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Assessing Deteriorated Concrete Structures

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ASSESSING DETERIORATED CONCRETE STRUCTURES

by

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B.A.Sc.

**University of Toronto
Toronto, Ontario, Canada
2009**

A project
presented to Ryerson University

in partial fulfillment of the
requirements for the degree of

Master of Engineering

in the Program of

Civil Engineering

Toronto, Ontario, Canada, 2010

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ABSTRACT:

Assessing Deteriorated Concrete Structures

Master of Engineering, 2010

Joseph Greto

Civil Engineering

Ryerson University

In today's society, concrete structures are deteriorating for a variety of reasons. In order to properly repair these structures, it is important to completely understand the root cause of each type of deterioration. Over the years, engineers have developed methods for identifying the causes of concrete failure. This paper recognizes the different forms of concrete deterioration, identifies the test methods which have been developed to locate these concrete defects (both non-destructive and destructive), reviews different case studies which have been performed on concrete parking structures implementing these test methods and draws conclusions from surveys which were conducted of professionals in the rehabilitation engineering field. Additionally, this research project develops a strategy which is meant to aid with the selection of concrete test methods to be used in diverse concrete deterioration situations.

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1.0 INTRODUCTION

1.0 Introduction

In today's society, tearing down and rebuilding concrete structures may not at all times be a feasible option. In order to demolish and replace an entire concrete structure, there is, to a great extent, a higher cost to the owner than restoring the existing structure. For this reason, numerous clients or entities are choosing to restore structures rather than replace them. To repair these concrete structures, the diverse forms of deterioration present must be identified to ensure that the correct repair approach may be performed. A lack of maintenance or testing for deterioration in concrete structures may result in disaster. Concrete deterioration, which should be identified, can be classified under three general headings: 1. *Visual Deterioration*, 2. *Loss of Strength*, and 3. *Hidden Flaws*. Condition surveys are the primary mode for discovery of these deteriorates.

Over the years, engineers have developed a variety of concrete testing methods for each of the different forms of concrete deterioration. It is important for engineers to understand which testing method would best identify the cause of the concrete deterioration in order for them to be able to recommend the optimal repair process. Though there are a multitude of methods to utilize to test the concrete, selecting the optimal method is not always an effortless decision. The objective of this research paper is to aid practitioners in the field of rehabilitation engineering with that decision and to familiarize the reader with present day practices exemplified through case studies.

This project will present a literature review of the different modes of deterioration, familiarize the reader with the different concrete testing methods which have been developed, examine two (2) separate concrete structure case studies and will review surveys which have been filled out by professionals in the rehabilitation engineering field. Once the aforementioned details have been discussed, a strategy will be developed to help engineers decide which test methods should be used in different situations. This information will be presented in the form of a flow chart.

1.1 Condition Survey

A condition survey provides an assessment of the physical concrete situation. It may fluctuate depending on the client's need of information and budget constraints. The results from the survey are recorded and then formulated into a report. Multiple surveys may be conducted to identify the different types of deterioration present in the structure under review. These surveys can be categorized as either: *Visual Surveys*, *Test Pits*, *Sounding Surveys*, and *Non-Destructive or Destructive Tests*. In spite of this, condition surveys do not necessarily result in preservation. These surveys may be used as tools to prove the structure is stable and safe, or it may confirm that the structure is in need of repair or demolition.

1.2 Structures

Condition surveys may be carried out on many different types of concrete structures. This paper will exemplify defects found and methods used on a variety of structures including, bridges, parking structures, and balconies. The case studies conducted in this paper will focus on how a condition survey is carried out for a *Parking Structure*. A parking structure is a building that is designed for multi car parking. There are two different types of parking structures: *Above Ground Parking Structures* and *Below Ground Parking Structures*. Above ground parking structures are exposed to the bare elements and have open side walls, whereas below ground parking structures are exposed to soil stresses which are applied to its closed walls. All parking structures are also exposed to heavy and shifting loads presented by moving vehicles.

1.3 Case Study - Condition Survey (7 King Street East, Toronto, Ontario)

An actual Condition Survey was conducted at 7 King St East in Toronto, Ontario. This survey was conducted in order to identify the scope of the work that would need to be carried out. Accordingly, the owner could finalize any decisions with regards to repairing the structure. Details of the assessment will be discussed later in **Section 3.1** this report.

1.4 Case Study - Detailed Construction Process (530 Lolita Gardens, Mississauga, Ontario)

When repairing a deteriorated concrete structure, there is a procedure that must be followed. Alone, the Condition Survey will not repair the structure. In order to better understand the entire construction process from initiation to the closing stages, a case study has been conducted for the parking structure rehabilitation at 530 Lolita Gardens in Mississauga, Ontario. Details of the case study assessment will be discussed later in **Section 4.1** of this report.

2.0 Literature Review:

Deterioration Modes and Assessment Methods

2.0 Literature Review - Deterioration Modes and Assessment Methods

2.1 Modes of Deterioration

2.1.1 Visual Defects

One of the simplest ways to understand what exactly causes concrete to fail would be performing a Visual Survey of the deteriorated concrete structure. When a client hires an Engineer to inspect a concrete structure, it is most likely because the owner visually noted some form of deterioration. This, technically, may be considered a preliminary visual test.

As one will later see, there are many different tests that are done to analyze a deteriorated structure. Usually, the first test an Engineer (or Inspector) will conduct is a Visual Condition Survey so as to get a sense of where, if any, additional testing should take place. This survey usually consists of the Engineer walking the site looking for any signs of deterioration that can be noticed visually and then noting them on the drawings or data report paper (See **Figure 1** below). The Engineer makes notes of both serious and non-serious defects to the concrete structure. In some cases, in view of the findings from the Engineer, the amount of work required to repair the structure may be determined. The more serious modes of deterioration that are found visually usually lead to further expensive investigation and repair.

Table 4.3 Some sources of defects in concrete structures

<i>Construction-related</i>	<i>In-service related</i>
Low cover	Weathering
Variable compaction/curing	Abrasion
Blow holes/voids	Erosion/cavitation
Honeycombing	Impact damage
Excessive bleeding	Joint leakage
Grout loss	Surface scaling
Construction joints	Leaching
Exposed spacers/tying wire	Exudations/surface defects

Figure 1 - Visually noticeable signs of deterioration

When the visual test is conducted by the engineer during the Condition Survey it is for the purpose of analyzing the visually noticeable deteriorations in the concrete structure. Some of these deterioration mechanisms are discussed below in more detail.

2.1.1.1 Scaling

Scaling is a form of disintegration and is due in most part to freeze-thaw reactions which cause the concrete to flake or peel off at the exposed surface (See **Figure 2 - Scaling** Left). Scaling is indicative of poor air-void characteristics of the concrete. This form of deterioration is detected through a Visual Condition Survey. The

passerby will first notice scattered small patchy sections of peeling or flaking surface. If this is not attended to, it will continue to grow and possibly cover a very large area. Scaling severity is rated on the basis of the depth of its defect. These different severities are categorized by way of visual analysis. '*Light*' scaling does not expose any coarse aggregate, '*Moderate*' scaling is approximately up to 0.4 inches (or 3 to 10 mm) in depth from the surface and exposes some coarse aggregates, '*Severe*' scaling has an even larger depth of flaking in which the coarse aggregates become very noticeable and there also may be some mortar loss surrounding the aggregate, and finally, '*Very Severe*' scaling is the loss of coarse aggregate particles, as well as surface mortar and surrounding aggregate (it generally is greater than 0.8 inches in depth) (Woodson, 2009 & NRMCA).

2.1.1.2 Spall

A spall occurs when large chunks of concrete break off of a concrete structure. The primary cause of spalling is the corrosion of the rebar within the concrete. The products of steel corrosion which are formed are expansive and thus, take up additional volume causing the concrete to be placed under stress, leading to cracks and eventual

spalls due to concrete breaking open and exposing the corroded rebar (See **Figure 3 Below**) (Neville, 1995). When a spall is noted during a visual survey, the rebar will usually be noticeable by the naked eye. Another cause for a spall is through the process of freezing and thawing of concrete. Water finds its way into the natural pores of the concrete mixture, which was most likely not designed with air-entraining admixtures. Freezing, an exothermic process occurs, causes the water to expand and the concrete to spall under pressure provided by the water on the surrounding concrete paste. As well, a common cause of spalls in a parking garage is large impacts caused by cars driving into the concrete structure's columns or walls. The force exerted by the cars is often enough to cause concrete to break away from the structure resulting in a spall (DNR, 1999). Usually spalling is common when you work with chert particles of low specific gravity, or with limestone that contains clay, or shale material (Mailvaganm, 1992). As one can imagine, it is not hard to notice when concrete has spalled, in fact, it is quite possibly the most obvious mode of deterioration and all that is required to determine that spalling has taken place is visual confirmation via the Visual Survey.

**Figure 3 - Cracking, Spalling and Delam Due to Reinforcement
Corrosion (Neville, 1995)**

2.1.1.3 Cracking

Cracking is exceptionally frequent in concrete structures. There are different types of cracking that may occur. Cracks are caused by numerous diverse reasons and can be detected by a Visual Analysis. It should be noted that all concrete structures go through the process of expansion and contraction as a result of the fluctuation in surrounding temperature (from night to day). The expansion of the concrete applies a tensile force on the concrete; once the force is beyond the cracking tension of the concrete, the concrete will crack. Another cause of cracking is the flex of the concrete members in different directions due to applied dead and live loads. Basically, if the Design Engineer does not account for these movements in his/her design of the concrete members (i.e. designing with control joints (telling the concrete where to crack)),

cracking will be inevitable (NRMCA). There are different types of cracks, and they can be categorized by their varying depths: on the surface, shallow depth, deep crack and a through crack. To determine the width of a crack there are three categories: a fine crack (less than 0.04 inches), a medium crack (0.04 to 0.08 inches) and a wide crack (greater than 0.08 inches). Surface cracks detected in the visual survey are usually categorized into two ways, '*Hairline*' cracks and '*Structural*' cracks. Hairline cracks are as the name depicts, fine as a piece of hair. Hairline fractures are only a few millimetres in width and depth and do not require immediate repair for there are no consequences to this natural flaw. The difficulty starts when hairline cracks turn into structural cracks. Structural cracks are normally wider than a quarter inch, and if they are not tended to in the very near future, they may affect the structural load capabilities of the concrete (DNR, 1999). During a Visual Condition Survey, if a thin, non-linear separation of the concrete is detected, it is a surface crack. Some cracks lead to delamination or even eventually lead to spall (See **Figure 3** Above); in order to be sure of the current state of the concrete at each surface crack that is detected, further testing (sound testing) should be performed (See **Section 2.2.3.1: Sounding Survey**).

2.1.1.4 Honeycombing

Honeycombing occurs when there is an improper concrete mix which is usually a short-term deterioration. It occurs when the concrete is poured and not properly consolidated. In order to fix honeycombing you need to remove and replace the concrete (Granite). A visual condition survey will identify areas of honeycombing by observation (DNR, 1999). An example of honeycombing can be seen below.

Figure 4 - Honeycombing

2.1.1.5 Surface Rust Stains

Rust stains are caused by corroded steel below the surface which has inadequate cover, or by corroded steel that is fully exposed due to a spall or poor craftsmanship. The process of corrosion of reinforcement yields oxidation products which expand and tend to flake. The expansion of the oxidation products leads to cracking and decreases the bond the concrete originally had with the rebar. Rust stains are brownish-red in colour and are detected using the visual survey. The identification of rust stains on the surface of a concrete structure is critical because they represent where corroded reinforcement is located below the surface. Ultimately, the process of visually noticing rust staining will save time and money necessary for non-destructive tests to determine the actual location of the corroded steel (Granite).

2.1.1.6 Efflorescence (or Leaching)

Efflorescence is formed as a result of water finding a way into concrete through cracks or openings at the surface, evaporating, and leaving behind salts and other minerals which were picked up on the way into or through the concrete (i.e. water may pick up contaminants from the soil before it reaches the exterior surface of the concrete). Efflorescence is detected by a visual survey and is a non-structural defect. It is located at the surface of concrete structure along cracks and is known for its white essence. The

amount of efflorescence present will distinguish how much water is entering the concrete structure. For this reason, it is important to record and include this defect in the visual survey. In the same manner as the rust stain visual survey, the visual detection of efflorescence helps Engineers with discoveries beyond the surface. In this case, the discovery of a great deal of efflorescence could mean that the concrete is very susceptible to freeze-thaw cycles. Since there are plenty of remnants present from seepage through the concrete, it means that water can easily access the concrete and thus it is easily saturated and subject to freeze-thaw situations (DNR, 1999).

2.1.1.7 Joint Deterioration

There are three types of joints that can be found in concrete structures today. The first are control joints, which are sawcut or tooled to weaken the concrete. Shrinkage

stresses will be directed to these joints. A second type is expansion joints, which provides thermal expansion in the concrete. The third type of joint is a construction joint. This is also known as a 'cold' joint and is caused by the transition between formwork or between concrete pours. In visual

surveys for a parking structure, among all of the different types of joints, expansion joints are of the largest concern for deterioration because they can lead to major leaks within the

structure. Expansion joints allow the concrete slabs/walls to expand and contract without causing the concrete members to shift laterally. A typical parking structure expansion

joint and its make up can be seen in **Figures 5a and 5b - Wabo-crete Joint Configurations (BASF)** above. When the rubber material of the expansion joint is cut or severed, water is easily allowed direct entrance into the joint and can stay there providing easy access into the concrete carrying with it any contaminants it may perhaps in its mixture. The easiest way to detect expansion joint deterioration is to visually inspect it. If the rubber (or other continuous sealing element) is cut, it must be replaced. Also, during a rainfall, if an inspector can see water coming through the expansion joint on the level below; there is a de-bonding issue or the joint is severed in multiple places (Trow).

2.1.2 Loss of Strength

When concrete begins to deteriorate, Engineers become most concerned with its strength. This is very important because once the concrete starts to lose its strength, it may not be able to support the design loads any longer and if there is further deterioration, it may end up having to be entirely replaced because the concrete can no longer support a sufficient load. The Engineer's early detection is able to discover strength loss, and is more likely to devise a strength restoration plan for the concrete structure. If this strength loss is discovered too late, then it is possible that there will be no means for restoration and the structure will have to be redeveloped (torn down and built up again).

2.1.3 Hidden Flaws

2.1.3.1 Delaminations

Delaminations are the cause of many different processes. The first process that may lead to a delamination is exceedingly similar to one discussed above in **Section 2.1.1.2** regarding spall; that is, the corrosion of subsurface steel rebar. As previously stated, the products of the corrosion process take up additional space applying pressure to the surrounding concrete. When the pressure does not cause the concrete to completely pop off (as in spalling), but it only causes the surface concrete to partially pop off, a delamination has been produced (See **Figure 3 Above**) (Neville, 1995).

2.1.3.2 Corrosion

One of the major deterioration mechanisms which can lead to the catastrophic failure of concrete structures is steel reinforcement corrosion. From a thermodynamic point of view, the corrosion of steel is inevitable. Iron ore, the parent of steel reinforcement, has a lower energy state, and naturally, the steel reinforcement wants to revert back to a generally lower energy state (Malhotra & Carino, 2004). The process works as follows: the passive layer on the steel reinforcing bars is broken down by the ingress of chloride ions as well as carbonation. With the passive layer deteriorated, the reinforcing bars become vulnerable to corrosion (Neville, 1995) (See **Figure 6 Below**). In a situation where corrosion is known to exist, it becomes important for Engineers to discover the state of the current corrosion as well as the rate at which the corrosion of the steel reinforcement is occurring; because, both are an integral component of concrete structures. Moreover, this knowledge is fundamental in safeguarding the public especially in situations when the concrete structure is utilized daily, such as in the case of a parking garage. There are many different methods, both direct and indirect, which can determine the extent of corrosion of the steel reinforcement in concrete structures. Three of the more common tests are considered non-destructive tests and include: The Half-cell Potential Test, Concrete Resistivity Test, and The Polarization Resistivity Test. These tests are discussed further in the **Section 2.2.3.3**.

Figure 6 - Corrosion Process (Trow)

2.1.3.3 Carbonation

Carbonation usually forms when carbon dioxide and sulphur dioxide gases penetrate the surface of the concrete and combine with the calcium hydroxide and other alkali hydroxides in the pores of the concrete. Carbonation leads to a reduction in the pH of the cement paste (Malhotra, 2004).

2.1.3.4 Alkali-Aggregate Reaction (AAR)

When aggregates in a concrete mix chemically react with the alkali hydroxides in the concrete cement paste, an expansive product is formed which puts pressure on the surrounding concrete leading to cracking over time (See **Figure 7 - AAR - Right**). AAR is an issue in some concrete structures, and it is important to identify it during any concrete structure condition survey or analysis (Concrete Technology).

2.1.3.5 Voids

The density of concrete directly affects its strength. For this reason, in order to identify the strength of a concrete structure, it is important to identify the concrete density by locating the amount of voids located beneath the surface (Concrete Voids). There are different methods to locate voids beneath the surface of a concrete member, these methods will be discussed in **Section 2.2.3.6** of this paper.

2.1.3.6 Fire Damaged Concrete Structures

Due to its natural incombustibility and low thermal diffusivity which slows the transmittal of sudden thermal changes through the structural members, concrete generally behaves well when exposed to high temperatures (Colombo et al, 2006). For cases such as a long term fire duration or thin member cross-sections, the ability for the concrete members to sustain loads may be weakened (Colombo et al, 2006).

2.2 Detection Methods

2.2.1 Visual

A visual survey is possibly the simplest concrete deterioration detection method available. It is very straight forward, and it returns results that are 100% conclusive. The only problem with the Visual Survey is that it does not apply to deterioration beneath a concrete surface, it is only able to determine surface defects. It is conducted by walking around the site in question with a drawing or a pad of paper and marking down all of the deteriorated areas which are visually noticeable. Once these defects are marked on the drawing, the total area or length of each defect type (i.e. cracks, spall) is tallied up. Using these totals, recommendations are made to the client with some solid evidence in the form of numbers.

2.2.1.1 Test Pits

Test pits can be done either by hand or machine, depending how big of a test pit is needed and how deep one needs to go. Test pits are usually conducted above an underground structure to determine the following: the type and depth of overlying materials, the presence and condition of protection board, the visual quality of concrete surface, performance of a drainage system, and presence and condition of a waterproofing membrane. Test pits are also done to perform a cover meter survey to obtain core samples of the concrete, to do a visual survey of the concrete structure or to perform a corrosion activity survey. Additionally, test pits are used when there is settlement taking place and the concrete needs to be inspected or when foundation walls are not performing well. In this case, a test pit is done behind the wall to determine the cause of the problem (Trow).

2.2.2 Loss of Strength

2.2.2.1 Schmidt Hammer Test

Invented in 1948 by Ernst Schmidt, The Schmidt Hammer Test also known as the sclerometer test is used to measure hardness by measuring the rebound distance resulting from impacting a hardened steel hammer on the concrete with the use of a spring. The hardness value that is obtained with this tool can then be correlated to a strength value. In his book, *Properties of Concrete*, A.M. Neville describes the test as, “a spring-loaded mass has a fixed amount

of energy imparted to it by extending the spring to a fixed position; this is achieved by pressing the plunger against the surface of the concrete

under test. Upon release, the mass rebounds from the plunger, still in contact with the concrete surface, and the distance travelled by the mass, expressed as a percentage of the

initial extension of the spring, is called the rebound number (See **Figure 8 - Schmidt Hammer Test Setup** Above) (Neville, 1995).” Using the rebound number obtained from the test and depending on the aggregate size, using specific graphs, one can define the compressive strength of the concrete. An example of one of the graphs found on page 626 of *Properties of Concrete* by A.M. Neville and is shown below in **Figure 9** (Neville, 1995). Some of the parameters that will directly affect the results are the surface finish of the concrete, the moisture content of the concrete, the temperature, the rigidity of the member, carbonation and the direction of impact.

Figure 9 - Obtaining Compressive Strength indirectly using Rebound Number (Neville, 1995)

2.2.2.2 Pull-out Test

A.M. Neville defines the Pull-out test as one that “measures, by means of a special tension jack, the force required to pull out a previously cast-in metal insert with an enlarged end” (See **Figure 10 - Pull-out test Below**) (Neville, 1995). When the

12.33 Diagrammatic representation of the pull-out test

metal insert is 'pulled out', pieces of concrete will come with it due to its enlarged bottom. The void that is left in the in-situ concrete will be

cone shaped. Using the force required to remove the metal insert from the concrete, Engineers are able to devise the concrete's compressive strength. A graph like the one shown below in **Figure 11**, aids the conversion (Neville, 1995).

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Fig. 12.34 Relation between compressive strength of cores and pull-out force for concrete structures^{12.106}

Figure 11 - Obtaining Compressive Strength using indirectly using Pull-out Force (Neville, 1995)

Though in Neville, Malhotra states that the Pull-out test is better than the Schmidt Hammer test for determining compressive strength because it uses a larger volume and depth of concrete, because of the use of more concrete for the test, more concrete must be replaced when testing is completed. This means that the pull-out test is a more costly test because the cost of repairs must be included. Engineers have become clever since this test was first implemented. Nowadays, tests are conducted for certain pull-out forces which relate to certain compressive strengths, and if the metal insert does not pull out when these forces are applied, the required compressive strength is still available and no permanent damage is done.

2.2.2.3 Windsor Probe Test

The Windsor Probe Test is another concrete strength test that operates with a specially designed gun which drives, "a steel probe into the surface of concrete with a precisely governed explosive charge," (NDT). In other words, it drives a hardened steel alloy rod or probe into the concrete and the extent of penetration of the probe is used as an indicator of the concrete strength. The deeper the probe is able to penetrate the concrete, the less strength the in-situ concrete has. This method for testing concrete strength is very adaptable and it can be used on almost any concrete member, including both flat and curved surfaces. Depending on the type of surface tested, the manner in which the test is carried out differs. For example, when testing on a flat surface, three probes are grouped together and used with a triangular probe-locating template; however, in the case of testing on a curved surface, the three probes are used individually with a single probe-locating template (NDT). NDT James Instruments Inc. defines the specific guidelines as to how and where the Windsor Probe Test should be used as follows:

1. The lowest strength concrete will be found in the top third of a column, flat or suspended slab, or even a molded test cylinder. (NDT)
2. When testing suspended slabs for form or shore removal, test the top side. This is the compression side in the structural element. (NDT)
3. To determine strength adequacy for stressing post-tension strands, test near bulkheads and approximate center of the member." (NDT)
4. Columns or walls should be tested near the top, center, and bottom. The result is usually stated as an average." (NDT)

2.2.2.4 Ultrasonic Pulse Velocity Test

The next non-destructive strength testing method used in concrete structures is the Ultrasonic Pulse Velocity Test (Sutan N.M. & Meganathan M., 2003). It is mainly used to study deterioration, cracking, location of unusually large voids in a hardened concrete mass, and to determine the general condition or strength of concrete structures. It works by transmitting high frequency sound or ultrasonics through the concrete using an

emitting probe and measuring the speed or rate at which these sound waves travel through the concrete material using a receiving probe. The frequency that is usually used ranges from 25 kHz to 150 kHz and is read by piezoelectric transducers. There are three ways that one can measure pulse velocity. One is direct transmission, another is semi-direct transmission, and the third one is surface transmission. The following graph in **Figure 12**, depicts how obtained pulse velocities relate to compressive strength. Once the results are obtained, Engineers are able to calculate the in-situ strength, the elastic modulus and even locate some internal defects such as internal cracking or honeycombing (Sutan N.M. & Meganathan M., 2003).

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Fig. 12.35 Relation between compressive strength and ultrasonic pulse velocity for hardened cement paste, mortar, and concrete, in a dry and a moist condition (based on Fig. 12.62)

Figure 12 - Obtaining Compressive Strength indirectly using Pulse Velocity (Neville, 1995)

A drawback of the test is that sometimes when the surface is rough or uneven, the results may yield unrepresentative results. In a situation like this a liquid gel should be spread over the concrete to create the effect of an even surface and then the readings can be

taken. Also, when there is water present within the cracks of a concrete member, the test behaves poorly. The water prevents the cracks from being detected because it tricks the UPV test into thinking there is solid concrete in this area. In situations like this, when the member being tested has plenty of water filled cracks which may cause the UPV test to fail, alternate tests should be used to confirm the findings.

2.2.2.5 Core Sampling Tests (Semi Destructive) and the Cover Meter Test

In order to understand the properties of the in-situ concrete of a structure, the most common form of sampling is a core sample. These core samples are drilled from in-situ concrete and can be used (i) in the lab to determine the in-situ strength of the structure; (ii) by visually looking at the sample to determine the material makeup of the slab (i.e. is there a concrete topping for sloping to the drains?, etc.); and (iii) in petrographic and chemical tests for characteristic determinations of the concrete itself (i.e. how permeable is it?, etc.) (Bungey et al, 2006).

As mentioned, the purpose of testing core samples, extracted from concrete structures for strength, is to obtain an indication of the actual strength of the structure as it stands. The tests carried out to attain this compressive strength would be the same test conducted for cast-in-place core samples (i.e. with fresh cores made). The major difference between the two tests is the results, and what they represent. The strength of the cast-in-place samples represents potential concrete strength, whereas, in-situ cores represent the in-situ strength which is usually up to 10% less than that of the cast-in-place (Bungey et al, 2006).

Other reasons core samples may be needed include: checking bond strength at construction joints, and to verify the thickness of an unknown concrete slab or section by coring all the way through the member (Neville, 1995).

Sometimes the Engineer wants to take core samples which include rebar and other times the Engineer may want samples without rebar. In order to ensure a sample includes what the Engineer wants, there is a device which will locate the rebar so that the Engineer may pick a location for a core. This device is known as a cover meter. The cover meter

sends out an electromagnetic wave into the concrete which will be attracted by the rebar. It induces an eddy current in the steel and then this eddy current is attracted by coils in the instrument. With a cover meter survey, the rebar diameter, depth and distance can be determined. It works by sliding the cover meter along the concrete and marking the points where there is a greater deflection (Mailvaganam, 1992).

2.2.2.6 General Lab Compressive Strength Tests

Compressive strength tests are conducted on in-situ core samples, cylinders and cubes. Core sample testing is the most standard way of measuring the strength of in-situ concrete. It is done by using a rotary diamond drill which cuts the cores in their shape. The cores are then conditioned for moisture content, capped and tested for the information needed. Core samples usually have a lower strength than cylinders, due to the curing on site. As the strength of the concrete increases then the strength ratio between core and cylinder strength decreases. Cores that are taken at the top of a concrete lift will be weaker than one taken at the bottom. This is due to bleeding and settlement of coarse aggregates. Cylinders are the most common test for measuring the compressive strength of fresh concrete. In North America the standard size of a cylinder is 150mm in diameter and 300mm long. Cylinders are made with molds placed on a level surface where concrete is then added to the mold. It is cured for a certain number of days, having an atmospheric temperature between 16 and 27 degrees Celsius. These cylinders are then capped and the compressive strength is then measured. Cube tests, however, are used mainly in Great Britain, Germany, and other parts in Europe. The requirement is a 150mm cubic mold that is filled in 3 layers, with each layer being rodded 35 times with a 25mm square rod. The cubes curing temperatures is at 20 ± 1 degree Celsius. The drawback with this test is that the friction between the plates and the specimens ends creates much more confinement in the specimen (Mindess et al., 2003).

2.2.2.7 Other In-Lab Core Tests

There is a great deal to conclude from laboratory work. The tests that are usually conducted in the laboratory are petrographic analysis including aggregate-cement paste reactivity, air void system determination, compressive strength, density, absorption, void content determination, chloride content determination, condition of rebar, ultrasonic pulse velocity determination, determination of cement and aggregate contents and aggregate size distribution, and chemical analysis of cement paste. All cores that are done on site are brought to the lab to be tested and can be tested using any of these methods depending on what the Engineer in charge is interested in discovering.

2.2.3 Hidden Flaws

2.2.3.1 Sounding Survey (Slabs & Columns/Walls)

A visual survey is an excellent tool for determining surface defects; however, in some cases it may not be able to identify or confirm every single surface defect. It may provide an indication that there is a defect at the surface or just below it, but this cannot be confirmed until a sounding survey has been conducted in the suspect area (Personal Work Experience). Sounding surveys provide confirmation of certain surface/subsurface visual survey results and also discover new subsurface defects which are not obvious enough to be detected using the visual survey. In order to find subsurface defects, a sounding test may be conducted in many ways. The two most popular sounding tests are the 'chain-drag' test and the 'hammer-tap' test. The chain-drag test is conducted by having the inspector move or drag a broad metal chain over the suspect concrete surface (See **Figure 13 - Chain-Drag** Above). The purpose of this test

is to identify any 'hollow-sounding' areas. These areas would be your defect. Since gravity works vertically, the chain-drag only applies to horizontal surfaces. For surfaces which are vertical or above the inspector, the hammer-tap test is used. Hammer tapping may also be used wherever the chain drag is not feasible or accurate (i.e. small spaces or to confirm a hollow sound that is not prominent with the chain-drag). Hammer tapping is done exactly as its name implies, tapping the concrete with a hammer (See **Figure 14** Below). Wherever the hammer tapping produces a 'hollow' sound, a concrete delamination usually resides (Personal Work Experience).

Figure 14 - Hammer-Tap

2.2.3.2 Delaminations

In the case of an uneven surface, a delamination may in fact be detected by way of a visual inspection, but in order to confirm the discovery, a sounding test of the questionable area should be conducted and almost always is (Personal Work Experience). If an Engineer is not satisfied with the results of the sounding survey for detecting delaminations, an advanced way to detect delaminations is using infrared thermography. In this case, the delamination interrupts the heat transfer through concrete causing an increase in temperature in this area (Neville, 1995). This infrared thermography will pick up these changes in the temperature and spot the areas where delamination is occurring. This process is discussed further in **Section 2.2.4.2** of this report.

2.2.3.3 Corrosion

2.2.3.3.1 Indirect Measurements

2.2.3.3.1.1 Half-Cell Potential

The half-cell potential test provides a measure of the electrical potential of the reinforcing steel in the concrete structure with respect to a reference half-cell (Bungey et. al, 2006). Usually, the test is performed using a voltmeter and either a copper copper-sulphate half-cell or a silver silver chloride half-cell (Bungey et. al, 2006). In order to achieve sufficient electrical contact, the surface of the concrete is usually wetted (Bungey et. al, 2006). Please refer to **Figure 15 - Half-Cell Setup** below for schematic of this test (Gu et al, 1998).

Interpreting the results of this test is actually quite simple, the only dependent variable is the type of half-cell used to conduct the test. If a copper copper-sulphate half cell is used, when readings are greater than -200 mV, it is expected to be greater than a

90% probability that there will be no corrosion present. However, if the readings are found to be less than -350 mV, the expected probability changes to greater than 90% that corrosion will be present; then, for the remaining range of -200 mV to -350 mV, the probability of corrosive activity is unknown.

A disadvantage provided by this test is the need for contact with the actual steel reinforcement within the structure. This contact helps to complete the electrical circuit necessary to obtain the readings in the voltmeter. There have been systems developed called 'two-cell' methods which do not rely on the connection with the reinforcing steel below the surface in order to obtain a complete circuit, however, these results are generally thought of as less reliable when compared to those obtained using a direct circuit with the half-cell potential test (Bungey, et. al, 2006). Conversely, the advantages

lie in the tests ease and ability to be applied quickly and on multiple test locations of a structure with negligible structural damage. The quick application of the test allows for inspectors or Engineers using the test to obtain an idea of the corrosion level present in a miniscule amount of time.

A newer development and a growing trend for half-cell potential results is the use of the obtained data for the purpose of plotting electrical potential contour maps (Bungey, et. al, 2006). With the electrical potentials plotted in this fashion, a comparative analysis can be carried out with regards to where *islands* of more negative potentials are located as these *islands* represent the areas in which corrosion of the steel is more likely. This also allows the user to remove the variability present with other results which occur with a more absolute half-cell test approach (Bungey, et. al, 2006).

2.2.3.3.1.2 Concrete Resistivity

The concrete resistivity test indirectly uses the concrete's ability to resist electric flow to obtain the likelihood of corrosion processes occurring within the concrete (Bungey, et. al, 2006). The test is conducted by placing four electrodes in a straight line and equidistant from each other and then connecting the inner two electrodes with a voltmeter and the outer two electrodes with an ammeter and a current supply (Bungey, et. al, 2006). Please refer to **Figure 16 - Concrete Resistivity Test Setup** to the right for a schematic of the test (U.S.Dept. of Trans.).

In order for the test to be conducted, four small equidistant holes must be drilled into the concrete for each reading. This can be credited as one of the disadvantages of the test because the preparation necessary for just one reading is one that may slow the rate at which the test can be performed.

In order to make an accurate assessment when interpreting the results for the corrosion present in the structure, the following conditions must be considered:

1. If there is steel reinforcement nearby or close to the location in which the measurements are taking place, there will be an underestimation of the concrete's resistivity;
2. Severe overestimations and underestimations may be made if there is the presence of surface layer due to carbonation or surface wetting;
3. Overestimations will occur if the test takes place at the edge of a member or if the member being tested is considered to be a small member; and
4. Rainfall and differences in ambient temperature have been known to affect the measurements of the test significantly (Bungey, et. al, 2006).

After the test has been conducted and the data necessary has been collected and interpreted using the conditions discussed above, an assessment or evaluation of the likelihood of corrosion may commence. Corrosion potentials can be considered very high, for readings less than 5 ohm-centimetres, or very low, with readings greater than 20 ohm-centimetres.

When compared, each indirect method (the half-cell potential and the concrete resistivity test) can be considered to have their own performance difficulties. This is evident for the half-cell potential test with its requirement for rebar contact and also for the concrete resistivity test with its extensive preparation to obtain a single measurement.

2.2.3.3.2 Direct Measurements

2.2.3.3.2.1 Polarization Resistivity

Direct techniques have been devised in order to discover the corrosion rate of the reinforcing steel in a concrete structure. One of the more common direct methods which should be considered is the linear polarization resistance method. This method is gaining recognition in the workplace, there are several difficulties which must be overcome in to obtain high quality results (Bungey, et. al, 2006). For this reason, this test method will

only be mentioned and not discussed in detail as the previous indirect methods of corrosion detection were.

Everyday, deteriorating reinforced concrete structure maintenance and replacement programs rely heavily on the ability of infrastructure agencies or inspectors to determine the level of corrosion present. The results obtained can also have a major impact with regards to the possibility of catastrophic failure. With today's advancements in technology, levels of corrosion are able to be defined, even if these levels are only on a relative scale to the rest of the structure, areas of deterioration become alienated and the overall structure is able to be properly maintained. In the case of parking garages this is especially useful. Structural steel elements for these garages such as the longitudinal reinforcement or the shear stirrups play an integral role in the structural systems ability to remain stable, especially when massive dead and live loads are applied to them every day. Thus, with the ability to locate potentially corroded steel areas, using the non-destructive methods described above, clients or agencies can save money, and act proactively in repairing their structures. The repair will benefit all users as the potential of avoiding a catastrophic failure with regards to the corrosion of reinforcing steel has been increased. In addition to these benefits, the minimal equipment necessary to complete the analysis and the quick speed at which each analysis can be performed increases the practicality of these tests and it becomes an integral part of the monitoring and maintenance evaluation requirements necessary for reinforced concrete structures.

2.2.3.4 Carbonation

A method in determining if there is carbonation present, one can stain a sawn surface with phenolphthalein. When the concrete is normal it will stain pink and when the concrete has a pH less than 10, the chemical remains colorless. For a well cured concrete which has a low w/c ratio, the depth of the carbonated zone will not exceed 25mm. However if there are micro cracks present in the concrete then carbonation will be accelerated. If carbonation reaches the re-bars, they will begin to corrode (Mindess et al., 2003).

2.2.3.5 Alkali-Aggregate Reaction (AAR)

Today, non-destructive in-situ tests for alkali-aggregate reactions (AAR) have not been developed to the point where they can accurately identify or quantify the alkali-aggregate reactions (Bungey et al, 2006). There are tests available such as the Accelerated Mortar Bar Test for quantifying the alkali-aggregate reactions taking place, but this test cannot be applied in-situ; thus, during a condition survey it is difficult to evaluate a concrete structures in-situ vulnerability to AAR. According to John H. Bungey, taking concrete cores from the structure in question and then testing them using expansion tests, alkali-immersion tests, cement content and petrographic analysis tests is the most common reliable way to test for alkali-aggregate reaction in existing concrete structures. Nevertheless, since taking concrete core samples are semi-destructive because a sample is removed and a void is left, this void must be refilled. This test method may not always be preferred by a client.

2.2.3.6 Voids

2.2.3.6.1 Impact-Echo

The impact-echo test uses transient waves to detect flaws within the concrete structure. By placing monitors in different locations, this will help screen and record the locations of any defects or flaws in the concrete. The test is an extension of sonic and ultrasonic non-destructive testing techniques, and it is considered a point-by-point non-destructive test (McCann et al, 2001). The fact that the test is conducted using point-by-point testing techniques, selecting the points is critical should represent the entire area of the structure in question. The more points used for this test, the more data the Engineer has to work and the stronger the results will be. With more points tested and more accurate results, Engineers or Inspectors who use this test are able to confidently arrive at conclusions regarding damage within the concrete member.

The test is performed by hitting the surface of a concrete structure with a small pulse hammer. This impact produces dilatational waves (P waves), distortional waves (S waves) and Rayleigh waves (R waves) which all propagate in the concrete. The P and S waves will reflect at any internal flaws and will be recorded with an accelerometer (McCann et al, 2001). The internal flaws are noticed because the reflected energy is transmitted into a frequency spectrum record. The peaks that are produced on this record correspond to the flaw depth resonant frequency (McCann et al, 2001). Thus, using these peak readings and the compressional wave velocity of concrete, the flaw depth can be calculated and found in relation to the monitors (McCann et al, 2001). By identifying the depth of flaws (such as voids) within the concrete, this allows Inspectors or Engineers to determine the location of voids in the different members of concrete structures (McCann et al, 2001). This is significant for locating conduits beneath the concrete surface in post-tensioned structures; because, the grout injected into these conduits for the purpose of post-tensioning must completely fill the conduit in order for maximum bond strength between the surrounding concrete and the post-tensioning cables to be obtained. Additionally, actual loading capacities for post-tension structures can be calculated by Engineers if the areas in which the post-tensioning cables are not completely covered are identified. Subsequently, any major repairs that may be required due to voids or the loss of bond between concrete and reinforcing steel may be revealed using this method. This test is still being developed today in hopes to become more efficient and more accurate.

Concrete quality may also be determined with the knowledge of how many voids are present. Concrete (non-air entrained) with less voids, will be of higher quality or strength than concrete (non-air entrained) with more voids. This can help agencies such as the Ministry of Transportation Ontario, MTO (which award payment depending on the performance specifications) render judgment as to the durability of concrete used in its projects and if payment should be awarded.

2.2.3.7 Fire Damaged Concrete Structures

Below, three techniques which are used to test the load bearing capacities for fire damaged concrete members from Colombo et al.'s paper titled, *New NDT Techniques for the Assessment of Fire-Damaged Concrete Structures*, will be discussed in detail.

2.2.3.7.1 Ultrasonic Pulse Velocity (UPV)

As earlier discussed, the UPV test implements emits and receives probes on the face of concrete members to measure pulse arrival times. By increasing the depth between the emitting and receiving probes, Engineers are able to formulate a plot of arrival time versus probe distance. This plot allows for deeper material layers to be investigated. Being able to normalize the plot with respect to asymptotic velocities allows Engineers to identify the intercept of the final asymptote. At this location, probe distance will be zero and the results will be strongly related to the thickness of the damaged concrete. The time delay which is present represents the 'slow' layers of damaged concrete. Therefore, the more time delay, the more potential for fire damaged concrete layers. For this test, the most important step taken is the development of the time versus probe distance plot. This task is very time consuming; consequently, this test should not be used when responses are required immediately.

2.2.3.7.2 Concrete Calorimetry

This technique is one of the simpler methods that can be used to identify fire damaged concrete. This technique uses a digital camera to take pictures of the concrete in question. It is said that the colour of concrete changes as the temperature of the concrete increases dramatically. Therefore, by using a digital camera to take pictures of different sections of the concrete, the colour can be compared to that of a fire damaged sample and results can be obtained. Of course, this method can be considered semi-destructive as it requires core samples to be taken in order for the section view to be available for photograph. For very thin members, this method is not recommended as it may compromise the structural capacity of the structure.

2.2.3.7.3 Real-Time Monitoring of Drilling Resistance

The test is conducted by measuring the real-time thrust needed to drill through the concrete, to a certain depth, at a constant rate. This method is advantageous because the protected virgin concrete is drilled at the end of the test preventing the need for calibration curves. Since calibration curves do not need to be developed, Engineers will save plenty of preparation time and the results can be delivered in a shorter time. Like the Concrete Calorimetry Test, this technique is semi-destructive and for that reason, concrete members should only be evaluated using this technique when there is concern of high levels of thermal damage.

2.2.4 Additional Non-destructive Tests

The following tests do not fit into any of the above categories, but may be used during a condition survey depending and for other purposes that the Engineer is interested in discovering.

2.2.4.1 In Placed Tensile Test on Concrete

In Placed Tensile Testing on Concrete is a relatively simple method to perform. As the name describes, it is used to discover the tensile strength of in-situ concrete members. The test is conducted by taking core samples and applying direct tension through end plates which are glued to the concrete. However, it is difficult to create these sample cores since they should contain the bond zone along the length of the core. Sometimes during coring there will be breakage at the bonding section instead of below the core which will lead to re-coring and more time spent on the field. ASTM has not yet developed standards for this type of testing yet (Mindess et al., 2003).

2.2.4.2 Thermographic Survey

These types of surveys can be conducted and will determine beforehand whether the concrete is going to fail or breakdown and the location of these potential failures. The test is conducted by heating the concrete member then sensing the heat transfer through the concrete with an infrared reader. Concrete absorbs heat and gives it back to the environment. The thermographic survey picks up all of this heat. In areas where water or a void (crack, delamination, etc.) is present below the surface, heat will be trapped and the infrared will read a lower temperature at the surface, since less heat is being emitted by the concrete in this location (Trow & Neville, 1995).

2.2.4.3 Chemical & Petrographic Analyses

This type of procedure should be used for the evaluation of concrete quality. Petrographic analysis deals with different procedures and methods for determining properties for rocks and minerals that have been used for concrete aggregates in a structure. This includes looking at thin sections of rocks. Petrographic and chemical analysis used on similar aggregates are powerful tools which can determine what causes cracking in concrete. These analysis will determine the specific cause such as chloride or sulphate attack, or alkali-aggregate reaction (Mailvaganam, 1992).

2.2.4.4 Steel Reinforcement Survey

Usually the condition of the reinforcing steel is done by visual inspection, by measurement, or by radiography. Physical and chemical properties of the steel can be determined whether or not the cross sectional area has changed on the reinforcing bar and will lead to a weakening of the concrete. The CSA has a list of standards of what properties are needed for reinforcement and how to conduct different tests. Another test that can be completed is the checking of the continuity of the concrete rebar/reinforcement. This is done by sending an electrical current through the support and measuring it at another end. The same reading should be obtained on the other side. If

there is no current that reaches the other end then this means that the rebar has failed and needs to be repaired (Woodson, 2009).

2.2.4.5 Monitoring

When there are defects in the concrete structure and they are not destructive, it is important to keep on monitoring the defect because it can eventually change and grow becoming a severe problem. Monitoring is usually done when progressive changes are occurring. It can prevent concrete from getting damage by treating it before something disastrous occurs. Monitoring can be done by visual inspections (which the client can perform under their own power without the need to hire an Engineer) or even by electronic sensors or fixed measuring tapes to determine how long a crack becomes (Trow).

3.0 Condition Assessment Process

3.0 Condition Assessment Process

3.1 Condition Survey Case Study - 7 King Street East

For the purpose of the case study, my Employer, Trow Associates Inc. allowed me to participate in a large scale condition survey of a parking garage in which many different types of surveys were conducted. A description of the structure, the different surveys conducted, the results obtained and any recommendations we have developed are discussed in the following sections. An appendix (**Appendix A**) of photos from the condition survey can be found at the end of this report.

3.1.1 Description of Structure

An actual condition survey was conducted to serve as a case study for this report. The condition survey took place in the below ground parking structure at 7 King Street East in Toronto, Ontario, Canada. This parking garage was three levels (Level P1, Level P2 and Level P3) and was made entirely of concrete. The structure would be analyzed in two parts, (1) the topside (top-surface) of each slab including the walls and columns for each level and (2) the underside of each slab. The concrete topside was covered in an elastomeric topping, whereas the walls and underside of the concrete slabs had been painted white.

3.1.2 Conducting the Survey

To start the condition survey, we first conducted a 'sounding' test for the topside of the Level P1 and Level P2 slabs. The survey was not conducted on Level P3's top surface because this level was on grade and the test is not as affective when conducted for slab-on-grade situations. The survey was done by dragging a chain over the topside of the concrete slabs and listening for voids beneath the surface (delaminations of the concrete), which when located, are very obvious due to their unique sound. While one person performed the 'chain-drag' locating possible delaminations, another person was following closely behind ("trailing"), documenting the areas discovered on the drawings (See **Figure 17** in **Appendix A**). This task can end up being very time consuming

depending on how many delaminations are found. Two sets of drawings were printed, one for the topsides and one for the undersides of each level. Once the chain drag of Levels P1 and P2 was completed, the visual and sounding surveys of the walls and columns commenced. One person would hammer tap the walls and columns listening for hollow sounding areas and the trailer would conduct the visual survey (looking for cracks, rust, spall...etc.). With the topside drawing handy, the trailer would mark anything he saw or the other person heard on the topside drawing (See Legend in **Figure 18** in **Appendix A**). This was completed on all levels of the parking garage for all solid concrete walls and every column. There were some walls made of cinder blocks where we could not perform the sounding survey due to their already hollow nature, but the visual survey was still conducted in these areas.

Once the visual and sounding survey of the walls and columns was completed and all of the quantities had been documented on the drawing, we moved onto the visual and sounding survey of the underside of Level P1 and P2. The underside of the building was covered in a drop panel ceiling; and therefore, the concrete condition could not be visually seen without accessing or removing the ceiling. Also, since the Level P3 slab was on grade, the underside could not visually be seen; thus, a visual survey for the underside of this slab was not conducted. This survey was conducted similarly to the visual and sounding surveys for the walls and columns. One person would tap the underside of the slab in suspect areas with a long piece of rebar (because the ceiling was too high to reach with only the hammer) listening for hollow sounding areas, while the trailer would look for visual deterioration and mark everything on the underside drawings (See **Figure 19** in **Appendix A**).

In addition to the 'underside' and 'topside' visual and sounding surveys, we required cores to be taken at different locations of the slab. The purpose of taking the cores was to (a) find out what the makeup of the slab is and (b) for use in compressive strength tests to determine the in-situ strength of the slab in the areas of the cores, this

would help us to understand the approximate or average strength of the concrete. Samples which did not include rebar were necessary because if the rebar were to be cut, it would result in the risk of reducing the total resistance of the concrete to tension in this area. In order to be sure that when the coring company (Graff) came to drill the cores, they did not hit any rebar, we had to map out a grid with chalk lines indicating where the rebar was located. The location of the rebar was initially determined using the cover meter (See **Figure 20** in **Appendix A**). To use the cover meter, first I aimed the sensing pad in the air, away from all metal objects and zeroed it. Next, I slowly moved it across the surface of the concrete and obtained readings. Once the reading was at its lowest point (lowest number), I marked the surface with chalk. I did this for about half a meter, and then I did the same thing in the perpendicular direction until I had formed a grid of chalk lines. Since I was able to layout a grid of rebar, it could be determined where the openings in the slab were and I was now able to pick a location for Graff to drill a core. This was done for multiple core locations.

Later, when Graff arrived on site, I showed them the locations to which we wanted the cores to be drilled. In order to confirm the markings from the cover meter, Graff used their own scanner (See **Figure 21** in **Appendix A**) to locate the rebar. The grid that Graff produced was about an inch offset from our grid (See **Figure 22** in **Appendix A**). The operator told me that the cover meter does not perform as well as their scanner, especially for thicker slabs and that may be the reason for such a large error. Even though Graff had laid out their grid for the rebar location, they still could not drill until they had confirmed there was no electrical power or buried conduits within the slab where they were drilling. In order to do this they had an additional scanner for power lines (See **Figure 23** in **Appendix A**) which they waved over the concrete until they heard the noise it produced become louder. Wherever this machine made the loudest noise Graff determined there was power and marked this on the ground in chalk as well. Now that the rebar and power location within the slab had been mapped out, Graff could drill the cores without the worry of compromising the strength of the

concrete or being electrocuted in the process. Ultimately, due to the preparation, Graff was able to take the cores without hitting any rebar or conduit (See **Figure 24** in **Appendix A**). An example of one of the cores Graff drilled can be seen in **Figure 25** of **Appendix A**.

The next order of business for the survey was the half-cell test. We needed to conduct the half cell test in order to determine the extent of the corrosion of the structural steel within the concrete. To do this we first had to pick a location and lay out a grid of holes to be drilled (See **Figure 26** in **Appendix A**), the results from the holes would be averaged to get an approximate idea of the overall corrosion beneath the slab. Once the grid was laid out, we continued by drilling 3" to 4" holes into the concrete using a drill which we borrowed from Graff (See **Figure 27** in **Appendix A**). Additionally, we needed Graff to core down to the surface of a rebar in one location in order for us to complete the circuit necessary for the half cell test. Once all of the holes were drilled and a rebar was exposed, the test could take place. We had a voltmeter, and one end of the volt meter would be connected to the exposed rebar, whereas the other end or reference electrode (conductive stick attached to a wet sponge) would be moved to each of the holes on the grid (which we filled with water to complete the circuit) and a reading would be recorded for each hole. This information would help the Engineers analyze the risk of corrosion based deterioration. The half-cell equipment used can be seen in **Figure 28** of **Appendix A**.

3.1.3 Survey Results

Overall, we did not detect a lot of delaminations. However, when we did find some, they were of significant size (i.e. an entire parking spot or wall panel in width). We did notice during the visual survey that there was many previously repaired patches that had been installed and a large number of them were coming loose. This was indicative of poor workmanship in these areas, as the deterioration was not prevented properly.

As far as the half-cell test was concerned, the concrete topping which is used to slope water into the drains was de-bonded from the structural slab, so that when we filled the drill holes with water to complete the electrical circuit, it would dissipate into the opening between the topping and the structural slab preventing the half-cell test from being conducted. Thus, the half-cell test was deemed inconclusive.

3.1.4 Possible Recommendations

After having conducted the different surveys and only being able to have a general feel for the visual and sounding survey results (strength tests still needed to be conducted on the chosen cores), my recommendations are as follows:

1. I would recommend a series of localized repairs in the areas which we identified on the drawing as being delaminated or spalled; and,
2. Since many of these areas have been previously repaired already, another recommendation is to select a different contractor for the work and have an Engineer or Inspector oversee all repairs, providing regular site visits during construction and documenting his/her observations in a report.

I am only able to base my recommendations on the limited information obtained in a one day condition survey, so by no means are these the correct or only necessary recommendations which should be submitted to the client. However, with the knowledge we have obtained from the surveys, I am confident in my recommendations and feel that if there was no supplementary testing conducted, the consultant (Trow) would have similar recommendations.

A real-life case study in which repair options have been decided on based on the results of a condition survey is discussed later in **Section 4.1** of this paper.

3.2 Engineering and Reporting

In order to analyze a structures condition, there is a series of processes which must be carried out. When a structure looks like it is damaged or has gone through a lot of deterioration, a client will approach an engineer and ask that their structure undergo a condition survey. However, aside from the onsite activities (or surveys) discussed above, there is also a lot of paperwork associated with each condition assessment project.

3.2.1 Prior to Condition Survey

First the client approaches the consultant engineer and asks for a proposal for the cost of analyzing the structure and overseeing the work to be done. In most cases the Engineer will submit to the client their knowledge of the client's site, their company's rehabilitation program (what surveys and tests that the company conducts), their approximate time to complete their portion of the work and their engineering fees. If the client is satisfied with the proposal they will award the work to the consultant and the condition survey will commence. Below see **Figure 29 & 30** for an example of Engineering Schedule and Fees as submitted in a proposal.

	Activity	No. of Weeks
Phase I	Detailed Condition Survey including report	3 - 4
Phase II	Preparation of Specifications & Contract Documents	*
Phase III	Tendering and Review of Bids	*
Phase IV	Construction Review & Contract Administration	**

Figure 29 - Engineering Schedule (Trow)

	Activity	Budget (Excluding GST)
Phase I	Detailed Condition Survey	
	Interior Suspended Slabs & Underside of Garage Roof Slab	
	a) Field work including report, photographs and AutoCad Drawings	
	b) Half Cell tests - (Optional)*	
	c) Coring & Patching of Core Holes (Trow to retain a coring contractor)	
	d) Laboratory Testing– (Chloride & Compressive strength Tests)	
	Sub-Total (Interior Suspended Slabs & Underside of Garage Roof Slab)	
	Garage Roof Slab- Top Surface of Slab	
	a) Field work including report, photographs and AutoCad Drawings	
	b) Coring & Patching of Core Holes (Trow to retain a coring contractor)	
	c) Laboratory Testing– (Chloride & Compressive strength Tests)	
	d) Test Pits	
	Sub-Total (Top Surface of Garage Roof Slab)	
Phase II	Preparation of Specifications & Contract Documents, (if requested)	***
Phase III	Tendering & Review of Bids, (if requested)	***
Phase IV	Construction Review & Contract Administration Services	***

Figure 30 - Engineering Fees (Trow)

3.2.2 During Condition Survey

During the condition survey, the inspector will note the location and length or area of each type of deterioration on the drawings. In the case of a complete structural assessment, he/she will examine the whole structure including topside, underside, walls and columns.

3.2.3 Post Condition Survey

The consulting Engineer will quantify each type of deterioration in terms of length and area by reading the drawings submitted by the inspector. These results are then included in a report along with a judgment to how much the deterioration or damage will affect the safety, serviceability, and the life span of the expected durability of the structure; recommendations of how to repair the structure; recommendations of what can

be added to the structure for more durability; and also recommendations in maintaining the structure after it has been repaired or if it was proven durable and safe. This report will be presented to the client for review and approval. The client will review the quantities and recommendations presented to them and will decide what course of action will be taken.

Once the client has decided on the course of action, they will inform the consultant to prepare a bid form using the quantities obtained in the condition survey. This bid form will be included in the specifications and then be distributed to the client and multiple contractors. The contractor with the lowest price will be awarded the job.

4.0 Detailed Condition Assessment, Analysis and Results

4.0 Detailed Condition Assessment, Analysis and Results

4.1 Detailed Case Study - 530 Lolita Gardens

Earlier in this report (**Section 3.1**), the condition survey at 7 King Street East in Toronto was discussed. This was an example of a condition assessment, but information on the repairs was not included as the rehabilitation process has not yet been decided. For this reason, another case study, 530 Lolita Gardens in Mississauga has been included in this report. With this case study, not only will the condition survey and testing methods be analyzed, but so too will the survey results, the repair methods, the rehabilitation process and cost of the work.

4.1.1 Condition Survey

After the client approaches the Engineer and informs them of their intentions to rehabilitate their concrete structure, the first step is to analyze the structure via a condition survey.

The condition survey process was analyzed earlier and may be referred to in **Section 3.1**.

At 530 Lolita Gardens in Mississauga, the following processes were conducted to analyze the structure (Trow):

1. Visual Survey.
2. Sounding Survey
3. Coring Program
4. Laboratory Testing
5. Photographs.

4.1.1.1 Visual Survey

A thorough visual survey was carried out so to document via drawings, data of deterioration on the top surface and underside of the roof and interior suspended slabs and the interior surface of columns, perimeter walls, drop panels, and stairwells. In the

drawings, records were kept of specific details of the deterioration, and defective flaws on the top and underside surfaces of the concrete slabs. Core locations were also noted on the drawings.

4.1.1.2 Chain Drag used for Sounding Survey

In order to determine or detect a hollow sounding debonded traffic topping system, and if any concrete delaminations existed, a chain drag survey of the reachable top surface area of the interior suspended slab and ramps was performed. A sounding survey was performed using the hammer test in construed areas of the underside, column bases and walls, where unsound areas were recorded on the sketches created.

4.1.1.3 Coring Program

Cores from the roof slab of the garage and interior suspended levels were removed manually to resolve:

1. Rebar condition.
2. Concrete quality (visual)
3. Waterproofing condition on roof slab
4. Traffic topping system condition on the top surface of the interior suspended parking slab.
5. Determine if hollow sounding areas on the interior suspended parking slab are due to debonded traffic topping system or concrete delaminations.

As a representation of the entire structure under investigation, and by means of diamond drilling, four (4) cores were acquired on the P1 Level slab, and four (4) cores were obtained on the garage roof slab in the asphalt paved areas. These cores were used for verification of the existence of delamination, and for visual observation purposes. Records of the details obtained from the cores acquired are recorded in Trow's core log on the next page of this report.

Core Log

Project: Parking Structure Condition Survey at 530 Lolita Gardens, Mississauga.
Project No.: BRBS00001988D

Date: July 11, 2007

Core No.	Membrane Thickness (mm)	Traffic Topping Wear Course Thickness (mm)	Condition of Waterproofing	Bond of Water-proofing to Concrete	Bond of Traffic Topping Wear Course to Membrane	Condition of Top Surface of Concrete Core	Visual Concrete Quality	Cover to Reinforcing Bar (mm)	Termination	Total Length (mm)	Additional Comments
RC1	4"	NA	Fair	Good	NA	Good	Good	NA	Sound Concrete	165	Core taken in Roof slab. 65mm of asphalt pavement over 55 mm of gravel over concrete slab.
RC2	6"	NA	Good	Good	NA	Good	Good	NA	Sound Concrete	162	Core taken in Roof slab. 70mm of asphalt pavement over 60 mm of gravel over concrete slab.
RC3	NA	NA	NA	NA	NA	Good	Good	NA	Through Slab	215	Full depth core taken in Roof slab. 80mm asphalt pavement over 50mm of gravel over concrete slab.
RC4	NA	NA	NA	NA	NA	Good	Good	NA	Sound Concrete	165	Core taken in Roof Slab. 80mm asphalt pavement over 50mm of gravel over concrete slab.
P1C1	1"	1"	Good	Good	Good	NA	Delamination near top of core.	NA	Concrete	160	Core taken in Level P1. Core extracted from hollow sounding area.
P1C2	1"	1"	Good	Good	Good	NA	Good	NA	Sound Concrete	160	Core taken in Level P1 from sound concrete area.
P1C3	1"	1"	Good	Good	Good	NA	Good	NA	Sound Concrete	160	Core taken in Level P1 from sound concrete area.
P1C4	1"	1"	Good	Good	Good	NA	Fair	NA	Sound Concrete	162	Core taken in Level P1 from sound concrete area.

* Rubberized asphaltic waterproofing membrane found on garage roof slab.

** Elastomeric Traffic Topping System found on the top surface of the Interior Suspended Parking Slab.

4.1.1.4 Laboratory Tests

Cores which were extracted from the roof and interior suspended parking level slabs were brought back to Trow's laboratory and tested to determine the following (Trow):

1. "Chloride ion content of the concrete (4 cores - 2 cores from each slab, one horizon at the reinforcing steel level from each core plus one background horizon per slab)".
2. "Compressive strength of the concrete (4 cores - 3 cores from the roof slab and 1 core from the interior suspended parking slab)" (Trow).

4.1.1.5 Photographs

Digital photographs were captured and developed to identify and record the exact and characteristic conditions of the parking structure and the deterioration noted.

4.1.2 Laboratory Testing

All samples of cores eradicated throughout the survey were returned to a designed laboratory for core testing in order to determine in-place properties of the concrete specifically referring to the compressive strength and chloride ion content characteristics.

4.1.2.1 Compressive Strength

Compressive Strength testing was tried on Four (4) cores. These tests fell in accordance with C.S.A. Standard A23.2-14c, 2004, "Obtaining and Testing Drilled Cores for Compressive Strength Testing".

4.1.2.2 Chloride Ion Content

In accordance with the Ministry of Transportation, Ontario Laboratory Testing Manual, and LS- 417 Method of Test for Determination of Total Chloride Ion in Concrete, four (4) cores were cut on a horizontal and a slice at the horizon 20-30 was tested for total chloride ion content.

4.1.3 Field Test Results

4.1.3.1 Sounding and Visual Survey

4.1.3.1.1 Topside of Roof Slab

The asphalt on the roof slab on the east and south side of the building was discovered to be in fine condition. Also, on the topside of the roof slab there was a garbage disposal concrete pad which was recently constructed, and it too was found in good condition.

4.1.3.1.2 Underside of the Roof Slab

The underside surface of the suspended roof slab, which is not painted, is to some extent on the outer side of the mark of the building. The section of the underside of the roof slab which is below the footprint of the building consists of a drywall ceiling.

On the whole, after visual observations made, the underside of the roof slab with an exposed concrete component was noted as being in good condition. Only a few regions of the underside of the roof slab were noted with areas of peeling paint associated to leached cracks.

Delaminated concrete was recorded in areas on the underside of the roof slab, specifically at Parking Stalls 177 & 178, and between adjacent parking booths 103 & 104. Noted in observations made, repairs were made to the underside of the parking structure roof slab and the concrete on the underside of the parking structure roof slab expansion joints, and appear to be in suitable condition at the time of this observation.

Refer to **Table 1** below, taken from Trow's Report, for a summary of observations noted on the underside of the parking structure roof slab;

Table 1
Sounding Survey
Underside – Roof Slab

Level	Total Suspended Area (m ²)	Delaminated and/or Spalled	
		Area (m ²)	%
Roof Slab (underside)	4,194	3.50	0.08

4.1.3.1.3 Exterior Entrance/Exit Ramp

The exit ramp consisted of an suspended concrete ramp of mastic asphalt traffic topping and entrenched within, an electrical heating system. Observed were several coarse cracks in the mastic asphalt, and toward the middle of the ramp there was a large fracture.

After an extensive chain drag survey, hollow sounding areas, either due to the delamination of concrete or debonded mastic asphalt, were reported on the upper side of the exterior entrance/exit ramp.

The exterior ramp's underside was in unsatisfactory condition consisting of an abundant number of leaching cracks, concrete delaminations, and numerous rust stains. This ramp had been previously repaired with some of the repairs noted as once again failing.

Management responsible for the upkeep of the building informed the surveying team that there was leaking amidst the concrete ramp slab and ramp walls joints, of which were provisionally repaired via the sealing of the joints with asphaltic material, and connecting the top surface of the mastic wear course and ramp wall. The ramp walls were generally in satisfactory to moderate condition with remote sites along the west side of the wall with signs of rust and exposed steel.

The trench drain which was located at the bottom of the parking structure exterior entrance/exit ramp was noted as being in reasonable condition; yet the drain was full of leaves, humus, and debris. The existence of the debris in the trench drain restricts its ability to competently collect and drain away water. Exposure of the suspended concrete slab was noted on the topside of the P1 floor slab adjacent to the trench drain with isolated spots of tattered and missing traffic topping.

The base of the exterior ramp trench drain was noted to be in poor condition with extensive concrete delaminations, prevalent rust stains and wide spread leached cracks. Local repairs appear to have been made in the past, however these repairs also show widespread deterioration and failure.

4.1.3.1.4 Interior Ramp

A chain drag survey for detection of hollow sounding areas was completed of the suspended segment of the interior ramp, and identified were several hollow sounding areas, noting that the traffic topping system has debonded from the top surface of the concrete slab or there is existence of delamination of concrete. Yet, on the underside of the interior suspended ramp it was noted to be in good condition. Finally, the topside of this ramp which connects the parking garage levels P1 to level P2 also appeared to be in fair condition.

4.1.3.1.5 Topside of the Suspended Floor Slab - Level P1

There is a thin elastomeric covering and traffic bearing surface shielding the topside of the suspended floor slab on Level P1 with wearing away or missing membranes in isolated locations, particularly within the driving aisles. The structural concrete slab has been open to the elements of the harmful effects of de-icing salt, and grimy water which can accelerate further concrete deterioration.

The suspended floor slab of Level P1 contains many exclusive locations of hollow sounding areas which could be credited to delaminated concrete or debonded elastomeric

traffic topping. The Chain Dragging test was conducted over the surface examined. Hollow sounding areas were identified, noted, and marked on the top surface of the traffic topping system with chalk. It is suspected that the hollow sounding areas on the topside surface of the elastomeric traffic topping system could be caused by traffic topping that has debonded from the top surface of the suspended P1 level slab or by the presence of delaminated concrete beneath the traffic topping system.

In front of the Parking Booth No. 76, 77, and 78, there were annotations of exposed concrete commonly found in the area down the drive aisle. Isolated remote areas show signs of delamination and spalled concrete particularly in the driving passageway toward the southeast corner of the garage, also, it was pointed out that the traffic wear course also worn away at this location.

In the elastomeric traffic topping system, there were many hairline cracks and raised surfaces with evident isolated areas of patch noted, specifically in the hollow sounding area. Water and moisture infiltration appear to have filtered through the fractures in the concrete and may have set off further debonding and/or concrete deterioration of the this area.

Results from this site visit are summarized in Table 2 from Trow's Report below. This table provides an approximate calculation of the total top surface hollow sounding area.

Table 2
Sounding Survey
Top Surface - Interior Suspended Level P1

Total Suspended Area		Total Accessible Area		Hollow Sounding Area		Missing Membrane	
Level	Area	Area	Percent	Area	Percent	Area	Percent
P1	4,194 m ²	3,619 m ²	86.29	506 m ²	13.98	5 m ²	0.13

The drains and drain grates on the P1 level floor look to be in generally poor condition. The metal drain grates were obstructed with debris and the grates were visibly corroded. Specifically, at one of the drain locations, the traffic topping wear course was tattered, and because of its deterioration, the waterproofing membrane is exposed to the traffic and the damaging effects of water contaminated by salt throughout the winter season. Although the drainage cast iron piping was generally observed to be in fair condition, at several, the drain was pressed down into the slab creating a potential tripping hazard. At a few of the locations, the drain grate was missing making it vulnerable to loss of footing.

On the P1 level, in areas where service piping penetrates the slab, the elastomeric waterproofing system may be compromised.

4.1.3.1.6 Underside of the Suspended Floor Slab - Level P1

Although the underside surface of the suspended floor slab of Level P1 was unpainted, it was found to be in adequate condition. Repairs made in the past were noted on the underside of the concrete delaminations, and appear to be functioning in a satisfactory manner at the time of the visit.

A summary of the defects noted on the underside of the P1 level slab are formulated in Table 3 from Trow's Report below, where Trow recorded the development of water and rust stains, as well as efflorescence and the presence of leached cracks with rust stains on the base of the P1 level slab which are visible in isolated locations.

Table 3
Sounding Survey
Underside Surface - Interior Suspended Level P1

Level	Total Suspended Area Area (m ²)	Delaminated	
		Area (m ²)	Percent
Level P1 (underside)	4,194	37	0.88

4.1.3.1.7 Expansion Joints

Located typically at the perimeter of the mark of the building structure, the parking tower suspended slabs are isolated from the high-rise building at expansion joints. Horizontal movement at the expansion joints is due to thermal expansion and contraction. The top surface of the parking structure roof slab expansion joints is not visible because it is buried beneath the asphalt pavement where landscaping overload the area. There was local repair made at one point in the past in the expansion joints on the underside of the roof slab. At the time of the examination of the parking structure roof slab, the expansion joints appeared to be in good condition; and furthermore, did not show signs of any water seepage.

The top surface of the P1 level expansion joints also appeared to be in generally good condition. It consisted of a compression seal measuring approximately 15 mm in width and appeared to be well bonded to the sides of the joint.

When observations were made on the underside of the P1 level slab expansion joints, it exhibited verification of water leakage with visible rust stains down a great extent of its length. Delaminations of concrete were observed in specific locations on the underside of the joint, with deterioration at some locations. At some locations along the joint, there were corroded electrical conduits note, and this confirms the passage of water through or along the joint. Also, it was annotated that these “hard” connections as discussed tend to

limit the free horizontal joint movement that it is designed for and for this reason is not suggested.

4.1.3.1.8 Columns and Walls

It was found that the walls of the perimeter of the foundation in the parking structure were in fair condition. Although, delaminated concrete was located in specified locations and documented, on both underground Level P1 & Level P2 of the parking structure, with some scaled concrete in isolated areas on Level P2 foundation walls, specifically at parking booths No. 48, 49, and 50.

It has also been noted that there was evidence of water leakage through an opening in the wall. This was at the electrical conduit locations where the power supply for the ramp heating system is located at the P1 Level exterior ramp wall.

The concrete columns on both levels P1 & P2 of the parking structure generally appeared to be in fair condition. There was noted delamination at isolated locations at the Column base.

4.1.3.1.9 Garage Lighting

Trow documented in its site notes that the garage had three different light fixtures in operation in the garage: incandescent light, high pressure sodium, and fluorescent tube fixtures. Trow recommends that these lighting levels in the underground parking structure be checked in order that they comply to By-Law codes.

4.1.4 Laboratory Test Results

4.1.4.1 Compressive Strength Results

Compressive Strength Tests were taken by selecting four (4) cores: three (3) cores were taken from the roof slab and one (1) from the interior suspended slab, to determine the compressive strength of the concrete. These cores were tested in a dry condition in a laboratory, and the results are noted below in Table 4 from Trow's Report:

Table 4
Compressive Strength Results

Core No.	Compressive Strength (MPa)
RC1 - Roof Slab	32.8
RC3 - Roof Slab	35.3
RC4 - Roof Slab	41.0
P1C3 – P1 Level Slab	35.3
Average	36.10

The original design strength of this structure was found on a drawing from May of 1975 to be 25MPa. From the results obtained through laboratory testing, the concrete strength of both the roof slab and suspended floor slab is notably elevated over the originating design.

4.1.4.2 Chloride Ion Content

Four (4) cores in total were extracted from the garage to test for Chloride Ion Content: Two (2) cores were obtained from the interior suspended Level P1 slab, and two (2) cores were extorted from the garage roof slab. The cores each had a 10 mm slice cut horizontally 20-30mm from the cores extracted from the roof slab and a horizon of 60 – 70mm from the cores extracted from Level P1 and each was tested for the total chloride ion content. The background chloride ion content was determined by taking an additional 10 mm horizontal slice at 155-190mm of core No. P1C1 for P1 slab.

The results are documented in below in Table 5 from Trow's Report, specifying the Core number, the level, the Chloride Ion Content, the Corrected Background Ion Content and Horizon, as follows:

Table 5
Chloride Ion Content - Percent Total chloride Ion Content (Acid Soluble) by Mass of Concrete

Core No.	Level	Chloride Ion Content	(Corrected for Background Chloride Ion Content)	Horizon
Suspended Garage Roof Slab				
RC2	Roof Slab	0.038	0.015	20-30
RC3	Roof Slab	0.046	0.023	20-30
Suspended Floor Slab P1 Level				
P1C2	P1 Floor Slab	0.436	0.413	60-70
P1C4	P1 Floor Slab	0.021	0.000	60-70
P1C4	P1 Floor Slab		0.023	180-190mm (background)

- Notes: 1. "The threshold value is approximately 0.025 percent total chloride ion content" (Trow).
2. "The chloride ion content of the slice at horizon 180 - 190 mm in Core No. P1C4 of 0.023% should represent the background content for the P1 slab. The final values given were corrected by subtracting the background content from the uncorrected values" (Trow).

Because it is believed that only the water soluble chloride ions are available to cause depassivation of steel embedded in concrete, The chloride ion threshold value considered necessary to allow corrosion to proceed in the presence of oxygen and moisture, is generally taken at approximately 0.025 percent total chloride ion by the mass of concrete inspected.

Trow interpreted the results in their report as follows:

"The interpretation of the chloride values is complicated by the fact that many of the ingredients in the concrete mix contain chlorides, some of which are not available to initiate corrosion. It is thus necessary to correct the results for the "background" chloride content. This chloride content, which is not water soluble, is measured by the test method using acid, but is not available to contribute to the depassivation of the embedded steel because it is believed that they are chemically bound. In hardened concrete, where data on the ingredients of the mix is not available, the "background" chloride should be measured by testing a horizon external surface and away from an area that could be contaminated by external chloride sources.

The test results indicated that the slice taken at horizon 180 mm to 190 mm in core No.P1C4 from the suspended P1 level slab contains a background chloride ion content of 0.023 percent by mass of concrete.

Taking into account the background chloride content, the corrected chloride ion content at the level of the depth of reinforcing steel in the garage roof slab and in Level P1 slab is generally lower than the threshold of 0.025 percent total chloride ion by mass of concrete within the exception of Core P1C2 taken in Level P1 which substantially exceeds the threshold value by 16.6." (Trow)

4.1.5 Summary and Discussion

4.1.5.1 Sounding and Visual Survey

4.1.5.1.1 Roof Slab – Topside

Over the garage roof top, the asphalt pavement is in satisfactory condition and there is no evidence of any deterioration at the time of the observation. On the front side of the building, the topside of the roof slab is in part concealed with an asphalt driveway and landscaping on the north side. Also, recently reconstructed, is a garbage disposal area

pad located on the roof slab on the south side of the building. The Property Management Group has informed Trow Inc., through various discussions, that the roof slab on the south side was repaired only a short time ago.

4.1.5.1.2 Roof Slab - Underside

The exposed concrete segments located on the underside of the garage roof slab were examined, and viewed to be not painted but in generally good condition. There were a few contained regions with leached cracks and delaminated concrete. Trow noted that the underside of the parking structure roof slab had isolated areas which were repaired earlier on, and these areas appeared to be functioning adequately at the time.

4.1.5.1.3 Exterior Ramp

An embedded electrical heating system was identified in the exterior suspended concrete ramp with a mastic asphalt traffic topping system. There was a large crack located in the mid area of the ramp and also observed were numerous cracks in the mastic asphalt pavement. During the winter season, these cracks in the top surface of the mastic asphalt traffic topping were large enough to penetrate a substantial damaging amount of contaminated water with de-icing salts. The effects of prolonged exposure of salt contaminated waters could in time cause corrosion in the reinforcing steel in the suspended concrete slab and the end result is concrete delaminations.

Hollow sounding areas came from the top surface of the exterior ramp most likely due to debonded mastic asphalt pavement or delaminated concrete. The exterior ramp slab also had areas of delaminated concrete, scaled and leached cracks that were noted. Tree leaves and debris were located and noted in the trench drain at the entrance of the parking garage Level P1 which was found to be in poor condition. The underside of the concrete trench drain was surrounded by large areas of leached cracks and concrete delaminations. The topside of the Level P1 floor slab adjoining the trench drain contained isolated sites of missing traffic topping which meant that the top surface of Level P1 concrete slab was exposed.

The transition joint connecting the ramp slab and ramp walls had temporary repairs made to it with the use of an asphaltic sealant installed on the top surface of the ramp slab at the joint between the mastic and ramp walls. Other than the exception of a few isolated locations of rust and steel which was exposed on the west side of the wall, the ramp walls were in fairly good condition. Trow suggested addressing these isolated locations in order to minimize continued damage along the concrete walls.

4.1.5.1.4 Interior Ramp

The topside of the suspended portion of the interior ramp connecting parking garage level P1 to level P2 was chain dragged and a large area of the ramp was found to be hollow sounding. The hollow sound may be due to the presence of traffic topping that has debonded from the top surface of the concrete slab or the presence of concrete delamination. The underside of the ramp was also reviewed and was found to be in good condition.

4.1.5.1.5 Level P1 - Topside

The visual review conducted of the top surface of Level P1's suspended slab identified that at isolated locations, particularly the driving aisles, the traffic topping system was worn away or gone and has locally uncovered the top surface of the suspended Level P1 concrete slab to the harmful effects of de-icing salts which occurs throughout the winter season.

Hollow sounding areas were revealed in the sounding survey of a total approximating 506 m² or 13.98% of the accessible P1 level floor slab area, and also, an area of 5 m² of the missing traffic topping system. Based on the observations and analysis done of the concrete core samples, the relation between waterproofing membrane and concrete was poor in hollow sounding areas. On the topside of Level P1 floor, there were a number of raised areas of the elastomeric traffic topping observed, and all were found to be hollow sounding. In fact, delamination of concrete was identified in some exposed concrete areas where the traffic topping system was missing.

The Level P1 drains were found to be in poor condition. Some drain locations were locally depressed creating a lip that could present a possible tripping hazard. Additionally, the floor drain covers were corroded and often blocked with debris. Trow also noted that three (3) drain grates were gone, once more creating a potential tripping hazard for the inhabitants and/or visitors to the building.

Through Trow's documentations made in this survey, it was noted that the waterproofing detail at service pipe floor slab diffusions was expected. The waterproofing membrane was not appropriately detailed at the Level P1 slab penetrations and is likely to fall short and permit water to seep out onto the parking area below (Level P2).

4.1.5.1.6 Underside of the Suspended Floor Slab – Level P1

The underside surface of the suspended floor slab of Level P1 is not painted and was found to be generally in fair condition. The underside of the P1 level slab contained several localized areas of leached cracks, rust stains, and efflorescence and delaminations of concrete. The underside of the Level P1 slab has been repaired earlier at isolated points on the underside of the suspended floor slab, and the concrete repairs appear to be performing satisfactorily at this time. There were many areas of corroded electrical conduits observed at isolated locations on the floor slab.

It was identified through the survey conducted that there was approximately 37 m² or 0.88% of the underside of the suspended Level P1 floor slab which contained eroded concrete.

4.1.5.1.7 Expansion Joints

The expansion joints in the parking structure roof slab appear to be in good condition at the time of the report. Trow noted that repairs were made to the concrete along the underside of the roof slab expansion joints, of which these repairs were completed at isolated locations, and appeared to be performing satisfactorily at the time of observation.

The Level P1 expansion joints are generally in poor condition, with exception to the seal along the top surface of the P1 level expansion joints which appeared to be in good condition and well bonded to the edges of the joint. However, the underside of the expansion joints shows evident proof of past leakage which was confirmed by the progress of rust stains along much of the joint. Spalling and delamination of concrete was visible along the concrete which runs along the underside of the joint, noting that the structural integrity of the joint may be negatively affected at these locations, and should be repaired.

4.1.5.1.8 Columns and Walls

In the underground parking garage Levels P1 & P2, the columns and walls are generally in fair condition. Spalling and delamination of concrete were noted by Trow on the foundation walls. Evidence of water leakage was noted at the exterior ramp wall where the heating system electrical conduits penetrate the wall. All concrete columns were painted and contained concrete delaminations close to the base of the columns at a small number of isolated sites.

4.1.5.1.9 Garage lighting

Trow documented in its report that the garage had three different light fixtures in operation in the garage: incandescent light, high pressure sodium, and fluorescent tube fixtures. Trow noted that some of these fixtures were inoperable during the time of the report. Trow recommends that these lighting levels in the underground parking structure be checked in order that they comply in accordance with the By-Laws of the City of Mississauga .

4.1.5.2 Chloride Ion Content

In regards to the background chloride content, the corrected chloride ion content at the level of the depth of reinforcing steel on the roof slab and Level P1 which are below the threshold levels necessary for corrosion of the reinforcing steel to happen, it has now been determined concrete with the free chloride ion exceeding a specific value

will result in depassivation of alkaline in the environment around the embedded steel and initiate corrosion of steel subjected to infiltration of moisture/water. An exception of core P1C2 taken in Level P1 was noted which substantially exceeds the threshold value. The growing volumes due to by-products from oxidation activity of embedded steel will cause additional concrete deterioration, such as spalling, cracking, and delamination of concrete.

Localized patching repairs of the concrete will get rid of delaminated concrete in hollow sounding areas; however, surrounding the specified repairs, concrete with high chloride content will remain, and may cause additional continuing deterioration of concrete.

One alternative is to replace after removal a definite depth of the entire concrete slab (for example, 100mm) as well as installation of new top mat of reinforcing steel. Conversely, the deterioration of the concrete has not seemed to have reached the level of deterioration that warrants this option.

Chemical treatment and electrochemical chloride extraction are some repair options that exist in the restoration industry which removes chlorides from concrete slabs, of which reduce the chloride content and increase the alkalinity environment around the reinforcing steel, yet it must be noted these methods characteristically involve high costs to carry out. If requested by the owner, consideration may be given to remove the chlorides in the slab.

4.1.6 Conclusions

Based on the data obtained from this survey, we are able to make the following conclusions:

4.1.6.1 Topside of Roof Slab

- Trow understands that the asphalt pavement on the parking structure roof slab was recently installed. The asphalt pavement on all sides of the building was found to be generally in good condition.
- Two (2) of the four (4) cores taken in the asphalt covered areas on the roof slab do not contain a waterproofing membrane, as noted in the Core Logs, concluding that it appears when the new asphalt was placed on the slab, the waterproofing was either not done or only localized areas were waterproofed. Based on the review of the two (2) concrete cores from the roof slab that contained a waterproofing membrane, it was found to be pliant and also bonded well to the concrete surface.
- It was found that through a laboratory test that the compressive strength of the concrete of roof slab is far above that of the preliminary design strength.
- Based on the laboratory test of chloride content at both core locations, the corrected chloride ion content at the level of the depth of reinforcing steel at the roof slab is lower than threshold.

4.1.6.2 Underside of Roof Slab

- It was observed that the underside of the suspended roof slab was in generally fair condition at the time of the report.
- Evidence of water leakage and a few concrete delaminations and cracks were noted on the underside of the slab and along the expansion joints.

4.1.6.3 Exterior Ramp

- On the topside off the exterior ramp mastic asphalt traffic topping, hollow sounding areas were detected.
- In the observations made, it was noted that there was a large longitudinal crack down in the middle of the ramp.
- On both sides of the ramp, the mastic to the concrete ramp wall joint have been previously temporarily sealed to attempt to address the water leakage below, along the wall/slab joint.
- Localized delamination and cracks of concrete were visible along the ramp walls.
- Leached cracks, scaled and delamination of concrete were visible along the underside of the exterior ramp.
- The trench drain at the bottom of the ramp is in extremely poor condition, and includes heavy concentrations of leached cracks, stains, scaled and extensive concrete delaminations.

4.1.6.4 Topside of Suspended Floor Slab – P1 Level

- Throughout the sounding survey, many hollow sounding areas were detected. The survey revealed that a total of approximately 506m² or 12.06% of the accessible P1 level floor slab area was hollow sounding. This hollow sounding areas may be characterized by debonded elastomeric traffic topping or delamination of concrete. Delamination in the slab was identified in Core No. P1C1 when taken in a hollow sounding area.
- Observations concluded that there were a few areas of exposed concrete at locations where the waterproofing and traffic topping were absent, which has left the uncovered concrete susceptible to the effects of de-icing salt contaminated

water, especially during the winter months, and in turn promote concrete deterioration.

- It was noted that there was in a few areas tattered traffic topping wear course.
- The chloride ion content was below the threshold value when the background chloride content was taken into account. There was an exception to this threshold value of core P1C2, which was taken on Level P1 and substantially exceeded the limit.

4.1.6.5 Underside of Suspended Floor Slab – P1 Level

- Generally, fair conditions were note on the underside surface of the suspended floor slab of Level P1. Approximately 37 m² or 0.88% of the underside of the suspended P1 level slab contained delaminated concrete at the time the survey was conducted.

4.1.6.6 Expansion Joints

- Generally, the parking structure roof slab expansion joints look to be in adequate condition at the time that this survey was conducted.
- The concrete along the underside of the expansion joints at Level P1 is in poor condition as a result of delamination of concrete, spalling, visual proof of water leakage, and rust stains. In part, a few of the joints demonstrate evidence of previous injections made in an effort to seal the joint.
- There is a compression seal at the P1 level expansion joint.

4.1.6.7 Columns and Walls

- It was noted through a visual observation and sounding survey that both the columns in P1 and P2 levels were in good condition, with the exclusion of a few

isolated columns which contained several cracks, delaminated and/or scaled concrete near the base of columns.

- It was observed that the perimeter foundation walls were generally in good condition with the exception of some isolated locations having water stains, scaling, and delamination of concrete.

4.1.6.8 Garage Lighting

- According to the City of Mississauga By-Law requirements, the garage lighting levels do not appear to meet code. Lighting levels should be confirmed to determine if extra lighting is necessary.

4.1.7 Repair Options

Once the condition survey had been completed, based on the field observations and test results, Trow presented the client with the following repair options to consider:

4.1.7.1 Roof Slab

With regards to the garage roof slab, Trow understands that the asphalt pavement is a recent installation. As can be seen in the Core Log in **Section 4.1.1.3**, two (2) of the four (4) cores which were taken through the asphalt into the roof slab show no sign of a waterproofing membrane being present. After assessing and observing the underside and considering how recent the asphalt pavement installation is, only localized topside repairs are necessary at leaking crack and expansion joint locations. Some repair locations are required in landscaped areas. It should be understood by the client that because a portion of the roof slab is not waterproofed, eventually, with additional water interaction, further concrete deterioration can occur in the future.

Understanding the above, the following repair methods are recommended:

4.1.7.1.1 Top Surface Repairs

a) Localized Strip Waterproofing Repairs - At the locations of leaking cracks and expansion joints, remove all overlying materials. Prepare the exposed concrete surface for waterproofing then reinstall asphalt or landscaping materials as required.

4.1.7.1.2 Underside Repairs

a) Localized Concrete Repairs - Where required, locally repair concrete. Chip down to steel bars, replace any severely corroded bars and patch area with compatible concrete.

b) Chemical Injection - Where required, inject all leaking cracks on the underside of the roof slab or foundation walls.

c) For aesthetic purposes, paint underside of the roof slab (Optional).

4.1.7.2 Exterior Entrance/Exit Ramp

To repair the ramp the client can chose one of two options. These options are outlined below:

4.1.7.2.1 Localized Repairs - Option 1

a) Install an asphalt sealer over the entire ramp surface once the cracks in the mastic traffic topping system have been sealed.

b) On the underside of the ramp, carry out any necessary concrete replacement repairs.

c) Additionally, for any leaking cracks on the underside of the ramp, perform chemical injection.

d) At the trench drain, any delaminated concrete should be repaired as a through slab type restoration. Once chipped through the slab, prepare the concrete, pour back, then waterproof the inside of the trench drain.

e) If any, rout and seal any exterior ramp wall cracks.

f) Where leaking is occurring at the wall where the heating cable penetrates, repair the waterproofing membrane.

4.1.7.2.2 Total Replacement of Waterproofing Membrane, Heating Cables and Mastic Asphalt - Option 2

a) Expose the concrete slab by removing all mastic traffic topping, heating cables and waterproofing membrane.

b) If any, repair concrete top surface deterioration (i.e. delaminations)

c) If any, repair concrete underside deterioration (i.e. delaminations/spall). Carry out these repairs as through slab repairs.

d) If any, rout and seal any cracks in the exterior ramp walls.

e) Where leaking is occurring at the wall where the heating cable penetrates, repair the waterproofing membrane.

f) Once the above is completed, prepare the concrete surface including the trench drain and waterproof the entire ramp slab.

g) After installation of the waterproofing, install asphalt pavement over the entire ramp slab, while at the same time embedding new electric heating cables.

4.1.7.3 Intermediate Suspended Slab (Level P1)

To repair the intermediate suspended slab the client can chose one of two options. These options are outlined below:

4.1.7.3.1 Localized Repairs - Option 1

- a) To prevent any hazards, ensure that all loose concrete located on the underside of the intermediate suspended slab is removed by a qualified contractor to assure falling debris is avoided.
- b) At debonded/hollow sounding areas and leaking crack locations, remove the elastomeric traffic topping exposing the concrete surface of the Level P1 slab.
- c) Any delaminated concrete on the top surface of the slab should be removed and repaired using compatible concrete.
- d) Unsound concrete on the underside of the slab should be repaired as a through slab repair using compatible concrete.
- e) In areas where rebar is exposed, remove and replace any severely corroded bars.
- f) If any, remove and replace damaged piping and drains. Where pipes penetrate the slab, provide curbs and waterproofing upturns.
- g) Remove all existing expansion joint materials on Level P1 and replace with new continuous rubber gland and elastomeric concrete nosings along the full length of the joints.
- h) After concrete, expansion joint and curb installation, prepare the concrete surface for localized elastomeric traffic topping installation which will overlap the existing elastomeric system.
- i) Where necessary, reinstate parking stall lines.

j) For aesthetic purposes, paint underside of the roof slab (Optional).

4.1.7.3.2 Remove and Re-Install Entire Elastomeric Traffic Topping System (includes Concrete Repairs) on Level P1 (Intermediate Suspended Slab)

a) To prevent any hazards, ensure that all loose concrete located on the underside of the intermediate suspended slab is removed by a qualified contractor to assure falling debris is avoided.

b) Remove the existing elastomeric traffic topping system on the intermediate suspended slab (Level P1).

c) After removal, remove and repair any concrete deficiencies on the top surface with compatible concrete, as required.

d) Repair any concrete deficiencies on the underside of the intermediate suspended slab as through slab repairs with compatible concrete.

e) If any rebar are exposed during concrete repairs, check for and replace any severely corroded steel bars.

f) If any, remove and replace damaged piping and drains. Where pipes penetrate the slab, provide curbs and waterproofing upturns.

g) Remove all existing expansion joint materials on Level P1 and replace with new continuous rubber gland and elastomeric concrete nosings along the full length of the joints.

h) After concrete, expansion joint and curb installation, prepare the entire concrete surface of the slab for elastomeric traffic topping installation.

i) Where necessary, reinstate parking stall lines.

j) For aesthetic purposes, paint underside of the roof slab (Optional).

4.1.8 Cost Estimates and Repair Priority

In addition to providing the repair options to the client, Trow (the Engineer) provides the client with approximate costs for the work. By providing the approximate cost to the client, the client can then identify if they are able to afford all of the repairs. If the client is unable to afford every repair at this time, Trow also presents the client with the priority for each repair. By providing the client with a list of priorities, the client is able to visually understand which repairs are the most important and where the money should be spent in order to maintain and restore the structural integrity of the concrete elements while at the same time reducing the potential of future deterioration of the structure. The cost estimates and repair priorities from Trow's Report which was submitted to the client for the job at 530 Lolita Gardens can be seen below in **Table 6**.

Table 6 - Cost Estimates + Repair Priority (Trow)

Location	Repair Option	Estimated Repair Cost	Repair Priority
Roof Slab (Underside)	Localized top surface waterproofing strip repairs and underside concrete repairs	\$30,000	2
	Paint underside of roof slab (Optional)	(\$30,000)	2
Exterior Entrance/Exit Ramps	Option 1 - Localized Concrete and Traffic Topping Repairs	(\$30,000)	1
	OR		
	Option 2 - Total Replacement of Heating Cables and Waterproofing including		

	Localized Concrete Repairs including Trench Drain and Associated Work as Required.	\$75,000	1
Intermediate Suspended. Slab (P1 Level)	Removal of Loose Concrete from the Underside of the Suspended Level P1 Slab.	\$2,000	ASAP
	Option 1 – Local Repairs.	(\$300,000)	1
	OR		
	Option 2 - Replacement of Traffic Topping System over the Entire Slab, including Localized Concrete Repairs and Associated Work As Required.	\$380,000	1
	Paint underside of P1 Level slab (Optional)	(\$35,000)	1
	Mobilization/Demobilization	\$38,000	
	Contingency	\$50,000	
	Tender Variance (+/- 10%)	+/- \$57,500	
	Sub-total	\$517,500 to \$632,500	
	G.S.T. (6%)	\$31,050 to \$37,950	
	Total	\$548,550 to \$670,450	

() These items are Not included in Total above.

Please Note: The above estimated costs do not take into account the cost of repairs to electrical conduits, mechanical systems, new garage lighting, removal of sound concrete which may be deemed to have high half cell corrosion potential or high chloride ion content.

The cost estimates which have been determined by Trow in the above table are based on prices obtained during competitive bidding for similar projects in the Toronto area. Any actual costs are determined during the tendering period from actual quantities submitted by contractors. For this reason a variance of 10% is included in the above table.

4.1.9 General Recommendations

It is important to reduce the rate at which the concrete elements of the parking structure deteriorates. Having done the condition survey and acknowledging that 13.98% of the total accessible area was hollow sounding, Trow recommends repairing the structure as soon as possible to substantially reduce the rate of deterioration.

For the intermediate suspended slab, Trow recommends using Option 2 to repair the entire slab by replacing the whole traffic topping system while at the same time locally repairing any concrete deficiencies. Additionally, Trow recommends that all of the expansion joints on Level P1 be removed and replaced and that a new waterproofing seal is added to each joint.

In regards to the Entrance/Exit Ramp, Trow recommends using Option 2 described above as the total removal and replacement of the waterproofing and heating cables including any localized concrete repairs or trench drain work.

During the assessment of the garage, Trow noticed that the lighting levels were low, for this reason, Trow recommends having the lighting levels reviewed to confirm that they meet the city by-laws.

Lastly, it is recommended that Trow be retained for the preparation of the specifications and the duration of the construction. Trow would review all construction processes and conduct any necessary tests. If a different consultant is chosen to review the construction and prepare the specifications, it is recommended that Trow is allowed to review the specifications to ensure that Trow's survey has been properly interpreted. If Trow is not

retained for review of the specifications, Trow is not responsible for any misinterpretation of the recommendations put forth in this report.

4.1.10 Documentation

After discussing the repair options with the client and understanding which options they wish to include in the repair contract, Trow prepared a set of specifications which outlined in detail the work to be done. A section of the specification is dedicated to pricing for any interested contractors. Trow sets up a site visit in which any interested contractors may attend to help them understand the scope of work. The contractors purchase a copy of the specifications and fill in the bid form with prices that they can perform the work for. The bid form is returned to Trow for review. Typically, the lowest bidder is awarded the contract, as was the case for this project.

The bid form which the successful contractor filled out can be found on the next page. Any company or personal information has been removed for privacy reasons.

APPENDIX "D"

Project Title: 2009 Parking Structure Repairs (Repair Priority 1)
Location: 530 Lolita Gardens
Mississauga, Ontario

Bid Submitted by: _____

Schedule of Unit Prices

The following is our Unit Prices for the items listed hereunder. The Unit Prices include the specified cost, overhead, profit and any applicable taxes in force at date of tender with the exception of the value added taxes (i.e., G.S.T.). The Goods and Services Tax (G.S.T.) is shown as a separate line item.

The Contractor shall prepare this bid in ink or typewriter. Any erasures or overwriting of the prices shall be initialed.

Item No.	Description	Unit	Est. Qty.	Contractor's Bid Per Unit	Contractor's Total Bid
Part 1- P1 Slab Expansion Joint Repairs					
1.1	Concrete Repairs				
1.1.1	Ledge Beam Repairs: Remove full depth concrete and deteriorated reinforcing steel. Supply and install new concrete beam and ledge as per Detail S1 including sliding bearing pad, formwork, shoring, and concrete.	Lump Sum	60 lin.m	N/A	30,000 ⁰⁰
1.1.2	Through Slab Repairs (Supported P1 Slab on Ledge): Repair slab as per Detail S1	Lump Sum	30 m²	N/A	32,500 ⁰⁰
1.1.3	Replace existing U-shaped bars with new 10 M U-shaped bars in ledge beam repair in item 1.1.1 above as per Detail S1	Lump Sum	300	N/A	5000 ⁰⁰
1.1.4	Add continuous 15 M top and bottom bars within Ledge beam repair in item 1.1.1 above as per Detail S1	Lump Sum	N/A	N/A	5000 ⁰⁰
1.1.5	Replace existing Beam Stirrups in ledge beam repair in item 1.1.1 above with new 10 M bars as per Detail S1	Lump Sum	N/A	N/A	5000 ⁰⁰

2009 Parking Structure Repairs (Repair Priority 1)
Phase II Area of 830 Lolita Gardens, Mississauga, Ontario
BRBS00001988P

Section 00 41 00
Bid Form (2008)

Item No.	Description	Unit	Est. Qty.	Contractor's Bid Per Unit	Contractor's Total Bid
1.1.6	Add new 10 M top bars in supported slab on ledge item 1.1.2 as per detail S1	Lump Sum	240 bars	N/A	5000
1.1.7	Replace existing bars in supported slab with new 15 M top and bottom bars as per Detail S1	Lump Sum	N/A	N/A	5000
1.1.8	Allowance for welding	Per Hour	N/A	85 ⁰⁰	\$2,500.00
	a) Welder	N/A	N/A	Cost + 10%	
	b) Materials	N/A	N/A		
1.2	Expansion Joint Waterproofing				
1.2.1	Remove and dispose a 1.5 m wide strip of the existing traffic topping centered over the slab expansion joint including existing seal in the expansion joint on Level P1 and supply and install a new Wabocrete gland expansion joint system and elastomeric traffic topping system, as specified and shown on drawings	Lump Sum	60 lin. m	N/A	24,600 ⁰⁰
				Sub-total Part 1	114,600 ⁰⁰
Part 2 – Concrete Repairs (directly adjacent to the expansion joint repair area)					
(Provisional)					
2.1	Topside Slab Delamination Repairs	Lump Sum	50m ²	N/A	12,500 ⁰⁰
2.2	Through Slab Repairs	Lump Sum	50 m ²	N/A	22,500 ⁰⁰
2.3	Column Base and Wall Repairs	Lump Sum	20 m ²	N/A	10,000 ⁰⁰
				Sub-total Part 2	45,000 ⁰⁰
Part 3 – Shoring					
3.1	Shoring (All Phase 2 Slab Area) Supply and install temporary single steel vertical post shores (minimum specified load capacity of 5000 pounds) connected to 2x10 continuous sill plates bearing on P2 concrete slab spaced at 4 feet on-centre maximum in two directions.	Lump Sum	N/A	N/A	12,000 ⁰⁰

2009 Parking Structure Repairs (Repair Priority 1)
Phase II Area of 530 Lolita Gardens, Mississauga, Ontario
BRBS00001988P

Section 00 41 00
Bid Form (2009)

Item No.	Description	Unit	Est. Qty.	Contractor's Bid Per Unit	Contractor's Total Bid
3.2	Monthly Rental of Post Shores	Per month	N/A	10 ⁰⁰	
3.3	Removal of Post Shores	Lump Sum	N/A	N/A	3000 ⁰⁰
Sub-total Part 3					15,000 ⁰⁰
Part 4 - Other Items					
4.1	Mobilization/demobilization including site safety, supervision, etc.	Lump Sum	N/A	N/A	25,000 ⁰⁰
4.2	Electrical Repairs (Allowance)				\$5,000.00
4.2.1	Electrician	Per Hour	N/A		
4.2.2	Materials	N/A	N/A	Cost + 10%	
4.3	Testing Allowance	Lump Sum	N/A	N/A	\$2,500.00
4.4	Building Permit (Allowance)	Lump Sum	N/A	N/A	\$1,000.00
4.5	Contingency Allowance				\$15,000.00
Sub-total Part 4					48,500 ⁰⁰
Priority 1 Sub-Total Parts 1 to 4:					223,100 ⁰⁰
GST (5%)					11,155 ⁰⁰
Priority 1 Total Parts 1 to 4:					234,255 ⁰⁰
Priority 1 Construction Schedule					10 weeks

End of Bid Form (2009), Section 00 41 00

4.1.11 Repairs - Actual Construction

Based on the bid form from preceding pages, repairs were carried out to the structure. A complete photo log (**Figure 31 to Figure 53**) of the repairs which were carried out, with captions, can be found at the end of this report in **Appendix B**.

4.2 Survey of Professionals

In addition to reviewing actual case studies, it is also important to understand the personal opinions of practicing engineers in order to truly understand which methods should be used to analyze a concrete structure.

During my research, I developed a two (2) surveys which can be seen starting on the next page of this report.

The first survey was distributed to professionals whom have been practicing in the field of engineering for multiple years. I feel as though it is extremely important to understand how to analyze concrete structures from different points of view, for this reason, the professionals given this survey come from different knowledge backgrounds (i.e. material science, building science, structural engineering). This survey is general in nature and was developed in order to understand which methods the different professionals prefer (non-destructive or destructive) with regards to determining in-situ strength or the presence of corrosion in a concrete structure.

The second survey was only distributed to a few professionals. These professionals deal with choices regarding testing methods everyday and I felt they would be able to relate to the questions that were asked and provide the most accurate responses. This survey deals with the cost, accuracy and reliability of the different test methods which determine in-situ strength and the presence of corrosion in a concrete structure.

NDT Methods Survey - Joseph Greto (Masters Project)

a. Name: _____

b. Designation(s) [i.e. P.Eng, C.E.T]: _____

c. Company: _____

d. Number of Different Engineering Firms you have worked for: _____

e. How long have you been practicing in the Engineering Field [years]?: _____

f. Different Structures which you have analyzed the condition of [i.e. parking structure/bridges etc.]: _____

g. Different Countries you have been employed or Countries with which you are familiar with their Non-Destructive Testing Methods and Processes: _____

h. What is your preferred method (non-destructive or destructive) to analyze the in-situ strength of a concrete structure [i.e. schmidt hammer/windsor probe, etc.] and why?: _____

i. What is your preferred method (non-destructive or destructive) to locate corroded rebar in a concrete structure [i.e. half-cell, concrete resistivity, etc.] and why?: _____

j. If either answer to question h or i is a destructive method, what would be your preferred non-destructive test method be for that question?: _____

NDT Methods Survey - Joseph Greto (Masters Project)

- a. Name: _____
- b. Designation(s) [i.e. P.Eng, C.E.T]: _____
- c. Company: _____

Loss of Strength

Please fill in the columns of the chart below:

Test Method	A) Affordable? [Y/N + Approx. Cost]	B) Accurate? (Does it ever work or provide satisfactory results?) [Y/N]	C) Reliable? (Does it work often?) [Y/N]	D) Rank (1 = most preferred)
Schmidt Hammer				
Pull-out Test				
Windsor Probe				
Ultrasonic Pulse Velocity				
Other: _____				

Hidden Flaws - Corrosion

Please fill in the columns of the chart below:

Test Method	A) Affordable? [Y/N + Approx. Cost]	B) Accurate? (Does it ever work or provide satisfactory results?) [Y/N]	C) Reliable? (Does it work often?) [Y/N]	D) Rank (1 = most preferred)
Half-cell Potential				
Concrete Resistivity				
Polarization Resistivity				
Other: _____				

4.2.1 Analysis (Survey Results)

The survey responses can be found in **Appendix C**.

Surveys were received by many different professionals in the field of engineering. Responses were collected from P.Eng's, C.E.T's, LEED Ap's and E.I.T's. A total of 155 years of experience and 7 different company experiences have been taken into account with these surveys. These professionals have analyzed the concrete condition for a vast number of different structures including parking structures, balconies, post-tensioned beams, TTC tunnels, hydro substations, high and low rise residential and commercial buildings, sewage and water treatment plants, bridges, dams, silos and train stations. Also, the experience which has been questioned is not only familiar with the testing or condition assessment methods in Canada, but there is experience and familiarity with methods from the Czech Republic, Hong Kong, China, Japan and the U.S.A.

For question h) "What is your preferred method (non-destructive or destructive) to analyze the in-situ strength of a concrete structure [i.e. schmidt hammer/windsor probe, etc.] and why?", responses included, coring, schmidt hammer, ultrasound and windsor probe. In general, the majority of the professionals agreed that destructive methods such as coring should be done as it provides the most accurate results, however, in order to determine if coring is necessary, a non-destructive method such as the schmidt hammer may be necessary.

For question i) "What is your preferred method (non-destructive or destructive) to locate corroded rebar in a concrete structure [i.e. half-cell/concrete resistivity, etc.] and why?", responses included, chain drag, half-cell, test openings and pneumatic removals. In general, the majority of the professionals agreed that non-destructive methods could only provide an indication of corrosion, not identify it with 100% accuracy. It was determined that with epoxy coated rebar, the half-cell method does not work and thus, is not reliable. After discussions with these professionals, the preferred method of discovering corroded rebar would be to first chain drag the surface for any hollow sounding areas then

performing a destructive opening to confirm the results. Coring may also be necessary to determine the contents of the concrete in "sound" locations to ensure that there is low corrosion potential in these areas.

The above determined preferences for determining the in-situ strength and presence of corrosion in concrete structures was reiterated by the condition assessment specialists in the second, more in depth survey. To determine in-situ strength, coring is the most accurate, it is affordable and it is reliable. Schmidt hammer may be useful in order to determine if coring is necessary. To determine if corroded rebar are present, a chain drag will provide you with an indication and destructive methods will confirm any suspicions.

4.3 Flow Chart Diagram

After having interviewed several professional engineers and practitioners in the field of concrete restoration related work and conducting a condition survey and reviewing the methods which were put in place to repair the deteriorated concrete, a flow chart (See **Appendix D**) has been developed to aid future engineers with decisions when it comes to which non-destructive/destructive test methods to use to identify concrete deficiencies.

5.0 Conclusion

5.0 Conclusion

There are multiple ways in which concrete deteriorates. If concrete is allowed to deteriorate without being maintained or assessed on a regular basis, it may lead to a catastrophe. Nowadays, concrete structures that are weakening can be assessed using different techniques. The techniques used in the field or on site depend strongly on the mode of deterioration that is being identified and the condition or current state of the structure (i.e. Half-cell cannot be used to identify corrosion if the rebar is epoxy coated).

After having studied and reviewed two separate case studies, one of a detailed condition survey process, the other of the overall construction process (from identification of the problem to the repair process), the amount of work required to assess a concrete structure's condition should now be understood.

Additionally, by interacting with practicing professionals in the field of condition surveys of concrete structures, the findings in this report are even further developed. These professionals perform assess concrete structures and make decisions as to which testing methods apply to different scenarios on a daily basis.

With the extensive coverage of the assessment of deteriorated concrete structures which has been presented in this paper, unless there is new developments with future technologies, an engineer should be able to refer to this document and decide how to assess any concrete structure in regards to visual defects, loss of strength (in-situ strength) or hidden flaws (corrosion).

6.0 References

6.0 References

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APPENDIX A

Figure 17 - General Drawing

Figure 18 - Topside Legend

Figure 19 - Underside Visual Deterioration (Delamination)

Figure 20 - Cover Meter (Gardco.com)

Figure 21 - Graff's Rebar Scanner

Figure 22 - Offset of Trow and Graff Results

Figure 23 - Graff Electricity Scanner

Figure 24 - Drilling Core

Figure 25 - Sample Core

Figure 26 - Drill Hole Grid for Half-Cell Test

Figure 27 - Drilling the Holes

Figure 28 - Half-Cell Equipment

APPENDIX B

Figure 31 - Once the existing traffic topping system is removed, the concrete members are sounded and hollow areas are marked out in spray paint.

Figure 32 - The concrete which has been marked out is then chipped out to expose the rebar.

Figure 33 - Any markings on the underside of a concrete slab are chipped all the way through the slab, leaving the rebar in place.

Figure 34 - Any hollow sounding walls or columns are chipped down to the rebar.

Figure 35 - Ledge beams in poor condition are completely removed, as is the slab edge which rests on the ledge beam.

Figure 36 - Once chipping is completed, forms are put in place and prep work for concrete reinstallation begins.

Figure 37 - If corroded rebar are found, additional new rebar are added.

Figure 38 - To ensure there is a gap for expansion of the concrete slab, Styrofoam is used as a form as it can easily be removed after concrete placement.

Figure 39 - The ledge beam and slab edge are formed separately.

Figure 40 - In locations where the ledge beam is sound, a structural engineer visits the site to mark out in spray paint where additional stirrups are required.

Figure 41 - Before placing the concrete, in areas where bonding is important, a bonding agent should be applied.

Figure 42 - Bonding agent should be applied where a beam intersects a wall.

Figure 43 - After formwork and bonding agent has been applied, concrete is placed.

Figure 44 - Vertical concrete members such as walls and columns are also formed and poured.

Figure 45 - After concrete placement, it is covered in wet burlap and allowed time to cure.

Figure 46 - After concrete has cured, it is ready to be prepared for traffic topping installation.

Figure 47 - Before the traffic topping system can be installed any cracks must be routed and sealed.

Figure 48 - Routed cracks are filled with an epoxy material.

Figure 49 - Expansion Joint Glands are installed.

Figure 50 - Wabocrete sealant material is installed to keep expansion joint gland in place.

Figure 51 - Cracks on vertical elements are chemically injected.

Figure 52 - Once concrete surface is prepared, elastomeric traffic topping system is installed.

Figure 53 - After installation of the traffic topping system, parking stall lines are painted and any additional painting that is required is completed.

APPENDIX C

NDT Methods Survey - Joseph Greto (Masters Project)

a. Name: LUCIO (Lou) Maragna, P. Eng.

b. Designation(s) [i.e. P.Eng, C.E.T]: P. Eng

c. Company: Trow Associates Inc.

d. Number of Different Engineering Firms you have worked for: 2

e. How long have you been practicing in the Engineering Field [years]?: 16

f. Different Structures which you have analyzed the condition of [i.e. parking structure/bridges etc.]:

Parking Structures, Balconies, Post-tensioned Beams in
office Buildings, TTC Tunnels & Hydro Substations

g. Different Countries you have been employed or Countries with which you are familiar with their Non-Destructive Testing Methods and Processes: None

h. What is your preferred method (non-destructive or destructive) to analyze the in-situ strength of a concrete structure [i.e. schmidt hammer/windsor probe, etc.] and why?: Destructive - in order
to obtain in-situ strength and better result/analysis for repair
recommendations.

i. What is your preferred method (non-destructive or destructive) to locate corroded rebar in a concrete structure [i.e. half-cell, concrete resistivity, etc.] and why?: Destructive (where possible)
as Half Cell alone cannot solely be relied upon. Cores taken
over rebar can provide actual physical evidence of corrosion or condition of
rebar.

j. If either answer to question h or i is a destructive method, what would be your preferred non-destructive test method be for that question?: For h, Schmidt hammer

NDT Methods Survey - Joseph Greto (Masters Project)

a. Name: SAVERIO PRATO

b. Designation(s) [i.e. P.Eng, C.E.T]: P. Eng.

c. Company: TROW ASSOCIATES INC.

d. Number of Different Engineering Firms you have worked for: TWO

e. How long have you been practicing in the Engineering Field [years]?: THIRTY THREE

f. Different Structures which you have analyzed the condition of [i.e. parking structure/bridges etc.]:

PARKING STRUCTURES, HIGH & LOW RISE RESIDENTIAL
AND COMMERCIAL BUILDINGS, SEWAGE & WATER TREATMENT PLANTS

g. Different Countries you have been employed or Countries with which you are familiar with their Non-Destructive Testing Methods and Processes: CANADA ONLY

h. What is your preferred method (non-destructive or destructive) to analyze the in-situ strength of a concrete structure [i.e. schmidt hammer/windsor probe, etc.] and why?:

SCHMIDT HAMMER

i. What is your preferred method (non-destructive or destructive) to locate corroded rebar in a concrete structure [i.e. half-cell, concrete resistivity, etc.] and why?:

NONE, DESTRUCTIVE ONLY (I.E. CORROSION PREVENTION
REMOVAL, ETC.)

j. If either answer to question h or i is a destructive method, what would be your preferred non-destructive test method be for that question?:

CHLORIDE CONTAMINATION (HALF CELL TESTS TENDENCY, NOT
SPECIFIC LOCATION).

NDT Methods Survey - Joseph Greto (Masters Project)

a. Name: Petr VEGH

b. Designation(s) [i.e. P.Eng, C.E.T]: P.Eng.

c. Company: Trow Associates

d. Number of Different Engineering Firms you have worked for: 4

e. How long have you been practicing in the Engineering Field [years]?: 28

f. Different Structures which you have analyzed the condition of [i.e. parking structure/bridges etc.]:

bridges, buildings, dams, parking garages, silos, ~~sea~~
sewage tanks

g. Different Countries you have been employed or Countries with which you are familiar with their Non-Destructive Testing Methods and Processes:

Czech Republic, Canada

h. What is your preferred method (non-destructive or destructive) to analyze the in-situ strength of a concrete structure [i.e. schmidt hammer/windsor probe, etc.] and why?:

testing of concrete cores - reliability, can get an
idea about concrete quality at the same time

i. What is your preferred method (non-destructive or destructive) to locate corroded rebar in a concrete structure [i.e. half-cell, concrete resistivity, etc.] and why?:

half-cell or actually exposing the rebar;
in some cases ferroskan

j. If either answer to question h or i is a destructive method, what would be your preferred non-destructive test method be for that question?:

schmidt hammer for concrete strength

NDT Methods Survey - Joseph Greto (Masters Project)

a. Name: Peter Waisaner

b. Designation(s) [i.e. P.Eng, C.E.T]: P.Eng.

c. Company: Trow

d. Number of Different Engineering Firms you have worked for: 2

e. How long have you been practicing in the Engineering Field [years]?: 35 years

f. Different Structures which you have analyzed the condition of [i.e. parking structure/bridges etc.]:

parking structures, condo buildings, office towers

g. Different Countries you have been employed or Countries with which you are familiar with their Non-Destructive Testing Methods and Processes: Canada only

h. What is your preferred method (non-destructive or destructive) to analyze the in-situ strength of a concrete structure [i.e. schmidt hammer/windsor probe, etc.] and why?: Coring (destructive)

is the best but I have used the Schmidt Hammer to determine if coring is necessary.

i. What is your preferred method (non-destructive or destructive) to locate corroded rebar in a concrete structure [i.e. half-cell, concrete resistivity, etc.] and why?: Half cell is the preferred method at Trow but epoxy coated rebar will require us to review other methods.

j. If either answer to question h or i is a destructive method, what would be your preferred non-destructive test method be for that question?: I have always believed that something that penetrates the concrete surface is more representative such as a Windsor Probe.

NDT Methods Survey - Joseph Greto (Masters Project)

a. Name: CARBON LEUNG

b. Designation(s) [i.e. P.Eng, C.E.T]: C.E.T. LEED AP

c. Company: TRON

d. Number of Different Engineering Firms you have worked for: 2

e. How long have you been practicing in the Engineering Field [years]?: 14

f. Different Structures which you have analyzed the condition of [i.e. parking structure/bridges etc.]:

parking structure, High rise, underground tunnel, bridges,
re train stations.

g. Different Countries you have been employed or Countries with which you are familiar with their Non-Destructive Testing Methods and Processes: Hong Kong, China.

Toronto, Canada, Tokyo, Japan.

h. What is your preferred method (non-destructive or destructive) to analyze the in-situ strength of a concrete structure [i.e. schmidt hammer/windsor probe, etc.] and why?: ultra sound.

windsor probe, generally faster, more efficient, dependent on the
age of the structure.

i. What is your preferred method (non-destructive or destructive) to locate corroded rebar in a concrete structure [i.e. half-cell, concrete resistivity, etc.] and why?: destruction - sampling.

new structures are built with epoxy rebar which prevent most
non-destructive test methods cannot accurately detect defects.

j. If either answer to question h or i is a destructive method, what would be your preferred non-destructive test method be for that question?:

NDT Methods Survey - Joseph Greto (Masters Project)

a. Name: Domenic Dell'Aquila

b. Designation(s) [i.e. P.Eng, C.E.T]: C.E.T.

c. Company: Trow Associates Inc.

d. Number of Different Engineering Firms you have worked for: 1

e. How long have you been practicing in the Engineering Field [years]?: 29

f. Different Structures which you have analyzed the condition of [i.e. parking structure/bridges etc.]:

parking structures, balconies, bridges

g. Different Countries you have been employed or Countries with which you are familiar with their Non-Destructive Testing Methods and Processes: U.S.A. similar concrete

Investigation process & rehabilitation.

h. What is your preferred method (non-destructive or destructive) to analyze the in-situ strength of a concrete structure [i.e. schmidt hammer/windsor probe, etc.] and why?: coring and

laboratory testing - most accurate method
of determining compressive strength. Ensure that
rebar is not cut during core drilling & patch work.

i. What is your preferred method (non-destructive or destructive) to locate corroded rebar in a concrete structure [i.e. half-cell, concrete resistivity, etc.] and why?: Chain drag will determine

horizontal fractures (delaminations) in the slabs which
has already occurred due to corrosion. In order to
determine adjacent corrosion potential of embedded rebar is

j. If either answer to question h or i is a destructive method, what would be your preferred non-destructive test method be for that question?: Schmidt Hammer.

"Sound" concrete
areas, the 1/2 cell
test provides
readings on
the "corrosion
potential"
of the rebar.

NDT Methods Survey - Joseph Greto (Masters Project)

a. Name: Domenic Dell'Aquila

b. Designation(s) [i.e. P.Eng. C.E.T.]: C.E.T.

c. Company: Trow Associates Inc.

Loss of Strength

Please fill in the columns of the chart below:

Test Method	A) Affordable? [Y/N + Approx. Cost]	B) Accurate? (Does it ever work or provide satisfactory results?) [Y/N]	C) Reliable? (Does it work often?) [Y/N]	D) Rank (1 = most preferred)
Schmidt Hammer	Yes	Yes however, may also require coring to confirm results	Yes	3
Pull-out Test	Yes	Same as above	Yes	2 (new const.)
Windsor Probe				4
Ultrasonic Pulse Velocity	No	No will determine voids	No	5
Other <u>Coring + Lab Testing</u>	Yes as above	Yes	Yes	1

Hidden Flaws - Corrosion

Please fill in the columns of the chart below:

Test Method	A) Affordable? [Y/N + Approx. Cost]	B) Accurate? (Does it ever work or provide satisfactory results?) [Y/N]	C) Reliable? (Does it work often?) [Y/N]	D) Rank (1 = most preferred)
Half-cell Potential	Yes	Yes	Yes (weather sensitive)	2
Concrete Resistivity				
Polarization Resistivity				
Other: <u>Cham drag</u>	Yes	Yes to determine delamination	Yes	1
<u>GPR (ground penetrating radar)</u>	No	limited concrete only. Yes to determine voids (delaminations) in conc.	Yes	3

APPENDIX D

Flow Chart - Decision Process