experiment. First of all, there were other methods that were intended to be included for comparison but were not completed due to time limitations and limited access to the laboratory related to the COVID-19 pandemic. Also, as mentioned earlier, the test specimens were designed with Impact Echo in mind. The tested points of interest were selected because they are things the IE method is expected to find. Therefore, there are advantages to the other methods that were not displayed in these tests. Corrosion is an example of a common deterioration process that was not investigated. It has barely any effects on Impact Echo or UPE until they expand and crack the material resulting in air around the reinforcement. In this case, GPR, which interacts with the reinforcement directly would be expected to perform much better at finding this type of defect. This could have been a good addition to the project, but causing predictable corrosion is a challenge, and three slabs were already planned. Another common situation not investigated is the presence of water in the pores and in the defects. The methods considered do not interact with water the same way as with air, and therefore it could have affected the IE results greatly.

There are also multiple upgrades or alternative methods that could have been added and tested with the device. For example, testing using the air-coupled sensor approach which removes issues with the surface preparation, but introduces other limitations. Testing of an upgraded prototype with two sensors could also enable measurements of perpendicular crack depths. Finally, it is also recommended to conduct field trials to validate the IE system for real structures containing naturally occurring defects outside of a controlled environment.

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