A DEFECT-BASED APPROACH FOR DETAILED CONDITION ASSESSMENT OF CONCRETE BRIDGES

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ABSTRACT

A Defect-Based Approach for Detailed Condition Assessment of Concrete Bridges

Sami Moufti

Bridge condition assessment is one of the most essential elements of Bridge Management Systems (BMS). This is owing to the fact that available inputs from assessment reports are constantly interpreted for maintenance decisions and budget allocation to critical bridges within a region's inventory. Thus, performing effective bridge assessment is vital to ensure safety and sustainability of the bridge infrastructure. In practice, the evaluation of concrete bridges is mostly conducted on the basis of visual inspection, which is associated with considerable subjectivity and uncertainty inherent in human judgments. Additionally, current bridge assessment practices were found to be oversimplified, with conclusions being often drawn in absence of in-depth review and consideration of critical factors.

To remediate the existing shortcomings and ameliorate the bridge assessment process, this study proposes a fuzzy Hierarchical Evidential Reasoning (HER) approach for detailed condition assessment of concrete bridges under uncertainty. The essence of the suggested framework addresses the treatment and aggregation of uncertain measurements of detected bridge defects, in a systematic manner, to establish an enhanced platform for reliable and detailed

bridge assessment. The significant features of this methodology can be summarized in the following points. First, the proposed approach utilizes a generic hierarchy that models the several levels of a concrete bridge under assessment; namely: bridge components, elements, and measured defects. Second, the proposed model is set to account for relative importance weights of all assessment factors in the hierarchical breakdown. Third, a novel HER assessment belief structure is employed to grip probabilistic uncertainty (ignorance) in bridge evaluation, whereas fuzzy uncertainty (subjectivity) is processed through a set of collectively exhaustive fuzzy linguistic variables. Forth, Dempster-Shafer (D-S) theory is eventually applied under the proposed HER framework for the purpose of accumulating supporting pieces of evidence in a comprehensive manner. The suggested model is implemented to arrive at detailed and informative bridge element condition ratings through data acquired from two case study bridges in Canada. As it benefits from a data oriented and structured algorithm, the developed defect-based model is believed to introduce a great deal of objectivity in an otherwise subjective area of infrastructure assessment. This falls within the ultimate goal of enhancing overall public safety and well-being.

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CONTENTS

LIST OF FIGURES

LIST OF TABLES

LIST OF ABBREVIATIONS

AASHTO	American Association of State Highway Officials
ACI	American Concrete Institute
AHP	Analytic Hierarchy Process
ASCE	American Society of Civil Engineering
ASTM	American Society for Testing and Materials
BHI	Bridge Health Index
BIM	Bridge Inspection Manual
BMI	Business Monitor International
BMS	Bridge Management System
CoRe	Commonly Recognized Bridge Elements
DOT	Department of Transportation
DS	Dempster-Shafer
ER	Evidential Reasoning
FHWA	Federal Highway Administration
GPR	Ground Penetrating Radar
HCP	Half-Cell Potential
HER	Hierarchical Evidential Reasoning
IE	Impact Echo
MADM	Multi-Attribute Decision Making
MIS	Manual of Inspection of Structures
MLE	Main Longitudinal Elements
MR&R	Maintenance, Repair & Rehabilitation
MTO	Ministry of Transportation of Ontario
MTQ	Ministry of Transportation of Quebec
NBI	National Bridge Inventory
NDE	Non-Destructive Evaluation
OSIM	Ontario Structures Inspection Manual
SD	Structural Deficiency
US	United States

I. INTRODUCTION

I.1 Research Background

Bridges, roadways, oil/gas/water pipelines, and transit networks are vital constituents of a country's civil infrastructure; all of which contribute to its social and economic welfare, and compose a substantial portion of its national economy. In Canada, the total value of infrastructure in 2012 was estimated to be around $ 538 billion according to the Canadian infrastructure report card (2012). However, approximately one third of the Canadian infrastructure is in a fair or worse condition as stated by the same report. Past and forecasted figures indicate that "transportation" forms the largest portion of the Canadian infrastructure industry value, with "roads and bridges" comprising its most significant worth share as per latest estimates by the Business Monitors International report on Canadian Infrastructure (BMI 2013). However, most of the nation's bridges were built during the boom period of infrastructure construction in the 60's and 70's of the past century (Adhikari et al. 2012). With many of the nation's bridges approaching or exceeding their 50-year design life, the aging problem of the bridge infrastructure imposes great bridge preservation challenges on provincial and municipal ministries of transportation in Canada.

Bridge aging concerns are similar in the United States. According to America's 2013 Infrastructure Report Card, the average age of the nation's 607,380 bridges is currently 42 years (ASCE 2013). Statistics by the Federal Highway

Administration's report further indicate that one in nine of the nation's bridges are rated as "structurally deficient" (FHWA 2012).

The growing problem of bridge aging and deterioration has created needs for a further detailed bridge element inspection system that is able to sufficiently provide in-depth inputs for maintenance decision making and budget allocation. In order to assist informative decisions, bridge managers have been advocating the use of an updated, performance based insight of the bridge breakdown that would emphasize the detailed geometric and functional characteristics of bridges (Rehm 2013). Within this context, the first edition of the "Guide Manual for Bridge Element Inspection" was published in 2011 to introduce an improvement on the widely used Commonly Recognized (CoRe) system of bridge elements, and to build on the concept of element-level condition rating (AASHTO 2011). The new guide reconfigures the condition rating scheme to comprehensively capture bridge elements' distress indicators; providing in-depth assessment of bridges, and promoting detailed data reporting to fully support agency decision making and inventory management. As a result, the FHWA has recently started works on updating the widely used 1995 National Bridge Inventory (NBI) coding guide to accurately reflect the condition and performance of highway bridges, and to integrate the newly developed guide for detailed bridge inspection data acquisition (Lwin 2012).

Accordingly, there has been a parallel trend among transportation agencies towards the adoption and implementation of element level inspection. This is due to the fact that a further in-depth inspection provides more details for later analysis and implementation in effective deterioration curves and performance models (Markow and Hyman 2009). Newly established or updated guidelines, as well as the recent state of the practice, suggest increased incorporation of a defect-based bridge element condition rating to achieve more insightful assessments.

I.2 Research Problem Statement

Although bridge assessment is a safety-critical process for the public, several limitations still exist within the current practice and implementation. First, studies have reported the involvement of poor accuracy and uncertain judgments in bridge inspection. This is attributable to the considerable association of human subjectivity and the amount of fuzzy information in the evaluation process; leading to vagueness in quantifying the real condition of a structure (Phares et al. 2001; Jain and Bhattacharjee 2011). Second, a considerable amount of uncertainty in the current bridge assessment practice stems from ignorance, lack of data, or inability to precisely assess bridge elements with subsurface deterioration. Third, there exist little or no direct incorporation of structural defects' measurements in the overall bridge condition rating process.

It is has been the subject of on-going research to improve the bridge rating practice and account for some of the above mentioned shortcomings. Yet, most of the implemented rating models in practice suffer from some limitations. The commonly used bridge conditions rating scales, such as the Bridge Condition Index (BCI), employ solid/deterministic grades that do not take into consideration the gradual transition from one condition state to another (Jain and Bhattacharjee

2011). In addition, the common practice still lacks efficient and effective treatment of the involved uncertainties and subtle subjective nature of the assessment. Also, common rating practices don’t account for the structural role and relative importance of different bridge components and elements towards the overall evaluation (Wang and Elhag 2008).

In summary, the research problem statement can be defined as:

“There is a need for an objective bridge condition assessment model that integrates weighted structural elements, incorporates detected bridge defects, and takes into account uncertainty and incomplete data”.

I.3 Research Objectives

To remediate the above mentioned shortcomings in the bridge condition assessment process, the main goal of this research is set to propose a systematic, objective, and data oriented approach for a comprehensive condition assessment of concrete bridges under uncertainty. To achieve this goal, research sub-objectives are defined as follows:

- Identify and study the different types of defects which develop in various elements of a concrete bridge.
- Develop a defect-based bridge condition assessment model that accounts for inspection uncertainties.

I.4 Summary of Research Methodology

This research aims at developing a comprehensive assessment tool that takes into consideration all the stated limitations and achieve the targeted objectives. To accomplish this, a comprehensive state of the art and practice review of literature is conducted; including an extensive overview of deterioration mechanisms of concrete bridges, commonly employed inspection techniques, ways of managing bridge information, and available bridge rating systems. This research also reviewed several topics related to the existing concerns of subjectivity in grading, inspection uncertainties, and the currently implemented assessment aggregation techniques.

The development of the hereby proposed bridge assessment framework went through several steps as follows:

- Identifying a generic condition assessment structural hierarchy of concrete bridges. This is achieved by breaking down the concrete bridge structure into its fundamental components and elements. Further, this step expands on identifying the major defects -on the basis of which- a bridge is going to be evaluated.
- Adopting a relative structural weighting approach to establish relative importance weights of the various bridge components, elements, and defects identified in the first step. Relative structural weights are obtained through expert surveys.
- Establishing a unified fuzzy grading scheme that will form the basis of the rating process and treat the subjective and judgmental nature of the

assessment. This is essentially done by collecting information about severity and extents of all possible bridge defects identified in step one. The fuzzy grading scheme will map defect extents to an order of descending fuzzy grades, laying grounds for a detailed assessment of all detected bridge defects.

- Constructing a generic hierarchical evidential reasoning bridge model for an overall assessment of the entire structure. Having the ability to handle probabilistic uncertainty or ignorance in the assessment, the D-S theory is implemented in this framework as an accumulating engine for all the supporting pieces of evidence in what should be a comprehensive multi-leveled condition assessment model for concrete bridges.

I.5 Thesis Organization

The thesis in hand is organized to represent the best product of the work undertaken through the course of this research endeavor. This includes a high end review of literature, data collection, research methodology, implementation and drawn conclusions and recommendations. It is structured according to the following chapters:

Chapter II compiles a comprehensive literature review about concepts in bridge management, inspection, and testing. This includes an overview of bridge inspection, inspection types and intervals, bridge inspection techniques, and condition rating practices. This chapter also offers an overview of the implemented Artificial Intelligence (AI) & Multi-attribute decision Making (MADM) methods in the proposed bridge condition assessment model. A thorough explanation is provided on the Analytic Hierarchy Process (AHP), fuzzy set theory, and the Hierarchical Evidential Reasoning (HER) algorithm. Additionally, this chapter reviews some of the pertaining previously developed assessment models as well as highlighting their limitations.

Chapter III fully describes the different steps adopted through the development of the proposed methodology. This entails the identification of different bridge assessment factors at different hierarchical levels, the development of a unified defect grading scheme, and the recursive assessment aggregation method implemented in the model.

Chapter IV demonstrates the undertaken procedures for model data collection; including survey layout and sections, survey respondents, and data acquisition/ analysis.

Chapter V lays out the resultant weighted hierarchical assessment factors for the proposed model. An illustration of the model assessment aggregation algorithm is additionally presented. Eventually, two case studies are showcased as a proof of concept and practical application of the suggested model.

Chapter VI draws relevant conclusions to the presented research. Further, it demonstrates the limitations of the proposed assessment model and application, as well as offering recommendations and hints for future enhancement.

II. LITERATURE REVIEW

II.1 Deterioration of Concrete Bridges

Concrete bridges are susceptible to many factors that may lead to deterioration over the course of their service lives. Environmental factors, such as freeze-thaw cycles and moisture attacks, usually bring about harmful effects on the exposed concrete elements. In addition, distress loads imposed by traffic and vehicular actions have their toll on the gradual wear of bridge elements in contact. The deterioration of a concrete bridge is commonly demonstrated in several types of defect mechanisms that may develop at the surface and/or subsurface of its elements. Figure 1 features an array of different potential deterioration factors.

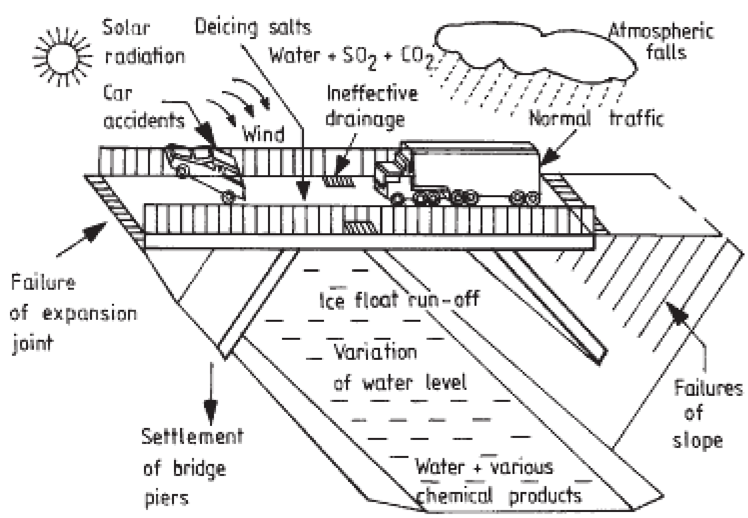


Figure 1: Various Factors Leading to Bridge Deterioration (Radomski 2002)

As much as factors imposed over a bridge life cycle can impact its condition, the efficiency of practices during the construction phase can be crucial as well to its future performance. The correct laying out of the reinforcement bars and the adequate compacting and curing of concrete can drastically influence its durability, so does the provision of appropriate cover and insulation. A combination of poor construction practices and severe exposure conditions may cause a bridge structure to rapidly deteriorate and prematurely reach an obsolete or deficient state (TN Zealand 2001).

Corrosion of steel reinforcement in reinforced concrete structures has always been regarded as the biggest problem that triggers and factors in many subsequent damages as the structure ages. In fact, reinforcing steel bars are naturally protected in newly constructed concrete structures as they are surrounded by highly alkaline cement environment. This helps in the formation of a passive oxide layer that wraps around the steel bars. The protective passive film can be preserved as long as the pH levels are kept above 9 (Penttala 2009). However, after some timespan, passivity gets normally broken and corrosion starts to formulate due to two main mechanisms; namely: carbonation, or chloride contamination.

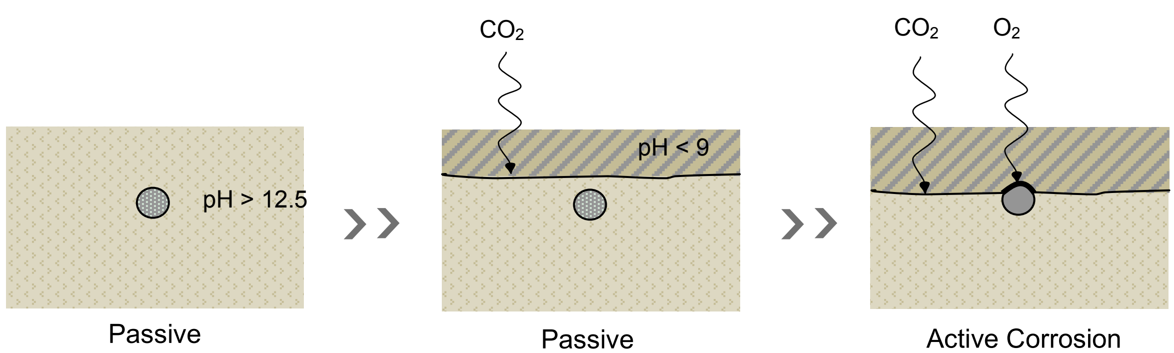


Figure 2: Initiation of Reinforcement Corrosion by Carbonation (adapted from Penttala, 2009)

Carbonation-induced corrosion commonly happens in concrete that is exposed to factors of high humidity and moisture penetration. As such, dissolving carbon dioxide CO_2 particles react with the alkali particles already present in concrete, such as calcium hydroxide $Ca(OH)_2$ and sodium hydroxide NaOH. As alkali properties in concrete were maintained by hydroxide particles, their reaction with CO_2 will result in reducing the pH levels in concrete to values less than 9 (Figure 2). This gradually causes disruption to the protective layer and exposes the steel bars to corrosion, provided sufficient presence of oxygen and moisture. Penttala (2009) reported four factors that were found to have influence on the carbonation process: volume of available CO_2 in the ambient air, permeability of the concrete cover over the top of reinforcement, moisture content in pores, and the concrete cover thickness.

The protective oxide layer can also be damaged by chloride. Chloride-initiated corrosion occurs when chloride content- in the pore water surrounding steel bars- reaches a certain threshold beyond the safe alkalinity level. Ingression of chloride ions through the concrete cover in bridge decks is usually brought about by many factors, such as salt contamination in near coastal regions, the presence of salts in aggregate or admixtures, or from the application of de-icing salts during winter (TN Zealand 2001). As the structure ages, the increased concentration of chloride ions Cl^- over the top steel bars will result in a higher negative charge (anode). The outcome of which will create a potential difference with the less negative bottom bars (cathode). With the available pore water acting as an active electrolyte, localized electric circuits form; allowing electrons and OH^- ions to flow between the anodic and cathodic poles. This type of the electrochemical configuration is referred to as corrosion macro-cell (Pincheira et al. 2008). A second type of chloride related corrosion; corrosion micro-cell, forms when both anode and cathode regions exist on the same steel bar. This can be made possible by cases of uneven chloride concentration over a reinforcing bar. Surface regions in the vicinity of high Cl^- ions turn into anodes forming a difference in potential with other less chloride-infected regions (cathodes). Figure 3 illustrates the typical electrochemical corrosion process. As can be seen from the figure, positive metal ions Fe^+ are generated and dissolved in pore water as ferrous particles lose electrons e^-. This process can be represented by the following oxidation reaction:

$$Fe \rightarrow Fe^{2+} + 2e^- \qquad [1]$$

While at the cathode, the incoming electrons react with the available oxygen and water particles to form hydroxyl ions OH^- as shown in the following reduction reaction:

$$O_2 + 2H_2O + 4e^- \rightarrow 4\ OH^- \qquad [2]$$

The resulting OH^- ions flow to the anode and, together with the abundant ferrous ions $Fe2^+$, form ferrous hydroxides $Fe(OH)_2$ that accumulates at the anode as rust. This electrochemical process will continue to get its driving force as long as the potential differences between anodes and cathodes are sustained in the presence of moisture and oxygen. The end product of this repeated process (rust) will continue to reproduce over cathodic regions of the reinforcement. This will yield to a substantial increase in the volume of steel bars over the years, which will subsequently subject the surrounding concrete material to overwhelming internal stresses. Thus, low-strength pockets of the surrounding concrete will get damaged in the form of internal cracks. As corrosion gets more severe, those internal cracks may gradually cause loss of bond and partial separations of concrete (delamination) over the reinforcement layer. The situation gets even worse as several delaminated regions form into spalls that escalate to the concrete surface, causing serious structural disintegration.

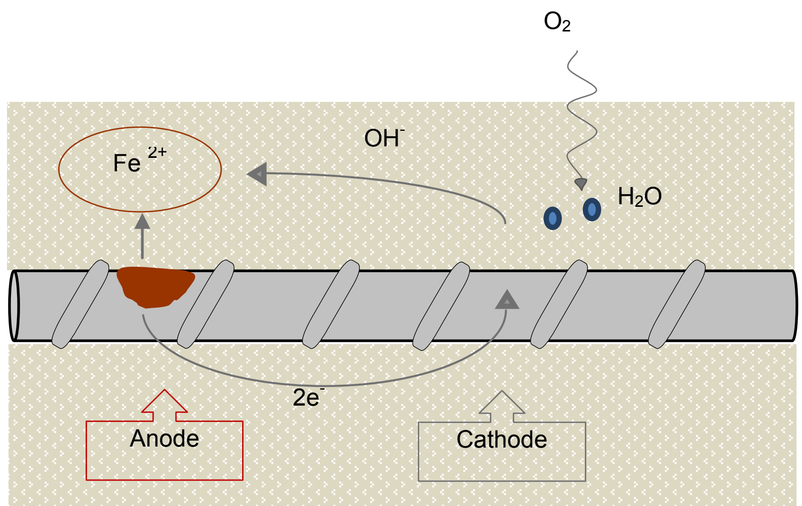


Figure 3: The Process of Steel Reinforcement Corrosion

II.2 Bridge Inspection

Bridge inspection aims at recording observations of the state of all bridge elements by well-trained and experienced personnel, as part of what should be a strong reporting system that forms the basis of any logical deductions and conclusions on the state of the inspected bridge (FHWA 2002). As one can anticipate, bridge inspection in the past did not exist in the same fashion that we have nowadays. Standards and methods for inspection have gone through ample evolution, and the modern bridge inspection philosophy had to benefit from accumulated experience and safety-critical incidents.

II.2.1 Bridge Inspection Overview

In an article that sheds light on one of the most tragic accidents in the history of bridge failures in the United states, Lichtenstein (1993) states the fact that bridge engineers and field practitioners were commonly in a normalized state until a dramatic collapse of a bridge hits the news; causing a "pendulum swing" type of reactions among federal and state agencies. As such, the accelerated and intense mobilization of engineering and research resources after such tragedies has brought about formations of national scale investigation committees and the eventual stipulation of new bridge design and inspection standards.

With no exception, the creation of the National Bridge Inspection Standards (NBIS) came along following the same trend. In December 1967, the famous "Silver Bridge" connecting West Virginia to Ohio collapsed into the Ohio River, killing 46 people, and leaving a tragedy that significantly altered the history of bridge safety in the US. The arousal of public and national interest in bride safety has prompted the US government to legislate the federal highway act of 1968, demanding the establishment of firm national bridge inspection standards (Alampatti and Jalinoos 2009).

As a consequence, the National Bridge Inventory Standards (NBIS), issued in 1971 by the Federal Highway Administration (FHWA), were set forth to establish a uniform program that regulates minimum requirements for bridge inspection and inventory reporting for all state departments of transportation in the US. As a consequence, and to fulfill the requirements of NBIS, structural bridge inventory data and appraisals have been collected from every state on an annual basis, and aggregated in a federal system that is called the National Bridge Inventory (NBI).

Continuous research in the area has flourished ever since the establishment of the national standards. Consequently, FHWA and the American Association of State Highway Officials (AASHTO) have issued several manuals and revisions for bridge inspection coding and rating, special type bridges, fracture critical members and culverts inspection. National attention was drawn once again; this time towards underwater inspection, with the unfortunate 1987 failure of the "Schoharie Creek Bridge" in New York. Following technical advisories on bridge evaluation for scour vulnerability, the FHWA issued significant revisions to NBIS with adjustments on inspection frequencies, increase in inspector qualification requirements and further guidelines on under water and scour inspections (FHWA 2002).

Along with the establishment of NBIS, several manuals were subsequently developed in the states. Among those to mention is the FHWA's Bridge Inspector's Training Manual (BRIM). Being first published in 1970 and recently revised in 2002, BRIM has established principles for inspector training courses covering inspection standards and procedures. Many updates were made previous to the most recent 2006 edition by the US Department of Transportation and the National Highway Institute (FHWA 2002).

The earliest forms of bridge inspection standards were incorporated in the AASHTO Manual for Maintenance Inspection of Bridges, which was released in 1970 and received several updates since then. Another major manual that helped establishing a common federal condition reporting code is the FHWA "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges", which was also under continues development since it's early publication in 1972 (FHWA 1995). The accumulated experiences over the years, in addition to learned lessons and advisories from state bridge experts, have always urged FHWA and AASHTO to accommodate upgrades in newly adapted inspection guidelines. The latest publication in this regard is the second edition of the AASHTO manual for bridge Evaluation (AASHTO 2011), that surpasses older AASHTO guides for bridge condition (AASHTO 1994; AASHTO 2003), and benefits from all gathered experience in guiding bridge agencies to adopt the latest inspection practices and evaluation procedures in accordance with NBIS.

On the higher managerial and agency levels, increased public accountability and the need towards a more efficient allocation of bridge maintenance funds have urged FHWA and Caltrans to sponsor the development of “Pontis” Bridge Management System in the early 1990’s (FHWA 2002). Offering a menu of as many as 160 bridge elements, Pontis has provided a greater level of details than the earlier minimum requirements of NBIS, and allowed different states to have common grounds in reporting their bridge management data to the federal inventory (Thompson and Shepard 2000). Shortly after the creation of Pontis, and based on the gained experiences, a new standard named “Commonly Recognized Bridge Elements (CoRe)” was developed under the FHWA guidance. Being accepted and published by AASHTO, (CoRe) would become a more generic guide for a more detailed inspection of bridge elements with fewer ties to Pontis. CoRe elements provide a uniform basis for data collection among the states and enable consistent data collection and reporting to the federal NBI system.

II.2.2 Inspection Types and Frequencies

Bridge inspection is considered as a main, and perhaps one of the most essential elements of a bridge management system (BMS). Inputs from bridge inspections are implemented for maintenance decision making and budget allocations to the deserving, intervention-needy bridges in a state’s or province’s inventory. Carrying out periodic inspections is vital to ensure the safety and sustainability of the bridge infrastructure, and is considered as a continuous condition assessment process. The goal of periodic field inspection is to report on the physical condition of bridges. This is mostly done on a routine basis by carrying out detailed visual inspections of structures on site to detect and evaluate defects or damages on their elements. In addition, urgent out-of-schedule inspections may take place on an ad-hoc basis after natural disasters such as earthquakes to evaluate the degree of caused damage to bridges in the affected region.

Table 1: US Federal Regulations for Inspection Intervals (Hearn 2007)

Inspection Type	Standard Frequency (months)	Maximum Frequency (months)
Routine	24	48
Underwater	60	72
Fracture-Critical Member	24	-

The most common practice for evaluating a bridge's health is through visual assessment and judgment. However, mere visual inspection might not be sufficient when investigating serious flaws or damages. For those instances, bridge owners might go beyond visual inspection and require the involvement of Non-Destructive Testing (NDT) to conduct more in-depth condition surveys. In a summary of factors that may influence the selection of inspection timing and procedures, AbuDayyeh et al. (2004) included bridge age, bridge size, traffic density, impact of traffic disruption, availability of equipment and personnel, geographic location, and/or environmental conditions (see Figure 4).

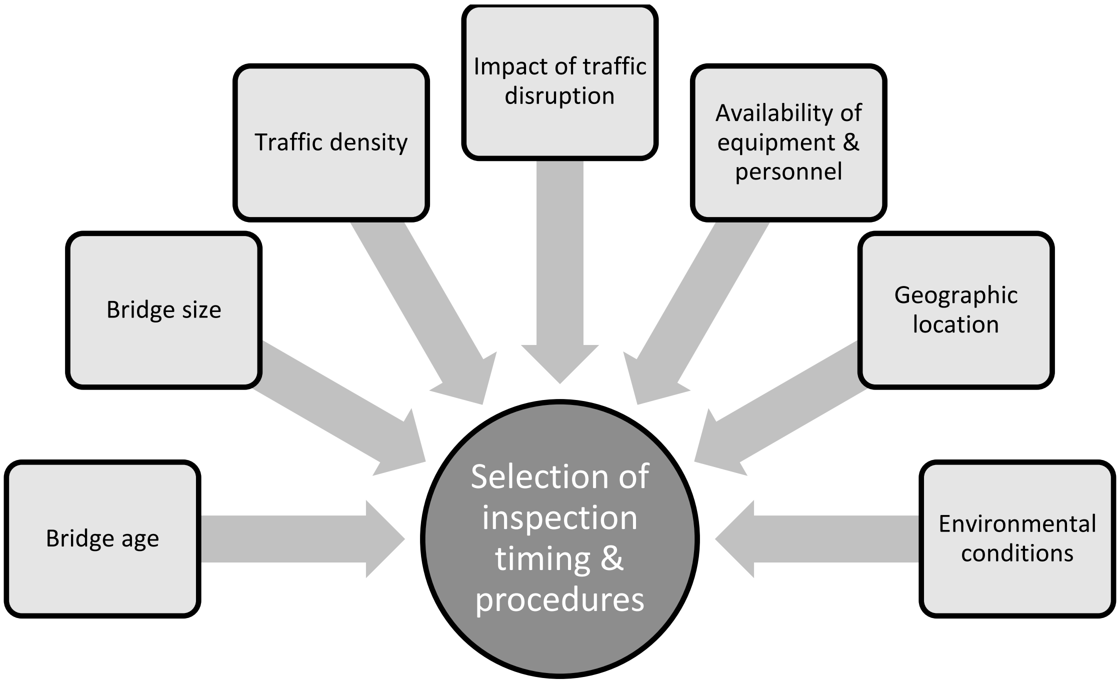


Figure 4: Factors that Affect the Selection of Inspection Timing and Procedures (AbuDayyeh et al. 2004)

According to Hearn (2007), eight different types of bridge inspections are established by the U.S. federal regulations, three of which should be conducted on strictly defined inspection intervals (Table 1). The several default types of bridge inspections are established to set consistent reporting standards among the state DOTs, with each inspection type being conducted at different time intervals and serving a distinct purpose. As a customary practice, most DOTs run routine inspection on the majority of their bridges on a 24-month basis. Routine inspection of bridges is a significant practice that is carried out by state or provincial transportation agencies to maintain a well-updated periodical assessment of their asset of bridges. Thus, this practice is framed into many significant aspects that are to be considered by both field professionals and operation managers. While some of those aspects are left for individual DOTs and bridge owners to manage and set the pace for, federal regulations have controlled four essential aspects of routine bridge inspections, namely: structure types, inspection frequency, inspector qualifications, and process regulations (Figure 5).

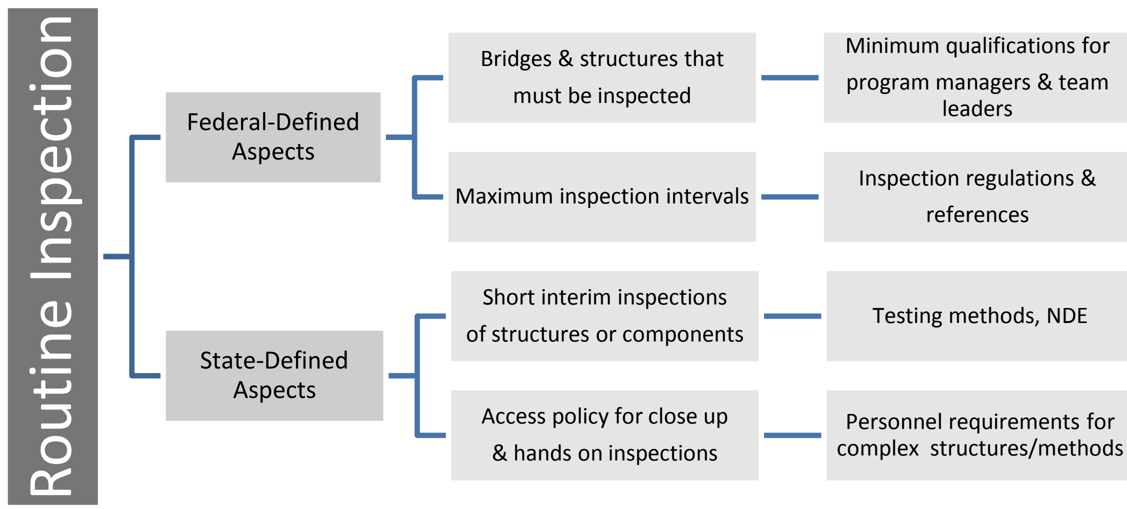


Figure 5: Federal and State-Defined Regulations for Routine Bridge Inspection in the U.S. (Hearn, 2007)

As mentioned above, there is a diversity of well-established types of bridge inspection in practice. The different inspection forms may vary in purpose, inspected portion of the bridge, and/or applied assessment tools. Some of these types include hands-on inspection, underwater inspection, damage inspection, etc. the interested reader may refer to (Hearn 2007) for a full list of various bridge inspection types.

II.2.3 Inspection Techniques

The Inspection of concrete bridges has evolved in the recent years. Although visual inspection is still widely accepted as a common practice, many NDE techniques have been developed and widely adapted to objectify the process and make it more fast and reliable. The following sections will discuss the most pertinent inspection techniques.

II.2.3.1 Visual Inspection

Visual inspection is considered the most basic, yet the most prevalent bridge inspection technique. The goal of field visual inspection is to report on the physical condition of bridges and to evaluate their status. This is mostly done on a routine basis by bridge engineers or inspectors, who carry out comprehensive visual inspections of structures on site to detect and evaluate the deterioration and spot damages on the different structural elements.

In general, visual inspection is required to follow well established procedures established by bridge inspection manuals and codes issued by transportation agencies. In addition, certain requirements are commonly set forth to regulate inspectors' qualifications and data recording formats (FHWA 2002; MTQ 2012; MTO 2008). The amounts of funds, time and efforts involved in experimental investigations render visual inspection more practical and appealing as a condition assessment strategy. Visual inspection can provide valuable information on a bridge's condition; especially that most bridge defects (such as cracks, spalls and leaching) can be visually detected. However, results obtained from visual inspection heavily depend on the expertise and judgments of bridge inspectors, yielding them to be primarily qualitative and subjective (Jain and Bhattacharjee 2011). Nevertheless, Data from Visual inspections are still regarded as the standard input to assist in maintenance decision making and evaluate needs for further investigations.

II.2.3.2 Non-Destructive Evaluation Methods

Due to the several drawbacks of visual inspection, many Non-destructive Evaluation (NDE) techniques have been introduced to augment the evaluation process. This section aims at providing the reader with a brief overview of some of the popular NDE testing methods that are implemented for onsite assessment of concrete structures. More focus was hereby given to NDE techniques that are most commonly used in practice, or predominantly cited in literature, as fitting the purpose of evaluating reinforced concrete bridges. Each of the below mentioned tests can be single-handedly applied to evaluate certain aspects of concrete bridges; but one test might as well be combined with a second or third test to cover a wider breadth of testing capabilities in a complimenting manner. Included in this section is the description of Half-Cell Potential test, Impact Echo test, and acoustic methods. Further NDE techniques are explained in detail in appendix A, including Concrete Resistivity, Infrared Thermography and Ground Penetrating Radar (GPR).

II.2.3.2.1 Half-Cell Potential Test (HCP)

This test is considered to be one of the most widely applied NDE tests for corrosion assessment and evaluation. It is much easier to conduct than many other methods including nuclear or radio-active tests, which are deemed to be more complicated. Relative ease of administration, fairly low cost, and simple data interpretation have promoted half-cell potential (HCP) to be a very popular test in measuring reinforcement corrosion in concrete structures. The test is conducted according to the configuration shown in Figure 6. As can be seen from the figure, the apparatus consist of the reference electrode (half-cell), connecting wires, and a high impedance voltmeter.

The principle of the test, in its basic form, relies on measuring the potential difference between steel re-bars and the concrete surface. This is achieved by wire connecting an exposed steel reinforcing bar to one terminal of the voltmeter, while having the other terminal linked to a reference probe which rests on the concrete surface and forms the other half of the cell. The concrete cover must be moist enough in order for it to act as an electrolyte. This will allow excess electrons to flow from the corroded rebar to the reference probe through the damp concrete cover due to difference in potential. Therefore, a rebar with a higher corrosion probability will be identified by a greater potential difference pointed out by the voltmeter.

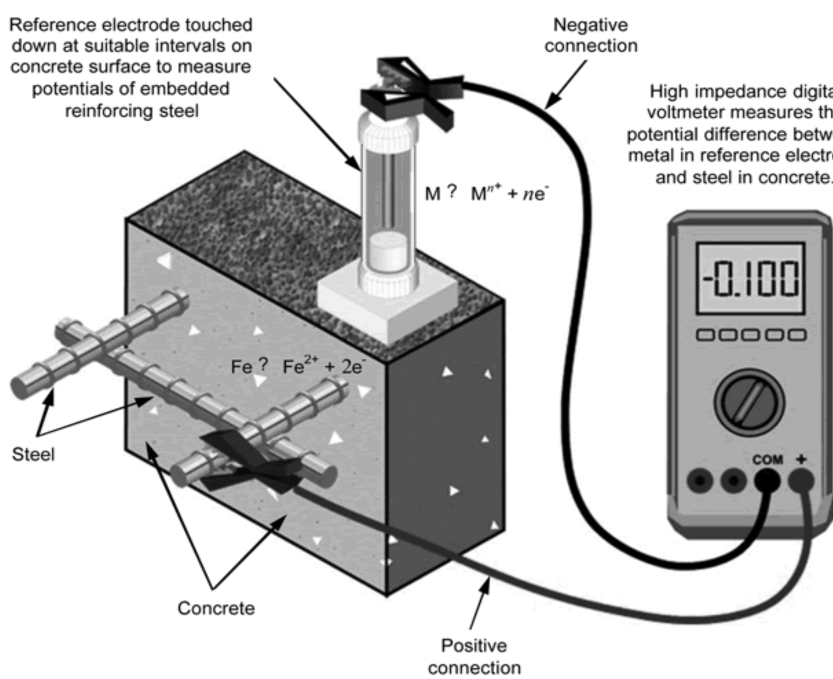


Figure 6: Half-Cell Potential Test Apparatus (ASTM C876-09)

In addition to the sufficient moisture required by this test, reinforcing steel bars should be free of any coating that might hinder their electrical connectivity (ASTM C876-09). Therefore, the test can't be run on epoxy-coated bars. By manually placing the reference probe on predefined grid points, a map illustrating corrosion potentials can be generated. ASTM C876 provides general guidelines on performing Half-Cell potential test to evaluate the probability of corrosion activity in reinforced concrete structures. According to those guidelines, potential readings are interpreted as per Table 2 to indicate corrosion probably. Gucunski et al (2010) stated that the HCP technique has been extensively utilized by bridge engineers as a standard corrosion measurement; however, the measured potential values may be influenced by concrete resistivity and cover thickness. In most traditional cases of performing HCP test, holes through concrete cover have to be dug so as to connect the bars to the voltmeter. Therefore, many sources, including Lai et al. (2012), argue that traditional HCP test is partially destructive and may not be looked at as a pure NDE technique. The later study proposed a modified HCP method that alternatively reports deferential potential with the use of two probes, by having both of them placed and moved on the concrete surface.

Table 2: HCP Readings and Interpretations (ASTM C876)

Half-Cell Potential Reading	Interpretation
More positive than -200 mV	90% probability of no active corrosion
Between -200 mV and -350 mV	Corrosion probability is uncertain
More negative than -350 mV	90% probability of active corrosion

II.2.3.2.2 Impact Echo Test

The Impact Echo (IE) test is one of the major techniques that belong to the ultrasonic tests family. It is generally based on seismic analysis and transmission-reception of low frequency impact-generated waves through the tested material. The IE test has found various applications in the depth measurement and internal flaws detection of concrete bridge decks. This includes the detection, location, and range approximation of subsurface concrete defects; such as: internal voids, honeycombing, and delaminations.

Standard guidelines for carrying out the IE test are fully described in ASTM C 1383. The test principle is based on an instant, mechanically induced, stress wave that propagates through the tested structural material, and gets reflected by internal defects or intruding substances. When the hemispherical fronts of the stress waves reach an internal interfaces or discontinuities, such as boundaries or voids, energy reflections (echoes) are mirrored in multiple directions within the structure; thus exciting local modes of vibrations that can be received and recorded by a transducer positioned near the small steel sphere that originates the impact. The transducer generates a voltage that is proportional to the received displacements or vibrations, and transfers a “voltage-time” signal to a processor where it is mathematically analyzed into a spectrum of amplitude vs. frequency. Frequency peaks in the spectrum are commonly associated with multiple reflections against thin or delaminated layers (Carino 2004).

Carino (2004) describes the test’s main principle as an analysis of the frequency of displacement waveforms. When the stress pulse is generated by mechanically impact the surface of the tested material, it propagates back and forth between the internal defect and the surface. The reflection of this pulse creates a characteristic downward displacement every time it arrives to the top surface (Figure 7). Thus, a periodic wave is formed with a known wave length (period) calculated by dividing the travel path (2L) by the wave speed. As wave frequency is equivalent to the inverse of the period, f of the characteristic displacement pattern is equal to:

$$f = \frac{C_{pp}}{2L} \quad [3]$$

Where C_{pp} is the plate P-wave speed; determined from performing the IE test on a part of the structure (or plate) with known thickness. Therefore, as the dominant frequency of the waveform is calculated, the depth or distance to the reflecting internal flaw can be determined as follows

$$L = \frac{C_{pp}}{2f} \quad [4]$$

A relatively straight forward application of the IE test is to determine the actual depth of plate-like concrete structures such as slabs. This application has been standardized ASTM C1383, particularly for plate like structures in which any lateral dimension is at least six times the thickness. Defect detection capabilities of IE range from cases of delamination or internal voids to rather complex cases of micro-cracking (Carino 2004).

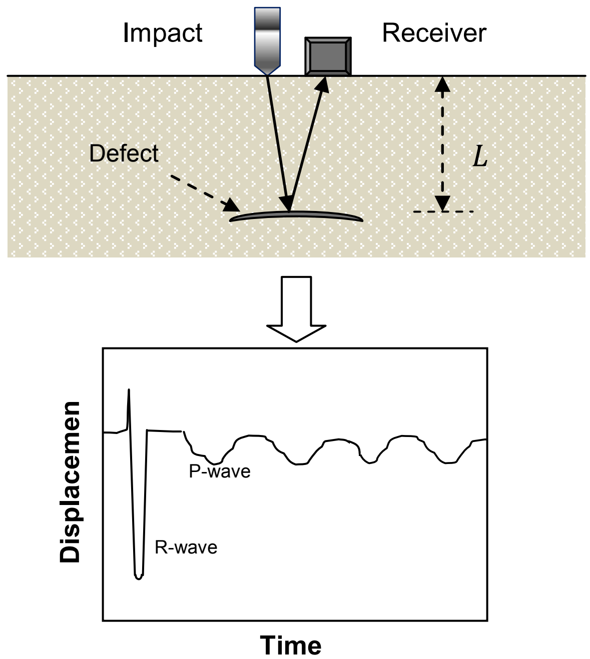


Figure 7: Impact-Echo Method

II.2.3.2.3 Acoustic Methods

Acoustic methods are based on the sound effect produced by a hammer or metal chain against the surface of concrete bridge elements. Metal chains are commonly used for approximating near surface delamination of concrete bridge decks, while hammer tapping is usually utilized to test vertical bridge elements. Chain dragging is perhaps the simplest and the most widely used test to detect areas of delamination over the top reinforcement bars right below the surface of exposed concrete decks. The test procedure and apparatus are described in the guidelines of ASTM D4580 with the latest version in 2012 (ASTM D4580-2012). It is mainly focused on detecting the subsurface delamination by dragging a steel chain over the top of concrete decks. While sound concrete areas will produce a clear ringing sound, areas of delamination can be recognized by the operator when encountering a dull or hollow sound (ASTM D4580-2012). This method is mostly not intended to be applied on bridge decks that are overlaid with asphalt, since the overlay might act as an insulator that hinders the transmission of sonic signals. Chain drag test is best utilized for uncovered concrete decks. It is still, however, applicable on concrete decks that have been overlaid with portland cement concrete mixtures (Scheff and Chen 2000).

II.3 Bridge Information Management and Condition rating

As discussed above, the inspection of bridges is a key task to establish essential knowledge about their condition. The timely reporting of information about condition and deterioration of individual bridges in an agency's network helps building a big picture about the overall network performance. All of which is mainly needed to assist engineers in managerial levels figure justifiable maintenance decisions within the accessible funds. However, inputs from qualified bridge inspectors are only considered subjective attempts to assess the condition of a bridge element or component numerically. With no calculations performed, a typical bridge inspection report merely quotes numbers or condition grades for the various bridge elements quantifying their deterioration levels. Those numbers are later interpreted, together with other factors, to calculate condition rating indices and decide on maintenance strategies and prioritizations (Ryall 2010).

II.3.1 National Bridge Inspection Standards (NBIS)

As a consequence of the "Silver Bridge" collapse in Ohio 1967, the Federal Highway Administration (FHWA) issued the National Bridge Inventory Standards

(NBIS) in 1971 to establish a uniform program that regulates the minimum requirements for inspection types and procedures, inspection intervals, inspector qualifications, and inventory reporting for all state departments of transportation in the US. Guidelines for conducting bridge inspections in accordance with the NBIS mandate were firmly set up in the “Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation’s Bridges” by the FHWA. Commonly referred to as “The Guide”, this manual helped establishing a common federal condition reporting system among transportation agencies in the states. It has been under continuous development since its early publication in 1972 (FHWA 1995).

The FHWA *Recording and Coding Guide* provides an alpha-numeric rating system to be used by bridge inspectors for rating bridge items (Markow and Hayman 2009). The rating system for bridge elements conditions and structural appraisals is defined by 9-point scale; with 9 being excellent/new condition, and zero implying absolute failure (Table 3).

Table 3: Rating Scale for Bridge Decks, Superstructure, and Substructures by the FHWA Guide (FHWA 1995)

Rating Code	Description
9	*EXCELLENT CONDITION*
8	*VERY GOOD CONDITION*: no problems noted.
7	*GOOD CONDITION:* some minor problems.
6	*SATISFACTORY CONDITION:* structural elements show some minor deterioration.
5	*FAIR CONDITION:* all primary structural elements are sound but may have minor section loss, cracking, spalling or scour.

4	*POOR CONDITION:* advanced section loss, deterioration, spalling, scour.
3	*SERIOUS CONDITION:* spalling, or scour have seriously affected primary structural Components. Local failures are possible. Fatigue cracks in steel or shear cracks in concrete may be present.
2	*CRITICAL CONDITION:* advanced deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present or scour may have removed substructure support. Unless closely monitored, it may be necessary to close the bridge until corrective action is taken.
1	*"IMMINENT" FAILURE CONDITION:* major deterioration or section loss present in critical structural components or obvious vertical or horizontal movement affecting structural stability. Bridge is closed to traffic but corrective action may put back in light service.
0	*FAILED CONDITION:* out of service—beyond corrective action.
N	Not applicable.

II.3.2 NBI Condition Rating

To fulfill the requirements of NBIS, structural bridge inventory data and appraisals have been collected from every state on an annual basis, and aggregated in a federal system that is called the National Bridge Inventory (NBI). Typically, NBI requires the condition rating of three main bridge components, namely: deck, superstructure, and substructure. Ratings should be in accordance with the FHWA's 9 point scale (Table 3). Each state is required to annually report on the ratings of all bridges in its local network to the FHWA. The NBI database is the main source of federal information and statistics on the nation's bridges, and is used by the FHWA in its biennial condition and performance report to the Congress (Markow and Hayman 2009). Further, NBI ratings form the basis on which federal funds are calculated and assigned to the deserving bridges nationwide.

Based on data from NBI, federal funds are allocated according to a bridge maintenance prioritization formula called Sufficiency Rating (SR) (Weykamp et al. 2010). The SR of a bridge is a numeric value that implies its sufficiency to remain in service. A bridge's SR is determined using a combination of four factors as shown in the following formula (FHWA 2002):

$$SR = S_1 + S_2 + S_3 + S_4 \qquad [5]$$

Where

S_1 = Structural adequacy and safety

S_2 = Serviceability and functional obsolescence

S_3 = Essentiality for public use

S_4 = Special reductions based on detour length, traffic safety and structure type

Information about the four factors used in the calculation of sufficiency rating is usually collected through updated inventory data. Those factors are combined according to different proportions, while taking into account many sub factors as displayed in Figure 8. The SR is used to determine the relative sufficiency of all of the nation bridges, and thus helping in efficient allocation of maintenance funds (FHWA 2002). In basic terms, eligibility for federal funding with the highway bridge rehabilitation program is established when SR ≤ 80, whereas eligibility for replacement is determined by SR< 50.

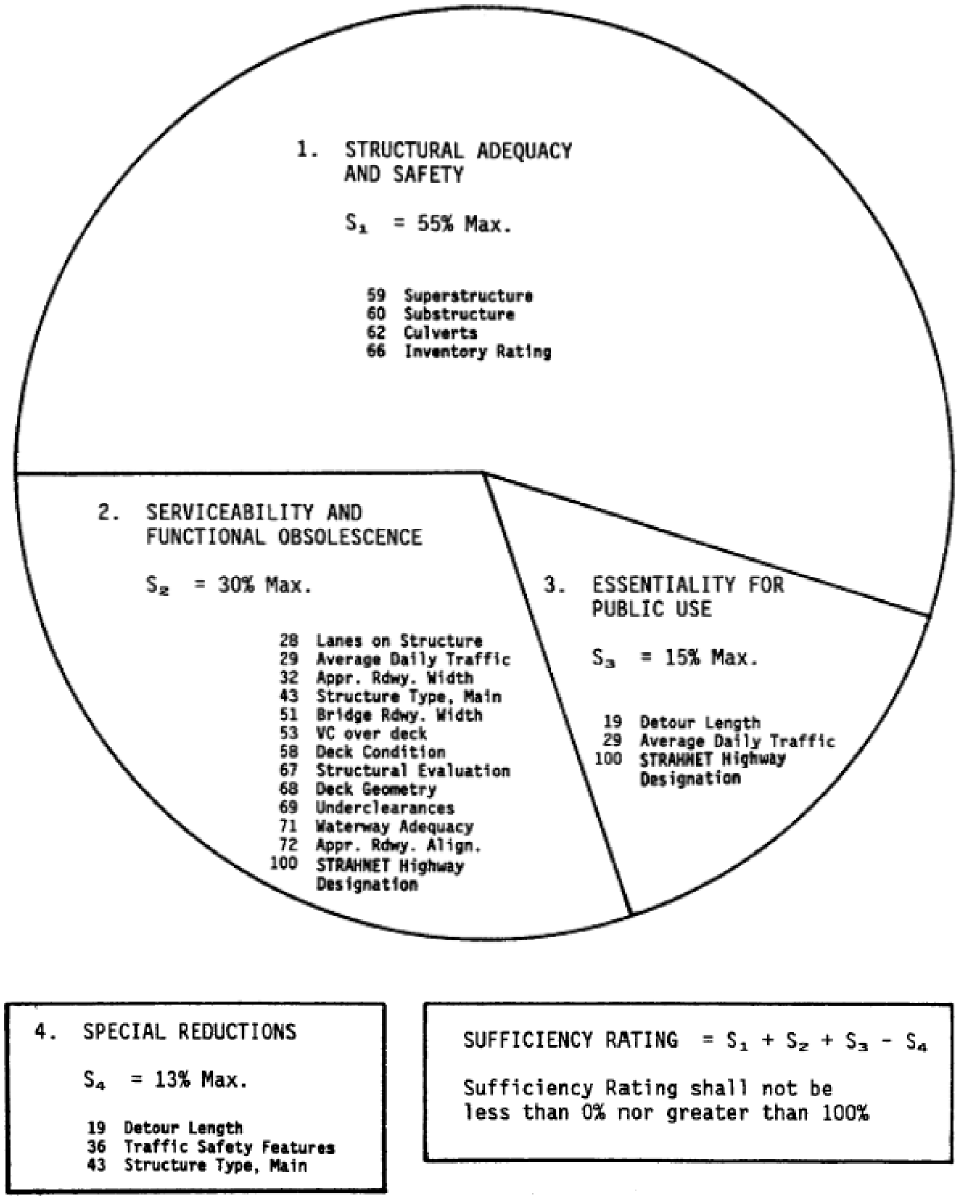


Figure 8: Calculation of Bridge Sufficiency Rating (FHWA 1995)

The SR has a maximum value of 100%, indicating perfect bridge sufficiency, and a minimum of 0%. As a general rule, the less a bridge's sufficiency rating is, the more eligible it is for maintenance or replacement funds. There are two other performance measures that are deduces from the NBI bridge ratings; namely, Structural Deficiency (SD) and Functional Obsolescence (FO). A bridge is considered structurally deficient if at least one of its elements is rated poor or worse (Markow and Hayman 2009). Table 4 shows the NBI criteria for considering a bridge as structurally deficient. It is implied from the SD rating approach that a bridge might be rendered structurally deficient whether one or more of its elements are rated below 5. SD classification doesn’t point out the structural causes of poor ratings. A poor rating may still result from deficiencies that do not really reflect serious effects on structural bridge integrity. Rather, SD can be looked at as a coarse measure that flags the need for additional investigation.

Table 4: NBI Criteria for Structural Deficiency (FHWA 1995)

NBI Rating Item #	Rated Structural Item	Criterion for Poor or Worse
58	Deck	<5
59	Superstructure	<5
60	Substructure	<5

The second bridge performance measure is Functional Obsolescence (FO). A bridge is considered functionally obsolete if it is not already classified as structurally deficient, and it has a deficiency in its load capacity, clearance, or geometry (Markow and Hayman 2009). As can be perceived from the definition, FO classification takes into account the NBI ratings for structural capacity items as well as items concerning evaluations of functionality. Functional adequacy can be inferred from assessing the existing load carrying capacities and geometric configurations against the currently demanded standards. A bridge may exhibit both structural deficiencies, that would imply structural deficiency, and functional problems that warrant functional obsolescence. The standard NBI reporting convention in this case is to identify the bridge as only structurally deficient, since SD is considered more serious and critical than FO (Markow and Hayman 2009). On the whole, both FO and SD are considered important bridge condition and performance measures because they do not only flag structural and functional bridge deficits, but also form a solid backbone of the federal and state bridge maintenance funding strategies.

II.3.3 PONTIS Condition Rating

Bridge management through the NBI program is still wildly in use in the US as the primary data source for distribution of federal and state Maintenance, Repair & Rehabilitation (MR&R) funds (Kelley and Rehm 2013). However, the NBI measures are rather general, with limited performance and analytical capabilities. Through applying NBI condition rating, the evaluation of bridges doesn't provide sufficient level of detail to identify repair strategies or cost estimates. It also usually provides a general measure of condition that is highly vulnerable to the subjective interpretations of bridge inspectors (Thompson and Shepard 2000).

NBI ratings and condition metrics (SR, FO, and SD) have been convenient in terms of general assessment, reporting, and fund allocation. However, they are not necessary good enough for analysis and predication of budgetary needs (Small et al. 1999). The development of Pontis BMS has introduced plenty of ideas to enhance the bridge management and maintenance efforts in the US.

II.3.3.1 Bridge Health Index

While starting to implement Pontis, element level condition data started to become available to decision makers. The newly available data, together with the need to accurately reflect bridge condition through a single number, have led to the development of the Bridge Health Index (BHI) by California Department of Transportation (Shepard and Johnson 1999). To California, the initiation of BHI was an attempt to establish a clear and reliable format of communicating bridge performance to higher managerial personnel (Thompson and Shepard 2000). It was an attempt to cover the limitations and to enhance the existing Pontis analytical tool.

The BHI is a single number ranging between 0-100, with 100% indicating the best state and 0% indicating the worst. In Pontis, BHI is being calculated by a two-step series of formulas. The first step determines the health index at element level by calculating every element’s health through its inspection information. The second step involves the computation of the overall BHI through weighted aggregation of individual element health indices.

According to Thompson and Shepard (2000), the HI can be developed for either a single bridge, or a group of bridges. The basic idea behind the HI is to consider the condition of an element or a bridge as a point along a continuum of 0-100. In most basic forms, the HI can be calculated as follows:

$$Health\ Index\ (HI) = (\Sigma\ CEV\ \div\ \Sigma\ TEV) \times 100 \qquad [6]$$

Where:

$$\text{Total Element Value } (TEV) = \text{Total Element Quantity} \times \text{Failure Cost of Element (FC)} \qquad [7]$$

$$\text{Current Element Value } (CEV) = (\Sigma[\text{Quantity in Condition State i} \times \text{WF}_\text{i}]) \times \text{FC} \qquad [8]$$

$$\text{Weighting Factor of condition } i \ (WF_i) = 1 - [(\text{i} - 1) \div (\text{Number of States} - 1)] \qquad [9]$$

As can be seen from equation 6, the HI of a bridge element is based on that element's total quantity, the quantity in each condition state, and the fail cost of that particular element. The HI of the entire bridge can be similarly evaluated as the weighted average of individual health indices of its elements. The aggregation is based on elements' weighting factors which are determined based on their relative economic values. Pontis system currently uses two weighting methods for elements aggregation; namely: the failure cost (FC) method, and the repair cost (RC) method (Jiang and Rens 2010).

II.3.4 Condition Rating in Quebec

The Ministry of Transportation of Quebec (MTQ) is accountable for the inspection, preservation and maintenance of approximately 10000 bridges in the vast province. This fairly large number of structures, with an impressively wide range of different bridge materials and structural systems, has made this job quit complex and challenging for the ministry. In addition, the existing structures are subject to a year-round exposure to harsh environmental factors. Thus, bridges in the province are under the continuous need for maintenance, which is augmenting the ministry's expenditure burden to meet the challenge of constant and efficient preservation of its bridge inventory (Vaysburd and Bissonette 2007).

With the ever growing challenge of bridge preservation in hand, MTQ has published several documents over past years in an effort to provide a structured guidance to the bridge inspection personnel in the province. The first comprehensive inspection document was issued in 1987 and titled “Inspection des Ouvrages d’Art”. This has led to the development of the first edition of the bridge inspection manual “Manuel d’Inspection des Structure” in 1993. The Manual for inspection of structures (or shortly “MIS”) has gone through several improvements since 1993, including an updated version in 2004. MIS has witnessed a huge upgrade with the introduction of the brand new version in 2012, which builds upon MTQ’s past experience and contains an impressively improved guidance to a comprehensive bridge inspection procedure (MTQ 2012). Updates include, but are not limited to, the addition of new inspection elements, development of new rules to calculate damaged portions, the introduction of a new 4-level grading scheme, and the integration with the ministry’s new bridge management system (Système de Gestion des Structures, or “GSQ”) (MTQ 2012).

II.3.4.1.1 Element Evaluation Scheme

Basically, bridge element evaluation principle has not changed much in concept. The material condition of a bridge element is assessed based on the severity and extent of the defects detected on the element’s surface or subsurface. This is mainly based on visual inspection of bridge elements, yet it may be more investigated by means of non-destructive evaluation. The evaluation of the material state of elements involves defining the level of degradation according to the newly defined 4-level grading system (Table 5). Since defects of different severity may be detected on a bridge element, the grade must be distributed so as to represent one or more evaluation grades according to the level of material deterioration at different locations of the element.

The four newly defined evaluation grades (A, B, C, and D) are set to match the four degrees of severity; namely light, medium, severe, and very severe. For deciding on the condition of the bridge element's material, an inspector is required to utilize general tables for material defects provided in the manual. Some specific evaluation criteria are also provided to aid the inspector rate material defects that are not available in the generic tables. One example is recording the extent of cracks that may be observed on concrete elements. For this case, the manual advises to report each 4 m of crack length as equivalent to 1 m^2 of defected surface.

Table 5: Material Condition Index (MTQ, 2012)

Condition rate	Severity degree
A	None or light
B	Medium
C	Severe
D	Very Severe

II.3.4.1.2 Bridge Element Evaluation: an Illustrative Example

As mentioned above, the material condition of a bridge element is assessed based on the severity and extent of the defects detected on the element's surface or subsurface. To illustrate the method for calculating the material condition index for a bridge element, an example of an inspected abutment wall is illustrated in the manual as shown in Table 6 (MTQ 2012). An abutment wall has been inspected and found to have 2 defects extending over two separate zones of the wall.

Table 6 Observed Material Defects on a Bridge Abutment Wall

Material State	Severity	Extent (%)
Disintegration	Severe	20%
Delamination	Severe	10%
Good state/Non-defected	None	70%

Having noted the severity and extent values for the observed defects, the material condition state of the abutment wall in this example is therefore calculated as follows:

$$D = 0\%,\ C = (20+10) = 30\%,\ B = 0\%,\ A = (100-30) = 70\%$$

It is noted that the new inspection manual eliminates the category of Auxiliary elements, and considers them as secondary elements. Thus, bridge elements are only classified as either primary (P) or secondary (S).

II.3.4.1.3 Evaluation of an Element's Performance

Designated as the "CEC" index, the evaluation of an element's performance is aimed at appraising the aptitude or of the element's structural role in the bridge.

The performance index measures the effect of a bridge element's defects on its structural capacity, functionality, and stability. This evaluation is of particular importance to the principle load carrying elements of the bridge. As shown in Table 7, The CEC index value ranges from 1 to 4, and can be generally determined by estimating the percentage reduction in structural capacity. An element attaining a CEC grade of "4" is one with defects that have very light or no impact on its structural capacity. A CEC grade of "1" is assigned to an element having defects that have a severe impact on its structural capacity, and an immediate intervention action need to be taken.

Table 7: General Performance Evaluation of Elements

Performance	**Decrease in the ability of an element to play its role**	
CEC Index	Principle Element (P)	Secondary Element (S)
4	0 to 10 %	0 to 10 %
3	10 to 20 %	10 to 30 %
2	20 to 30 %	30 to 50 %
1	>30 %	>50 %

II.3.5 Condition Rating in Ontario

The Ontario Structures Inspection Manual (OSIM) sets standards and provides uniform approaches for visual and detailed inspections and condition evaluation for all types of bridge structures. OSIM has been used for inspecting bridges in the province of Ontario since 1985. Several versions have been consecutively published, including the major modification in the year 2000. Some minor revisions took place in 2003. The manual received its latest update in the year 2008 (MTO 2008).

II.3.5.1 Philosophy

The philosophy adopted by OSIM, and stated in its introductory clause, is the "severity and extent" approach. The purpose is to simplify the process of recording inspection procedures, and to use the information in estimating bridge rehabilitation needs and implied costs. Defects to be detected on deferent bridge components are classified per material type, with quantitative data required to be collected and recorded during the condition evaluation process. Four condition states have been established, namely: Excellent, Good, Fair, and Poor. An element can be evaluated to more than one condition states due to different defects detected over quantities within that element. In case of evaluating an element to multiple condition states, an amount (length, area or unit) should be estimated and recorded for each of the respective condition grades. Some of the additions in the 2008 version include condition state tables, quantity estimation tables, and new inspection forms.

II.3.5.2 Material Condition States:

Relevant of the severity of material defects, condition states are defined to categorize the state of the inspected bridge element. As a general Rule, four condition states are established for condition evaluation: Excellent, Good, Fair, and Poor. OSIM provides tables to describe the four condition states for every possible bridge material type; including wood, steel or concrete. States are also defined for special bridge elements such as Bearings, wearing surface and coating.

II.3.5.3 Material Defects:

OSIM recommends an element-level inspection to identify the quantities of the element falling in each condition state. Material defects are distress indicators that are detected and recorded through a visual inspection. Defects' extents and severities are measured to reflect the condition state of the bridge element or quantities within it. Provided tables explicitly defining limits for severity extents of defects detected on various bridge material types, including concrete, steel and timber. Table 8 gives an insight on grading concrete bridge elements.

Table 8: Suggested Grading for Concrete elements (MTO 2008)

Excellent Condition	Good Condition	Fair Condition	Poor Condition
No observed material defects	Light scaling	Medium scaling	Severe to very sever scaling, erosion and disintegration
	Rust stains on concrete due to corroding rebar chairs	Rust stains on concrete due to corroding reinforcing steel	Medium to very severe corrosion of reinforcing steel
	Surface carbonation (Reaction with CO_2 , associated discolouration, shrinkage and cracks)	Surface defects such as stratification, segregation, cold joints, abrasion, wear, slippery surfaces, wet areas and surface deposits (except on soffits).	
	Light honeycombing and pop-outs	Medium honeycombing and pop-outs	Severe to very severe honey combing and pop-outs
	Hairline and Narrow cracks	Medium cracks	All wide cracks
	Light alkali-aggregate reaction	Medium alkali-aggregate reaction	Severe to very severe alkali-aggregate reaction
		Stable relative displacement between precast units. Leaking between precast units.	Active relative displacement between precast units
			All delaminated and spalled areas
		Active wet areas on soffit without cracks	Active wet areas or leachate deposits on soffit with associated cracks

II.3.6 Condition Rating in Alberta

Development of the inspection and maintenance standards in Alberta started in mid-1980’s, with the inspection component of bridge inspection manual (BIM) first introduced in 1987 (Alberta Transportation 2008). BIM is the inspection and maintenance constituent of Alberta’s Transportation Infrastructure Management System (TIMS), which has been implemented in 2005, and integrated the former Bridge Information System (BIS) with Culvert Information System (CIS).

Alberta has a two volume manual for Bridge Inspection and Maintenance (BIM), corresponding to level 1 and level 2 inspections. While Level 1 describes the monotonous visual inspection practices, Level-2 inspection is more in-depth and can require sampling and several kinds of testing. Several versions of BIM have been released, including the version 3.1 updated in March 2008. Level-1 inspection deals with standard visual inspection data and stipulates inspection standards, procedures, and 25 specifically tailored forms for inspection reporting. Level 1 only requires reporting the worst part of each element among each category in a bridge. Level 2, however, pertains to in-depth inspections conducted for specific components; requiring specific equipment and/or expertise. Level-2 inspections are quantitative and require filling out corresponding forms, provided in BIM level 2 manual, by experienced inspectors.

BIM rates individual inspection elements as well as their respective categories. An element’s rating represents both the current condition of the element and its functionality. That is, how deteriorated the element is and how much it is serving its intended purpose, respectively. It also reflects safety concerns and priority for maintenance. As previously mentioned, the rating of an element is determined by the worst item within the group. The inspector may describe why an element group was assigned a specific rating by indicating the worst element's rating and its location in the "explanation of condition" slot of the inspection form. BIM adapts a numerical rating system ranging from 1 to 9, with 9 indicating very good condition and 1 necessitating immediate action; in the same manner as the NBI rating system previously described.

For rating an entire bridge, inspection is performed through breaking it down to a set of defined element groups/categories by the BIM manual. The inspector is required to assign a grade to each individual element within the defined general bridge categories. Elements required to be rated as well as their respective categories are illustrated in Table 9. BIM manual provides instructions for rating each element, in addition to possible distress indicators that may appear to the inspector for that specific element. Amounts, types, and extents of distress indicators -if found- have predefined effects on the condition rating of bridge elements they are detected on. An example is illustrated through Table 10.

Table 9 Bridge Categories and Their Elements (Alberta Transportation 2008)

Category	Elements
Approach Roads	Horizontal alignment, Vertical alignment, Roadway width, Approach bump, Guardrail, Drainage
Superstructure	Wearing surface, Deck top, Deck Readability, Deck joints, Deck drainage, Wheel guards, Bridge rails & posts, Sidewalk, Stringers, Concrete girders, Truss members, Steel girder/beam, Diaphragms, Paint, Bearings, Sub deck

Substructure	Abutment and pier bearing seats/caps/corbels, Backwalls/breastwalls, Wingwalls, Abutment bearing piles and pier shaft/piles, Paint/coating, Abutment/pier stability, Scour/erosion, Bracing/struts/sheathing, Nose plate, Debris
Channel	Channel, Slope protection, Guidebanks/Spurs, Adequacy of opening
Grade Separation	Road alignment, Traffic safety features, Slope protection, Bank stability, Drainage

Categories are then assigned a grade that is governed by the rating of its most critical element in terms of load carrying, functionality or safety. For instance, general rating for the category of substructure should be governed by the rating for structural load carrying elements, bearing seat or cap, pile, stability, or back wall of rating 2 or less.

Table 10 Deck Top Rating Guidelines (Alberta Transportation 2008)

Deck Top Distress Indicators	Rating
Deck top surface is without defects or cracks and is relatively smooth	9
Deck top surface is in relatively good condition but has some form of hair-line shrinkage cracking	7 or less
Speed has to be reduced due to potholes, etc.	4 or less
Narrow cracks in concrete surface	5
Wide cracks in concrete surface	4 or less
Severe scaling (aggregate exposed), spalled or deboned areas.	3 or less

II.3.6.1 Limitations of Current Bridge Management Practices

Although BMSs facilitate the management of bridge networks to a great deal, some studies have reported certain shortcomings experienced by their users. One of the commonly reported limitations of current BMSs is their shortage of supporting custom decision making processes by decision makers, who would sometimes favor using their own personalized analytical tools; thus, limiting the BMS software usage to rather only be for bridge data storage (Wang et al. 2010). For instance, Kansas DOT uses its own priority formula for maintenance, repair and rehabilitation of its bridge inventory, though they are running PONTIS (Scherschligt and Kulkarni 2003). Also, among other observed issues is the inflexibility to incorporate data obtained by some of the developed inspection technologies by local DOTs; which is caused by the BMSs' quite rigid data input structures. Additionally, an abundance of data may accumulate on the network level that may challenge the limited simultaneous perception ability of human beings when making critical prioritization decisions. Other limitations may be ambiguity, conflicting input, and inspection measurement uncertainty.

II.4 Artificial Intelligence & Multi-attribute Decision Making

A considerable magnitude of research has been devised towards the use of artificial intelligence (AI) methods in the enhancement of Multi-Attribute Decision Making (MADM) problems with inherent uncertainties; including numerous applications in condition assessment of infrastructures. This includes, but is not limited to, the analytical hierarchy process (AHP) and its variations, the fuzzy set theory, artificial neural networks (ANN), multiple regression techniques or expert systems, and most recently the Evidential Reasoning (ER) approach (Wang and Elhag 2007). This section illustrates those techniques that have been utilized within the scope of this research.

II.4.1 The Analytic Hierarchy Process (AHP)

The Analytic Hierarchy Process (AHP) is a decision support and analysis tool that has found extensive applications in multi-attribute decision making (MADM) problems. It was first developed by Professor Saaty in 1980 as an approach to relative measurement (Saaty 1994). The technique is based on modeling decision problems into multiple layers of criteria and sub criteria to form a decision hierarchy. This is followed by a series of pair wise comparisons among elements in the same layer to decide on their relative importance/influence. The relative influence of one element over another is determined through judging the degree of importance in a pair-wise comparison process with respect to a higher level element that is called “parent” or “control” criterion. Judgments of relative importance can be provided using the 9-point fundamental scale suggested by Saaty (Table 11).

Following, the relative importance/priority weights are obtained through the Eigen vector approach. Normalized weights for the criteria are calculated by dividing the horizontal sums of each row in the comparison matrix by the total of all horizontal sums. The process is repeated to obtain relative importance weights of all criteria and sub criteria in the hierarchy, following a bottom up approach (Al-Harbi 2001). Eventually, the available alternatives can be compared against each other in terms of importance; thus, optimizing the decision making process.

Table 11: Saaty's Fundamental Scale of Absolute Numbers for Pairwise Comparisons (Saaty 1994)

Saaty's Scale		
Intensity of Importance	**Definition**	**Explanation**
1	Equal importance	Both elements contribute equally to the control criterion.
3	Moderate importance	An element has moderate importance over the other element in comparison
5	Strong importance	An element has strong importance over the other element in comparison.
7	Very strong importance	An element has very strong importance over the other element in comparison.
9	Extreme importance	An element is extremely more important/ affirmatively dominates the other element in comparison
2,4,6,8	Intermediate values	Intermediate values representing ascending order of importance

AHP has been extensively applied in optimizing assessment problems that can be formulated into a hierarchy of criteria (or elements). Examples include supplier assessment (Handfield et al. 2002), environmental impact assessment (Ramanathan 2001), cross country pipelines assessment (Dey 2002), and the assessment of concrete bridge conditions (Sasmal and Ramanjaneyulu 2008).

II.4.2 Fuzzy Logic in Condition Assessment

Most of the assessment attributes in MADM problems are of qualitative nature, making it uneasy to provide absolutely precise numeric assessments to represent human judgments. Hence, the evaluation of quantitative attributes is well suited to, and can be best represented by, linguistic terms of the natural language. For example, assessment grades such as “excellent” or “good” can be a decent representation of an inspector's evaluation of the state of a bridge member or pipe section to be actually in excellent or good condition, respectively. However, having the judgment encapsulated in linguistic terms may result in vagueness caused by the possible overlap in meanings (Carlsson and Fuller 1996). Therefore, it is fairly natural to define adjacent evaluation grades as two dependent fuzzy sets.

Since its introduction by Zadeh, the fuzzy set theory has proven its ability to effectively model uncertain linguistic variables using the concept of fuzzy membership and approximate reasoning (Zadeh 1965; Emami et al. 1998). The theory expands on the traditional set theory by allowing components in a set to have partial membership values falling in the interval [0, 1]. If H_n is a fuzzy set that represents the evaluation grade of an attribute in a condition assessment environment, then the rating membership function R_n can be of the following general form (Emami et al. 1998):

$$R_n = \mu_n(H_n) | H_n \;; \quad 0 < \mu_n < 1 \;,\; n = 1, \ldots, N \qquad [10]$$

Where a membership value μ_n depicts the degree of membership to that grade/fuzzy set H_n. Equation [10] represents the ambiguity in the condition rating, where the rating can be described by partial membership to two or more adjacent fuzzy grades. The implementation of fuzzy sets and fuzzy numbers aggregation has found its way to a wide plot of assessment applications, including areas of risk assessment (Tah and Carr 2000; Xu et al. 2010), water quality management (Mujumdar and Sasikumar 2002; Dahiya et al 2007), and condition assessment of infrastructures (Liang et al. 2001; Rajani et al. 2006; Sasmal et al. 2006; Kang and Chen 2010).

II.4.2.1 Fuzzy Synthetic Evaluation

On the basis of fuzzy sets, the Fuzzy Synthetic Evaluation (FSE) technique is an emerging approach for condition assessment and decision making under fuzzy uncertainties. FSE based models have proven utility in the analysis of numerical intensive systems with multiple levels and attributes, such as the evaluation of air and water quality (Onkal-Engin et al. 2004; Dahiya et al 2007), risk assessment of public private partnerships PPP (Xu et al. 2010) and most recently the performance evaluation of suppliers (Pang and Bai 2011). As an improved version of the traditional synthetic evaluation techniques, an FSE model is capable of dealing with data sampling uncertainties. It normally involves three steps, namely: fuzzification, aggregation, and defuzzification (Lu et al. 1999). It works well with systems that can be represented by hierarchical forms.

II.4.3 The Hierarchical Evidential Reasoning (HER) Framework

The Evidential Reasoning (ER) is one of the recently developed, and fairly complex, MADM tools. It is considered to be a breakthrough in handling hybrid (quantitative/qualitative) MADM problems with inherent uncertainties. This is achieved through its capability of assigning belief/credibility to the evaluation of the various assessment attributes. The application of the ER approach in the schematic evaluation process of MADM problems was first proposed by Yang and Singh (1994), and has recently gained popularity in the domain of infrastructure management. ER, in its general form, has been implemented in many assessment/evaluation applications that deal with multiple source/evidence aggregation under uncertainty, such as environmental quality assessment (Wang et al. 2006) and organizational self-assessment (Siow 2001).

ER is established on the basis of Dempster-Shafer (D-S) theory of combining multiple sources of information, known as the D-S theory of evidence. By employing the D-S theory, the ER methodology is able to combine both numerical input and qualitative data with uncertainty as evidences towards a rational and comprehensive assessment. In addition, the ER algorithm can be employed in a multi-leveled structure of attributes and sub-attributes (factors), making it powerful in analyzing problems that can be represented by a hierarchy of different levels that branch into several attributes.

In the cases of applying ER in a hierarchical format, the method becomes commonly referred to as Hierarchical Evidential Reasoning (HER). HER offers a strong hierarchal analysis algorithm that is founded on a multi-level evaluation and aggregation process; thus, it has been widely emerging as a significant approach for infrastructure condition assessment under uncertainties (Yang and Xu 2002). The method employs a belief configuration that assigns degrees of belief to the evaluation of elements in the hierarchy. Consecutively, The D-S theory of evidence is profoundly utilized under the HER framework for the purpose of accumulating supporting pieces of evidence in a comprehensive and systematic manner.

HER approach runs the analysis in a bottom up manner through the modeled hierarchy structure. In other words, a complex engineering system is decomposed to an order of levels or a hierarchy. A higher level attribute is assessed through attributes/factors that are directly below it in a hierarchical manner. The generic HER framework has been put into use in many in many applications that handle aggregation of various bodies of evidence in multi attribute hierarchies. For instance, a model based on a combination of the analytic hierarchy process (AHP) and HER was developed to establish an assessment for E-commerce security (Zhang et al. 2012). Other recent applications include the use of HER in ranking the most influencing factors for tender selection (Chowdury et al. 2012); the development of an optimized expert system for stock trading (Dymova et al. 2010); aggregating quantitative and qualitative capability measurements for the ranking and assessment of weapon systems (Jiang et al. 2011) and ship turbines (Gaonkar et al. 2010); grading and ranking regional hospitals based on their solid wastes (Abed-Elmdoust and Kerachian 2012); developing a multi attribute decision analysis model for the quality assessment of electrically commuted motors (Boškoski et al. 2011); establishing risk assessment models for sea port infrastructure (Mokhtari et al. 2012) and inland waterway transportation systems (Zhang et al. 2013).

II.4.3.1 Dempster-Shafer (D-S) Theory of Evidence

The D-S was proposed as a generalized advancement of the Bayesian probability theory for the fusion or aggregation of uncertain information. The theory was first suggested by Dempster in 1967 and subsequently enhanced and perfected by Shafer in 1976. A major improvement that the D-S theory has put in handy to the MADM problems lies in its capacity to accept incomplete information about attributes in cases where complete data are not available. It can also account for ignorance in data acquisition, solving the common dilemma of data scarcity in MADM problems (Sentz and Ferson 2002). That is, the theory is able to handle incomplete description of an attribute or total absence of data concerning an attribute in a system. Moreover, the D-S theory offers a solution to subjective data uncertainties by being able to cope with cases where the available evidence may partially support different propositions of an attribute. This is particularly helpful in accommodating data of vague or fuzzy nature.

The D-S theory has been utilized in the Artificial Intelligence (AI) domain as an alternative to the traditional Bayes theory. D-S theory-based methods have been developed to model a wide range of applications in engineering and other fields, such as diagnostic reasoning in medicine (Gordon et. Al 1984), and estimating the risk of contaminant intrusion through water in distribution networks (Sadiq et al. 2006).

In the D-S theory, a finite set of mutually exclusive and collectively exhaustive propositions is denoted by Θ and commonly referred to as *the frame of discernment*. Let $\Theta = \{H_1, H_2, .., H_n\}$ be an exhaustive set of n independent propositions or assessment grades, where the number of possible subsets of H_n (Ψ) in its domain is given by 2^{Θ}. The set which contains all possible subsets (Ψ) of the original frame of discernment is called *the power set*, with its individual subsets sometimes being referred to as focal elements (Yager et al. 1994):

$$2^{\Theta} = \{ \Phi, \{H_1\},\ldots, \{H_N\}, \{H_1 \cup H_2\},\ldots,\{H_1 \cup H_N\},\ldots, \Theta \} \quad [11]$$

Where $H_1, H_2,\ldots, H_N$ are independent prepositions or assessment grades.

II.4.3.2 Distributed Belief Structure

It is common in MADM applications to have evaluation factors assessed to a crisp numerical score (quantitative) or a linguistic grade (qualitative) based on a certain assessment scale or evaluation system. This might, however, overlook the involved subjectivity in the inspector's opinion and not truly reflect the diversity of the real condition state. Given the uncertainty of a human judgment, an expert may not be completely sure that one evaluation grade will be representing the real condition state of the assessed item. Thus, expert judgments on condition might rather be better represented by more than one assessment grade with degrees of confidence or belief ascribed to each grade. This is, in fact, the essence of the distributed belief structure in the ER approach

Suppose that an alternative is being assessed based on *K* number of attributes that can be evaluated based on L number of identified factors that can be directly measured or observed (Figure 9). Let's say that a factor e_i , $i \in \{1, 2, \ldots, L\}$, is evaluated to a grade H_n with a degree of belief $\beta_{n,i}$, it follows that the condition rating "S" for that given factor "e_i" can be written as (Bai et al. 2008):

$$S(e_i) = \{(H_n, \beta_{n,i}), n = 1, \ldots, N\} \; ; i \in \{1, 2, \ldots, L\} \quad [12]$$

$$\beta_{n,i} \geq 0 \text{ and } \sum_{n=1}^{N} \beta_{n,i} \leq 1 \quad [13]$$

Therefore, the *k-th* attribute can be assessed through a factor e_i^k to a grade H_n with a degree of belief $\beta_{n,i}$. The assessment is known to be complete if $\sum_{n=1}^{N} \beta_{n,i} = 1$, and incomplete if $\sum_{n=1}^{N} \beta_{n,i} < 1$. In the exclusive case of $\sum_{n=1}^{N} \beta_{n,i} = 0$, it can be assumed that total ignorance prevails and there is absolutely no information concerning the assessment of that particular factor.

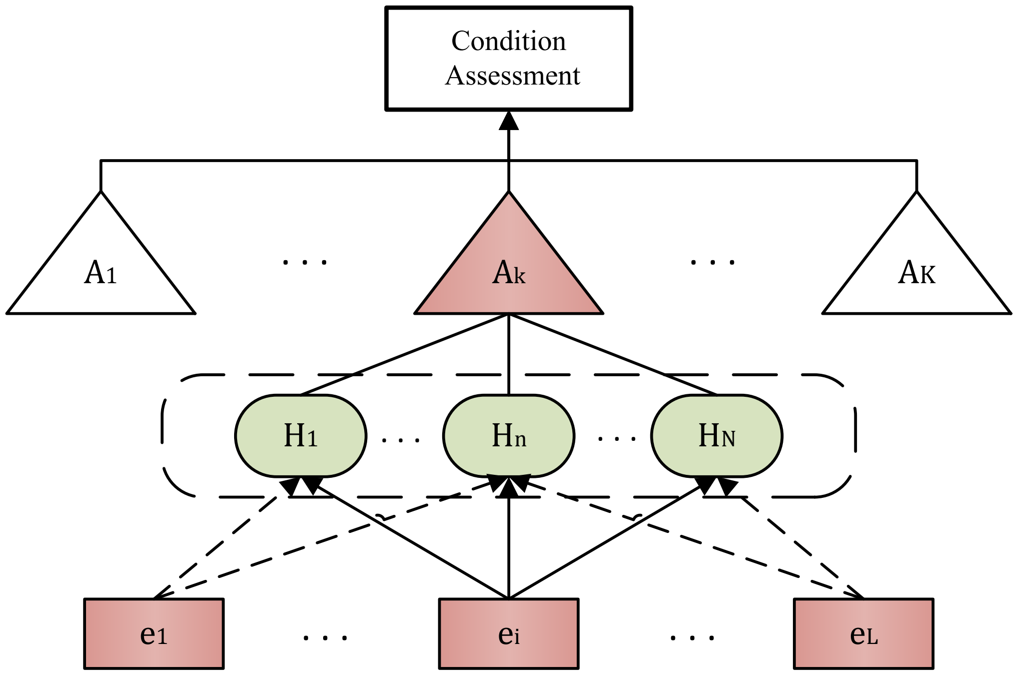


Figure 9: Generic ER Bottom-up Assessment

II.4.3.3 Relative Importance of Assessment Factors

In the ER framework, factors vary in their importance with respect to their parent attributes. An assessment factor can perhaps be more important than a second factor, yet it can be less important than a third factor that would be more significant than the first two; and so on. Different assessment factors therefore possess different relative importance weights with regards to the assessment of their parent attribute, and thus towards the assessment of the whole system. Suppose that for every attribute A_k, relative weights of all L factors e_i , $i \in \{1, 2, \dots, L\}$, can be provided by $w = \{w_1, w_2, \dots, w_L\}$. Those relative importance weights of the assessment factors should be normalized towards their parent attribute, by satisfying the following:

$$0 \leq w_i \leq 1\ ;\ \sum_{i=1}^{L} w_i = 1 \qquad\qquad i \in \{1, 2, \dots, L\} \qquad [14]$$

II.4.3.4 The Basic Probability Assignment (BPA)

The basic probability assignment (BPA) represents the degree of belief/confidence assigned to a certain preposition in Θ and reflects the extent to which the existing evidence supports that preposition. It may also be referred to as the assigned probability or probability mass. BPA can be represented by a function m: $2^{\Theta} \rightarrow [0.1]$, which satisfies the following (Yang et al. 2006):

$$m(\Phi)=0\ \ ;\ \ 0 < m\,(\Psi) < 1\ \ ;\ \ \sum_{\Psi \subseteq \Theta} m(\Psi) = 1\,,\ \text{for all } \Psi \subseteq \Theta \qquad [15]$$

with Φ denoting an empty set, and 2^{Θ} representing the power set of the frame of discernment, i.e the set of all possible subsets of H_n (or Ψ) in Θ. The assigned probability mass (m) to Ψ signifies the strength of evidence support and measures the exact belief portion that a given body of evidence provides to Ψ. An m value assigned to any subset $\Psi \subseteq \Theta$, and should take a value from the interval [0, 1]. The total sum of all BPAs assigned to subsets in Θ should be unity, and the empty set is always assigned a BPA of zero (Yang et al. 2006). The amount of probability allocated to Θ “i.e. m (Θ)”quantifies the degree of ignorance in the available body of evidence. The magnitude of m (Θ) reflects the portion of the total belief that remains unassigned after allocating BPAs to the subsets Ψ in the frame of discernment. This designates the intrinsic ignorance in the case where incomplete evidence is provided (Bai et al. 2008).

II.4.3.5 (D-S) Rule of Combination

As discussed earlier, each source of evidence results in a different set of basic probability assignments (m-values) to the mutually exclusive propositions or condition states. The acquired sets of BPAs can be referred to as bodies of evidence. The D-S theory provides a scheme for aggregating multiple bodies of evidence on the basis of the D-S rule of combination. Alternately referred to as the orthogonal sum rule, the combination operation, represented by the operator $\oplus$, is able to compile two or more bodies of evidence as follows:

$$m_{12}(\Psi) = \begin{cases} 0 & \Psi = \Phi \\ \frac{\sum_{A \cap B = \Psi} m_1(A) m_2(B)}{1 - \sum_{A \cap B = \Phi} m_1(A) m_2(B)} & \Psi \neq \Phi \end{cases} \qquad [16]$$

for $\forall\, A, B \subseteq \Theta$

where $m_{12}(\Psi)$ is equivalent to $m_1(\Psi) \oplus m_2(\Psi)$; A and B are any subsets of Θ whose intersection is equal to the subset Ψ in the numerator, and to Φ in the denominator. The combined BPA for the subset Ψ, i.e. $m_{12}(\Psi)$, is calculated by summing all the products of the BPAs of subsets A and B whose intersection = Ψ, and dividing that by the compliment of summing the BPAs of subsets A and B of which the intersection is a void set. The D-S rule of combination is commutative and associative, allowing for any order of combining pieces of evidence. Moreover, the combination scheme can be expanded to cover the aggregation of more than two bodies of evidence (Yager et al. 1994).

II.4.4 Previous and Related Work

HER is a strong hierarchal analysis approach that is founded on a multi-level evaluation and aggregation throughout a modeled hierarchy structure; Thus, it has been widely emerging as a significant approach for infrastructure condition assessment under uncertainties. HER based models have proven to be beneficial in the analysis of data intensive systems that can be represented by several levels of attributes and condition factors. This section reviews some of the proposed HER assessment models in literatures and highlights some of their limitations.

Bai et al. (2008) developed a systematic ER approach for the condition assessment of water mains. The study intelligently interprets a comprehensive model for aggregating several deterioration indicators that can be detected through internal and external inspection of the pipe material. The proposed assessment model provides treatment of data incompleteness as well as an efficient translation of inspection results into a condition rating of the evaluated infrastructure. Wang and Elhag (2008) developed an ER based approach to assess the existing state of concrete bridges, and consequently rank different bridge alternatives using a utility based condition scale. The ER model developed in their study is proposed as an alternative to the traditional bridge condition assessment models. They suggest that the power of applying ER for bridge condition assessment is reflected in its ability to efficiently deal with uncertainties inherent in the subjective evaluation process. In addition, their model is proposed to potentially account for uncertainties caused by ignorance; solving the issue of data scarcity while allowing for incomplete data to be accepted by the assessment model. Bolar et al. (2013) have built an HER model for a comprehensive condition assessment of concrete bridges. The model builds upon condition indices that are eventually grouped together in an overall bridge condition index. Primary, secondary, tertiary, and safety-critical indices, as suggested by the study, are evaluated based on classifying bridge elements in the respective order of importance. Reliability or importance factors are introduced in the proposed model to capture both structural importance and data reliability.

The latter two studies (Wang and Elhag 2008; Bolar et al. 2013) offer a great advent in bridge management through the direct application of an ER approach to model the condition assessment process. The proposed approaches represent an element level bridge evaluation that may be sufficient for a routine bridge inspection. However, for a deeper and perhaps more accurate evaluation, there exists an auxiliary need to develop an assessment tool that can handle the prescribed uncertainties in the bridge inspection process, improve the quality of data obtained from bridge inspection reports, and objectively translate the real condition of a bridge through further in-depth measurements.

II.5 Overall Limitations in Bridge Condition Assessment

According to a state of the practice survey (Rolander et al. 2001), conducted by the Federal Highway Administration's (FHWA's) Non-destructive Evaluation Validation Center (NDEVC), some of the current practices may affect the reliability of bridge visual inspection (VI) results. The study targeted State DOTs, local level DOTs and inspection contractors. Encompassed questionnaire included questions related to the composition of bridge inspection teams, impacts of administrative requirements, and the use of Non-destructive evaluation. Some of this study's findings can be summarized in the following points:

- Beside some NDE inspection techniques used by state DOTs, VI is the most prevalent practice for bridge inspection.
- Although it is a prerequisite for conducting inspection in some other industries, vision testing almost doesn't exist as an administrative requirement to perform bridge inspections for the surveyed transportation agencies and inspection contractors.
- Topics of improvement in the bridge inspection process suggested by respondents are generally related to bridge management and operational areas, with most common suggestions being to allow direct incorporation of inspection data in the bridge management software and improving access to bridges.

Uncertainties exist in currently implemented BMSs (Attoh-okine & Chajes 2003); especially through collecting inspection data, where subjective judgments and inaccurate inputs may occur. Besides, inspectors may not be able to provide complete data sets which cause lack of quantitative input. Other uncertainties may be ambiguity, conflicting input, and measurements uncertainty.

The treatment of assessment subjectivity has been the subject of many previous research studies. Abu Dhabous et. al (2008) proposed a probabilistic bridge element condition index that is based on fuzzy membership functions of condition states and Monte Carlo simulation method. Sasmal et al (2008) proposed a fuzzy based AHP model for a systematic condition assessment of concrete bridges under uncertainty. Wang and Elhag (2008) employed an ER model to treat the uncertainties incorporated in element assessments of concrete bridges.

The above mentioned studies offered decent approaches to treat assessment subjectivities on an element level; however, there is a tangible need for a more in-depth assessment that is able to more accurately reflect the condition of bridges as close to reality as possible. Current bridge assessment practices were found to be oversimplified, with conclusions being often drawn in absence of in-depth review and consideration of critical factors. A deeper and perhaps more accurate evaluation should incorporate direct measurements of observed bridge defects into the condition assessment process.

III. RESEARCH METHODOLOGY AND MODEL DEVELOPMENT

III.1 Overview

This research proposes a detailed model for a systematic and procedural condition assessment of concrete bridges. The novelty of the hereby developed model lies in its capability of objectively translating field-recorded bridge defects into an overall assessment of condition. As can be noted from the literature review of current practices, bridge assessment is predominantly associated with a considerable amount of uncertainties and subjectivities inherent in the human being's judgments. It is also marked that the growing challenge of bridge deterioration have created an auxiliary need to ameliorate the current level of assessment details, with expert voices increasingly advocating more emphasis on performance based and in-depth bridge condition evaluations. Therefore, it is attempted here to introduce a detailed (defect-based) assessment model, while featuring a substantial solution to the issue of subjectivity.

The assessment model idealizes the concrete bridge under assessment into a breakdown of components, elements, and defects. Relative structural weighting of all elements in the assessment hierarchy are established on the basis of processing expert surveys through AHP. The next step involves accustoming the model with a systematic, uniform methodology of accepting information about defects and, and mirror those information on the condition rating grades. This is achieved through developing a grading system that is able to map reported defect measurements onto a unified fuzzy grading scheme.

Consecutively, a comprehensive HER algorithm is adopted for weighted multi-level aggregation of the condition data inputs towards obtaining an overall educated, objective bridge condition assessment. The proposed assessment model is eventually applied to the assessment of two case studies for testing and validation purposes. Figure 10 displays a flow diagram illustrating the different steps involved in developing the presented research methodology.

III.2 Literature Review

A comprehensive state of the art and practice review of literature was conducted through the course of this research. This included an extensive overview of deterioration mechanisms of concrete bridges, commonly employed inspection techniques, ways of managing bridge information, and available bridge rating systems. The literature review also touches on several topics related to existing concerns of subjectivity in assessment grading, inspection uncertainties, and currently implemented assessment aggregation/condition rating techniques. The existing infrastructure assessment models in literature were used as a starting point while embarking on different Artificial Intelligence (AI) and Multi-Attribute Decision Making (MADM) techniques.

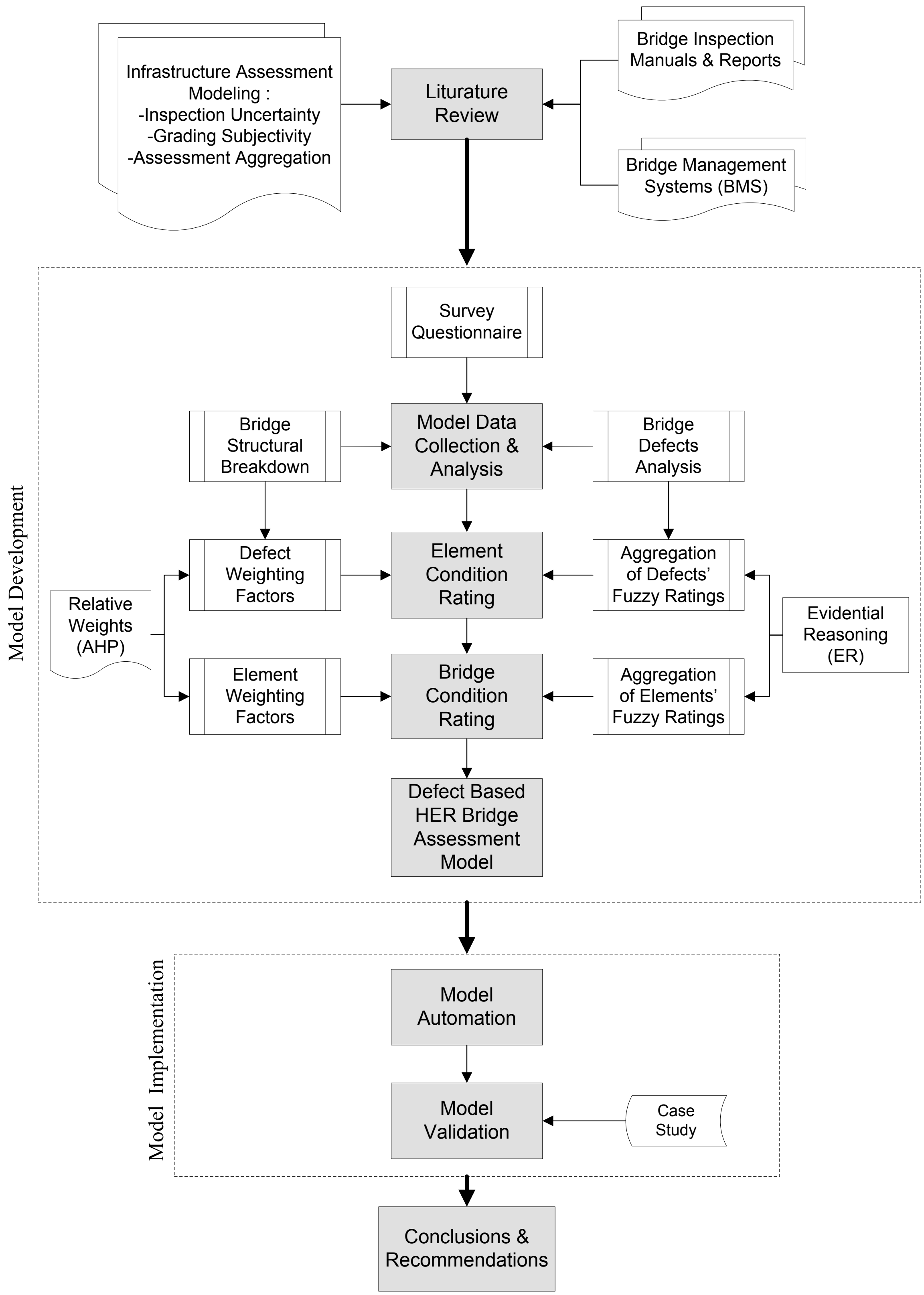


Figure 10: Schematic Flow Diagram of the Research Methodology

III.3 Data Collection

Given the scope of this research, and in order to build a reliable evaluation model that is consistent with the predominant bridge assessment practice, data used to construct the model had to be drawn from the population of bridge inspection/engineering/management practitioners in Canada. In general, all the acquired data for this study can be categorized into three main types. The first type pertains to the identification of the different bridge assessment factors. The proposed generic model is set to identify the different components, elements, and defects; on the basis of which a bridge will be assessed.

The second type of data draws on expert opinions of the targeted population to deduce the relative importance weights of all assessment factors in the hierarchy. This has been materialized by designing a questionnaire survey of two parts, and consecutively processing responses from the surveyed experts. The third type of data encompasses detailed inspection reports/condition surveys of existing concrete bridges. These reports were carefully studied to extract information related to the extents and severities of the detected defects on the tested bridges. These data can be concluded from close-up bridge inspections that are supported by various means of Non-Destructive Evaluation (NDE). The data collection chapter provides the reader with more detailed descriptions on the data collection procedures, designed survey structure, analysis of responses, and obtained data for case studies.

III.4 Identifying Detailed Condition Model Assessment Factors

As an integral part of the condition model building process, this research attempts at the realization of a generic structural hierarchal factors for the condition assessment of concrete bridges. This is achieved by breaking down the concrete bridge structure into its fundamental components and elements. Further, this step expands on identifying major defects on the basis of which a concrete bridge is going to be evaluated.

III.4.1 Bridge Structural Breakdown

In order to efficiently manage bridge inspection and assessment, it is essential to have common grounds as to how to breakdown and classify the different bridge elements and components. This is to establish standard terminology in identifying and describing the various composing parts of a bridge; thus, having inspectors properly identify and recognize the bridge structural breakdown in a consistent manner.

However, as seen in the literature review, different transportation agencies/departments have slightly varying definitions regarding the breakdown of concrete bridges, with many of them having their own agency-specific definitions. Normally, the definition of general and specific bridge components and elements can be found in details in bridge inspection manuals published by those agencies/departments of transportation. But in some way or another, most bridges can be divided into three major parts or components, namely:

- Bridge Deck

- Super Structure
- Substructure

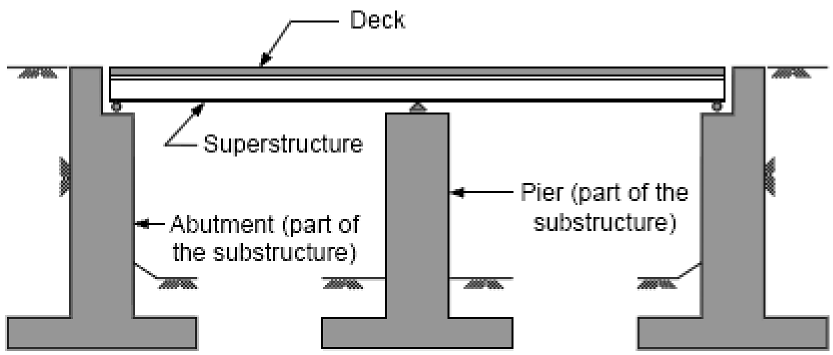


Figure 11: Main Bridge Components (FHWA 2002)

As mentioned earlier, there has been a trend among transportation agencies towards the adoption and implementation of element level inspection. This is due to the fact that a further in-depth inspection provides more details for later analysis and implementation in maintenance management and performance models (Markow and Hyman 2009).

In the United States, most of the departments of transportation have been successfully using the “Commonly Recognized (CoRe) Elements for Bridge Inspection” as a standard federal guideline for conducting bridge inspections and collecting data about field condition. With the introduction of the CoRe guide in the 1990s, more comprehensive inspection details were established for an element-level bridge evaluation (Kelly and Rehm 2013). The (CoRe) elements standard has been a preferred data collection basis in the U.S., as it allows for uniformity of collected information, facilitates inspectors training, and permits data sharing for research purposes (Thompson and Shepard 2000).

Table 12 shows the typical bridge structural elements demonstrated by the National Bridge Inventory (NBI) system. Similarly, the Bridge Inspection Manual (BIM) issued by Alberta recommends that the inspection of a bridge must be done through breaking it down to a set of defined elements and categories. Elements required to be rated as well as their respective categories are illustrated in Table 13.

Table 12: Typical Bridge Breakdown in NBI (Wang and Elhag 2008)

Bridge Components	Bridge Elements
Deck	Wearing surface, deck topside, deck bottom side, deck underside, SIP forms, curbs, medians, sidewalks, parapets, railing, expansion joints, drainage system, lighting, utilities
Superstructure	Stringers, floor beams, floor system bracing, multibeams, girders, trusses, arches, cables, paint, bearing devices, connections, welds, timber decay, concrete deterioration, steel corrosion, collision damage, LL deflection, vibration, member alignment, utilities
Substructure	Abutments (piles, footing, stem, bearing seat, backwalsl, wingwalls), piers and bents (piles, footing, column(s)/stem(s), cap), scour/undermining, settlement, substructure protection, fender system, collision damage, high-water mark, timber decay, concrete deterioration, steel corrosion, paint

The Ontario Structures Inspection Manual (OSIM) defines a list of the most common elements that can possibly be found in all types of bridges. It is then followed by a cross table that has most common bridge types alongside a list of all possible bridge elements. Illustrated in Table 14 is a deductive summary of elements and their respective element groups for a general bridge according to OSIM (MTO 2008).

Table 13: Bridge Categories and their Elements (BIM 2008)

Category	**Elements**
Approach Roads	Horizontal alignment, Vertical alignment, Roadway width, Approach bump, Guardrail, Drainage
Superstructure	Wearing surface, Deck top, Deck Readability, Deck joints, Deck drainage, Wheel guards, Bridge rails & posts, Sidewalk, Stringers, Concrete girders, Truss members, Steel girder/beam, Diaphragms, Paint, Bearings, Sub deck
Substructure	Abutment and pier bearing seats/caps/corbels, Backwalls/breastwalls, Wingwalls, Abutment bearing piles and pier shaft/piles, Paint/coating, Abutment/pier stability, Scour/erosion, Bracing/struts/sheathing, Nose plate, Debris
Channel	Channel, Slope protection, Guidebanks/Spurs, Adequacy of opening
Grade Separation	Road alignment, Traffic safety features, Slope protection, Bank stability, Drainage

Table 14: Typical Bridge Elements and Element Groups (MTO 2008)

Bridge Element Group	**Bridge Elements**
-Decks	Wearing surface, Deck top, Soffit, Soffit-inside boxes, Drainage system
-Joints	Seals/Sealants, Concrete end dams, Armoring/Retaining devices
-Sidewalk/Curb	Sidewalk and Medians, Curbs
-Barriers	Barrier/Parapet Walls, Railing Systems, Posts, Hand railings
-Beams/Main Longitudinal Elements (MLE)	Girders, Floor beams, Stringers, Inside boxes (sides & bottoms), Diaphragms (concrete), Diaphragms (steel, wood, etc.), Bracing

-Trusses/Archs	Top chords, Bottom chords, Verticals/diagonals, Connections
-Coating	Structural steel, Railing systems/Hand railings
-Abutments	Abutment walls, Ballast walls, Wingwalls, Bearings
-Piers	Shafts/columns/Pile Bents, Caps, Bearings
-Foundations	Foundation (below ground level)
-Retaining Walls	Walls, Drainage, Railing system on walls, Barrier systems on walls
-Culverts	Inlet components, Outlet components, Barrels
-Embankments & Streams	Streams and Waterways, Embankments, Slope protection
-Accessories	Electrical, Noise Barriers, Signs, Utilities, Other
-Approaches	Wearing surface, Approach slabs, Drainage system, Curb and gutters, Sidewalk/curb

III.4.2 Model-Incorporated Hierarchy for Bridge Components and Elements

For the purpose of this study; and as a simplified representation of the structure, a concrete bridge is broken down into a hierarchy of four main components, which are subdivided into different element sets (Figure 12). The four main components that compose the concrete bridge include the Deck, Beams/Main longitudinal Elements (MLE), Abutments, and Piers. In addition, components are subdivided into elements (for instance, “Deck” is divided into: Wearing surface, Deck top, Soffit, and Drainage system). The hierarchies are further expanded to cover element-specific sets of common possible defects for every bridge element.

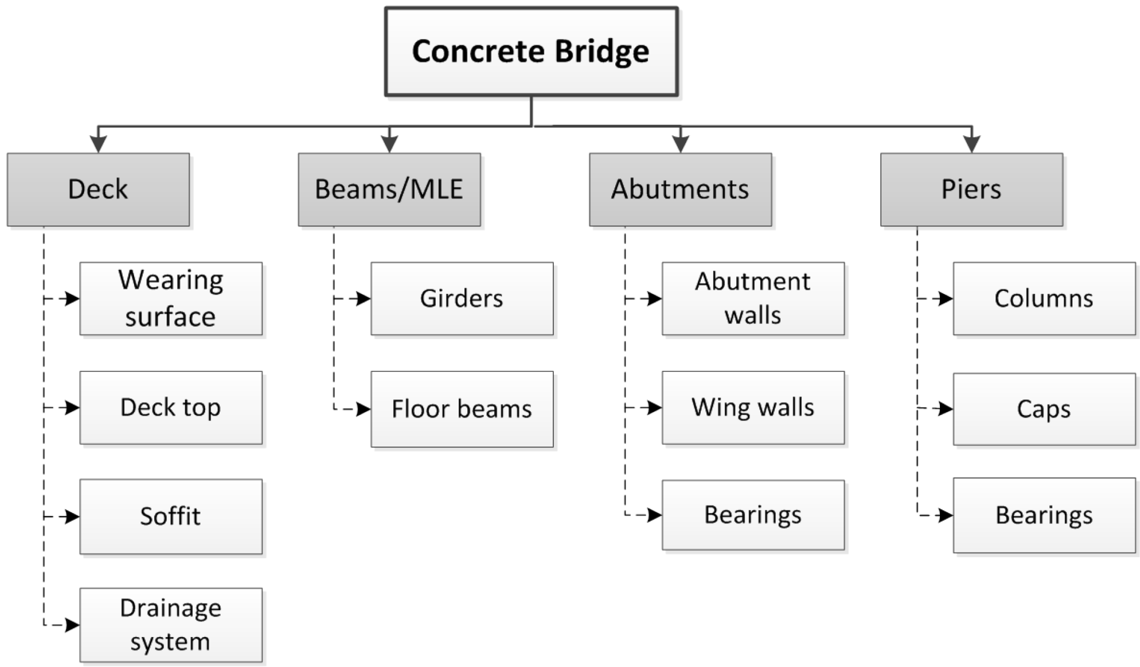


Figure 12: Model-Incorporated Hierarchy of Bridge Components and Elements

The first major assessment component is the bridge deck. It is the part of the bridge that carries and allows for smooth passing of traffic/ live loads. It performs the structural function of transferring all applied live loads, in addition to its own dead load, to the supporting components below. The deck component consists of several elements, including the wearing surface, deck top, soffit, and drainage system.

The second major model component incorporates the Beams/Main longitudinal Elements (MLE). Beams or MLE are major bridge superstructure components that support the bridge deck above, carrying all its applied loads, and transferring the resulting forces to the supports below. Girders and beams are grouped into the MLE component.

Moving to the substructure, this category typically consists of all elements that support and transmits loads from the superstructure elements down to the soil or foundation. Typical substructure components include piers and abutments. Abutments can be typically subdivided into main abutment walls, wing walls and bearings. Besides, elements that collectively form bridge piers are commonly recognized as pier columns, pier caps, and bearings. Bearings exist as accessories on both abutments and piers for the girders to rest on and transmit their loads down to the bridge's substructure.

III.4.3 Structural Defects

As mentioned in literature review, concrete bridges are commonly susceptible to many deterioration factors over the course of their service lives. Those factors range from environmental conditions, such as freeze and thaw actions, to human imposed factors, such as traffic load action and excessive application of de-icing salts in winter. Over the years, those factors result in progressive deterioration of the bridge elements, which gets reflected in a host of structural defects. During bridge field inspection, several defects may be detected on the surface and/or subsurface of bridge elements indicating different distress and deterioration mechanisms. Detailed description of possible defects that may occur in different bridge materials (concrete, wearing surface, etc.) can be found in bridge inspection manuals issued by provincial or state transportation departments in the United States and Canada. While it is the purpose of this section to briefly identify the various types of bridge defects and defect groups, a more thorough explanation can be found in appendix C.

III.4.3.1 Concrete Defects

Several types of defect mechanisms may develop on or below the surface of concrete bridge elements as they progress in service life. Some of these defects can be superficial without warranting serious threat to the bridge load rating capacity, such as surface flaking (scaling) or light non-structural cracks. The progressive corrosion of steel reinforcement commonly results in a substantial increase in the volume of steel bars in reinforced concrete elements. This subsequently imposes overwhelming internal stresses, causing low-strength pockets of the surrounding concrete to get damaged in the form of internal cracks. As corrosion gets more severe, those internal cracks may gradually cause loss of bond and partial separations of concrete (delamination) over the reinforcement layer. The situation gets worse as several delaminated regions form into spalls that escalate to the concrete surface, causing serious structural disintegration (pop-outs).

Defects discussed in this section may potentially occur at the surfaces or subsurface of concrete bridge decks, concrete girders and beams, concrete pier elements, and concrete abutment elements. Surface defects include: scaling, cracking, erosion, pop-outs, spalling and wet areas; whereas defects falling under the subsurface category include delamination and reinforcement corrosion. Table 15 provides a brief definition of the defects that can possibly develop on the various concrete bridge elements

Table 15: Concrete Defects

Defect Group	Defect	Description
Concrete Defects	Scaling	local flaking, or loss of surface portions of concrete or mortar as a result of the freeze-thaw deterioration of concrete
	Corrosion of Reinforcement	Rust & deterioration of steel reinforcement by electro-chemical reactions. Might initially appear as rust stain on concrete surface
	Pop-outs	Conical, shallow depressions caused by small fractured portions of the concrete surface, due to the expansion of some aggregates and/or frost action
	Cracking	Linear fractures caused by external loads inducing tensile and/or compressive stresses in concrete
	Delamination/ Spalling	Partial separation of the top concrete layer due to substantial reinforcement corrosion
	Erosion	Deterioration of concrete brought about by water-borne sand and gravel particles, and/or flowing ice, scrubbing against concrete surfaces. Usually at the footing level
	Wet areas	Salty/white exudations on concrete surface

III.4.3.2 Wearing Surface Defects

The concrete bridge deck could be left bare as traffic vehicles move on top of its surface. Yet in most cases, the concrete deck is covered with a wearing surface overlay to act as a protective layer and to ease traffic movement. The wearing surface seals the underlying concrete deck against water and salt ingression, which plays a great role in determining the durability and service life of the bridge deck. A well designed pavement should be highly resistant to traffic skid and permanent deformations. Besides, it should be sufficiently bonded with the deck top while assuring good absorption and transfer of traffic loads. In general bridge construction practices, a bridge deck pavement system consists of a bituminous primer layer, a water proofing membrane, and surface asphalt layers (Alberta Transportation 2003).

The deterioration of asphalt wearing surface stems from many factors that range from poor placement practices to the aggressive surrounding environment. Such asphalt-durability related factors may be coupled with stresses induced from the deterioration of the concrete deck top underneath the asphalt. In this context, OSIM (2008) distinguishes two main groups of asphalt defects; namely, top-down defects and bottom-up defects. Top-down defects are ones that originate in the asphalt material itself, such as isolated asphalt cracks, wheel rutting, and loss of bond. On the other hand, Bottom-up defects are rather rooted in the underlying concrete deck and successively reflected in the asphalt layer. Examples of bottom-up defects include pattern cracking, wide transverse and longitudinal cracks, and potholes.

Table 16: Asphalt Wearing Surface Defects

Defect Group	Defect	Description
Asphalt Wearing Surface Defects	Cracking	Longitudinal, transverse or mapped linear surface fractures
	Potholes	Conical holes in the pavement caused by freezing-thawing and/or vehicular actions
	Rutting	Longitudinal depressions caused by truck wheels
	Rippling	Transverse Crinkles (asphalt waves & valleys) caused by traffic movements and/or poor pavement mix
	Loss of Bond	Detachment areas between the asphalt layer, water-proofing, and/or deck top

III.4.3.3 Drainage System defects

Surface drainage systems are commonly installed on bridges as vital elements to insure efficient collection and diversion of surface water from concrete bridge decks (Ryall 2010). Deck drains vary in size, shape, and material. Commonly, deck drains are channeled along deck curbs and extend away from the structure components through a discharge system connected to storm sewers (MTO 2008).

A comprehensive drainage system typically consists of several components (pipes), connections, and fasteners. These system elements are prone to deterioration over the bridge's service life due to the surrounding environmental and impact factors. Deterioration might be manifested in the form of Loosening of any of the drainage elements, or breakage along the surface area of the drainage pipes. Defects of the drainage system are considered serious due to their later contribution in water leaching and salt ingress to the concrete bridge elements below.

Table 17: Drainage System Defects

Defect Group	Defect	Description
Drainage System Defects	Pipe Breakage	A break along the surface area of the drainage pipe
	Loosening/Deterioration of Components or Connections or Fasteners	Loss of stability/ corrosion and/or weakening of a component, connection or fastener in the drainage system

III.4.3.4 Bearings' Defects

Bearings are structural accessories used to transmit load reactions from a bridge's superstructure to its substructure, while accommodating structural design requirements for transitional or rotational deck movements (Ramberger 2002). Over time, bridge bearings may be subject to different deterioration mechanisms. While steel rockers and rollers may develop corrosions and scouring, neoprene bearings are subject to shear bending and deformations. Bending or cracking may also occur to the welds or bolts at the bearing plate. Given that bearings play a significant structural role in bridges, a thorough bridge inspection should check the bridge bearings and record the several types of apparent defects. OSIM (2008) lists some of the common bearing defects as follows: lack of lubrication, cracked or broken parts, loosening or deformation of welds or bolts, steel corrosion, cracks or splits in the elastomeric pads, or Damage in stainless steel surfaces

Table 18: Bearings' Defects

Defect Group	Defect	Description
Bearings' Defects	Cracking	Variable sized linear fractures in elastomeric pads and/or steel plates
	Deformations	Shear bending in elastomeric pads and/or rocker/roller support plates
	Scouring/Scratches	Erosion and scouring in the TFE and/or stainless steel layers
	Corrosion	Rusting in steel layers and/or end support plates
	Bending/Cracking of Anchor Bolts/welds	Lateral deformations of anchor bolts, or cracking of welds

III.4.4 Model-Incorporated Sets of Bridge Element Defects

The proposed bridge assessment model in this research is of a detailed nature. Presented earlier where the different component and element hierarchies to idealize the generic breakdown of an assessed concrete bridge. However, and to attain the desired degree of detail, hierarchies are further expanded to cover element-specific sets of common possible defects for every bridge element. For instance, the deck wearing surface element of the deck component is expanded to cover the following potential defects: cracking, potholes, rutting, rippling, and loss of bond. Figures 13 through 16 illustrate the defect-based bridge element hierarchies incorporated in the proposed assessment model.

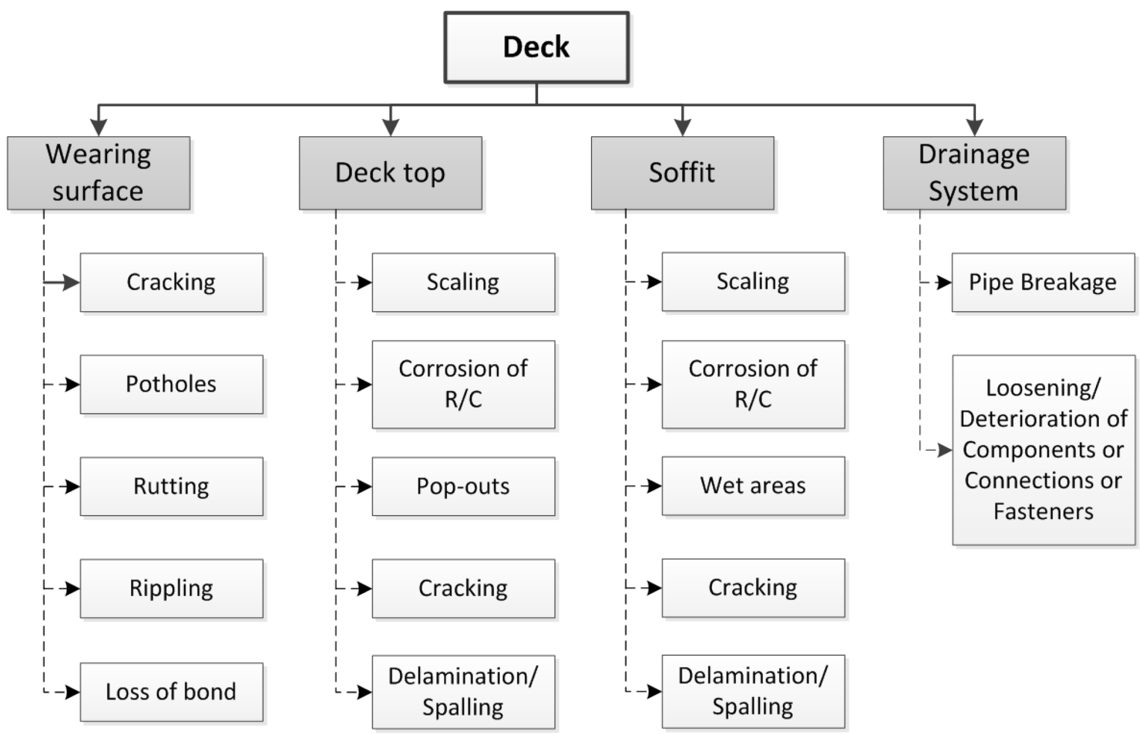


Figure 13: Deck Elements' Defects

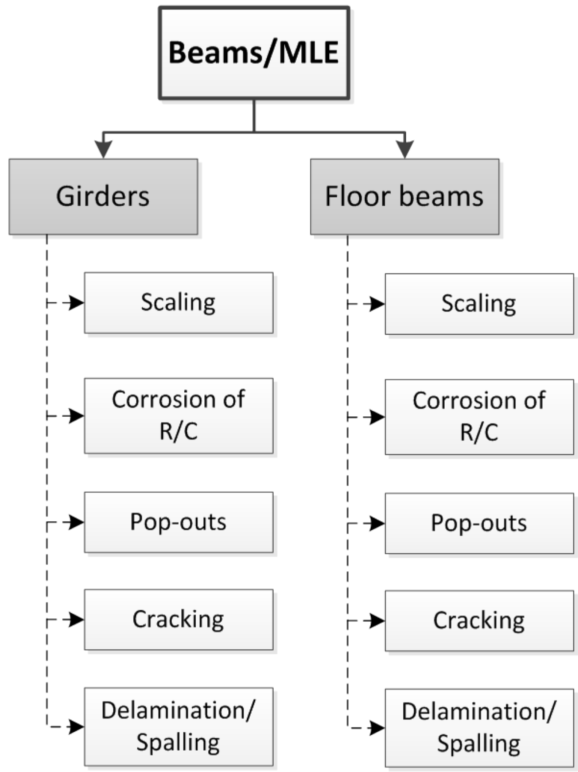


Figure 14: Beams/MLE Elements' defects

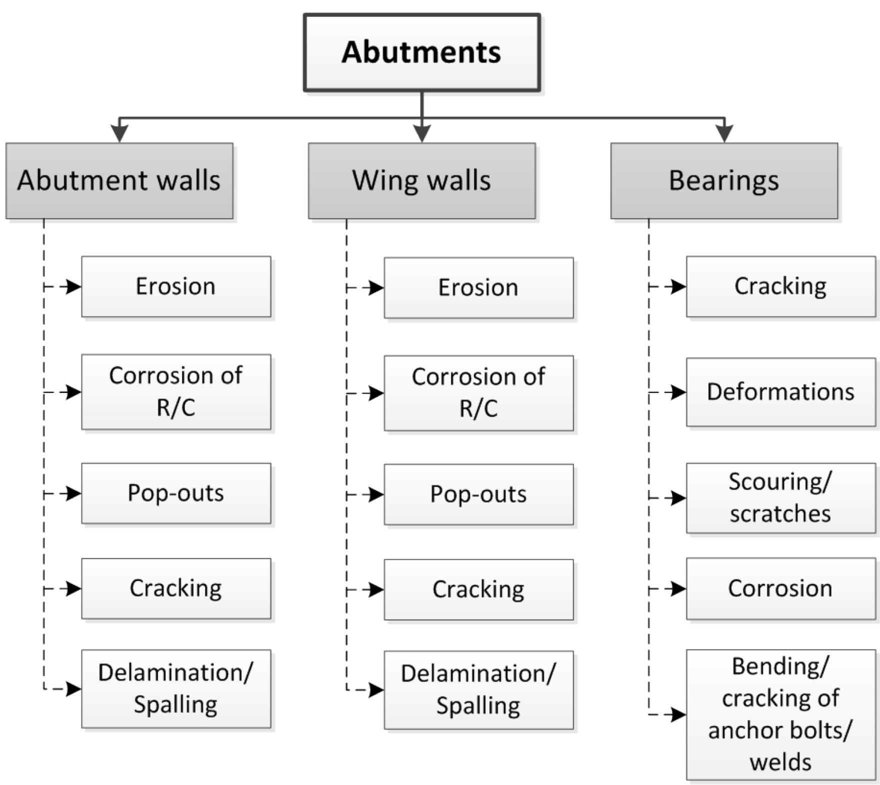


Figure 15: Abutment Elements' Defects

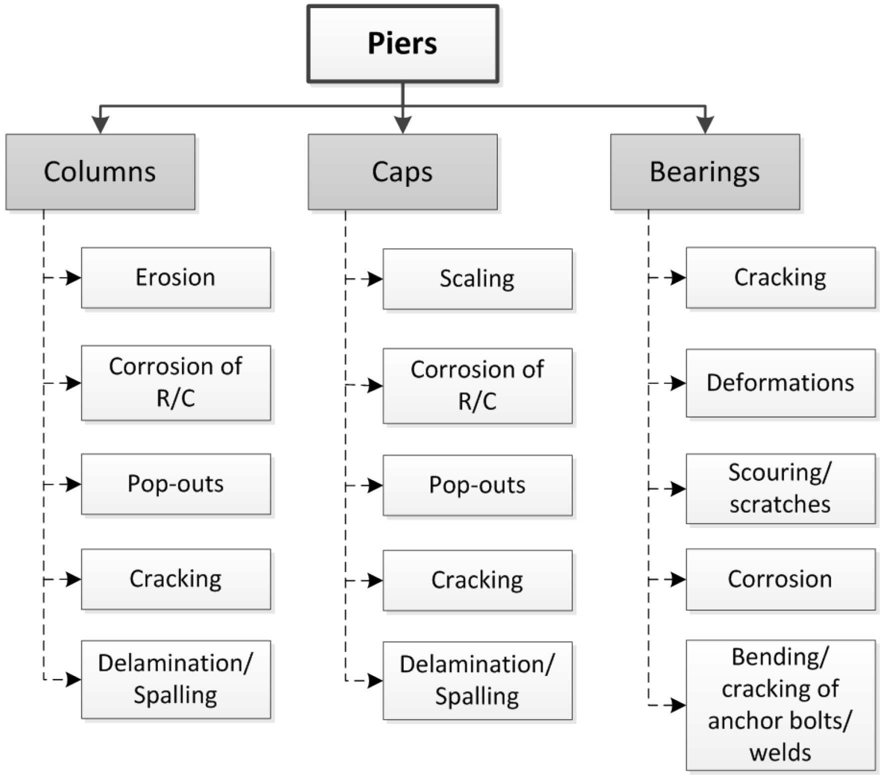


Figure 16: Pier Elements' Defects

III.4.5 Definition of Defects' Severities

After listing the possible material defects, a verbal definition is usually provided for each defect to help the inspectors correctly distinguish it (MTO 2008; MTQ 2012). Some manuals expand on defects' severity, answering the question of how severe a defect is. Verbal or numerical defect extents are defined in some cases to aid in classifying a defect in the right degree of severity (none, light, severe, very severe). Table 19 lists some of the most common concrete bridge defects that are used for the purpose of this study, along with respective measurement extents defining their level of severity (MTO 2008). The interested reader may refer to the OSIM manual for an exhaustive definition of defect severity definitions.

Table 19: Common Defects in Bridge Elements with Severity Definitions (MTO 2008)

Defect Group	Defect	Severity (none, light, medium, severe, very severe)
Concrete Defects	Scaling	(none , depth < 5mm , 6mm < depth < 10mm , 11mm < depth < 20mm , depth > 20mm)
	Corrosion of Reinforcement	(none , light stain rust , exposed reinforcement with rust stains and section loss < 10 % , exposed reinforcement with rust stains and section loss from 10% to 20% , exposed reinforcement with rust stains and section loss > 20%)
	Pop-outs	(none , hole depth < 25mm , 25mm < hole depth < 50mm , 50 mm < hole depth < 100 mm , hole depth > 100 mm)
	Cracking	(none , width < 0.1mm , 0.1 mm < width < 0.3mm , 0.3 mm < width < 1.0mm , width > 1.0mm)
	Delamination/ Spalling	(none , area < 150 mm^2, 150 mm^2 < area < 300 mm^2, 300 mm^2 < area < 600 mm^2 , area > 600 mm^2)
	Wet areas	(none , exist without cracks, exist with some cracks, exist with many cracks, exist with severe cracks)
Wearing Surface Defects	Cracking	(none , 1mm < width < 5mm , 5mm < width < 10mm , 10mm < width < 20mm , width > 20mm)
	Rippling	(none , few noticeable bumps , several bumps , numerous bumps , numerous bumps leading to imminent danger)
	Potholes	(none , depth < 10mm , 10mm < depth < 20mm , 20mm < depth < 40mm , depth > 40mm)
	Rutting	(none , depth < 10mm , 10mm < depth < 20mm , 20mm < depth < 40mm , depth > 40mm)
	Loss of Bond	(none , area < 150 mm^2, 150 mm^2 < area < 300 mm^2 , 300 mm^2 < area < 600 mm^2, area > 600 mm^2)
Drainage System Defects	Loosening/ Deterioration of Components or Connections or Fasteners	(none, up to 20% , 20% to 60 % , more than 60%)
	Pipe Breakage	(none , exist)

III.5 Determining Relative Importance Weights

To represent their contribution to structural integrity and general condition, relative importance weights for the various bridge components, elements, and defects are consequently determined. The Analytic Hierarchy Process (AHP) introduced by Saaty (1994) and previously demonstrated in the literature review chapter is implemented to accomplish this task. Through pairwise comparison surveys, expert judgments are synthesized to represent the inner and outer dependence among various elements in the hierarchy. Expert opinions are solicited by using the 9 points importance rating scale suggested by Saaty to numerically represent their judgments (Saaty 1994). A full explanation of the survey data collection for this task is thoroughly illustrated in the following data collection chapter. Defects' weight factors will be representing the defects' influence on the safety and structural integrity of their respective elements. Similarly, Weight factors will be representing the bridge elements' and components' relative structural importance towards the bridge structure.

III.6 Establishment of a Uniform Fuzzy Grading Scheme

Liang et al. (2001) indicate that the application of fuzzy logic has considerable significance in the condition assessment of existing bridges. This is attributable to the involvement of judgmental experience and the amount of fuzzy information in the evaluation process. It has been also suggested that using fuzzy based techniques help in defining a measure of exceeding predefined numerical limits with memberships to adjacent linguistic sets/grades (Sadiq and Rodriguez 2004).

The next step in the development of the proposed assessment model is the establishment of a unified fuzzy grading scheme, which will form the basis of the rating process and treat the subjective and judgmental nature of the assessment. This is essentially done by collecting information about severities and extents of all possible bridge defects (see Table 19). The scheme will map defect extents to an order of descending fuzzy grades, laying grounds for an objective, uniform assessment of all detected bridge defects. In this application, it is assumed that every defect measurement is associated with an underlying fuzzy set (frame of discernment) H, which is defined by 4 fuzzy linguistic grades ranging over the defect extent. The 4-grades scale is based on material condition rating (MCR) defined by Quebec Ministry of Transportation (MTQ), with A and D being best and worst states, respectively (MTQ 2012).

$$H = \{A, B, C, D\} \tag{17}$$

Figure 17(a) shows the instant grade mapping of bridge defects based on the type of element (primary, secondary, or auxiliary). To facilitate fuzzy representation of the deterioration process, linear triangular fuzzy numbers (TFN) were constructed to help in remediating the vague/overlapping nature of the 4 linguistic grades. Intermediate interval values of the prescribed limits of affected defect area were considered to define ranges pertaining to the evaluation grades for “light”, “medium”, “severe”, and “very severe” defects. The Membership functions presented in Figure 17(b) will map a given defect-affected area onto the interval [0, 1] for primary elements; indicating a defect measurement’s degree of belonging to each of the assessment grades. Similar functions were generated for secondary and accessory elements.

Note that exclusively for the case of detecting “delamination/spalling” on concrete members, or “Loss of Bond” in the wearing surface, a grade “C” is immediately assigned without going through the fuzzification process. This is due to the severity of such defects on the structural integrity of bridge members, as suggested by MTO (2008). The hereby proposed fuzzy grading system treats the inherent subjectivity of structural defect measurements, and secures uniformity in mapping all the detected defects on the widely used 4-level grading system.

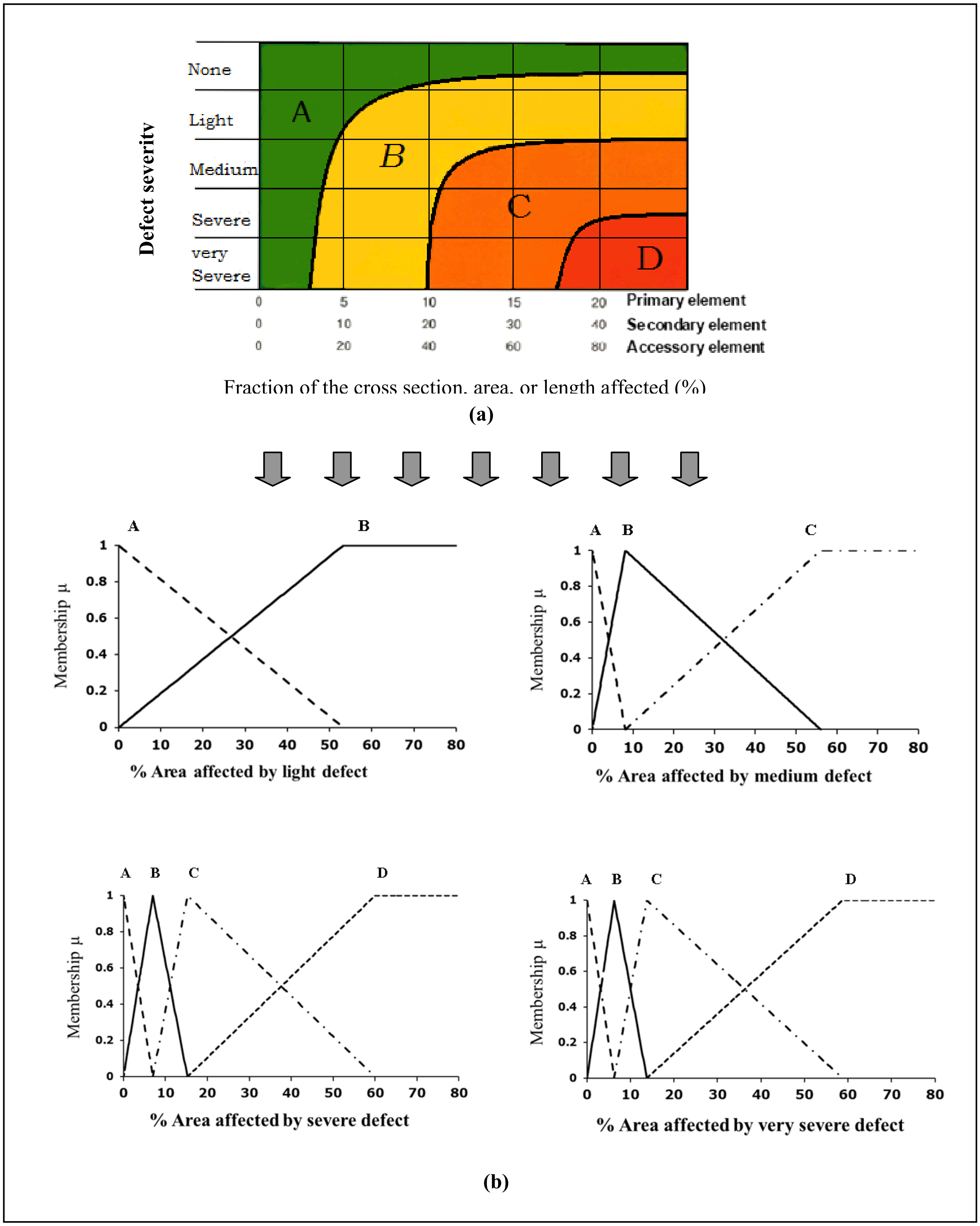


Figure 17: (a) Grade Mapping Over Defect Extent (b) Fuzzy Membership Functions for the Evaluation of Defects of Different Severity Levels.

III.7 Multi-level Aggregation of Bridge Assessments

This section is intended to show the systematic aggregation and value interpretation of the concrete bridge defect measurements; filling in the final step in the hereby proposed development of a defect based condition assessment model for concrete bridges. By this stage, the concrete bridge modeled hierarchy structure has already been built, the relative importance weights of elements in the hierarchy have been established through surveys, and fuzzy logic based processing of the various field-detected bridge defects has been performed. At this point, the obtained fuzzy defect assessments will be aggregated in a systematic, bottom-up manner.

III.7.1 The FSE Approach

The Fuzzy Synthetic Evaluation (FSE) technique can be used as a simple approach for multi-level aggregation of fuzzy numbers. Therefore, the FSE is employed in this section to translate concrete bridge defect measurements into a comprehensive condition assessment. This approach will be able to synthesize the evaluation of defects, elements, and components in a concrete bridge structure.

The multi-level weighted aggregation of fuzzy assessment can be performed as follows (Rajani et al. 2006):

- Aggregation of defect ratings towards their respective bridge elements using equation 18:

$$R_e = [\omega 1 \ldots \quad \omega i \ldots \quad \omega n] \bullet \begin{bmatrix} \mu 6(1) & \mu 5(1) & \mu 4(1) & \mu 3(1) & \mu 2(1) & \mu 1(1) \\ \vdots & \vdots & \vdots & \vdots & \vdots & \vdots \\ \mu 6(i) & \mu 5(i) & \mu 4(i) & \mu 3(i) & \mu 2(i) & \mu 1(i) \\ \vdots & \vdots & \vdots & \vdots & \vdots & \vdots \\ \mu 6(n) & \mu 5(n) & \mu 4(n) & \mu 3(n) & \mu 2(n) & \mu 1(n) \end{bmatrix} \quad [18]$$

Where:

R_e : Element Rating

ω_i : Weight of defect i , $i \in (1,2, \ldots, n)$

n : Number of defects

$\mu_3(i)$: Membership of defect "i" to the evaluation grade 3, for instance

"." : Scalar matrix multiplication

- Similarly, Aggregation can be performed for bridge elements towards their respective components using equation 19:

$$R_c = [\omega 1 \ldots \quad \omega i \ldots \quad \omega n] \bullet \begin{bmatrix} \mu 6(1) & \mu 5(1) & \mu 4(1) & \mu 3(1) & \mu 2(1) & \mu 1(1) \\ \vdots & \vdots & \vdots & \vdots & \vdots & \vdots \\ \mu 6(i) & \mu 5(i) & \mu 4(i) & \mu 3(i) & \mu 2(i) & \mu 1(i) \\ \vdots & \vdots & \vdots & \vdots & \vdots & \vdots \\ \mu 6(n) & \mu 5(n) & \mu 4(n) & \mu 3(n) & \mu 2(n) & \mu 1(n) \end{bmatrix} \quad [19]$$

Where:

R_c : Component Rating

ωi : Weight of element "i" , $i \in (1,2, \ldots, n)$

$\mu_4(i)$: Membership of element i to the evaluation grade 4, for instance

n : Number of elements

"." : Scalar matrix multiplication

The overall condition rating of the bridge can be obtained using similar calculations that factor in both components weights and their individual fuzzy ratings. The initial effort of this research targeted the implementation of the FSE approach for the multi-level fuzzy rating aggregations as it features a simple and systematic procedure. However, while analyzing the approach, a number of practical limitations have been encountered while applying it to bridge condition assessment under uncertainties.

FSE works perfectly when having full information about all the inputs in the model. However, if a defect measurement is missing, the matrix multiplication process cannot be soundly carried on. In the FSA aggregation method, a missing measurement of a defect implies that the defect is assumed to be of excellent condition (non-existing). This could lead to misleading results, particularity in the cases of rating bridge elements with nonparent subsurface defects.

Alternately this study attempted to implement a more powerful approach that can handle probabilistic types of uncertainties in addition to fuzzy uncertainties. The following section describes the application of the Hierarchical Evidential Reasoning (HER) method as a comprehensive condition assessment and aggregation algorithm.

III.7.2 HER Model for Bridge Condition Assessment

As mentioned in the literature review, The Hierarchical Evidential Reasoning (HER) algorithm has shown strong potentials in the analysis of hierarchal MADM assessment models that are based on multi-level evaluation and aggregation of attribute assessments throughout modeled hierarchy structures. The method is powerful in combining information that has inherent epistemic uncertainty; by employing a belief structure that is able to take into account incomplete information or ignorance about some assessment attributes. This is made possible by employing the Dempster-Shafer (D-S) theory of evidence combination. The HER algorithm is profoundly utilized in this research as an aggregation engine in what should be a comprehensive multi-leveled condition assessment of concrete bridges.

Suppose that the condition of a concrete bridge is being assessed based on F number of components, which can be evaluated on the basis of K number of elements that, in turn, are evaluated on L number of directly observed or measured defects (Figure 18). Let's say, for instance, that the "soffit" element in the "Bridge Deck" component is evaluated on the defect/factor of "Cracking D_2". Further, let's assume that filed inspection of the deck soffit revealed that the state of the developed cracking is evaluated to the grade $H = B$ with a degree of belief $\beta_{B,2} = 60\%$, and to the grade $H = C$ with a degree of belief $\beta_{C,2} = 40\%$. It follows that this distributed form of evaluation can be represented by the general form of (Bai et al. 2008):

$$S(D_2) = \{(H_A, \beta_{A,2}), (H_B, \beta_{B,2}), (H_C, \beta_{C,2}), (H_D, \beta_{D,2})\} \qquad [20]$$

Therefore, the *k-th* bridge element can be assessed through a defect factor of D_i^k to a grade H_n with a degree of belief $\beta_{n,i}$ .The assessment is known to be complete if $\sum_{n=1}^{N} \beta_{n,i} = 1$, and incomplete if $\sum_{n=1}^{N} \beta_{n,i} < 1$. The case of $\sum_{n=1}^{N} \beta_{n,i} = 0$ represents ignorance in the assessment, and reflects lack of information concerning the assessment of that particular defect.

III.7.2.1 Basic Probability Assignment (BPA)

The basic probability assignment (BPA) represents the degree of belief/confidence assigned to an evaluation grade in *H*, and mainly signifies the extent to which the existing evidence supports that grade. Essentially, Basic probability assignments (BPAs) to evaluation grades, denoted $m_i(H_n)$, are obtained from every available piece of evidence (defect measurement).

When a defect D_i is evaluated by an inspector, a new set of probability masses is assigned to the evaluation grades. This is achieved by scaling down the degrees of belief ($\beta_{n,i}$) in the evaluation grades determined from the defect's condition rating, to the relative importance weight (w_i) of that defect (Yang and Xu 2002) :

$$m_{n,i} = m_i(H_n) = w_i\, \beta_{n,i}\,, \qquad n \in \{1, 2, \dots, N\}; i \in \{1, 2, \dots, L\} \qquad [21]$$

$$m_{H,i} = m_i(H) = 1 - \sum_{n=1}^{N} m_{n,i} = 1 - w_i \sum_{n=1}^{N} \beta_{n,i} \qquad i \in \{1, 2, \dots, L\} \qquad [22]$$

$$\bar{m}_{H,i} = \bar{m}_i(H) = 1 - w_i \qquad i \in \{1, 2, \dots, L\} \qquad [23]$$

$$\tilde{m}_{H,i} = \tilde{m}_i(H) = w_i \left(1 - \sum_{n=1}^{N} \beta_{n,i}\right) \; i \in \{1, 2, \dots, L\} \qquad [24]$$

$$m_{H,i} = \bar{m}_{H,i} + \tilde{m}_{H,i} \qquad [25]$$

$$\sum_{i=1}^{L} w_i = 1 \qquad [26]$$

Where $m_{n,i}$ denotes the BPA assigned to the $n\text{-}th$ evaluation grade through the assessment of defect D$_i$; it measures the exact portion of belief that a given body of evidence provides to that particular grade. The ignorance in the assessment is embodied in $m_{H,i}$, which represents the BPA assigned to the whole set of *H* and not to any of the individual grades

$m_{H,i}$ involves two quantities: ($\bar{m}_{H,i}$) is caused by differences in the relative importance, and ($\tilde{m}_{H,i}$) illustrates the incompleteness of the assessment (degrees of confidence not summing up to 1). The reader may refer to Yang and Singh (1994) for a more detailed description on the topic.

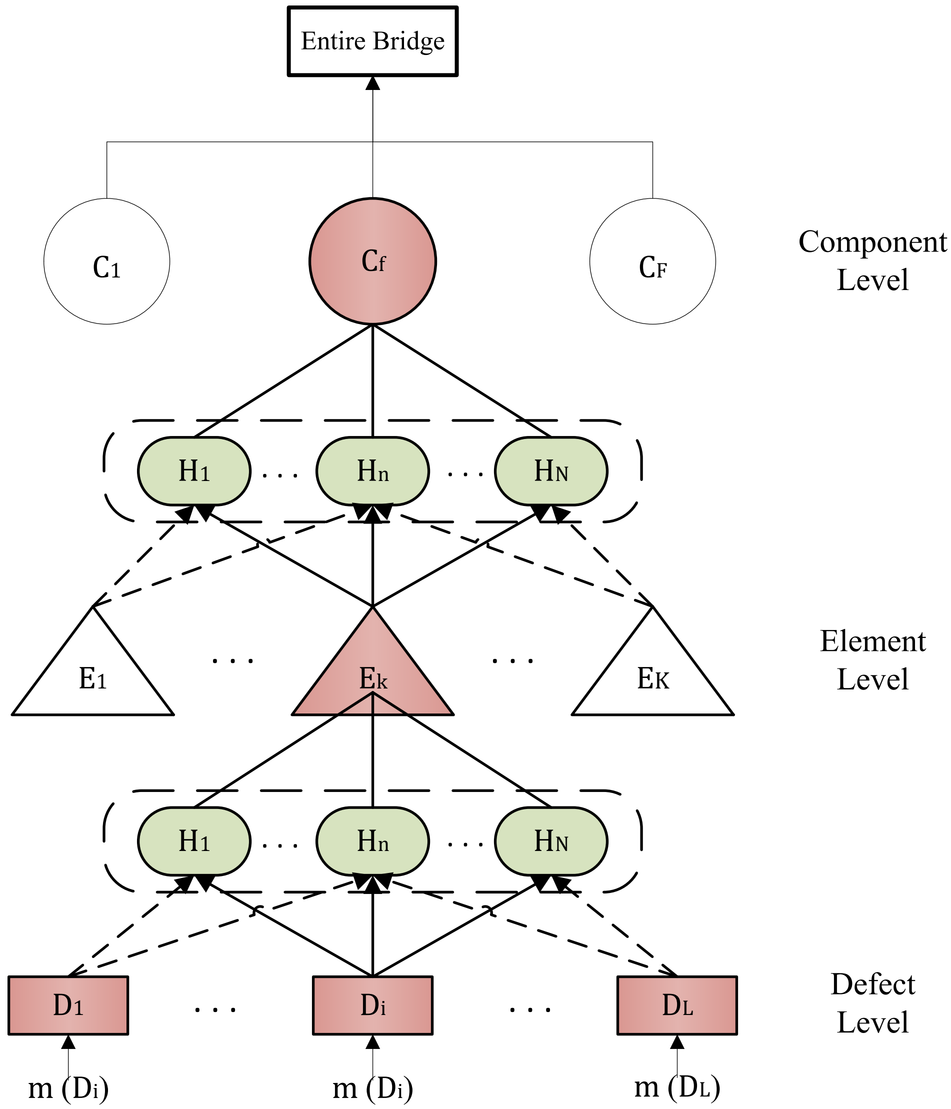


Figure 18: Assessment Aggregation Process for the HER Bridge Model.

III.7.2.2 The Recursive ER algorithm for multi-level assessment aggregation

Through the detection and evaluation of all different defects, several sets of basic probability assignments (m-values) to the evaluation grades are generated in the process. Those acquired sets of BPAs can be referred to as bodies of evidence provided to the ER bridge condition model. At this point, The D-S combination rule is utilized under the ER framework for the purpose of accumulating supporting pieces of evidence towards obtaining a comprehensive and systematic condition rating of the concrete bridge under appraisal. The model commences by assessing defects at the bottom of the hierarchy structure and aggregating the resulting bodies of evidence towards the respective elements, which in turn are aggregated to rate their parent components and eventually the entire bridge.

Aggregation is performed through the recursive ER algorithm for combination of evidence. Every iteration I involves combining BPAs of an assessment i with those of assessment $(i+1)$. The recursive combination process is illustrated in the following equations (Yang and Xu 2002):

$$\{H_n\}: m_{n,I(i+1)} = K_{I(i+1)}\left[m_{n,I(i)}\, m_{n,i+1} + m_{H,I(i)}\, m_{n,i+1} + m_{n,I(i)}\, m_{H,i+1}\right] \quad [27]$$

$$m_{H,I(i)} = \tilde{m}_{H,I(i)} + \bar{m}_{H,I(i)} \qquad n \in \{1, 2, \dots, N\} \quad [28]$$

$$\{H\}: \tilde{m}_{H,I(i+1)} = K_{I(i+1)}\left[\tilde{m}_{H,I(i)}\, \tilde{m}_{H,i+1} + \bar{m}_{H,I(i)}\tilde{m}_{H,i+1} + \tilde{m}_{H,I(i)}\, \bar{m}_{H,i+1}\right] \quad [29]$$

$$\{H\}: \bar{m}_{H,I(i+1)} = K_{I(i+1)}\left[\bar{m}_{H,I(i)}\bar{m}_{H,i+1}\right] \quad [30]$$

$$K_{I(i+1)} = \left[1 - \sum_{t=1}^{N} \sum_{\substack{j=1 \\ j \neq i}}^{N} m_{t,I(i)}\, m_{j,i+1}\right]^{-1}, i = 1, \dots, L-1 \quad [31]$$

Where $m_{n,I(i)}$ designates the combined probability mass, aggregating relative support of the evaluation of i number of defects to the evaluation grades in H. Every iteration will result in updated BPAs allocated to the evaluation grades. Following this recursive fashion, a set of BPAs obtained from iteration will be regarded as a prerequisite for the next iteration, where it will be combined with another set of BPAs. The process is continuously repeated until all available assessment defects are taken into consideration.

Eventually, after aggregating all L defects, the degrees of belief allocated to the assessment grades H_n and to the set H are obtained through the following (Yang and Xu 2002):

$$\{H_n\}: \beta_n = \frac{m_{n,I(L)}}{1 - \bar{m}_{H,I(L)}}, n = 1, \dots, N \qquad [32]$$

$$\{H\}: \beta_H = \frac{\tilde{m}_{H,I(L)}}{1 - \bar{m}_{H,I(L)}} \qquad [33]$$

Where β_n represents the degree of belief or likelihood to which H_n is supported, whereas β_H denotes the degree of belief that remained unassigned to neither of the evaluation grades (ignorance).

IV. DATA COLLECTION

IV.1 Overview

This research aims to develop a detailed condition assessment model for concrete bridges. As mentioned in the previous chapter, data solicited for the proposed assessment model consist of three main categories (Figure 19):

1. Bridge assessment factors. The proposed model is set to identify the different components, elements, and defects. The 3-level hierarchy of factors forms the basis on which a bridge will be assessed.
2. The relative importance weights of all the assessment factors in a generic concrete bridge's hierarchical model. This covers weights of the different components and elements, in addition to the various structural defects. These weights are evaluated using the Analytic Hierarchy Process (AHP) method.
3. Information pertaining to the extents and severities of the detected defects on tested bridges. Detailed inspection reports/condition surveys of existing concrete bridges are required for gathering such types of inputs and supplying them to the assessment model.

The first data category was explained in detail in the previous chapter. It is the focus of this chapter to lay out all the adopted procedures to obtain the rest of the above outlined data sets. This includes the survey questionnaire formulation, survey data analysis, particulars of the survey respondents, and data collection for case studies.

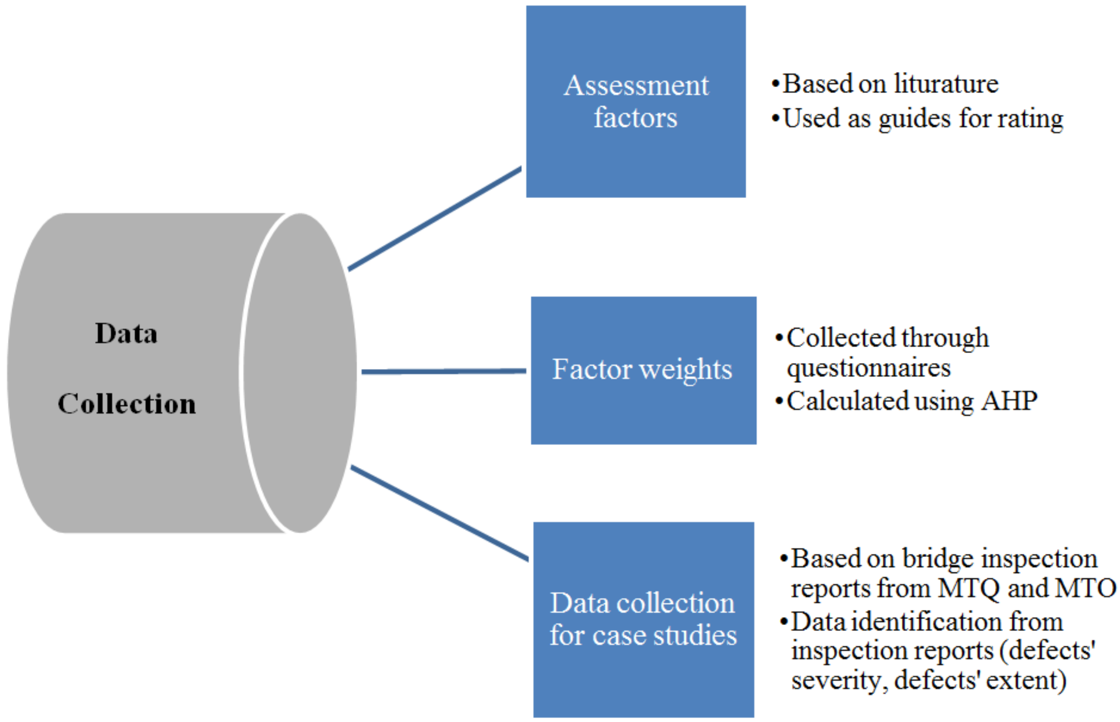


Figure 19 Assessment Model Data Collection

IV.2 Assessment Factors

The first type of data pertains to the identification of the different bridge assessment factors from literature. The proposed generic bridge assessment model is set to identify the different components, elements, and defects -on the basis of which- a bridge will be evaluated (refer to section III.4.2). In addition, among the main objectives of this research was to expand on the identification and analysis of the various types of deterioration mechanisms/ defects that may develop on the various concrete bridge elements. This includes identifying and grouping the defects to their respective bridge elements (refer to section III.4.3); in addition to identifying and collecting defect measurement extents that define their level of severity (refer to section III.4.5).

IV.3 Survey for Relative Weights of Factors

As mentioned earlier, it is intended for the comprehensive bridge assessment model to account for the relative importance weights of all the assessment factors in the hierarchical breakdown structure. This covers weights of the different bridge components and elements identified earlier. Moreover, weights are to be calculated for the various structural defects. These weights are evaluated using the Analytic Hierarchy Process (AHP) method. AHP requires carrying out pairwise comparisons among factors of the same level with respect to a common factor/criterion of the upper level in the hierarchy. Thus, a survey questionnaire of two parts was designed and executed for this task.

IV.3.1 Survey Procedure

The survey questionnaire was designed and structured into an easy to understand and user-friendly approach. A brief background about the research is provided at the top of the survey package to introduce respondents to the purpose behind the research and get them familiar with the theme of the study. Survey questions were formulated to guide respondents through a concise and straightforward pairwise comparison procedure, with two comparable items at a time. A simply defined rating scale was suggested to the survey participants to facilitate the quantification of their preferences. A chart showing slots of the compared items in the hierarchy was supplemented at the beginning of every section to visually aid respondents in comprehending section-specific hierarchical breakdowns. Additionally, a brief definition table was appended at the end of the questionnaire package for reference.

An example on the nature of performed pairwise comparisons is shown in Figure 20. At every step, the comparison is made between two elements (A) and (B) with respect to a higher element/criterion (C). Respondents were advised to carry out the pairwise comparison as follows:

1. If the comparison indicates that element (A) is equally important to element (B) with respect to the control criterion (C), please mark “1”.
2. If the comparison indicates that element (A) is more important than element (B) with respect to the control criterion (C), please indicate so by choosing a value from the middle to left, using the ascending level of importance scale.
3. If the comparison indicates that element (B) is more important than element (A) with respect to the control criterion (C), please indicate so by choosing a value from the middle to right, using the ascending level of importance scale.
4. Should it be felt that intermediate values (2, 4, 6, 8) better represent your comparison judgement, please indicate the value between the right limits in the table.

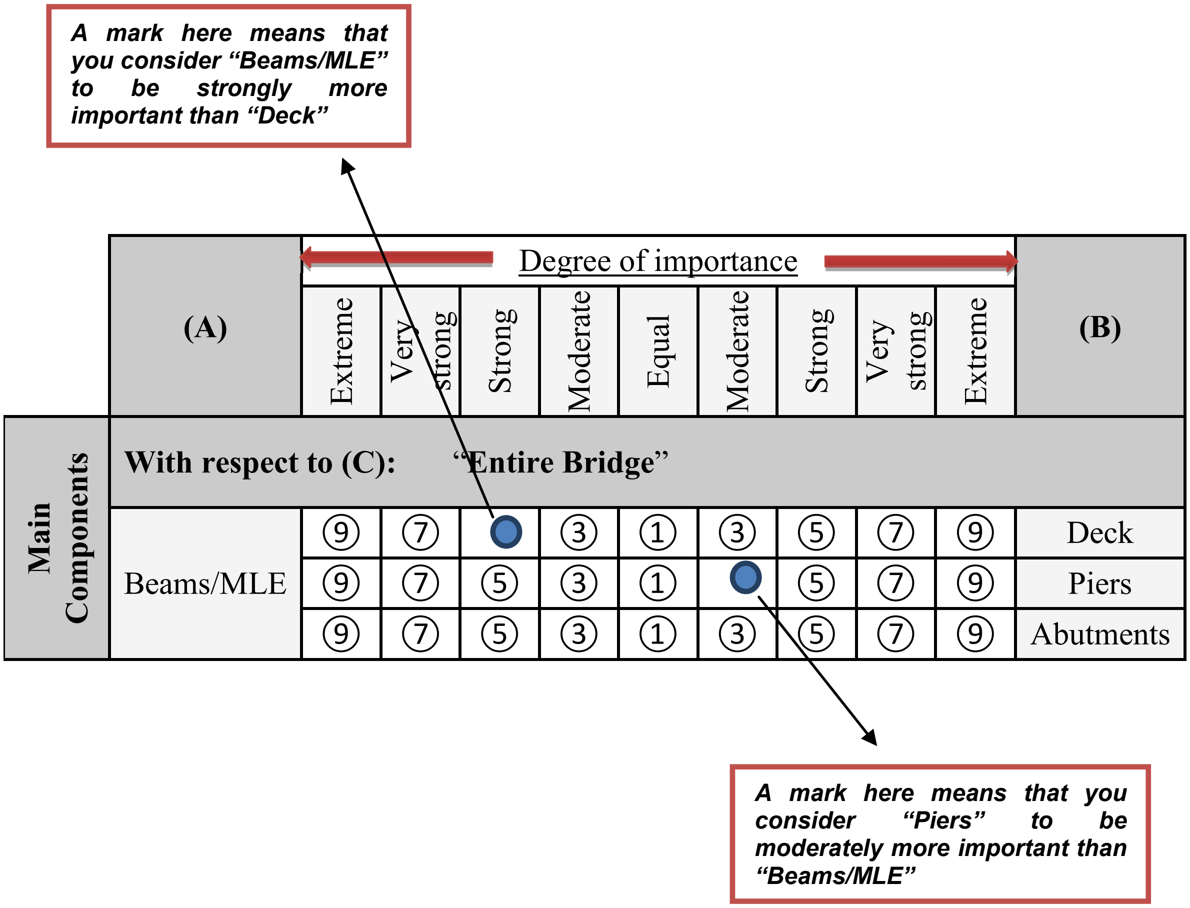


Figure 20: An Example Illustrating the Comparison Process

IV.3.2 Online Questionnaire Interface

Given the geographic barrier between the researcher and targeted survey respondents, it was decided to adapt to an online version of the survey to go in parallel with the hardcopy packages. The survey interface was uploaded to an online server, aiding the purpose of facilitated circulation towards the targeted respondents, making it easy to access and fill out. A snapshot from the survey online interface can be found in Figure 21.

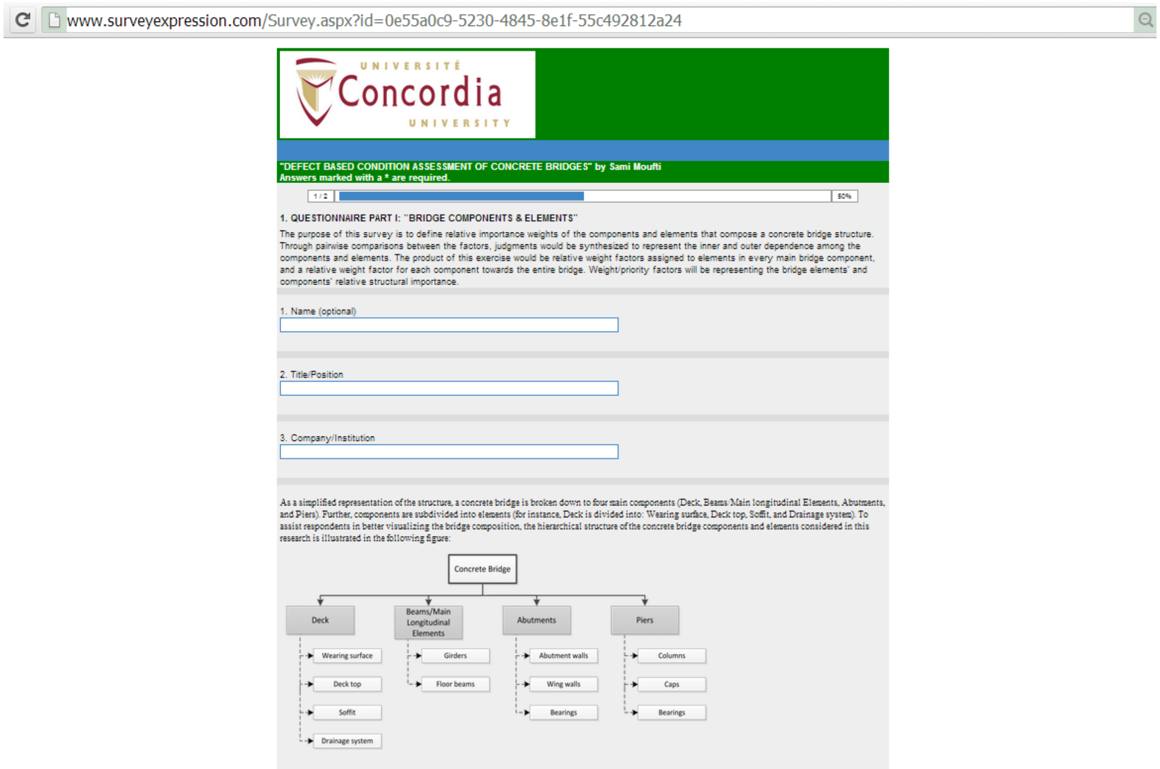


Figure 21: Survey Online Interface

A link to the online survey interface was provided to the targeted respondents through personal emails or social media platforms. A pilot test of the survey was performed by sending it to two independent experts before its official lunch. The purpose being to ensure clarity, and increase the credibility of the survey.

IV.3.3 Survey Respondents

The survey questionnaire was distributed and circulated to bridge engineers/ inspectors/ managers in an effort to procure responses from field-related experts. The questionnaire, in both paper and online versions, was sent out to 52 experts located throughout different Canadian provinces. However, only 21 responses were successfully collected, resulting in a response rate of around 40 %. Figure 22 summarizes the geographic allocation of the survey respondents.

Respondents ranged from bridge inspectors and structural engineers to bridge managers and bridge network directors. In addition, the respondents held executive positions at both public and private organizations.

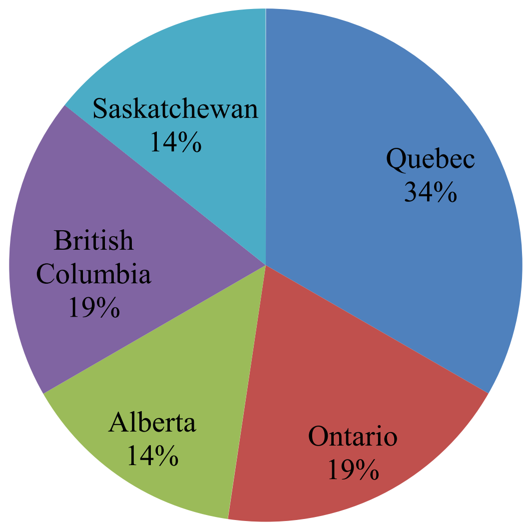


Figure 22: Geographic Distribution of Survey Respondents.

IV.3.4 Questionnaire Part I

The purpose of this part of the questionnaire is to define relative importance weights of the components and elements that compose a concrete bridge structure. Through pairwise comparisons between the factors, judgments would be synthesized to represent the inner and outer dependence among the components and elements in the bridge hierarchical breakdown. As mentioned earlier, experts' opinions are solicited by using the 9 points importance rating scale suggested by Saaty (1994) to numerically represent their judgments. The pairwise comparison between two bridge components could be the answer to the question of “How much component (A) is more/less important than component (B) in terms of structural contribution to the entire bridge?”

The product of this exercise would be relative weight factors assigned to elements in every main bridge component, and a relative weight factor for each component towards the entire bridge. Weight/priority factors will be representing the bridge elements’ and components’ relative structural importance. Please refer to appendix D for a full demonstration of the survey.

IV.3.5 Questionnaire Part II

The purpose of this part of the questionnaire is to define relative importance weights of the possible defects that may be detected on the concrete bridge elements. Experts’ judgments are similarly quantified using Saaty’s scale. An example of the pairwise comparison process between, let’s say cracking and scaling on bridge beams, can be reflected in the answer to the question “ What is the relative importance of cracking as compared to scaling in impact on the structural integrity of the bridge beam?”

Through pairwise comparisons between the defects, and as in part 1, judgments would be synthesized to represent the inner and outer dependence among the defects. The product of this exercise would be relative weight factors assigned to defects with respect to their relative elements. Weight/priority factors will be generated for every defect type in terms of importance on the safety and structural integrity of their respective elements. More details can be found in appendix D.

IV.3.6 Analysis of the Survey Results

IV.3.6.1 Calculating Relative Weights

Based on the received survey responses, pairwise comparison matrices are developed for each hierarchy level based on the AHP technique. The weight of every element in the hierarchy is consecutively calculated using the Eigen vector approach. Typically, pairwise comparison judgments (obtained through each survey response) are filled in comparison matrices in the form of fractions denoted by aij ; representing the degree of importance of element i relative to element j, with respect to their parent criterion. As a sample of pairwise comparison matrix calculations, Table 20 illustrates pairwise comparisons among the various bridge deck elements in order to calculate their respective relative importance weights. In the same manner, relative importance weights are calculated for the following:

1. Bridge components with respect to the entire bridge
2. Bridge elements with respect to their respective components
3. Structural defects with respect to their bridge elements.

Table 20: Pairwise Comparisons and Relative Weight Calculations of Bridge Deck Elements

	Deck Top	Wearing Surface	Soffit	Drainage System	Sum	Weight
Deck Top	1	5	3	7	16.00	0.60
Wearing Surface	1/5	1	3/5	7/5	3.20	0.12
Soffit	1/3	5/3	1	7/3	5.33	0.20
Drainage System	1/7	0	0	1	2.29	0.08
					26.82	1.00

IV.3.6.2 Data Reliability

To ensure reliability of the questionnaire responses, a reliability analysis using Cronbach's alpha approach is used to verify the questionnaire outputs. According to Wei et al. (2007), Cronbach's alpha approach is the most widely applied measure of statistical reliability. Otherwise called the coefficient of reliability, Cronbach's alpha (α) is a measure of internal consistency of items in an assessed instrument/questionnaire. It can be calculated according to equation 34:

$$C_{\propto} = \frac{n}{n-1}\left(1 - \frac{\sum V_i}{\bar{V}}\right) \qquad [34]$$

Where:

n = number of points

V_i = variance of scores for each point

$\bar{V}$ = total variance of overall points

Cronbach's alpha (α) coefficient has a maximum value of 1 and a minimum of 0. The reliability coefficient generally increases in value with higher correlation between the items. The lower acceptable limit of (α) is 0.5, with values below that being considered less reliable (Reynaldo 1999). A value of 0.7 or higher is typically judged to be of acceptable reliability. Table 21 provides a summary of reliability interpretations of different value ranges of (α).

Table 21 : Interpreting Cronbach's Alpha Values (Reynaldo 1999)

Cronbach's alpha	Interpretation
0.9 and greater	High reliability
0.8-0.89	Good reliability
0.7-0.9	Acceptable reliability
0.65-0.69	Marginal reliability
0.5-0.64	Minimal reliability

A full reliability analysis was conducted on relative weight data sets from the collected surveys. To verify reliability, each group variance is measured against the overall variance using equation 34. It was deemed appropriate to screen the data sets to achieve a minimum α value of 0.7, so as to ensure acceptable data reliability and consistency. This resulted in the elimination of one received response that was noticeably inconsistent; thus, a total of 20 responses were retained eventually. The following tables (22-27) summarize the resultant relative importance weights based on the AHP synthetic calculations, which were performed on the pairwise comparisons obtained from every survey respondent. The Cronbach's reliability coefficient is calculated for every group of component or element weights by measuring the variance of each respondent against the overall variance according the above explained equation 34. It is noted that higher Cronbach's alpha values indicate increasing similarity of views held by the responding experts.

Table 22: Relative Weights of Bridge Components and Elements Obtained From Collected Surveys

Components and Elements	Respondents																				
Main Components	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	**AVG**
Beams/MLE	0.44	0.33	0.65	0.16	0.25	0.39	0.54	0.31	0.33	0.54	0.33	0.58	0.43	0.33	0.33	0.31	0.16	0.58	0.06	0.38	**0.37**
Deck	0.44	0.55	0.13	0.04	0.25	0.13	0.11	0.06	0.55	0.18	0.55	0.12	0.09	0.57	0.55	0.31	0.04	0.19	0.39	0.13	**0.27**
Piers	0.06	0.07	0.13	0.40	0.25	0.39	0.18	0.31	0.05	0.18	0.07	0.19	0.43	0.05	0.07	0.31	0.40	0.12	0.28	0.38	**0.22**
Abutments	0.06	0.05	0.09	0.40	0.25	0.08	0.18	0.31	0.07	0.11	0.05	0.12	0.06	0.05	0.05	0.06	0.40	0.12	0.28	0.13	**0.15**
Cronbach's Alpha	0.83																				
Deck	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	**AVG**
Deck Top	0.41	0.65	0.43	0.20	0.60	0.56	0.38	0.40	0.60	0.38	0.65	0.39	0.32	0.60	0.65	0.52	0.20	0.65	0.54	0.43	**0.48**
Wearing Surface	0.41	0.13	0.06	0.03	0.12	0.06	0.13	0.06	0.12	0.38	0.13	0.08	0.32	0.12	0.13	0.22	0.03	0.13	0.18	0.09	**0.15**
Soffit	0.05	0.13	0.43	0.45	0.20	0.19	0.38	0.40	0.20	0.13	0.13	0.39	0.32	0.20	0.13	0.18	0.45	0.13	0.18	0.43	**0.25**
Drainage System	0.14	0.09	0.09	0.32	0.09	0.19	0.13	0.13	0.09	0.13	0.09	0.13	0.05	0.09	0.09	0.08	0.32	0.09	0.11	0.06	**0.12**
Cronbach's Alpha	0.96																				
Beams/MLE	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	**AVG**
Girders	0.75	0.83	0.75	0.88	0.83	0.88	0.50	0.88	0.75	0.83	0.83	0.83	0.50	0.17	0.83	0.75	0.88	0.75	0.83	0.50	**0.74**
Floor beams	0.25	0.17	0.25	0.13	0.17	0.13	0.50	0.13	0.25	0.17	0.17	0.17	0.50	0.83	0.17	0.25	0.13	0.25	0.17	0.50	**0.26**
Cronbach's Alpha	0.97																				
Abutments	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	**AVG**
Abutment Walls	0.45	0.74	0.11	0.65	0.14	0.47	0.65	0.76	0.65	0.60	0.74	0.20	0.47	0.11	0.74	0.43	0.65	0.14	0.74	0.33	**0.49**
Wing Walls	0.09	0.15	0.11	0.13	0.14	0.07	0.13	0.08	0.13	0.20	0.15	0.07	0.05	0.11	0.15	0.37	0.13	0.14	0.11	0.33	**0.14**
Bearings	0.45	0.11	0.78	0.22	0.71	0.47	0.22	0.15	0.22	0.20	0.11	0.73	0.47	0.78	0.11	0.20	0.22	0.71	0.15	0.33	**0.37**
Cronbach's Alpha	0.90																				

Piers	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	AVG
Pier Columns	0.33	0.65	0.11	0.68	0.71	0.47	0.43	0.47	0.60	0.23	0.65	0.45	0.33	0.09	0.65	0.33	0.68	0.45	0.71	0.60	**0.48**
Pier Caps	0.33	0.22	0.41	0.10	0.14	0.47	0.43	0.47	0.20	0.69	0.22	0.45	0.33	0.45	0.22	0.33	0.10	0.09	0.14	0.20	**0.30**
Bearings	0.33	0.13	0.48	0.23	0.14	0.05	0.14	0.07	0.20	0.08	0.13	0.09	0.33	0.45	0.13	0.33	0.23	0.45	0.14	0.20	**0.22**
Cronbach's Alpha	0.89																				

Table 23: Relative Weights of Deck Elements' Defects Obtained From Collected Surveys

Deck Elements' Defects	Respondents																				
Wearing Surface Defects	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	**AVG**
Potholes	0.29	0.50	0.55	0.52	0.29	0.33	0.45	0.08	0.52	0.57	0.50	0.28	0.56	0.06	0.50	0.55	0.50	0.23	0.39	0.50	**0.41**
Cracking	0.06	0.10	0.08	0.10	0.06	0.11	0.09	0.32	0.10	0.11	0.10	0.09	0.11	0.41	0.10	0.08	0.07	0.46	0.08	0.10	**0.14**
Rutting	0.06	0.17	0.11	0.10	0.10	0.11	0.15	0.03	0.10	0.08	0.17	0.04	0.11	0.06	0.17	0.08	0.10	0.05	0.39	0.17	**0.12**
Rippling	0.06	0.07	0.08	0.10	0.04	0.11	0.15	0.25	0.10	0.11	0.07	0.06	0.11	0.06	0.07	0.18	0.17	0.05	0.06	0.07	**0.10**
Loss of Bond	0.54	0.17	0.18	0.17	0.50	0.33	0.15	0.32	0.17	0.11	0.17	0.53	0.11	0.41	0.17	0.11	0.17	0.23	0.08	0.17	**0.24**
Cronbach's Alpha	0.95																				
Deck Top Defects	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	**AVG**
Delamination/ Spalling	0.38	0.53	0.14	0.27	0.48	0.24	0.55	0.32	0.27	0.23	0.53	0.20	0.35	0.38	0.48	0.55	0.56	0.31	0.45	0.37	**0.38**
Cracking	0.38	0.08	0.54	0.06	0.10	0.05	0.11	0.04	0.06	0.05	0.08	0.37	0.12	0.08	0.10	0.08	0.11	0.06	0.15	0.07	**0.13**
Corrosion of R/C	0.13	0.11	0.14	0.56	0.16	0.43	0.18	0.53	0.56	0.08	0.11	0.37	0.14	0.08	0.16	0.08	0.11	0.52	0.15	0.12	**0.24**
Pop-outs	0.05	0.18	0.14	0.06	0.16	0.24	0.08	0.06	0.06	0.57	0.18	0.04	0.25	0.38	0.16	0.18	0.11	0.05	0.15	0.37	**0.17**
Scaling	0.05	0.11	0.03	0.06	0.10	0.04	0.08	0.05	0.06	0.08	0.11	0.03	0.14	0.08	0.10	0.11	0.11	0.06	0.09	0.07	**0.08**
Cronbach's Alpha	0.92																				

Soffit Defects	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	AVG
Delamination/ Spalling	0.28	0.53	0.40	0.57	0.55	0.38	0.52	0.17	0.53	0.52	0.53	0.13	0.35	0.16	0.53	0.56	0.57	0.29	0.27	0.52	**0.42**
Cracking	0.28	0.08	0.40	0.11	0.08	0.13	0.10	0.03	0.11	0.10	0.08	0.36	0.35	0.38	0.08	0.11	0.08	0.29	0.05	0.10	**0.17**
Corrosion of R/C	0.28	0.11	0.06	0.11	0.11	0.38	0.17	0.38	0.18	0.10	0.11	0.36	0.07	0.38	0.11	0.11	0.11	0.29	0.55	0.10	**0.20**
Wet Areas	0.09	0.18	0.06	0.08	0.18	0.05	0.10	0.38	0.08	0.10	0.18	0.13	0.12	0.03	0.18	0.11	0.11	0.03	0.09	0.17	**0.12**
Scaling	0.06	0.11	0.08	0.11	0.08	0.05	0.10	0.04	0.11	0.17	0.11	0.03	0.12	0.03	0.11	0.11	0.11	0.10	0.04	0.10	**0.09**
Cronbach's Alpha	0.95																				
Drainage System Defects	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	AVG
Pipe Breakage	0.50	0.83	0.83	0.75	0.75	0.50	0.75	0.75	0.75	0.50	0.83	0.83	0.83	0.25	0.83	0.75	0.75	0.50	0.88	0.83	**0.71**
Loosening/ Deterioration	0.50	0.17	0.17	0.25	0.25	0.50	0.25	0.25	0.25	0.50	0.17	0.17	0.17	0.75	0.17	0.25	0.25	0.50	0.13	0.17	**0.29**
Cronbach's Alpha	0.97																				

Table 24: Relative Weights of Beams/MLE Defects Obtained From Collected Surveys

Beams/MLE Defects	Respondents																				
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	AVG
Delamination/ Spalling	0.38	0.53	0.14	0.19	0.50	0.15	0.20	0.35	0.19	0.28	0.53	0.18	0.45	0.12	0.53	0.56	0.56	0.34	0.53	0.53	**0.36**
Cracking	0.38	0.09	0.63	0.19	0.17	0.42	0.60	0.04	0.19	0.28	0.09	0.40	0.15	0.36	0.09	0.11	0.11	0.05	0.09	0.09	**0.23**
Corrosion of R/C	0.13	0.11	0.05	0.53	0.10	0.26	0.07	0.50	0.53	0.16	0.11	0.35	0.15	0.36	0.11	0.11	0.11	0.50	0.11	0.11	**0.22**
Pop-outs	0.05	0.18	0.14	0.04	0.17	0.15	0.07	0.07	0.04	0.21	0.18	0.04	0.15	0.12	0.18	0.11	0.11	0.04	0.18	0.18	**0.12**
Scaling	0.05	0.09	0.05	0.04	0.07	0.02	0.07	0.04	0.04	0.06	0.09	0.04	0.09	0.04	0.09	0.11	0.11	0.07	0.09	0.09	**0.07**
Cronbach's Alpha	0.91																				

Table 25: Relative Weights of Abutment Wall Defects Obtained From Collected Surveys

Abutment Walls' Defects	Respondents																				
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	**AVG**
Erosion	0.53	0.36	0.09	0.12	0.30	0.30	0.09	0.09	0.13	0.41	0.36	0.53	0.16	0.29	0.36	0.15	0.43	0.10	0.07	0.36	**0.26**
Cracking	0.11	0.05	0.64	0.31	0.04	0.30	0.09	0.03	0.35	0.05	0.05	0.11	0.47	0.29	0.05	0.05	0.14	0.10	0.33	0.05	**0.18**
Corrosion of R/C	0.11	0.12	0.09	0.31	0.11	0.06	0.27	0.52	0.19	0.08	0.12	0.18	0.16	0.06	0.12	0.15	0.14	0.38	0.07	0.12	**0.17**
Pop-outs	0.08	0.12	0.09	0.03	0.25	0.04	0.09	0.09	0.05	0.06	0.12	0.08	0.05	0.29	0.12	0.05	0.14	0.03	0.07	0.12	**0.10**
Delamination/Spalling	0.18	0.36	0.09	0.25	0.30	0.30	0.45	0.27	0.28	0.41	0.36	0.11	0.16	0.06	0.36	0.60	0.14	0.38	0.47	0.36	**0.29**
Cronbach's Alpha	0.81																				

Table 26: Relative Weights of Bearings' Defects Obtained From Collected Surveys

Bearings' Defects	Respondents																				
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	**AVG**
Bending/Cracking of anchor bolts/welds	0.56	0.19	0.28	0.19	0.19	0.36	0.27	0.17	0.19	0.52	0.19	0.23	0.41	0.64	0.19	0.08	0.53	0.30	0.23	0.19	**0.30**
Cracking	0.11	0.53	0.28	0.37	0.53	0.12	0.09	0.04	0.37	0.10	0.53	0.23	0.41	0.09	0.53	0.23	0.11	0.30	0.33	0.53	**0.28**
Deformations	0.11	0.11	0.09	0.37	0.11	0.36	0.27	0.04	0.37	0.10	0.11	0.23	0.06	0.09	0.11	0.23	0.08	0.06	0.37	0.11	**0.17**
Corrosion	0.11	0.04	0.28	0.04	0.04	0.05	0.27	0.36	0.04	0.10	0.04	0.08	0.06	0.09	0.04	0.08	0.11	0.30	0.04	0.04	**0.11**
Scouring/ Scratches	0.11	0.14	0.06	0.04	0.14	0.12	0.09	0.40	0.04	0.17	0.14	0.23	0.06	0.09	0.14	0.38	0.18	0.04	0.03	0.14	**0.14**
Cronbach's Alpha	0.85																				

Table 27: Relative Weights of Pier Elements' Defects Obtained From Collected Surveys

Pier Elements' Defects	Respondents																				
Pier column Defects	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	**AVG**
Cracking	0.28	0.14	0.68	0.17	0.11	0.57	0.11	0.04	0.17	0.26	0.14	0.15	0.48	0.09	0.14	0.10	0.48	0.14	0.10	0.14	**0.23**
Delamination/ Spalling	0.28	0.41	0.08	0.33	0.43	0.08	0.33	0.30	0.33	0.26	0.41	0.05	0.13	0.09	0.41	0.47	0.10	0.30	0.38	0.41	**0.28**
Corrosion of R/C	0.28	0.16	0.08	0.42	0.16	0.08	0.33	0.39	0.42	0.26	0.16	0.36	0.13	0.45	0.16	0.10	0.10	0.39	0.03	0.16	**0.23**
Pop-outs	0.06	0.18	0.08	0.04	0.11	0.08	0.11	0.04	0.04	0.08	0.18	0.03	0.10	0.09	0.18	0.30	0.16	0.03	0.10	0.18	**0.11**
Erosion	0.09	0.10	0.10	0.04	0.18	0.19	0.11	0.22	0.04	0.14	0.10	0.41	0.16	0.27	0.10	0.03	0.16	0.14	0.38	0.10	**0.15**
Cronbach's Alpha	0.77																				
Pier Cap Defects	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	**AVG**
Delamination/ Spalling	0.38	0.53	0.18	0.27	0.57	0.24	0.52	0.27	0.27	0.33	0.53	0.18	0.14	0.38	0.53	0.57	0.48	0.22	0.33	0.38	**0.37**
Cracking	0.38	0.09	0.56	0.09	0.10	0.45	0.10	0.04	0.09	0.11	0.09	0.40	0.55	0.08	0.09	0.08	0.10	0.07	0.21	0.08	**0.19**
Corrosion of R/C	0.13	0.11	0.18	0.55	0.11	0.24	0.17	0.56	0.55	0.11	0.11	0.35	0.14	0.08	0.11	0.11	0.10	0.59	0.05	0.08	**0.22**
Pop-outs	0.05	0.18	0.04	0.05	0.11	0.04	0.10	0.09	0.05	0.33	0.18	0.04	0.05	0.38	0.18	0.11	0.16	0.04	0.33	0.38	**0.15**
Scaling	0.05	0.09	0.04	0.04	0.10	0.03	0.10	0.04	0.04	0.11	0.09	0.04	0.14	0.08	0.09	0.11	0.16	0.07	0.07	0.08	**0.08**
Cronbach's Alpha	0.89																				

IV.4 Data Collection for Case Studies

As a proof of concept and utility, the proposed bridge assessment model is implemented to achieve detailed and informative bridge element condition ratings through data acquired from two case study bridges in Canada. The targeted data for case studies pertain to the extents and severities of detected bridge defects through careful reviewing of detailed bridge inspection/ condition survey reports. The first case study is based on information gathered from a detailed inspection in 2012 of a 4 span, pier-supported reinforced concrete bridge constructed in 1965 and located in Quebec, Canada. Summary reports of the detailed inspections/ condition surveys were provided by Quebec Ministry of Transportation (MTQ) during the course of a joint research project with Concordia University (Figure 23).

Figure 23: General Overview of Case Study 1 Bridge (Photo Courtesy of Sami Moufti 2013)

The second case study of this research models the assessment of the deck and east abutment elements of the Attwell Drive Overpass in Toronto, Ontario. The study application is based on Information about structural defects collected from a summary report of a detailed condition survey performed in 2003, and obtained from the central region office of Ontario Ministry of Transportation (MTO). Constructed in 1973, the bridge's superstructure is composed of a single span 8" reinforced concrete slab overlaid with asphalt, and resting over pre-stressed concrete girders. The span length of the bridge is approximately 24.8 m, with a total deck surface area of 1196 m^2 (including curbs, median, and the driving surface of the east and west bound lanes). A general overview photograph of the structure is shown in Figure 24.

Figure 24: Attwel Drive Overpass, Toronto (Courtesy of MTO 2003)

V. RESULTS, ANALYSIS & IMPLEMENTATION

V.1 Overview

This chapter offers a demonstration of the results and analysis of data collected through the course of this study. Particularly, the final acquired sets of hierarchical factor weights are hereby presented, in addition to an explanation of defect data modeling, and the aggregation algorithm in the HER model. Also presented in this chapter is the practical application of the proposed assessment method to model the condition of two case study bridges located in the Canadian provinces of Quebec and Ontario.

V.2 Weighted Bridge Structural Hierarchies

As mentioned in chapter III, the proposed HER bridge condition model idealizes an assessed concrete bridge into a set of components, elements, and potential defects. Generic representations of these factors were established in hierarchal formats to be incorporated in the multi-level evaluation process of the proposed model (Figures 12 - 16).

The comprehensive bridge assessment model is set to account for relative importance weights of all the assessment factors in the hierarchical breakdown structure. Calculated weights ultimately represent the bridge components', elements', and defects' relative structural importance and contribution to the general bridge condition. As it was fully demonstrated in the previous chapter, a survey questionnaire was conducted to accomplish this task. Consecutively, and for every obtained survey response, AHP calculations were performed on the pairwise comparison matrices between factors at various hierarchical levels. The reader may refer back to section II.4.1 for full details regarding the Eigen vector approach for AHP relative weights calculations. The following sections demonstrate final weight values based on the average input of 20 different bridge engineers, inspectors, and managers throughout Canada.

V.2.1 Relative Weights for Bridge Components and Elements

This part illustrates the average relative importance weights of the different bridge elements towards their respective components, and those for the bridge components with respects to the entire concrete bridge structure. The relative weight distribution of bridge components and their respective elements are shown in Table 28 as per the hierarchical distribution defined in chapter III.

Table 28: Average Relative Weights of Bridge Components and Elements

Components (Level 1)	Weights	Elements (level 2)	Sub-weights
Deck	0.27	Deck Top	0.48
		Wearing Surface	0.15
		Soffit	0.25
		Drainage System	0.12
MLE	0.37	Girders	0.74
		Floor beams	0.26
Abutments	0.15	Abutment Walls	0.49
		Wing Walls	0.14
		Bearings	0.37
Piers	0.22	Pier Columns	0.48
		Pier Caps	0.30
		Bearings	0.22

As can be noticed from Table 28, the main longitudinal elements (MLE) attained the highest relative weight (0.37) among other bridge components. Bridge Decks, Piers and abutments respectively followed in importance towards the overall bridge structural condition (Figure 25).

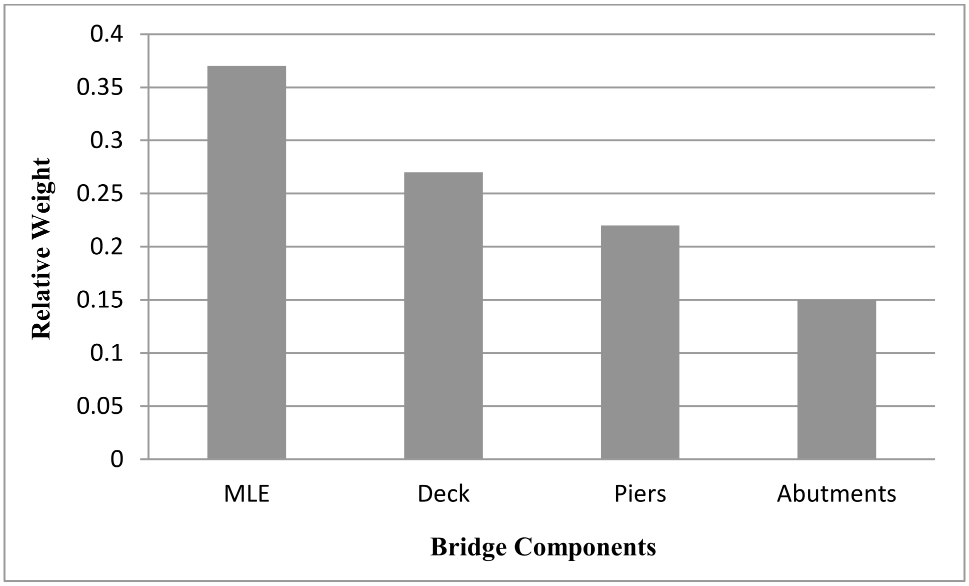


Figure 25: Relative Weights of Bridge Components

The “deck top” element attained the highest average relative weight (0.48) in competition with other bridge deck elements. Other elements that scored highest relative weights are “girders” for the “MLE” component (0.74), “abutment walls” for the “abutments” component (0.49), and pier columns for the “piers” component (0.48).

V.2.2 Relative Weights for Elements and Defects

Similar calculations were extended to attain average values of the survey generated relative importance weights of element-specific sets of potential defects. The determined defects’ relative weights numerically quantify the average judgment of the panel of experts on the partial contribution of those defects on the structural integrity of their respective bridge elements. Table 29 shows average relative weights for the decomposition of a bridge deck, as an example. It can be noticed that weights of defects belonging to a particular element sum up to 1. Please refer to section IV.3.6 for full information on the relative weights of all defects considered in this research.

Table 29: Average Relative Weights of Bridge Deck Elements and Defects

Deck Elements (level 2)	Weights	Deck Defects (level 3)	Sub-weights
Wearing Surface	0.15	-Potholes	0.41
		-Cracking	0.14
		-Rutting	0.12
		-Rippling	0.10
		-Loss of Bond	0.24
Deck Top	0.48	-Delamination/Spalling	0.38
		-Cracking	0.13
		-Corrosion of R/C	0.24
		-Pop-outs	0.17
		-Scaling	0.08
Deck Soffit	0.25	-Delamination/Spalling	0.42
		-Cracking	0.17
		-Corrosion of R/C	0.20
		-Wet Areas	0.12
		-Scaling	0.09
Drainage System	0.12	-Pipe Breakage	0.71
		-Loosening/Deterioration of Components or connections or fasteners	0.29

As can be noted from Table 29, defect types scored different relative importance weights toward their respective bridge deck elements. "Potholes", for instance, attained the highest relative weight (0.41) in contribution to the structural integrity and condition of the "wearing surface element" according to the average judgment of the surveyed panel of experts. Other defect types that scored highest weights were determined to be “delamination/spalling” for the “deck top” element (0.38), “delamination/spalling” again for the “deck soffit” element (0.42), and “pipe breakage” for the “drainage system” element of a bridge deck (0.71).

It should be noted that the relative weights presented here reflect a general sense of how important a defect type/category is in comparison to other defect types/categories within a bridge element, according to the surveyed panel of bridge experts. It is not the intention here to account for different cases or forms of a particular defect, which would rather be reflected in an inspector’s judgment on that defect’s severity and extent.

V.3 Execution of the HER Defect-Based Assessment Aggregation

As presented in chapter III, This study proposes a Hierarchical Evidential Reasoning (HER) approach as a comprehensive defect-based bridge condition assessment platform. This is facilitated through multi-level evaluation of defects, elements, and components in a concrete bridge structure.

To obtain an overall condition rating for the bridge structure, the structural breakdown represented by the previously shown bridge hierarchies will be utilized. The assessment starts on a defect level by mapping every defect on the proposed fuzzy grading scheme (Figure 17) to determine fuzzy grade assessments of the detected defects. Fuzzy defect grades will contribute to the rating of the elements on which they are detected. Proceeding is the aggregation of fuzzy elements’ assessments to determine their respective bridge component's rating. Eventually, the model aggregates bridge components' fuzzy assessments towards obtaining a fuzzy overall bridge condition.

To demonstrate the implementation procedure of aggregating different assessments via the evidential reasoning approach, the aggregation of only two defect assessments is explained here for simplicity. Say that an inspector has detected cracking (D_1) of medium severity over 18% of a bridge beam under assessment, and very severe pop-outs (D_2) affecting 5% of that same beam. Using the fuzzy grade membership charts presented in Figure 17 (B), it follows that:

$S(Cracking\ "D_1"\ ) = \{(A, 0), (B, 0.79), (C, 0.21), (D, 0)\}$

$S(Pop\ outs\ "D_2") = \{(A, 0.2), (B, 0.8), (C, 0), (D, 0)\}$

The Basic Probability Assignments (BPA) in the evaluation grades can therefore be calculated by factoring in the relative weights of cracking and pop-outs (w_1= 0.23 and , w_1= 0.12, respectively) according to equations [21] to [26], resulting in:

$m_{A,1} = 0 \quad m_{B,1} = 0.18 \quad m_{C,1} = 0.05 \quad m_{D,1} = 0 \quad \bar{m}_{H,1} = 0.77 \quad \tilde{m}_{H,1} = 0$

$m_{A,2} = 0.02 \quad m_{B,2} = 0.10 \quad m_{C,2} = 0 \quad m_{D,2} = 0 \quad \bar{m}_{H,2} = 0.88 \quad \tilde{m}_{H,2} = 0$

Thus, we can now apply the recursive equations [27]-[31] to calculate the combined probability masses as follows:

$$K_{I(2)} = \left[1 - \sum_{t=1}^{4} \sum_{\substack{j=1 \\ j \neq i}}^{4} m_{t,I(1)}\, m_{j,2}\right]^{-1}$$

$= [1 - (0 + \cdots + 0 + m_{B,1}\, x\, m_{A,2} + m_{C,1}\, x\, m_{A,2} + m_{C,1}\, x\, m_{B,2} + 0 + \cdots + 0)]^{-1}$

$= [1 - (0 + \cdots + 0 + 0.18\, x\, 0.02 + 0.05\, x\, 0.02 + 0.05\, x\, 0.1 + 0 + \cdots + 0)]^{-1}$

= 1.0097

Then we have:

$m_{A,I(2)}$ = $K_{I(2)}(m_{A,1}\ m_{A,2} + m_{A,1}\ m_{H,2} + m_{H,1}\ m_{A,2})$ = 0.0155

$m_{B,I(2)}$ = $K_{I(2)}(m_{B,1}\ m_{B,2} + m_{B,1}\ m_{H,2} + m_{H,1}\ m_{B,2})$ = 0.2559

$m_{C,I(2)}$ = $K_{I(2)}(m_{C,1}\ m_{C,2} + m_{C,1}\ m_{H,2} + m_{H,1}\ m_{C,2})$ = 0.0444

$m_{D,I(2)}$ = $K_{I(2)}(m_{D,1}\ m_{D,2} + m_{D,1}\ m_{H,2} + m_{H,1}\ m_{D,2})$ = 0

$\tilde{m}_{H,I(2)} = 1.0097(0) = 0$

$\bar{m}_{H,I(i+1)} = 1.0097(0.77x0.88)$ = 0.6842

Thus, the final degrees of belief allocated to the assessment grades H_n and to the set H(ignorance) are obtained through equations [32]-[33] as follows:

$$\beta_A = \frac{m_{A,I(2)}}{1 - \bar{m}_{H,I(2)}} = 0.0491$$

$$\beta_B = \frac{m_{B,I(2)}}{1 - \bar{m}_{H,I(2)}} = 0.8103$$

$$\beta_C = \frac{m_{C,I(2)}}{1 - \bar{m}_{H,I(2)}} = 0.1406$$

$$\beta_D = \frac{m_{D,I(2)}}{1 - \bar{m}_{H,I(2)}} = 0$$

$$\beta_H = \frac{\widetilde{m}_{H,I(2)}}{1 - \bar{m}_{H,I(L)}} = 0$$

The above performed calculations resulted in a distributed assessment over the 4 grades based on two defects (available evidences). In the same manner, the bridge element assessment can be updated and refined as more evidences (defect assessments) are collected. The recursive aggregation algorithm becomes notably more complex with greater number of available defect assessments.

In order to handle the recursive HER recursive assessments aggregation for the proposed bridge condition assessment model, a student version of the IDS© software was utilized (Xu and Yang 2005). IDS© (Intelligent Decision Systems) is a windows-based software platform that is developed to model and handle MADM problems through implementing the HER algorithm. Figure 26 illustrates the modeling of concrete bridge assessment using IDS©. The IDS based tool is configured to account for all the assessment features proposed in this research as demonstrated in chapter III.

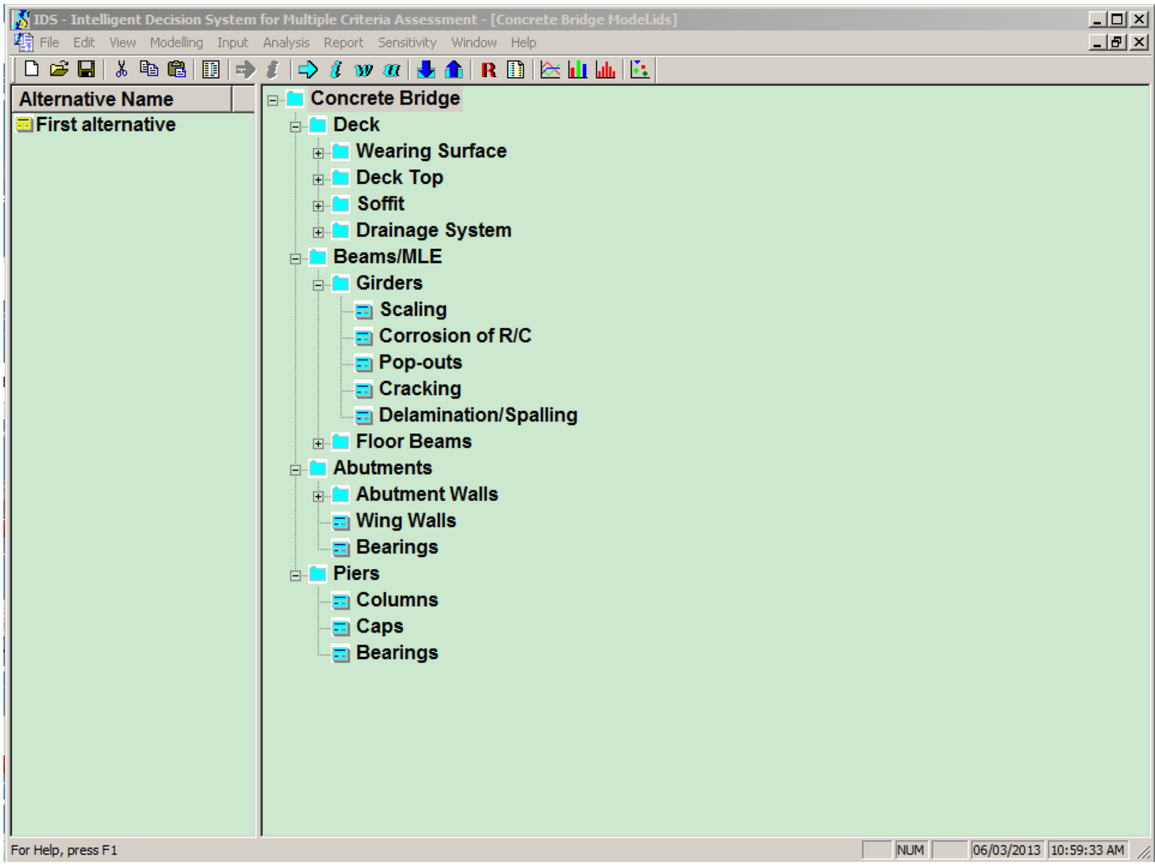


Figure 26: Detailed Bridge Assessment Application via IDS©

V.4 Defuzzification

As the HER model produces fuzzy assessment outputs (which may have memberships to several grades); a proposed final touch is to covert those outputs into a simple index value that would be easy to grasp in an intuitive sense. To this end, several methods concerning the translation of fuzzy condition ratings to crisp values were found in literature. Yager (1980) proposed deffuzification using the centroidal method where the deffuzified crisp value needs to be re-mapped on the adjacent fuzzy memberships. Tee et al. (1988) suggested the use of fuzzy weighted average computation for bridge condition assessment. However, the same method was criticized for its probability of yielding a non-convex fuzzy set (Abu Dabous and Alkass 2010). Deffuzification can also be achieved by selecting the fuzzy set that has retained the highest membership value as follows (Cheng and Lin 2002):

$$R = max\ (\mu\mathrm{A}, \mu\mathrm{B}, \mu\mathrm{C}, \mu\mathrm{D}) \quad [35]$$

Alternatively, the deffuzified crisp grade can be found by assigning weights/coefficients to the membership values of the condition vector (Lu et al. 1999):

$$R = [C1 \quad \cdots \quad Ci \quad \cdots \quad Cn] \,.\, [\mu(1) \quad \cdots \quad \mu(i) \quad \cdots \quad \mu(n)\,]^T \quad [36]$$

Where:

R : Crip Rating Value

Ci : User-Defined Coefficients corresponding to the membership value μ(i) ,

$\mathrm{i} \in (1,2, \dots, \mathrm{n})$

n : Number of membership functions

For the purpose of making the final crisp condition rating value compatible with the 4-grade rating scale used in this study, the coefficient vector is set to **C = [4 3 2 1]** . Using this method, an assessment with 100% membership to grade A, i.e. $S = \{(A, 1), (B, 0), (C, 0), (D, 0)\}$, will have a final crisp grade equal to “4”. Similarly, an assessment with 100% membership to grade D, i.e. $S = \{(A, 0), (B, 0), (C, 0), (D, 1)\}$, will have a final crisp grade of “1”. This method is chosen for this study to transform an assessment of distributed nature to a crisp value between 1-4, with “4” being the highest and “1” being the lowest.

V.5 Case Study I

V.5.1 Overview

In this section, the first case study is presented as a proof of concept and illustration of the presented HER approach for a comprehensive defect based bridge assessment. Data pertaining to the severities and extents of detected defects were collected from a 2012 inspection report of the bridge deck component of 4 span, pier-supported reinforced concrete bridge located in Quebec, Canada. The illustrated bridge was constructed in 1965 and has a total deck area of 827 m^2.

V.5.2 Data Analysis

Data shown in Table 30 depict measurements of defects that were spotted on various elements bridge deck. The presented details were interpreted from notes or comments left by bridge inspectors, reporting on the severity (refer to Table 19) and extent (i.e. proportion of the affected deck area) of defects. Defects are essentially detected by means of visual inspection or non-destructive evaluation. Similar information can commonly be retrieved from in-depth bridge inspection reports/condition surveys, where visual inspection is supplemented by non-destructive evaluation methods. Note that while deck-top and soffit are considered primary elements, the wearing surface and drainage system are secondary. The level of reinforcement corrosion was measured using the half-Cell Potential test (HCP), where areas of different corrosion activity levels were identified. Other applied NDE tests included hammer sounding to delineate areas of subsurface delamination.

Table 30: Defects Detected on the Studied Bridge Deck

Elements	Defect	Observation (Severity & Extent)
Wearing Surface (E_1)	Potholes (D_{11})	**V.severe** (30%)
	Cracking (D_{12})	**Severe** (65%)
	Rutting (D_{13})	**None**
	Rippling (D_{14})	**None**
	Loss of Bond (D_{15})	**Exists**
Deck Top (E_2)	Delamination/Spalling (D_{21})	**Exists: severe**
	Cracking (D_{22})	**Medium** (33%)
	Corrosion of R/C (D_{23})	**Light** (62%)**, Medium** (33.6%), **Severe** (3.7%), **V.severe** (0.14%)
	Pop-outs (D_{24})	**Severe** (30%)
	Scaling (D_{25})	**Medium** (20%)
Soffit (E_3)	Delamination/ Spalling (D_{31})	**Exists**
	Cracking (D_{32})	**None**
	Corrosion of R/C (D_{33})	**Light** (70%)**, Medium** (30%)
	Wet Areas (D_{34})	**None**
	Scaling (D_{35})	**V.Severe** (10%)
Drainage System (E_4)	Pipe Breakage (D_{41})	**None**
	Loosening/ Deterioration of Components or connections or fasteners (D_{42})	**None**

While most of the presented defects where detected and measured by visual inspection, subsurface defects such as delamination and loss of bond couldn't be measured to exact extents by inspectors, leaving only comments about their existence and approximate severity (if available). The asphalt wearing surface was in a generally bad condition, with severe cracks covering about 65% of the total surface area. Potholes are noticeable at several locations too (Figure 27). The deck top element suffered from cracking and scaling of medium severity, in addition to areas of varying reinforcement corrosion levels. Very severe scaling is noticeable on a localized area of soffit, with an evident existence of light delamination (Figure 28).

Figure 27: Deterioration of Deck Wearing Surface (Photo Courtesy of Sami Moufti 2013)

Figure 28: Localized Scaling Area on the Deck Soffit (Photo Courtesy of Sami Moufti 2013)

V.5.3 Assessment Calculations and Output

The obtained defect measurements are fed to the proposed HER assessment model after passing a fuzzy treatment process to attain uniform fuzzy grades according to the unified scheme presented in section III.6. This is essentially done through the fuzzification of every measured defect through their severity-relevant fuzzy membership functions presented earlier (see Figure 17). Resulting will be degrees of belief, assigned through every defect, to the 4 evaluation grades (A, B, C, D) defined in equation [17] (Table 31).

Basic probability assignments (BPAs) are generated, using equations [21]-[26], by scaling down the degrees of belief in evaluation grades, determined from defects' evaluations, to the relative importance weights (w_i) of defects (from Table 29). Next, BPAs (or m-values) are aggregated utilizing the recursive HER process towards their respective elements (equations [27]-[31]). For instance, the wearing surface (E_1) element was evaluated on the basis of aggregating the assessments of potholes D_{11}, cracking D_{12}, rutting D_{13}, rippling D_{14}, and loss of bond D_{15}. Consecutively, the obtained evaluations of different deck elements are in turn aggregated towards obtaining a comprehensive assessment of the bridge deck in question.

Table 31: Fuzzy Defect Grading Scheme for Case study 1

Wi	E_i	w_{ij}	D_{ij}	observation (Severity & Extent)	Evaluation: Light Defects				Evaluation: Medium Defects				Evaluation: Severe Defects				Evaluation: V.Severe Defects			
					A	B	C	D	A	B	C	D	A	B	C	D	A	B	C	D
0.15	Wearing Surface (E_1)	0.41	Potholes (D_{11})	**V.severe** (30%)	n/a				n/a				n/a				0.00	0.00	0.94	0.06
		0.14	Cracking (D_{12})	**Severe** (65%)	n/a				n/a				0.00	0.00	0.13	0.87	n/a			
		0.12	Rutting (D_{13})	**None**	1	0	0	0	1	0	0	0	1	0	0	0	1	0	0	0
		0.1	Rippling (D_{14})	**None**	1	0	0	0	1	0	0	0	1	0	0	0	1	0	0	0
		0.24	Loss of Bond (D_{15})	**Exists**									0.00	0.00	1.00	0.00				
0.48	Deck Top (E_2)	0.38	Delamination /Spalling (D_{21})	**Exists: severe**	n/a				n/a				0.00	0.00	1.00	0.00	n/a			
		0.13	Cracking (D_{22})	**Medium** (33%)	n/a				0.00	0.48	0.52	0.00	n/a				n/a			
		0.24	Corrosion of R/C (D_{23})	**Light** (62%), **Medium** (33.6%), **Severe** (3.7%), **V.severe** (0.14%)	0.00	1.00	0.00	0.00	0.00	0.47	0.53	0.00	0.47	0.53	0.00	0.00	0.98	0.02	0.00	0.00
		0.17	Pop-outs (D_{24})	**Severe** (30%)	n/a				n/a				0.00	0.00	0.67	0.33	n/a			
		0.08	Scaling (D_{25})	**Medium** (20%)	n/a				0.00	0.75	0.25	0.00	n/a				n/a			
0.25	Soffit (E_3)	0.42	Delamination /Spalling (D_{31})	**Exists**									0.00	0.00	1.00	0.00				
		0.17	Cracking (D_{32})	**None**	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00
		0.2	Corrosion of R/C (D_{33})	**Light** (70%), **Medium** (30%)	0.00	1.00	0.00	0.00	0.00	0.54	0.46	0.00	n/a				n/a			
		0.12	Wet Areas (D_{34})	**None**	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00
		0.09	Scaling (D_{35})	**Severe** (10%)	n/a				n/a				0.00	0.64	0.36	0.00				
0.12	Drainage System (E_4)	0.71	Pipe Breakage (D_{41})	**None**	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	1	0	0	0	1.00	0.00	0.00	0.00
		0.29	Loosening/ Deterioration of Components or connections or fasteners (D_{42})	**None**	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00	1.00	0.00	0.00	0.00

Unknown evaluation input (of subsurface defects)

In order to handle the recursive HER recursive assessments aggregation for all the assessment factors (defects and elements) of the bridge deck in case study 1, a custom HER model of the bridge deck assessment was constructed using the IDS© software (Xu and Yang 2005). The hereby developed IDS-based tool is configured to account for all features of the proposed HER assessment algorithm as explained earlier (section V.3).

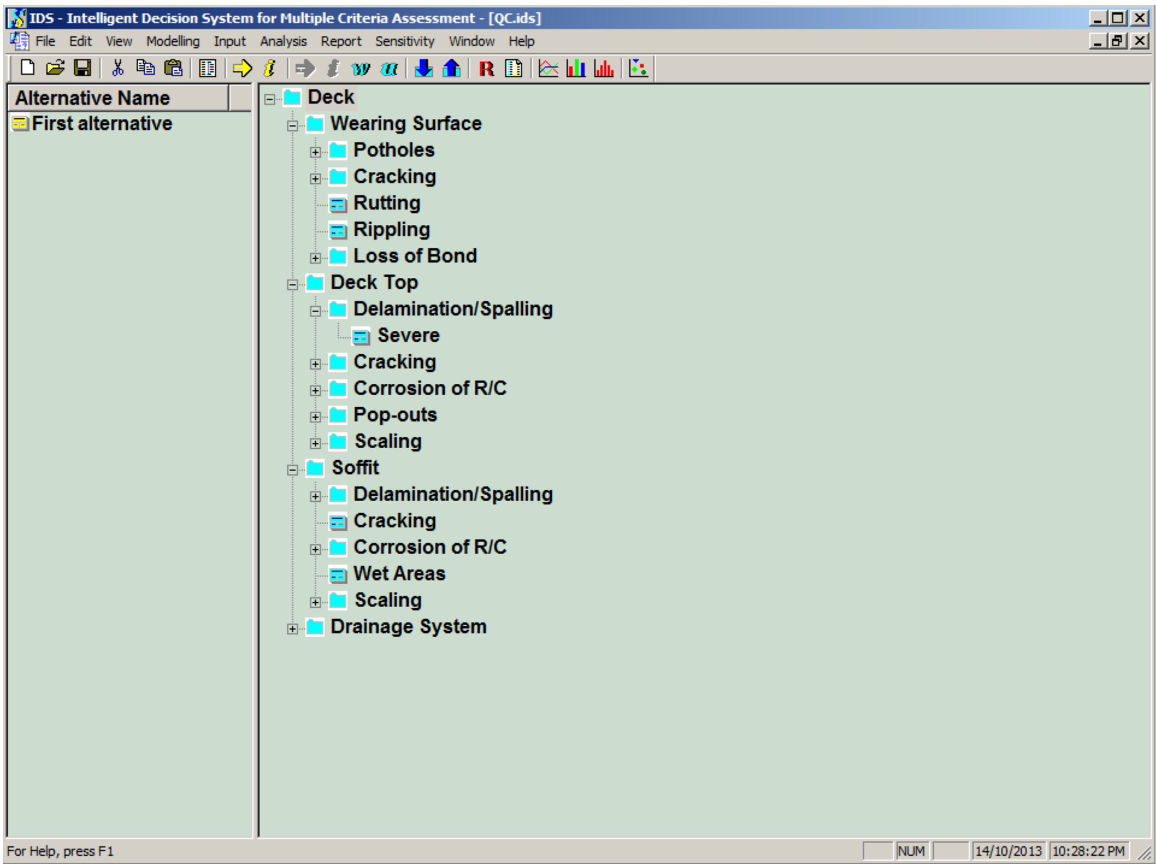


Figure 29: Detailed Bridge Deck Assessment Application

V.5.3.1 Assessment Output

Resulting condition vectors for the deck elements are as follows:

$\boldsymbol{S(Wearing\ Surface)} =$

$\{(A, 17.93\%), (B, 00.00\%), (C, 57.77\%), (D, 12.90\%), (Unknown, 11.40\%)\}$

$\boldsymbol{S(Decktop)} =$

$\{(A, 06.83\%), (B, 20.89\%), (C, 68.07\%), (D, 04.21\%), (Unknown, 00.00\%)\}$

$\boldsymbol{S(Soffit)} =$

$\{(A, 29.67\%), (B, 21.24\%), (C, 23.79\%), (D, 00.00\%), (Unknown, 25.30\%)\}$

$\boldsymbol{S(Drainage\ System)} =$

$\{(A, 100.00\%), (B, 00.00\%), (C, 00.00\%), (D, 00.00\%), (Unknown, 00.00\%)\}$

And the final aggregated assessment output for the bridge deck is:

$\boldsymbol{S(Deck)} =$

$\{(A, 21.00\%), (B, 15.83\%), (C, 53.92\%), (D, 03.63\%), (Unknown, 05.61\%)\}$

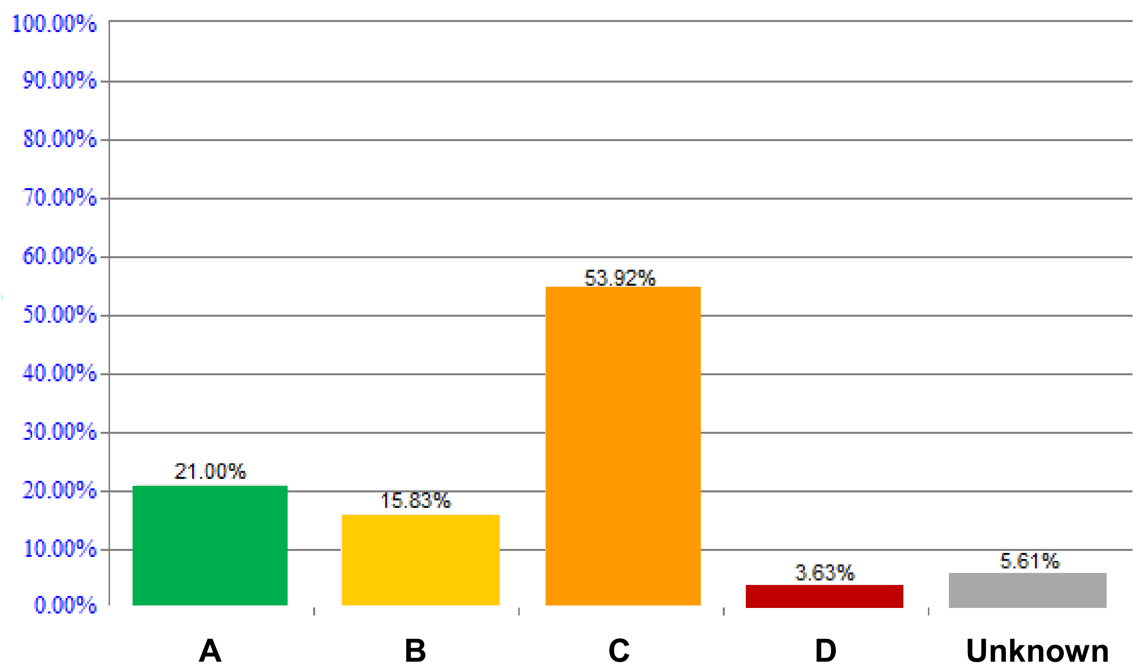


Figure 30: Assessment Output for Deck in Case Study 1

As can be seen from the condition assessment outputs, grade C or “poor” attained the biggest share of belief percentage as suggested by the collected bodies of evidence (defect measurements). This is believed to be sourced from the fact that most of the detected defects were of medium to high severity, which should be flagging attention for proper maintenance actions. Only, 5.61% of overall belief could not be assigned to any of the grades due to incomplete information available on the severity of some subsurface defects (loss of bond in the wearing surface and delamination in the soffit). Using equation [36], a final crisp assessment index can be calculated to a value of “2.43” out of 4.

In general, The proposed HER bridge condition assessment model provides a platform for continuous updates on a bridge’s condition as more data on its defects become available; Thus, providing an objective and informative assessment tool.

V.6 Case Study II

V.6.1 Overview

The second case study features the application of the proposed HER condition assessment model to the Attwell Drive Overpass in Toronto, Ontario. The bridge’s superstructure is composed of a single span 8” reinforced concrete deck (length ~ 24.8 m) overlaid with asphalt, and resting over pre-stressed concrete girders. The bridge was constructed in 1963; with a total deck surface area of 1196 m^2 (including curbs, median, and the driving surface of the east and west bound lanes). The developed evaluation method is conducted on the asphalt covered concrete bridge deck, as well as the bridge's east abutment. Assessment of both components will be presented in the following sections.

V.6.2 Data Analysis

Information about structural defects were collected from a summary report of a detailed condition survey performed in 2003, and acquired from the central region office of Ontario Ministry of Transportation (MTO). The assessment of this bridge systematically relies on details, measures, remarks and digital photos retrieved from the provided report. In general, the condition survey was described to have been conducted in accordance to guidelines and procedures defined in MTO's Structural Rehabilitation Manual. It particularly encompassed observing, measuring, and recording the severity and extents of surface deterioration, delamination, corrosion potentials, physical testing of concrete cores, etc.

As to the used equipment, inspectors relied on the use of auditory methods to detect subsurface delamination in concrete elements. This included the chain drag method for all the horizontal concrete surfaces, in addition to hammer sounding for vertical elements. Half Cell Potential test was also implemented for measuring the corrosion level of steel reinforcement.

V.6.2.1 Bridge Deck

Data shown in Table 32 depict the reported measurements of defects that were spotted on various elements of the 827 m^2 bridge deck. The presented details were interpreted from a summary report of detailed condition survey of the bridge deck.

Table 32: Defects Detected on the Bridge Deck of Case Study 2 Bridge

Elements	Defect	Observation (Severity & Extent)
Wearing Surface (E_1)	Potholes (D_{11})	None
	Cracking (D_{12})	**Medium** (15.8%), **Severe** (13.5%)
	Rutting (D_{13})	None
	Rippling (D_{14})	None
	Loss of Bond (D_{15})	None
Deck Top (E_2)	Delamination/ Spalling (D_{21})	None
	Cracking (D_{22})	None
	Corrosion of R/C (D_{23})	**Light** (91.6%)**, Medium** (7.9%), **Severe** (0.5%)
	Pop-outs (D_{24})	None
	Scaling (D_{25})	None
Soffit (E_3)	Delamination/ Spalling (D_{31})	**Exists**
	Cracking (D_{32})	**Medium** (0.08%)
	Corrosion of R/C (D_{33})	None
	Wet Areas (D_{34})	None
	Scaling (D_{35})	None
Drainage System (E_4)	Pipe Breakage (D_{41})	None
	Loosening/ Deterioration of Components or connections or fasteners (D_{42})	None

In practice, surface defects described in the table above were mainly detected by visual inspection and close-up observations by inspectors. However, subsurface defects were mostly measured with the aid of NDE equipment. This included performing the Half Cell Potential test on the bridge deck to measure the corrosion of steel reinforcement bars, in addition to the chain drag for the detection and approximation of potential delamination areas.

The deck inspection summary sheets reported that the wearing surface element was in about “fair” condition, with the only reported/observed defects being longitudinal and transverse surface cracks (Figure 31). The concrete deck top element was reported to be in “good” condition. Corrosion potential values obtained from half-cell tests ranged from –0.02 to –0.28 V on the deck surface area, with an average corrosion potential of –0.07 V. This indicates a low overall corrosion potential, with the entire subject area being less negative than -0.35 V. The soffit element of the bridge deck was noted to be in “fair” condition with some localized spalling (12.4 m^2), delamination (7.2 m^2) and cracking (0.9 m^2), predominantly in areas adjacent to the center gap (Figure 32).

Figure 31: Surface Cracks on the Asphalt Wearing Surface (Courtesy of MTO 2003)

Figure 32: Typical Condition of Deck Soffit (Courtesy of MTO 2003)

V.6.2.2 Bridge East Abutment

The east abutment was reported to generally be in fair condition; however, it exhibited notable cracking, localized delamination and spalling, in addition to some areas of surface staining (Figure 33). The available data on detailed condition assessment and defect recordings were only reported for the main abutment wall (total surface area= 244.5 m^2); thus, the modeled evaluation carried out here will be limited to this particular element of the “abutments” component (Table 33).

Table 33: Reported Defects on the Abutment wall of Case Study 2 Bridge

Element	Defect	Observation (Severity & Extent)
Abutment wall (E_1)	Erosion (D_{11})	**None**
	Cracking (D_{12})	**Medium** (10.56%)
	Corrosion of R/C (D_{13})	**None**
	Pop-outs (D_{14})	**None**
	Delamination/Spalling (D_{15})	**Exists: medium**

Medium cracks were reported to cover 103.3 m (equivalent to 103.3/4 = 25.83 m^2) of the abutment’s wall surface area. A localized medium delamination/

spalling area was also noted. Those inputs were graded according to the suggested fuzzy grading scheme (Figure 17), and subsequently inputted into a structured HER assessment model for the bridge abutment.

Figure 33: East Abutment Wall of Case Study 2 Bridge (Courtesy of MTO 2003)

V.6.3 Assessment Calculations and Output

In a similar way of the assessment model application to the first case study, the acquired defect measurements were fed to the HER assessment tool through their severity-relevant fuzzy membership functions (Figure 17(b)). This will render the assignment of degrees of belief to the 4 evaluation grades through every inputted defect. Basic probability assignments (BPAs) are then generated by scaling down the degrees of belief in evaluation grades to the relative importance weights (w_i) of defects. BPAs (or m-values) are in turn aggregated utilizing the recursive HER process towards their respective bridge deck or abutment elements.

The resulting condition vectors of the deck elements are as follows:

- $\boldsymbol{S(Wearing\ Surface)} =$

  $\{(A, 91.94\%), (B, 08.06\%), (C, 00.00\%), (D, 00.00\%), (Unknown, 00.00\%)\}$

- $\boldsymbol{S(Decktop)} =$

  $\{(A, 88.33\%), (B, 06.34\%), (C, 05.34\%), (D, 00.00\%), (Unknown, 00.00\%)\}$

- $\boldsymbol{S(Soffit)} =$

  $\{(A, 62.16\%), (B, 01.50\%), (C, 13.29\%), (D, 00.00\%), (Unknown, 23.05\%)\}$

- $\boldsymbol{S(Drainage\ System)} =$

  $\{(A, 100.0\%), (B, 00.00\%), (C, 00.00\%), (D, 00.00\%), (Unknown, 00.00\%)\}$

With the final aggregated assessment output for the **bridge deck** being:

- $\boldsymbol{S(Deck)} =$

  $\{(A, 88.47\%), (B, 03.74\%), (C, 04.40\%), (D, 00.00\%), (Unknown, 03.40\%)\}$

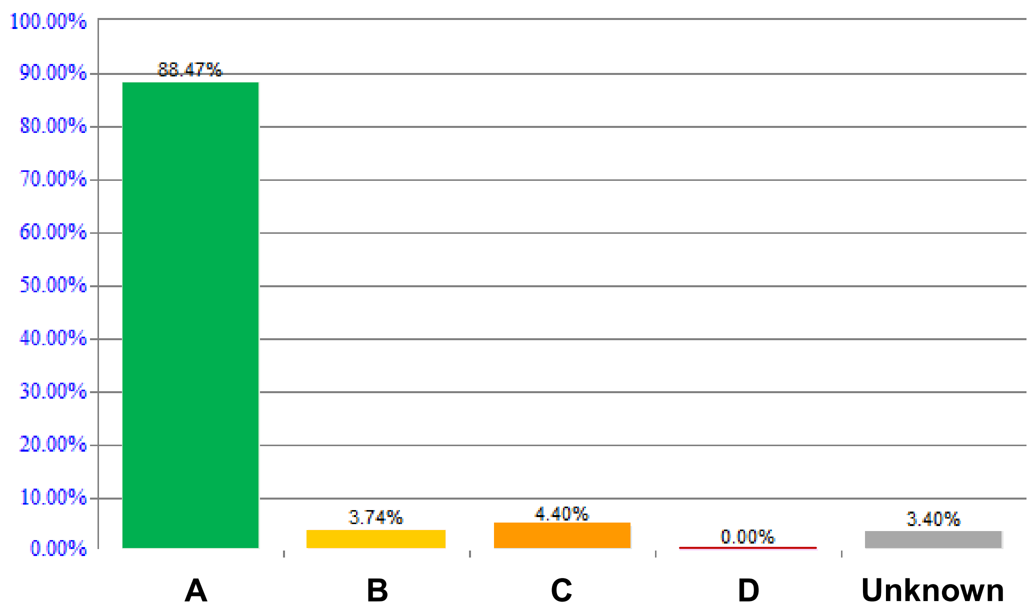


Figure 34: Assessment Output for Deck in Case Study 2

The good condition of the bridge deck (in general), and the little presence of defects was well reflected in the assessment output. As can be seen from Figure 34, grade A attained the biggest elect of belief. This is due to the nonexistence of most of the defects, which would render full belief in grade A. The presence of some defects on the deck top, wearing surface, and soffit has contributed towards distributing some percentages to grades B, C. 3.40% of the overall belief could not be assigned to any of the grades due to incomplete information available on the severity of the subsurface defect depicted in the localized delamination/spalling at the soffit. A final crisp assessment index can be calculated according to equation [36] to a value of “3.74” out of 4.

The **east abutment wall** of the same bridge was graded to the following:

- $\boldsymbol{S(Erosion)} =$

  $\{(A, 100.00\%), (B, 00.00\%), (C, 00.00\%), (D, 00.00\%), (Unknown, 00.00\%)\}$
- $\boldsymbol{S(Cracking)} =$

  $\{(A, 00.00\%), (B, 95.00\%), (C, 05.00\%), (D, 00.00\%), (Unknown, 00.00\%)\}$
- $\boldsymbol{S(Corrosion\ of\ R/C)} =$

  $\{(A, 100.00\%), (B, 00.00\%), (C, 00.00\%), (D, 00.00\%), (Unknown, 00.00\%)\}$
- $\boldsymbol{S(Pop - outs)} =$

  $\{(A, 100.0\%), (B, 00.00\%), (C, 00.00\%), (D, 00.00\%), (Unknown, 00.00\%)\}$
- $\boldsymbol{S(Delamination/Spalling)} =$

  $\{(A, 00.00\%), (B, 00.00\%), (C, 100.00\%), (D, 00.00\%), (Unknown, 00.00\%)\}$

With the final aggregated assessment output for the abutment wall being:

- $\boldsymbol{S(Abutment\ wall)} =$

  $\{(A, 56.13\%), (B, 14.47\%), (C, 29.41\%), (D, 00.00\%), (Unknown, 00.00\%)\}$

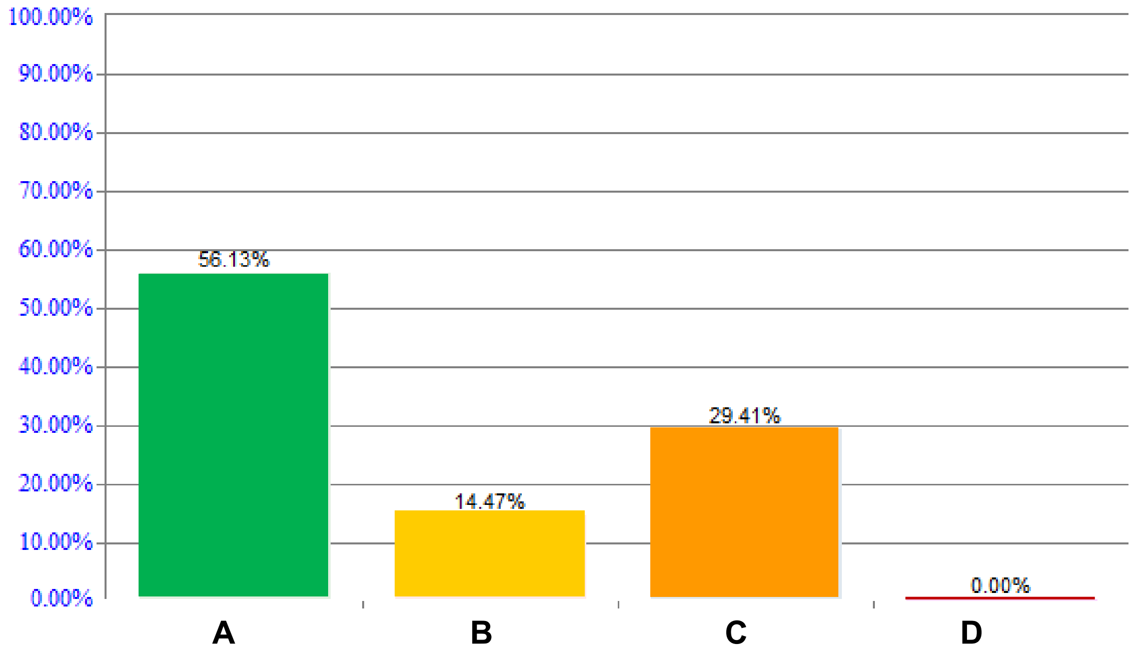


Figure 35: Assessment Output for East Abutment Wall in Case Study 2

While grade A attained the highest degree of belief, a considerable share (29.41%) of the total assessment was assigned to grade C. This is, in fact, resulting from the collected wall defect measurements as reported earlier. The defect-based output should bring attention for proper retrofitting or maintenance actions addressing repair of the defected wall regions. A rating index value of “3.27” out of 4 is calculated for this component, as per equation [36].

V.7 Summary and Comparison

While inspection reports tend to rate bridge elements based on the observation and subjective judgment of bridge inspector(s), the proposed model outputs were generated on the basis of objective and weighted aggregation of defect assessments. The obtained bridge element assessments reflect a data oriented and computational approach; in comparison to the mere approximations and judgments that are commonly practiced. Hence, there can be some variations between the suggested assessment outputs and the existing ones. It is argued, however, that the proposed model works in a more objective and data attentive manner, signifying highly credible and educated assessments of bridge elements.

As described in section V.4, the distributed assessment evaluations that are obtained for bridge elements and components can be consolidated in a single crisp value index. This value will show a simple crisp grade value out of 4. To this end, table shows the determined crisp evaluations of those elements and components that were the subject of the above explained case studies.

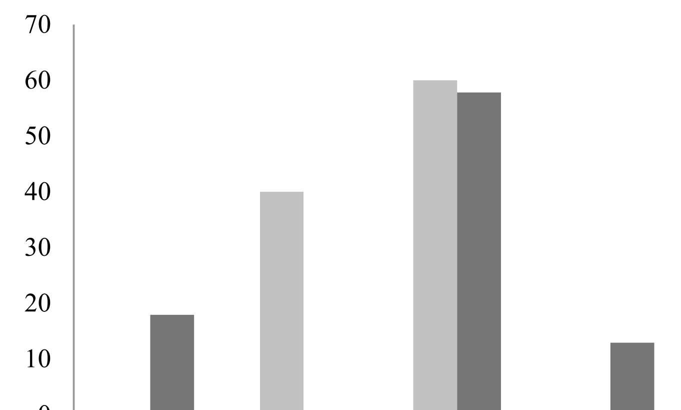
Table 34: Crisp Value Evaluation for Case Study 1 Bridge

Element	
Wearing surface	2.00
Deck Top	2.30
Soffit	2.30
Derainage System	4.00
Component	
DECK	2.43

Table 35: Crisp Value Evaluation for Case Study 2 Bridge

Element	
Wearing surface	3.92
Deck Top	3.83
Soffit	2.80
Derainage System	4.00
Component	
DECK	3.74
ABUTMENT WALL	3.27

Figure 36 summarizes the assessment output of the “wearing surface” element of the bridge in case study 1 (section V.4). The bridge inspection report suggests that the evaluation of wearing surface is estimated to fall in grades B and C, with 40% of the element in the former and 60% in the latter grade. While the assessment output from the HER assessment tool showed a wider grade distribution, the biggest portion of belief percentage (57.8%) was allocated to grade C. With only (3.8%) percentage difference, both results show close support to the preposition of evaluating the element to grade C.

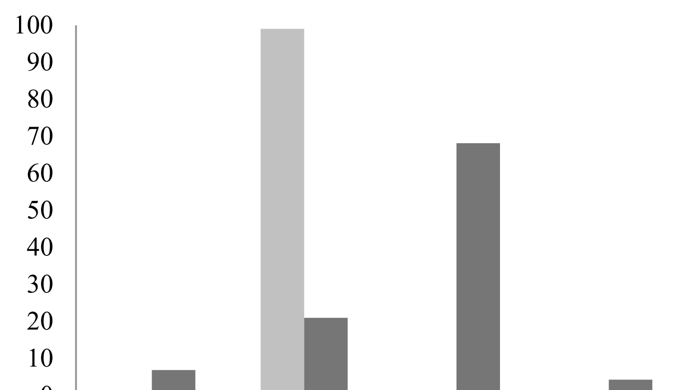


	A	B	C	D
Inspection Report	0	40	60	0
HER model	17.9	0	57.8	12.9

Figure 36: Distributed Grade Assessment of the Wearing Surface Element in Case study 1 Bridge

For the “deck top” element of the same case bridge, a higher difference in the resulting assessment was attained. The numerous reported defects of medium to high severity on the element rendered the highest portion (68.1%) of the assessment distribution to be assigned to grade C. However, the same element was almost fully judged to fall in grade B by the provided inspection report. In this case, the bridge inspector’s judgment was a little optimistic, given the resulting model output that suggests stronger support to a worse condition (Figure 37).

While traditional bridge inspection assessments might incorporate high uncertainty and a subjective judgmental nature, the proposed HER model benefits from an objective assessment platform that bases the evaluation on a data oriented and structured algorithm.



	A	B	C	D
Inspection Report	0	99	1	0
HER model	6.8	20.9	68.1	4.2

Figure 37: Distributed Grade Assessment of the Deck Top Element in Case study 1 Bridge

VI. CONCLUSIONS & RECOMMENDATIONS

VI.1 Summary and Conclusions

This research targeted the development of a comprehensive bridge condition assessment methodology that is able to objectively translate possible defect measurements into detailed and informative bridge element condition ratings. The presented tool can handle the fuzzy nature of detected bridge defects, as well as the varying relative importance of the different factors involved in the assessment hierarchy. Moreover, the proposed approach is proposed to assist in reducing sources of data uncertainty in the bridge inspection process; whether originating from the subjectivity involved in visual inspection, or from the partial ignorance in identifying defect-specific severities. Overall, the presented model aims at making a condition assessment sense of the detected bridge defects, which is ultimately believed to be effective in assisting educated bridge management decisions.

The following research conclusions are drawn:

- The results obtained through execution of the proposed assessment model warrant excellent consistency and uniformity.
- Through application to case studies, the proposed framework exhibits to be effective in securing objective, well-informed condition indices of the assessed bridge components.

- The proposed methodology places good emphasis on sufficiently exhaustive sets of assessment factors to generally model concrete bridges.
- Application of the suggested HER assessment framework can be re-configured based on the end-user needs. This can enable decision makers gain better insight into customizing the bridge breakdown structure, region-specific types of bridge defects, or the level of relative importance of the assessment factors.

VI.2 Research Contributions

The proposed bridge assessment framework is expected to augment to a great extent the existing practice in bridge condition rating. It is able to aggregate objective data concerning direct bridge defect measurement, and translate them into an informative condition index. The suggested model can either be applied to assess bridge components or elements of interest, or it can be geared towards developing an overall condition index for an entire bridge.

While bridge engineers, inspectors, and decision makers are calling for a further detailed and comprehensive bridge assessment, it is strongly believed that the hereby proposed assessment framework is able to meet those expectations and greatly contribute to the advancement of the state of the art as well as the practice in this vital area. The developed methodology is projected to introduce a great deal of objectivity in an otherwise subjective area of infrastructure management.

Key contributions of the presented research can be categorized as follows:

- A comprehensive up-to-date review of the current practices in the USA and Canada in the areas of bridge inspection techniques, bridge condition rating, and bridge management systems (BMS).
- Well-developed expert-based values of relative structural importance weights for the various bridge components, elements, and defect categories.
- The establishment of an innovative fuzzy grading methodology that treats the inherent subjectivity of structural defect measurements, and secures uniformity in mapping all the detected defects on the widely used 4-level grading system.
- The provision of a comprehensive defect-level assessment framework that takes advantage of the well-established ER methodology towards providing high end, objective, and data oriented bridge condition assessment.

VI.3 Limitations

The proposed methodology is able to translate possible defect measurements into a general bridge condition rating, while helping to reduce sources of data uncertainty in the bridge inspection process. However, some limitations of the presented approach are noted as follows:

- The bridge hierarchal breakdown, considered for the purposes of the presented assessment model, is limited to only 4 major bridge components and 12 composing elements.
- For simplicity reasons, the potential bridge element defects considered in this model are generally limited to sets of 5 most pertinent defects per element.
- Due to limited resources and timeframe, the calculated relative importance weights in the presented model are based on expert surveys that were numerically limited to 20, and geographically limited to Canadian provinces. An upgrade might consider a larger pool of experts and a wider geographical dispersion.
- The presented approach bases the condition assessment on the presently detected structural defects without looking into their timely development and causing deterioration factors.
- The proposed framework is developed to for the assessment of concrete bridges; however, it doesn’t cover other types of bridges (steel or timber).

VI.4 Future Work and Recommendations

This research targeted a basic, yet fundamental area of bridge management. Some of the recommendations and prospects for future work can be summarized in the following:

- A further elaborated extension of the bridge break down structure can be achieved by attempting to expand to additional components and elements. Additional components may include approaches, embankments, and accessories. More elements can similarly be introduced to the assessment break down of bridge components, such as barriers for the bridge deck.
- A suggested improvement on the existing model would be to investigate and consider more types of defects that may develop on bridge elements.
- The proposed bridge assessment model is intended to evaluate the bridge based on its general deterioration state and material condition. However, it doesn't account for the evaluation of bridge load capacity ratings, or user serviceability measures. A more comprehensive assessment framework can be developed in the future to combine material condition with load capacity and user satisfaction metrics.
- The presented approach bases the condition assessment on the presently detected structural defects without looking into their timely development and causing deterioration factors. Further research can attempt to investigate and ultimately incorporate defect causing factors into the assessment framework.

- The proposed condition assessment framework was developed for concrete bridge elements. However, the same methodology /model development steps can be utilized and implemented in the condition assessment of steel or timber bridge elements.
- The developed tool is practically based on two steps: mapping the defect extents on fuzzy grade membership functions using spreadsheets, and then feeding the distributed defect grades to a separately maintained HER assessment model on the IDS© software. Future work is recommended to develop a single software platform that is able to combine both processes in a seamless interface.

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APPENDIX A Alternative Non-Destructive Evaluation Methods

Concrete Resistivity Test

The rate of corrosion in steel reinforcement bars is highly dependent on the electrical conductivity of concrete. Given this fact, a test has been developed to measure the ability of currents that cause corrosion to flow through concrete. This is achieved by measuring the resistivity of concrete to imposed current flows. The basis of resistivity testing lies in the fact that a more receptive concrete to current flow is more vulnerable to the electrochemical corrosion reaction. Since half-cell potential test do not provide a rate for corrosion, a concrete resistivity test can be a good complementation. The most common form of in-situ resistivity is called the Wenner four-probe technique (as can be seen in Figure 38).

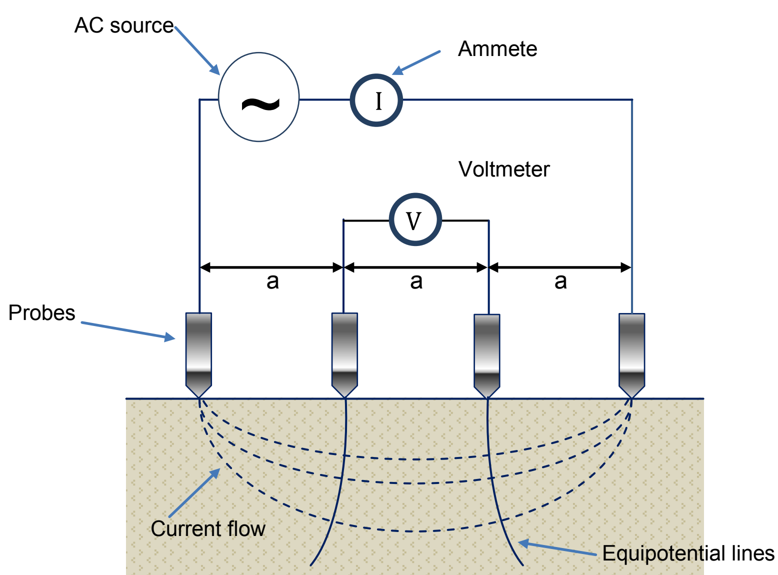


Figure 38: General Arrangement of Concrete Resistivity Test

The apparatus consists of four electrodes placed at equal spacing in a straight line on the concrete surface (Bungey et al. 2006). An Alternating electrical current with low frequency is allowed to flow between the two outer electrodes, while having a voltmeter connected to the two inner probes. The voltmeter will measure the difference in potential between the inner electrodes. This will facilitate the calculation of the apparent resistivity (in $\Omega.\mathrm{cm}$ or $\mathrm{K}\Omega.\mathrm{cm}$) as follows (Bungey et al. 2006):

$$\rho = \frac{2\pi a V}{I}$$

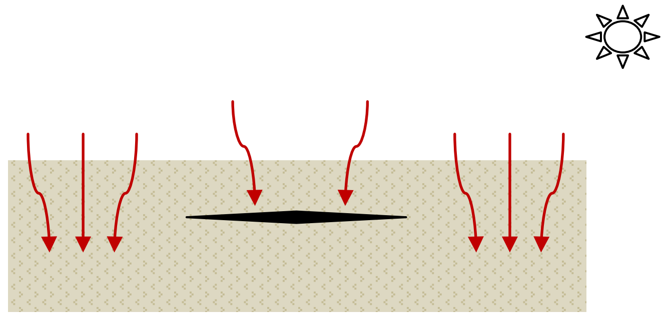
Where a is the spacing between the electrodes, V is the voltmeter reading of the voltage drop, and I is the ammeter reading of the applied current. In conjunction with half-cell potential readings showing probable corrosion, the value obtained by resistivity test can be a good representation of the likelihood of significant corrosion (Table 34).

Table 36: Interpretation of Concrete Resistivity Measurements (Bungey et al. 2006)

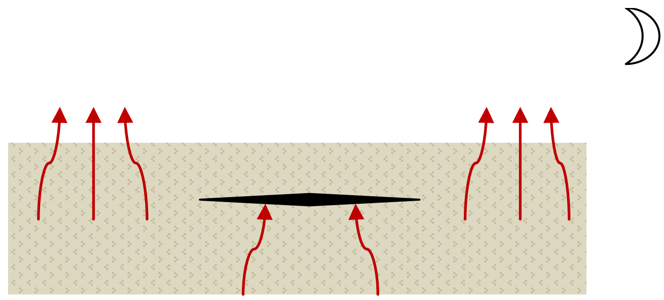
Resistivity	Likelihood of significant corrosion (for non-saturated concrete)
< 5	Very High
5-10	High
10-20	Low/moderate
>20	Low

Infrared Thermography Method

This technique is among the most effective, convenient, and economical methods of testing concrete. It has proved powerful capabilities in detecting internal anomalies in concrete structures such as bridge decks, garage pavements and concrete walls (Weil 2004). The technique is fundamentally based on the principle of localized differences in surface temperature between sound and defected concrete (Bungey et al. 2006). In general, the concrete surface temperature changes due to temperature variations during a given day; the surface is normally heated up by sunlight (especially in summer) and cooled down at night. But as we go below the surface level, heat usually decreases with depth during the day and vice-versa during the night. However, the presence of internal flaws imposes a direct effect on those temperature gradients by altering the thermal conductivity properties of concrete. Sound concrete is supposed to have minimal resistance to thermal flow; whereas the internally defected concrete material experiences lower rates of energy conduction due to the various thermal properties of the present anomalies and their interruption of the thermal convection currents (Weil 2004). These anomalies may include delamination in concrete over corroded steel reinforcement, honeycombing voids caused by poor concrete integration, and water infiltrations. Therefore, location and extent measurements of those defects can be achieved by the detection of localized temperature variations on the concrete surface.



(a) Hot surface resulting in inwards thermal flow



(b) Cold surface resulting in outwards thermal

Figure 39: Effect of Internal Defects on Thermal Flow (adapted from ACI 228.2R-98)

For instance, if a concrete bridge deck is provided with a sufficient and even distribution of heat over its surface by sunlight; then the surface temperature right above internal flaws would be noticeably higher than the surrounding sound surface temperature. The opposite is true if thermal measurements are carried on at night. The emitted thermal radiation and localized heat differentials can only be technically observed in the spectral range of infrared, hence the method's name.

High-resolution infrared radiometers are commonly used to scan large concrete surface areas, with the resulting data being pictures consisting of areas with varying color tones to indicate differential surface temperature. The implementation procedures for investigating internal defects in concrete bridge decks have been standardized and provided by ASTM D4788-03. An infrared thermography system can be broken down to four major components (Weil 2004):

1- An infrared sensor head unit for scanning/detection
2 A real-time microprocessor connected to a display monitor
3 A data acquisition and analysis component
4 An image recording and retrieving device.

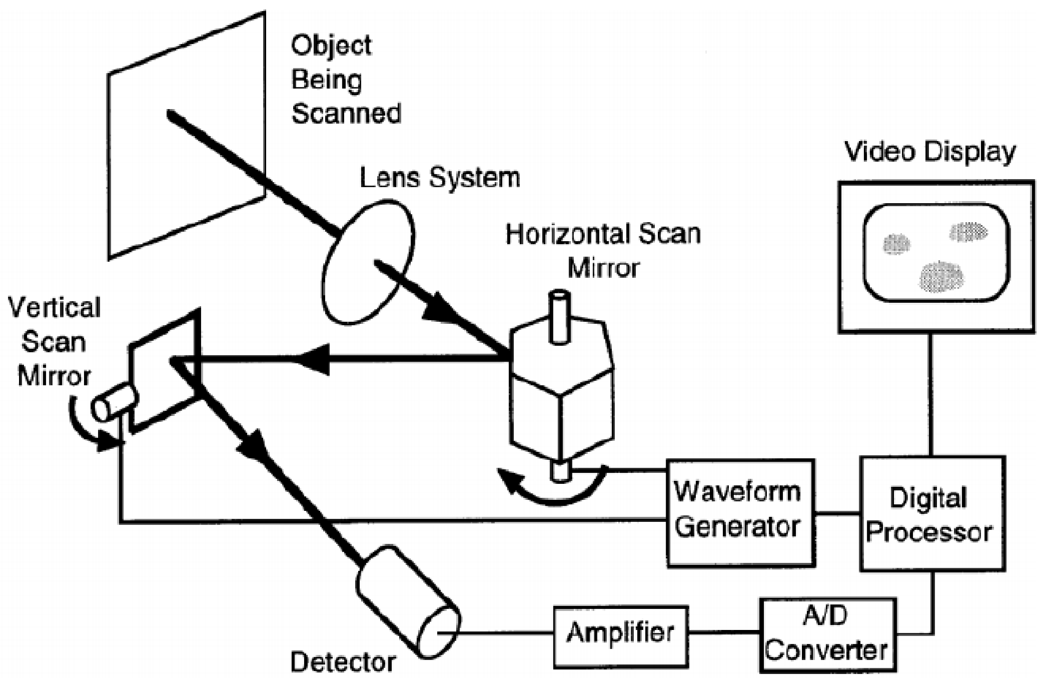


Figure 40: A Schematic of the Infrared Scanner System (ACI 228.2R-98)

Figure 40 shows a schematic arrangement of the test's system apparatus to facilitate the thermal surface scan of a large concrete surface, such as a bridge deck or a highway section, the test apparatus can be fully mounted on a moving vehicle.

Weil (2004) reported several factors that might have an impact on the test's accuracy. These factors can be classified into three groups; namely, subsurface configuration, surface conditions, and the environment. The first group includes thickness, concrete thermal conductivity, types of anomalies, and the heat source. The second group concerns factors such as surface heat emissivity. Finally, the significant environmental factors that are found to affect the test are: cloud cover, solar radiation, wind speed, and surface moisture.

Ground Penetrating Radar

Ground Penetrating Radar (GPR) is one of the emerging, powerful NDE technologies that found numerous applications in engineering. It is a rapid geophysical method that is based on electromagnetic waves propagation through tested objects to evaluate their subsurface features. Also referred to as Ground Probing Radar, the method is founded on the principle of varying microwave speed and amplitude from one material to another (Clemeña 2004). This feature enabled a variety of early geospatial GPR applications, such as measuring the depth of sea ice (Campbell and Orange 1974), profiling subsurface geology/archeology (De Vore 1998), studying of bedrocks and ground water (Azevedo and McEwan 1997), and even to analyze the subsurface of the moon (Porcello et al. 1974).

GPR typically produces graphical images of features below the surface of the tested materials, which are useful in interpreting subsurface layouts and deterioration forms. The produced electromagnetic waves are usually emitted in very short pulses; hence the common alternative name "Short-Pulse Radar".

One of earliest civil engineering applications were reported by Bertram et al. (1974), related to studying and investigating internal air voids in pavements. In the recent years, many studies have been showing the robustness of applying GPR as a subsurface investigation tool for the deterioration analysis of concrete structures, such as concrete retaining walls (Hugenschmidt et al. 2010), concrete water tunnels (Arosio et al. 2012), and most particularity; concrete bridges (Cruz et al. 2010). The applications of GPR in concrete structures range from the nondestructive detection of subsurface defects, to measuring the thickness of concrete elements, or determination of water content in concrete (Clemeña 2004). GPR applications in the various concrete structures include depth measurement and location of reinforcement layers, thickness measurement of concrete elements, corrosion investigation of reinforcement bars, and delamination/voids detection below concrete surface.

The propagation of electromagnetic waves through concrete is controlled by two main physical properties: the electrical permittivity affecting signal velocity, and the electrical conductivity which determines the attenuation of the signal (Bungey and Millard 1993). Reflections of radar waves will arise at interfaces between different materials, or from internal anomalies within a single material.

The Use of GPR for testing concrete bridges has gained wide recognition, particularly to evaluate bridge decks. The methodology is practically applied to assist detailed condition surveys in the U.S. and Canada, along with being well established in standards such as ASTM D4748 and ACI 228.2R . A GPR system for bridge deck inspections typically consists of a control unit, an antenna unit (for transmitting and receiving the electromagnetic signal), a power supply/converter unit, and a signal recorder. As the system moves along the bridge deck, received radar signals will be recorded for later processing and analysis (Figure 41). To inspect a bridge deck, the GPR system could either be manually dragged over the surface, or attached to a 3-wheeled vehicle (Figure 42). Advanced GPR systems can be mounted on traffic vehicles, allowing for faster scans and excluding the need to for traffic interruptions.

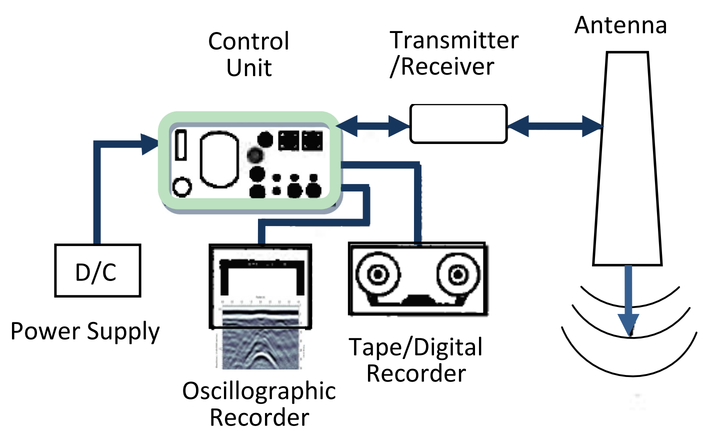
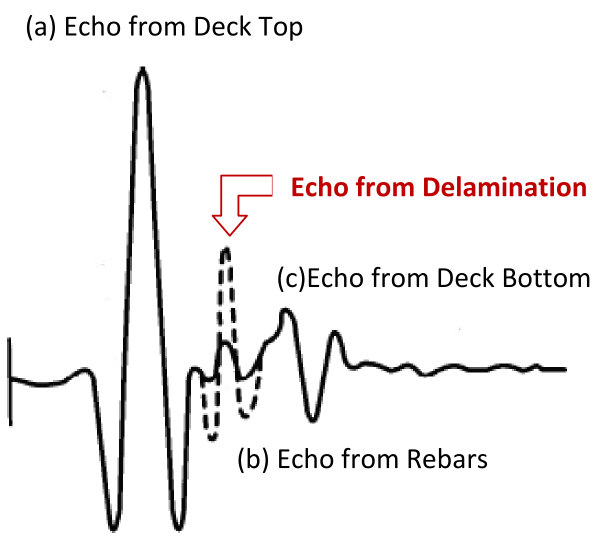


Figure 41: Typical GPR System Components (Clemeña, 2004)

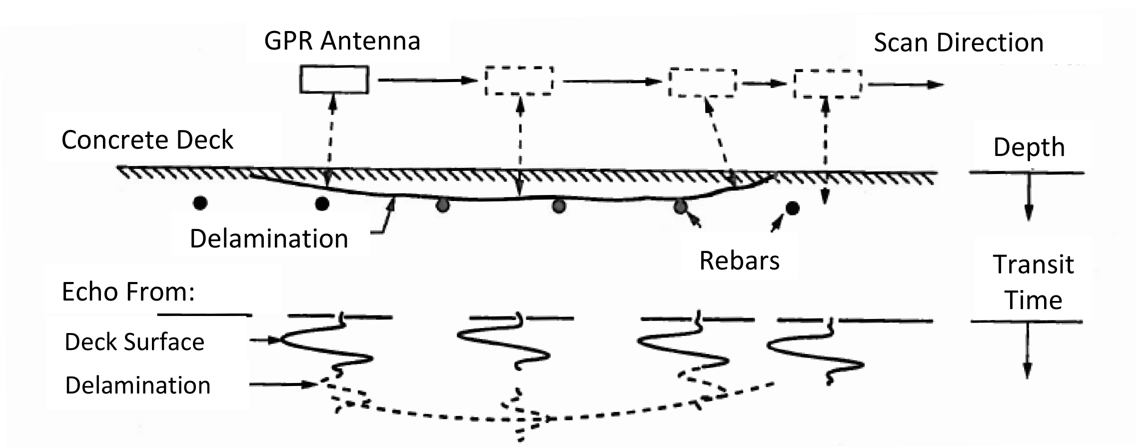
As the control unit triggers the antenna to generate high frequency pulses, a short electromagnetic wave is transmitted into the surveyed structure. The antenna will subsequently receive several reflections corresponding to interfaces between materials of different dielectric properties (Bungey et al. 2006). Those reflections will be received by the antenna at different time instants, suggesting varying depths of the detected subsurface interfaces.

Figure 42: Wheel Mounted GPR System

One of the most valuable applications of GPR is the detection of delaminations areas above the reinforcement layer in concrete bridge decks. When GPR waves are sent through the deck, typical portions of the received signal will indicate reflections from the deck top layer of reinforcement (if applicable), bottom layer of reinforcement (if applicable), and the bottom of the deck slab. However, the presence of delaminated areas will generally produce additional reflections (Figure 43(a)). Usually of negative polarity, those additional reflections serve to indicate the presence of delamination areas above reinforcement (Clemeña 2004). Scan lines are recorded as the antenna passes along the bridge deck for latter analysis of the collected signals. A bridge deck could be divided into a number of longitudinal scan lines that depends on the instrument's coverage power or signal strength. The product would be a set of waveforms that featuring topographic features of the scanned deck to delineate the delaminated areas. As a suggested interpretation of the collected radar signals, signal depressions that indicate delamination could be joined to approximate an idealized radar delamination signature (Figure 43(b)). To sum up, the GPR has proved robust abilities in scanning both bare and overlaid concrete bridge decks for subsurface investigation. The technique is continuously gaining rapid acceptance as a reliable non-destructive technique. So far, the testing bridge decks using GPR has proven to be fast, easy to carry out, and convenient for traffic.



(a)



(b)

Figure 43: (a) Effect of Delamination on Radar Echoes from a Reinforced Concrete Deck Section, (b) the Development of an Idealized Depression Signature for Delamination

APPENDIX B Bridge Management Systems (BMS)

Since bridges are normally inspected on a biannual basis, most of bridge management agencies are expected to run inspections on about half of their bridge inventory every year. Bearing in mind the budget constraints, not all bridges would get maintenance attention, and even fewer bridge cases would get prioritized for actual maintenance operations. Therefore, given the ample amount of inspection inputs on a project level, and substantive limitation of funds available for the network, decision makers in a particular bridge management agency would need to have powerful analytical tools to run critical maintenance tradeoffs among bridges under their authority.

Overview of BMS

In an effort to globally manage bridge inventories as well as retain a detailed review of each bridge in the system, DOTs in the U.S and Canada have developed or adapted Bridge Management Systems (BMS). In the US, PONTIS as part of AASHTOware package is the most widely implemented BMS. While in Canada, different provinces have their own BMS packages, such as the OBMS in Ontario, and the GSQ in Quebec. Those systems are designed to keep track of individual bridge details and inspection records, develop deterioration forecasts, and prioritize maintenance actions.

Pontis

In the late 1980's, the increasing gap between maintenance needs and available funds have triggered a nation-wide advocacy to more detailed and analytical form of bridge management. The need for more efficient allocation of bridge maintenance funds has urged FHWA and Caltrans to sponsor the development of “Pontis” Bridge Management System in the early 1990’s (FHWA 2002). Offering a menu of as many as 160 bridge elements, Pontis has provided a greater level of details than the earlier minimum requirements of NBIS, and allowed different states to have common grounds in reporting their bridge management data to the federal inventory (Thompson and Shepard 2000).

The Pontis BMS has received wide acceptance in the US. The Pontis full-featured BMS is currently in use by 40+ state DOTs, providing sufficient flexibility for individual agencies or organization based on their custom inventory management needs (Markow 2008). As shown in Figure 44, Pontis offers a number of bridge management and resource allocation capabilities as follows:

- Bridge inventory: It provides a platform for establishing and maintaining the bridge inventory information, with the capacity to exchange data with other agency systems.
- Inspection management: through bridge inspection schedules; import of inspection data; production management reports such as the Structure, Inventory, and Appraisal (SI&A) reports; and ability to render NBI output files to be annually submitted to the FHWA.
- Assessment of needs and strategy development: it offers continuous estimation and updating of bridge element deterioration and intervention cost models based on agency-specific experience; it develops long term network level policies for structure preservation and improvement; it

performs assessments of current and future preservation needs; it evaluates alternative bridge program investment scenarios; it accounts for technical, economic, and policy-related factors.

- Project and program development: Pontis develops projects to respond to inspector recommendations together with agency standards; it also evaluates the impacts of different project alternatives on structural performance; It ranks projects and develops budget constrained maintenance programs; It tracks project status and completion schedule.

In addition to the strong analytical capabilities of Pontis, it offers plenty of user defined features to suit agency specific needs (Markow 2008). For instance, the definition and classification of bridge elements can be customized, the classification of bridge actions can be re-defined, cost indices can be reviewed, and internal analytical formulas can be changed to accommodate different agency policies. Among the reported limitations in literature, it was pointed out that Pontis solely depends on cost-benefit analysis for project prioritization and selection, while ignoring other potential measures like asset customer value or serviceability (Scherschligt and Kulkarni 2003).

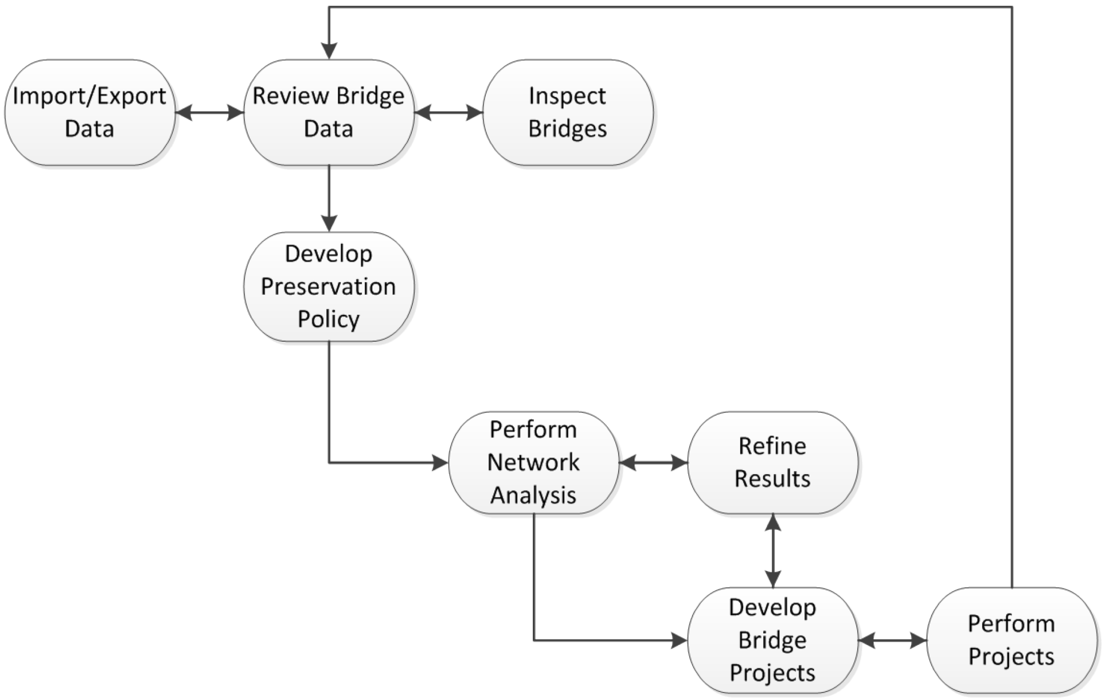


Figure 44: The Analytical Process of Pontis BMS (Cambridge Systematics 2004)

BMS in Canada

Similar to the United States, Canada is facing complex challenges to preserve and manage its existing bridge infrastructure. Throughout a vast geographic allocation, bridges in Canada are of wide range of structural systems and materials. The fairly harsh weather conditions and extensive use of deicing salts in winter have been imposing great bridge preservation challenges on the provincial and municipal ministries of transportation.

The growing social and economic accountability to maintain their bridge inventories, along with the need to efficiently allocate maintenance funds, have encouraged many Canadian provinces start implementing Bridge Management

Systems (BMS). However, the available systems in the different Canadian provinces vary in their functionalities, interfaces, and analytical capabilities (Khanzada 2012). This section attempts to review some of the predominant provincial bridge management practices in Canada.

- Quebec

An inventory of about 10000 provincial and municipal structures are owned and sustained by the Ministry of Transport of Quebec (MTQ). Similar to other agencies, the ministry is continuously striving to maintain the aging infrastructure inventory (Ellis et al. 2008). MTQ's first BMS finds its roots in 1987, and with the constant improvement, the currently implemented "Système de Gestion des Structures (GSQ)" stands out today as a state of the art BMS. In addition to the inventory and inspection module to store and handle the bridge database, GSQ has employed the strategic planning module (MPS) to facilitate network level analysis and decision making for MR&R actions (Khanzada 2012). The MPS module offers analytical abilities for element, project and network levels as shown in Figure 45. It also provides a lot of decision support tools, including life cycle costs and what-if scenario analysis. For instance, a user defined rehabilitation action will directly imply changes on 10 year deterioration and annual maintenance cost forecasting.

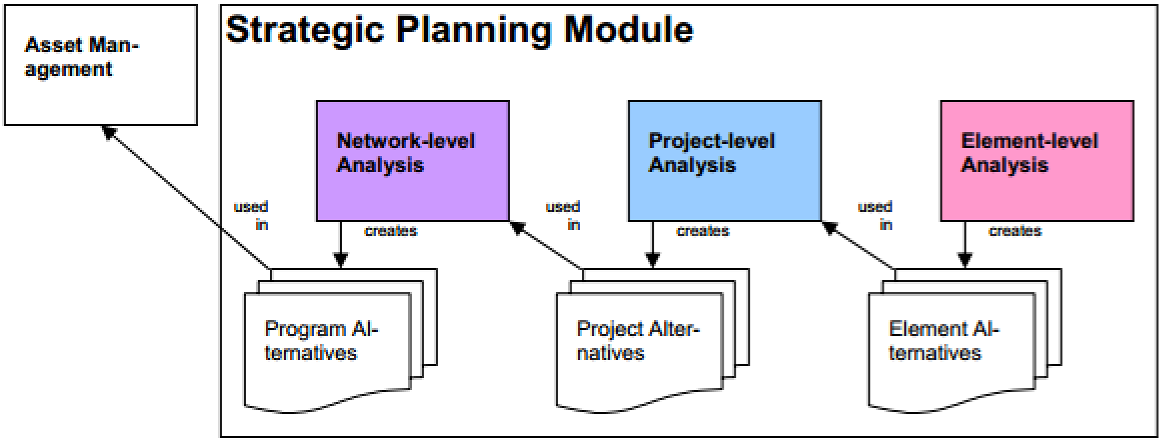


Figure 45: Analysis Levels by MPS (Ellis et. al. 2008)

- Ontario

According to the 2009 Annual Report of the Office of the Auditor General of Ontario, the province has about 14800 bridges; approximately 12000 of which are managed by local municipalities, while the rest fall under provincial jurisdiction. The ministry of transportation of Ontario (MTO) follows a disciplined approach to keep track of its provincial bridge inventory through the Ontario Bridge Management System (OBMS). Implementation of OBMS first started in the year 2000 as a tool for bridge inspection data management, with the analytical tools being introduced in 2002 (Thompson et al. 2003). The system has become fully populated with element level inspection data of bridges according to the OSIM specification mentioned earlier. On network level, OBMS features graphical trade off analysis of MR&R strategies.

- British Colombia

The Bridge Management Information System (BMIS) is implemented to manage inspection data entries for bridges in the province. BIMS has been used to manage the province’s inventory of about 4500 structures. The system has been developed over the last 20 years and received its last update in the year 2000. (Hammad et al. 2007).Inspection data are managed by a module for gathering on-site inspection data (BRIDE), which is an integral part of the BMIS. Additional modules include locating structures on a map (BIG), and creating custom ad-hoc reports (Discoverer). The system’s inspection forms include 6 different structure types - Bridges, Suspension/Cable Stayed Bridges, Culverts, Tunnels, Retaining Walls, and Sign Structures. It allows the inspector to rate each component to different condition states with respective percentages. The condition rating system comprises of 5 states.

- Nova Scotia

Nova Scotia Department of Transportation and Public Works (NSTPW) developed its own BMS in 1999-2003, named the Nova Scotia Bridge Management System (NSBMS) to help the department safely manage the province’s inventory of around 4000. NSBMS is mainly based on Ontario BMS. With respect to inspection, OSIM is used for inspection methodologies. It follows the severity and extent philosophy, requiring the inspector to measure the defects’ quantities in all the condition states. Deficiencies in performance are also noted for bridge components. The system uses a 4 grade scheme, following the Ontario model (Speiran et al. 2004).

APPENDIX C Concrete and Wearing Surface Defects

Concrete Defects

Cracking

Cracks are defined as linear fractures caused by external loads inducing tensile and/or compressive stresses in concrete. Cracks can be classified based on different set of criteria. By size, concrete cracks can range from hairline cracks barely observed by the naked eye to significantly wide cracks. Measuring gauges are commonly used to classify and document cracks into hairline, narrow, medium and wide. In addition, Cracks can be categorized based on the structural nature. Structural cracks are caused by the superimposed live and dead loads, which induce flexural and shear stresses in the structure. In fact, flexural cracks usually occur where concrete members are burdened with tensile stresses that exceed the tensile strength of concrete, and can be seen at tension zones before tension is taken up by the reinforcing steel. Shear cracks are usually found near member ends where the shear forces are at peak. Non-structural cracks are usually caused by temperature shrinkage and expansion of the concrete mass.

Contrary to structural cracks that may undermine the load carrying capacity of the structure; Non-structural cracks are smaller and structurally less significant. However, they may lead to future problems such as water seepage and contamination. Both structural and non-structural cracks should be measured and documented for maintenance actions. Since cracks may facilitate water leakage through concrete and along with contaminating salts, they usually act as the initial step leading to a set of other progressive deteriorations such as reinforcement corrosion and disintegration.

Scaling

Scaling can be defined as local flaking, or loss of surface portions of concrete or mortar at the top surface levels. It is mainly caused by the actions of freezing and thawing on the concrete surface, causing superficial disintegration of weak surface mortar layers that may eventually expose aggregates in some severe cases. The application of de-icing salts further accelerates surface scaling as well. Despite the fact that weak surface layers may result from poor concrete finishing and curing, scaling may often indicate insufficient air entrainment (Iffland and Birnstiel 1993).

Corrosion

Corrosion of the steel reinforcement bars is the causing trigger of many subsequent damages as the concrete elements age. Corroded reinforcement bar develop a gradual increase in volume, due to the accumulation of rust, inducing augmented internal stresses that result in damages to the low strength pockets of the surrounding concrete material. Corrosion can be initiated by either carbonation or chloride contamination; both of which cause the reduction of safe levels of alkalinity which promotes the electrochemical reactions leading to corrosion. A full explanation of both types of corrosion along with the accompanying electrochemical processes can be found in the literature review chapter.

Delamination and spalling

One of the serious internal deterioration mechanisms in concrete is delamination. It can be basically defined as the development of partial separation areas of the concrete cover at or near the outermost layer of steel reinforcement (FHWA 2002). In fact, delamination is a result of substantial and advanced-level corrosion in the top steel bars. The progression of rust on the bars causes them to swell multiple times of their original volume. This creates a surge of internal stresses that leads to cracking and deboning of the surrounding concrete in contact with the bars. Consecutively, deterioration results in the form of localized aerial separations of the concrete cover from the top reinforcement layer.

At severe deterioration levels, the subsurface delamination areas may escalate to the concrete surface causing rapture or spalling of chunks from the cover. The partially separated concrete cover will have less flexural capacity, which may lead to the cover being broken or spalled under repetitive vehicular loading (Hoensheid 2012). The presence of delamination areas is commonly approximated by the production of a hollow sound when tapped with a hammer.

Subsequent to severe cases of delamination, several delaminated regions form into spalls that escalate to the concrete surface, Spalls are parallel to surface depressions in concrete resulting from the separation and detachment of surface portions (FHWA 2002). In most cases, spalling is an advanced corrosion-induced deterioration in concrete. It is a subsequent perpetuation of the delamination process where severe deboning and separation of the concrete cover from reinforcement bars contributes to the breaking off and rapture of surface concrete portions. Apart from reinforcement corrosion, OSIM (2008) distinguishes several factors that cause spalling or local breading of concrete pieces. Those factors include vehicular impact, formation of ice in delaminated areas, or localized compression

Pop-outs

Pop-outs are shallow conical depressions caused by the fracture of small portions at the concrete surface. Pop-outs occur due to the expansion of some aggregates by moisture absorption. They may also be caused by frost prone aggregates; the formation of expanding ice crystals in water saturated aggregates may induce internal stresses that may exceed the aggregate’s tensile strength and eventually lead to its fracture. Shattered aggregate particles are typically found adhering to the bottom of the pop-out cone (Manning 1985). Pop-outs are not looked at as a serious structural concern; however, they can cause a rougher vehicular ride and assist in speeding up the rate at which water percolates to the steel reinforcement layer (INDOT 2010).

Erosion

Erosion is a common type of deterioration that occurs at the footing or ground level of piers columns or abutment walls. It is defined in the Ontario Structures Inspection Manual (MTO 2008) as a type of concrete deterioration which is caused by water-borne gravel or sand particles scrubbing against the concrete element’s surface. Alternatively referred to as scour; erosion may also be caused by flowing ice or streams hitting the supports of a waterway crossover. Moreover, gradual surface wearing away can be due to abrasion by wind-carried particles.

Wet Areas / Surface Deposits

Staining, surface efflorescence, and wet areas may be noticed on bridge several bridge elements as they advance in their service lives. Leakage of surface water through deck joints, seepage of ground water through cracks, or inefficient drainage system could be factors that contribute to the progressive development of wet areas or surface deposits on a bridge element’s surface (TN Zealand 2001). Wet areas or stains commonly observed on deck soffits due to water infiltration through the bridge deck. The presence of cracks in concrete further permits the permeation of water, which may carry dissolved salts or chemicals. The seeping water usually leaches a solution of salts and chemicals through the concrete element and leaves them behind as surface deposits as the water evaporates. Common types of surface deposits are (MTO 2008): white and powdery salts (efflorescence), liquid or gel-like discharges (exudation), hard crust (incrustation), or downward icicle-shaped formations (stalactite).

Wearing Surface Defects

Cracking

Asphalt cracks are commonly observed as linear fissions that extend in longitudinal, transverse, or mapped patterns; partially or completely throughout the asphalt pavement. They can be caused by many factors, including: poor quality, vehicular action, temperature induced freezing and thawing ...etc. cracks are mainly distinguished by their appearance and development direction, such as transverse cracks and alligator cracks.

Potholes

Potholes are of the commonly observed, serious asphalt defects. They can be easily distinguished as they appear as localized conical downward depressions in the pavement. One of the major causes of potholes in asphalt is the freezing and thawing action following progressive water penetration. Also, potholes may form due to the excessive wear that is being caused by vehicular movements. Pavements that suffer from extensive patterned cracks and raveling have more chances to subsequently develop potholes too.

Rutting and Rippling

Asphalt pavements might also be susceptible to rutting. Caused repetitive compaction and lateral shoving action of heavy vehicles, the asphalt pavement can suffer from longitudinal depressions at wheel track locations. Moreover, heavy wheel frictions and the applied braking forces can lead to rippling in asphalt. Rippling can be identified by the existence of substantial transverse undulations (crests and valleys) in the asphalt pavement.

Loss of Bond

In addition to these defects in asphalt which may be observed on the surface, there might exist another defect that is rather subsurface: the loss of bond. As both the bridge deck and its topping asphalt layer age in service, the bond may lose its strength. This is depicted by detachment areas that occur between asphalt layer, water-proofing, and/or deck top. Those Areas of detachment cannot be directly measured visually; rather, they can either be approximated by hammer sounding or further investigated by means of non-destructive evaluation.

APPENDIX D Survey Questionnaire

DEPARTMENT OF BUILDING, CIVIL & ENVIRONMENTAL ENGINEERING

CONCORDIA UNIVERSITY, MONTRÉAL, QC

``DEFECT BASED CONDITION ASSESSMENT OF CONCRETE BRIDGES``

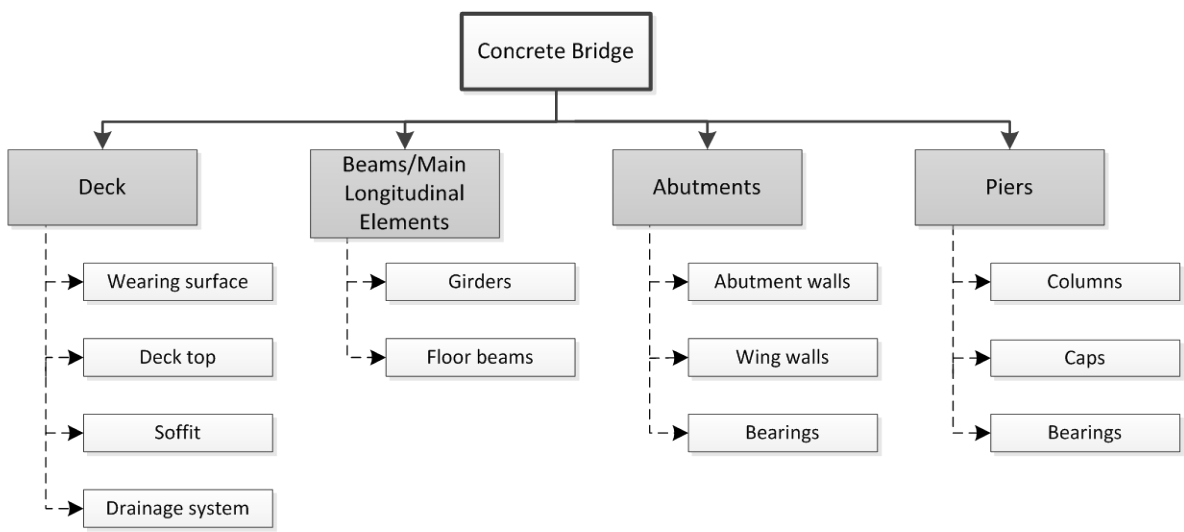
BY: SAMI MOUFTI

Respondent's Info	
Name	
Position	
Company/Institution	
Date	

QUESTIONNAIRE PART I: RELATIVE IMPORTANCE WEIGHTS OF BRIDGE COMPONENTS & ELEMENTS

The purpose of this survey is to define relative importance weights of the components and elements that compose a concrete bridge structure. Through pairwise comparisons between the factors, judgments would be synthesized to represent the inner and outer dependence among the components and elements. The product of this exercise would be relative weight factors assigned to elements in every main bridge component, and a relative weight factor for each component towards the entire bridge. Weight/priority factors will be representing the bridge elements' and components' relative structural importance.

As a simplified representation of the structure, a concrete bridge is broken down to four main components (Deck, Beams/Main longitudinal Elements (MLE) , Abutments, and Piers). Further, components are subdivided into elements (for instance, Deck is divided into: Wearing surface, Deck top, Soffit, and Drainage system). To assist respondents in better visualizing the bridge composition, the hierarchical structure of the concrete bridge components and elements considered in this research is illustrated in the following figure:



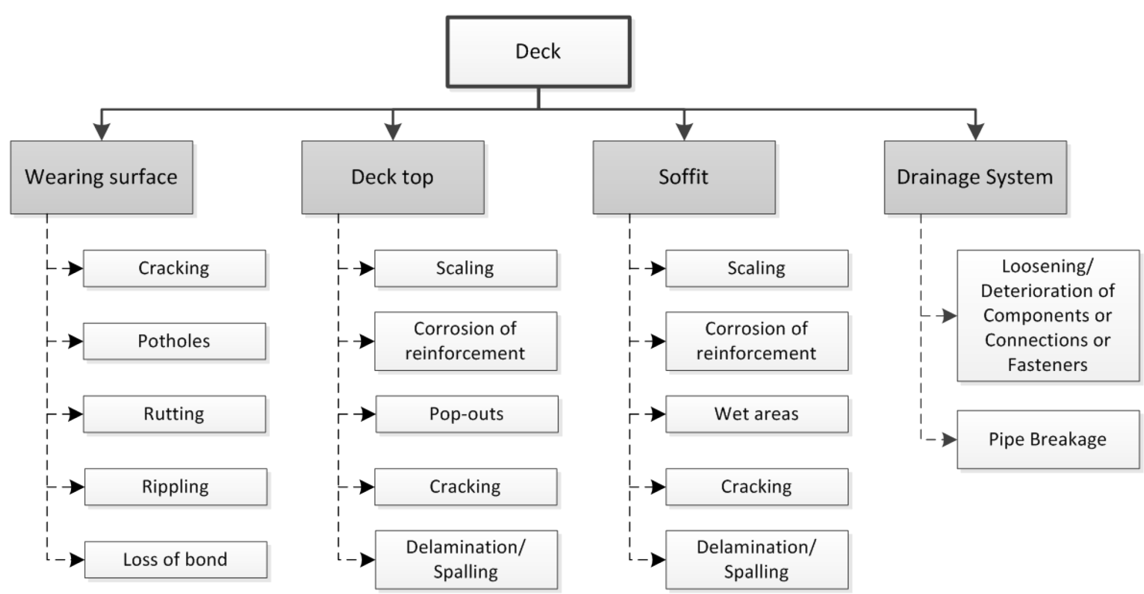
QUESTIONNAIRE PART I: "BRIDGE COMPONENTS & ELEMENTS"

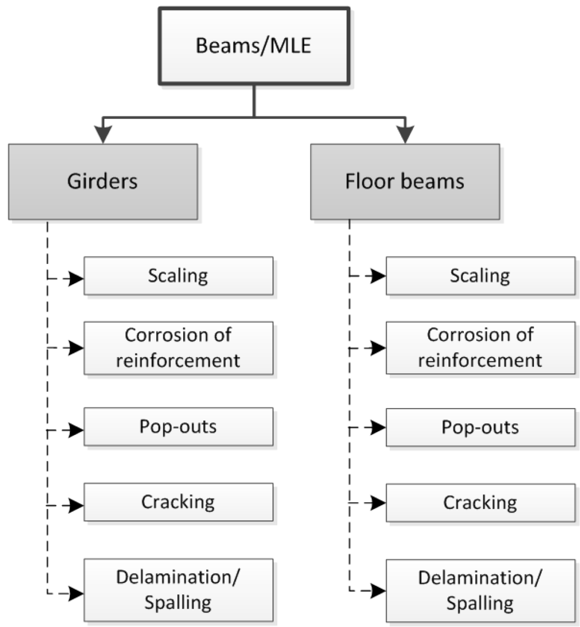
	(A)	Degree of importance									(B)
		Extreme	Very strong	Strong	Moderate	Equal	Moderate	Strong	Very strong	Extreme	
Main Components	**With respect to (C): "Entire Bridge"**										
	Beams/MLE	9	7	5	3	1	3	5	7	9	Deck
		9	7	5	3	1	3	5	7	9	Piers
		9	7	5	3	1	3	5	7	9	Abutments
Deck Elements	**With respect to (C): "Deck"**										
	Deck Top	9	7	5	3	1	3	5	7	9	Wearing Surface
		9	7	5	3	1	3	5	7	9	Soffit
		9	7	5	3	1	3	5	7	9	Drainage System
Beams/Main Longitudinal Elements	**With respect to (C): "Beams/MLE"**										
	Girders	9	7	5	3	1	3	5	7	9	Floor Beams
Abutment Elements	**With respect to (C): "Abutments"**										
	Abutment walls	9	7	5	3	1	3	5	7	9	Wing Walls
		9	7	5	3	1	3	5	7	9	Bearings
Pier Elements	**With respect to (C): "Piers"**										
	Columns	9	7	5	3	1	3	5	7	9	Caps
		9	7	5	3	1	3	5	7	9	Bearings

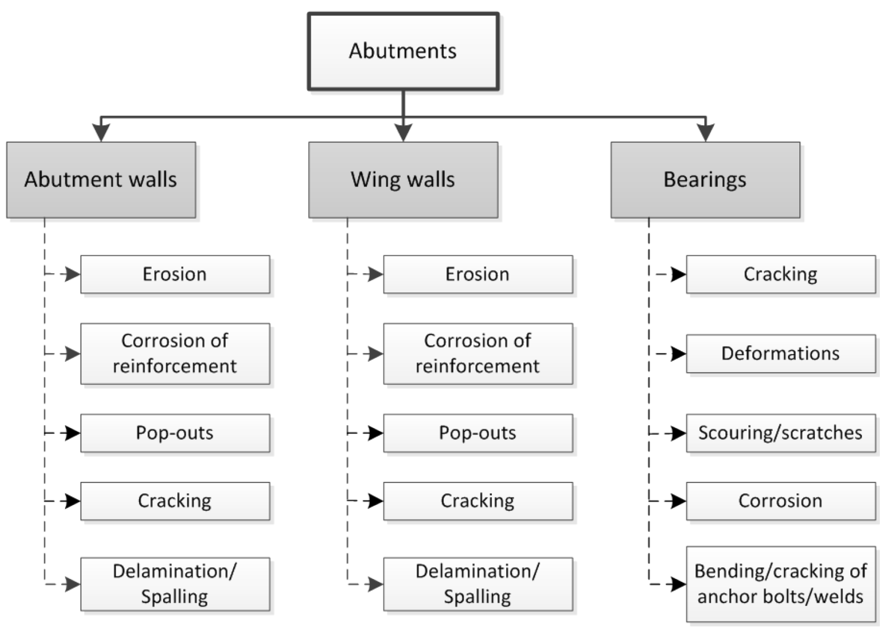
QUESTIONNAIRE PART II: RELATIVE IMPORTANCE WEIGHTS OF BRIDGE DEFECTS

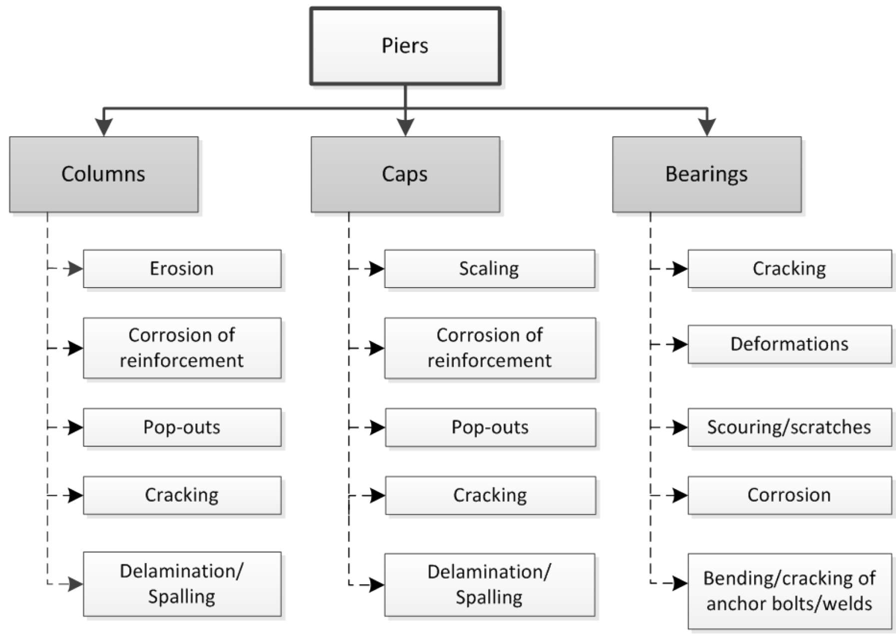
The purpose of this survey is to define relative importance weights of the possible defects that may be detected on the concrete bridge elements. Through pairwise comparisons between the defects, and as in part 1, judgments would be synthesized to represent the inner and outer dependence among the defects. The product of this exercise would be relative weight factors assigned to defects with respect to their relative elements. Weight/priority factors will be representing the defects' importance on the safety and structural integrity of their respective elements. Should it be needed, please refer to "Appendix" for detailed definitions of the various bridge defects presented in this research.

The structural hierarchy presented earlier is further extends to cover possible defects on every element of the concrete bridge. See the following figures:




Beams/MLE
Girders
Floor beams
Scaling
Corrosion of reinforcement
Pop-outs
Cracking
Delamination/ Spalling
Scaling
Corrosion of reinforcement
Pop-outs
Cracking
Delamination/ Spalling


Abutments
Abutment walls
Wing walls
Bearings
Erosion
Corrosion of reinforcement
Pop-outs
Cracking
Delamination/ Spalling
Erosion
Corrosion of reinforcement
Pop-outs
Cracking
Delamination/ Spalling
Cracking
Deformations
Scouring/scratches
Corrosion
Bending/cracking of anchor bolts/welds


Piers
Columns
Caps
Bearings
Erosion
Corrosion of reinforcement
Pop-outs
Cracking
Delamination/ Spalling
Scaling
Corrosion of reinforcement
Pop-outs
Cracking
Delamination/ Spalling
Cracking
Deformations
Scouring/scratches
Corrosion
Bending/cracking of anchor bolts/welds

QUESTIONNAIRE PART II"BRIDGE DEFECTS"

	(A)	Degree of importance									(B)
		Extreme	Very strong	Strong	Moderate	Equal	Moderate	Strong	Very strong	Extreme	
Wearing Surface Defects	**With respect to (C): "Wearing Surface"**										
	Potholes	9	7	5	3	1	3	5	7	9	Cracking
		9	7	5	3	1	3	5	7	9	Rutting
		9	7	5	3	1	3	5	7	9	Rippling
		9	7	5	3	1	3	5	7	9	Loss of Bond
Deck top Defects	**With respect to (C): "Deck Top"**										
	Delamination/ Spalling	9	7	5	3	1	3	5	7	9	Cracking
		9	7	5	3	1	3	5	7	9	Corrosion of R/C
		9	7	5	3	1	3	5	7	9	Pop-outs
		9	7	5	3	1	3	5	7	9	Scaling
Soffit Defects	**With respect to (C): "Soffit"**										
	Delamination/ Spalling	9	7	5	3	1	3	5	7	9	Cracking
		9	7	5	3	1	3	5	7	9	Corrosion of R/C
		9	7	5	3	1	3	5	7	9	Wet areas
		9	7	5	3	1	3	5	7	9	Scaling
Drainage System Defects	**With respect to (C): "Drainage System"**										
	Pipe Breakage	9	7	5	3	1	3	5	7	9	Loosening/ Deterioration of Components or Connections or Fasteners

Beam Defects	**With respect to (C): “Beams”**										
	Delamination/ Spalling	⑨	⑦	⑤	③	①	③	⑤	⑦	⑨	Cracking
		⑨	⑦	⑤	③	①	③	⑤	⑦	⑨	Corrosion of R/C
		⑨	⑦	⑤	③	①	③	⑤	⑦	⑨	Pop-outs
		⑨	⑦	⑤	③	①	③	⑤	⑦	⑨	Scaling
Abutment wall Defects	**With respect to (C): “Abutment Walls”**										
	Erosion	⑨	⑦	⑤	③	①	③	⑤	⑦	⑨	Cracking
		⑨	⑦	⑤	③	①	③	⑤	⑦	⑨	Corrosion of R/C
		⑨	⑦	⑤	③	①	③	⑤	⑦	⑨	Pop-outs
		⑨	⑦	⑤	③	①	③	⑤	⑦	⑨	Delamination /Spalling
Pier Column Defects	**With respect to (C): “Pier Columns”**										
	Cracking	⑨	⑦	⑤	③	①	③	⑤	⑦	⑨	Erosion
		⑨	⑦	⑤	③	①	③	⑤	⑦	⑨	Corrosion of R/C
		⑨	⑦	⑤	③	①	③	⑤	⑦	⑨	Pop-outs
		⑨	⑦	⑤	③	①	③	⑤	⑦	⑨	Delamination /Spalling
Pier Cap Defects	**With respect to (C): “Pier Caps”**										
	Delamination/ Spalling	⑨	⑦	⑤	③	①	③	⑤	⑦	⑨	Cracking
		⑨	⑦	⑤	③	①	③	⑤	⑦	⑨	Corrosion of R/C
		⑨	⑦	⑤	③	①	③	⑤	⑦	⑨	Pop-outs
		⑨	⑦	⑤	③	①	③	⑤	⑦	⑨	Scaling
Bearing Defects	**With respect to (C): “Bearings”**										
	Bending/ Cracking of anchor bolts/welds	⑨	⑦	⑤	③	①	③	⑤	⑦	⑨	Cracking
		⑨	⑦	⑤	③	①	③	⑤	⑦	⑨	Deformations
		⑨	⑦	⑤	③	①	③	⑤	⑦	⑨	Corrosion
		⑨	⑦	⑤	③	①	③	⑤	⑦	⑨	Scouring/ Scratches