Integrated Decision Support Methodology for Bridge Deck Management under Performance-Based Contracting

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ABSTRACT

Integrated Decision Support Methodology for Bridge Deck Management under Performance-Based Contracting

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Bridges are vital elements of the civil infrastructure system in terms of mobility, environment, economy, and development of communities. Maintaining bridges at sufficient functional and safety levels is an important mandate to ministries of transportation. The 2016 Canada infrastructure report card alarmed that more than 26% of bridges in Canada have deteriorated and the bridges are mostly rated as fair, poor or very poor (CIRC 2016). In the United States, the report card on America's infrastructure assigned grade "C+" to bridge infrastructure (ASCE 2017). Hence, developing rational decision support methods that can assist in managing the vast bridge infrastructure is of paramount importance. This research aimed toward developing a decision support methodology for concrete bridges capable for optimizing the Maintenance, Repair and Replacement (MRR) actions under Performance-Based Contracting (PBC) arrangement through implementing the following steps: i) develop an integrated condition assessment and rating model, ii) develop a forecasting model to assess bridge condition reliability and predict future deteriorations/improvements, iii) develop short- and long-term optimized rehabilitation plans, and iv) design a PBC-based framework for rehabilitation decisions. Upon studying bridge inspection standards and current practices, the research introduces the Quality Function Deployment (QFD) theory and Weibull Distribution Function (WDF)

to produce novel methods to rate the current bridge conditions and forecast future performance. These methods integrate data extracted from visual inspection and Ground Penetrating Radar (GPR) surveys. The *k*-means clustering technique is utilized to develop a rating index that recommends suitable MRR actions based on an integrated condition rating. The Genetic Algorithm (GA) optimization technique is applied to select the best combinations of rehabilitation strategies under the PBC scheme. The integrated rating along with the GA optimization ultimately develop a recommended work program that considers the identified performance triggers and budget constraints. The research contributes a novel PBC-based decision support framework to the area of bridge management that enhances efficiency in implementing MRR strategies while maintaining the delicate balance between the different stakeholders' requirements and goals. The developed methodology is implemented and tested on data extracted from bridge inspection reports and GPR scans, mainly on bridges in Quebec, Canada. Ministries of transportation can benefit from the condition rating and deterioration modeling to assess their bridges' condition and to interfere and do a rehabilitation action before reaching the end of useful service life. The GA-based model provides the maintenance contractors with optimized interventions plans that specify what type of MRR actions to do and when. Further, it assists the ministries to set the budget for such projects. The PBC framework is expected to assist both the transportation agencies and maintenance contractors in arriving at a fair contract value while maintaining the desired bridge performance.

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List of Abbreviations

ADM: Administrative costs

AHP: Analytical Hierarchy Process

AI: Artificial Intelligence

AIP: Average Invalidity Percentage

AVP: Average Validity Percent

ANN: Artificial Neural Networks

ANP: Analytical Network Process

ASCE: American Society of Civil Engineers

B/C: Benefit/Cost ratio

BHI: Bridge Health Index

BIM: Bridge Inspection Manual

BMS: Bridge Management System

CBR: Case-Based Reasoning

CIRC: Canadian Infrastructure Report Card

CMI: Cote de Matériau Intégré

CoRe: Commonly Recognized

DIS: Disincentives

DOT: Departments of Transportation

EAs: Evolutionary Algorithms

FHWA: Federal Highway Administration

FO: Functionally Obsolete

FST: Fuzzy Set Theory

GA: Genetic Algorithm

GPR: Ground Penetrating Radar

HCP: Half-Cell Potential

HER: Hierarchical Evidential Reasoning

HOQ: House of Quality

ICI: Integrated Condition Index

IDC: Ideal Deterioration Curve

IE: Impact Echo

INC: Incentives

IR: Infrared thermography

KPI: Key Performance Indicator

LCCA: Life-Cycle Cost Analysis

LOS: Level(s) of Service

MAE: Mean Absolute Error

MRR: Maintenance, Repair and/or Replacement

MSL: Minimum Service Life

MTO: Ministry of Transportation of Ontario

MTQ: Ministère des Transports du Québec

NBI: National Bridge Inventory

NDE: Non-Destructive Evaluation

NPV: Net Present Value

PBC: Performance-Based Contracting

PDC: Predicted Deterioration Curve

PI: Performance Indicator

PMI: Project Management Institute

QFD: Quality Function Deployment

RAC: Rehabilitation Actions Costs

RMSE: Root Mean Squared Error

SD: Structurally Deficient

SL: Service Life

SR: Sufficiency Rating

TRAC: Total Rehabilitation Actions Costs

TQM: Total Quality Management

UDC: Updated Deterioration Curve

UPE: Ultrasonic Pulse-Echo

USL: Useful Service Life

VI: Visual Inspections

WCCM: Weighted Comprehensive Criterion Method

WDF: Weibull Distribution Function

CHAPTER 1: INTRODUCTION

1.1. BACKGROUND AND RESEARCH MOTIVATION

Deterioration is the foremost problem that affects the performance of any structure during its operation. It occurs due to aging, excessive usage, poor maintenance, environmental impacts, and other factors. Because of deterioration, conditions of more than 26% of Canada's bridges are rated fair, poor, and very poor and an investment of $50 billion is required to replace all of these bridges (CIRC 2016). The aging problem of bridge infrastructure is similar in the United States where the American Society of Civil Engineers (ASCE) report card graded bridges' conditions as "C+" which refers to a mediocre condition that requires immediate attention (ASCE 2017). According to the Federal Highway Administration (FHWA), around 24% of the total bridge decking area in the US is either structurally deficient (SD) or functionally obsolete (FO) (FHWA 2016). ASCE estimated that $20.5 billion of annual investment is needed to eliminate the country's bridge deficient backlog by the year 2028. The largest portion of this expected expenditure is allocated to bridge decks (Dinh et al. 2015). With a large number of aging bridges in North America and the growing problem of deterioration across the globe, managing bridges have been the subject of ongoing research. There is an overwhelming amount of maintenance, repair, and replacement (MRR) activities to be done but the resources (funding, staffing, equipment) available are too limited and selecting a suitable MRR strategy to achieve a better standard of infrastructure facilities is one of the most challenging tasks for decision makers. Limited budget is even making the decision-making process more challenging. The growing needs and limited resources to maintain the transportation networks have resulted in motivating transportation agencies to expand the amount of contracting they do. Projects that have been outsourced under strong competition and with longer durations tend to perform better (Anastasopoulos et al. 2010).

With the traditional method-based contracts, the owner agency specifies techniques, methods, materials, quantities, along with the time that the maintenance activities should be executed (Alyami and Tighe 2013). This causes a limitation on the use of new materials and innovative techniques and technologies while maintaining the infrastructure asset (Panthi et al. 2008). Fulfilling required performance level through innovating in such new and unproven technologies can be achieved by extending the short maintenance period to a longer period so that the performance levels can be satisfactorily measured over a longer duration (Panthi 2009). This type of agreement is called Performance-Based Contracting (PBC) which is a type of contract that pays a contractor based on the results achieved, not on the methods for performing the work. Such contracts allow the maintenance contractors to innovate and improve the efficiency of the services provided to the public (de la Garza et al. 2009). Incentives are introduced to the contractors, in this type of contact, as an increase of payments owing to exceeding on a pre-defined performance measure or indicator. However, the maintenance contractors will be penalized for failing to comply with performance indicators or to rectify revealed deficiencies promptly. By looking into the literature, it is observed that many transportation agencies established and defined many desired performance indicators extensively for roads maintenance (Haas et al. 2009; Alyami and Tighe 2013; Galenko et al. 2013; Abu Samra et al. 2016). Yet, in the area of bridges management, only a few attempts have addressed these indicators. Having no proper decision support framework that identifies performance indicators, their Levels of Service (LOS), and defines contractors' payments and penalties is challenging transportation agencies and maintenance contractors in making successful contractual agreements using innovative contracting (PBC).

Selecting maintenance, repair or replacement activities is based on current and future element conditions. Thus, condition assessment and deterioration modeling should reflect the accurate performance of the bridge. In North America, the commonly used techniques to assess bridge conditions is through visual inspection and close observation to bridge elements because it is inexpensive. Visual examinations involve using specific techniques to provide valuable information on bridges physical condition. This inspection process can evaluate defects such as cracking and spalling. However, this practice is still limited to detect surface defects and external flaws. Subsurface defects are mostly measured with the aid of Non-Destructive Evaluation (NDE) techniques. Moreover, NDE technologies are involved in objectifying the inspection process and making it more reliable. These technologies are promising in term of providing improvements to the traditional inspection processes; however, no integrated method has the capability of assessing the bridge condition in terms of detecting surface and subsurface defects. Although condition assessment is performed regularly, some inspection data may be missing or unavailable. Deterioration modeling can be used to estimate the current condition if such data is not existing; additionally, deterioration models can predict future conditions. In any Bridge Management System (BMS), it is essential to include an integral deterioration model to forecast the future conditions of a bridge structure as it will help in planning and budgeting purposes. Besides, accurate deterioration module throughout the bridge life-cycle is a necessity for determining the appropriate MRR decisions.

1.2. PROBLEM STATEMENT

The challenge of maintaining the transportation infrastructure within the acceptable limits of safety and serviceability while the available budget is limited has promoted innovative contracting approaches. Transportation agencies have increased private sector involvement through performance-based contracts or long term warranty contracts. Since PBCs are relatively new contractual arrangement considered by departments of transportation, there are very limited studies to assess this approach and to integrate it into the decision-making process. For instance, the standard approach for determining the optimal schedule of rehabilitation during the warranty period discussed mostly in the literature is based on the life-cycle costing (Panthi et al. 2008). Additionally, the availability of reliable and comprehensive sets of guidelines to evaluate the effectiveness and efficiency of this type of contract is limited (de la Garza et al. 2008; de la Garza et al. 2009). Therefore, there is a need to study potential of integrating the new contractual arrangements within the design of decision support systems including the development of a framework that identifies performance indicators, the establishment of a payment system, and the selection of a maintenance contractor in order to facilitate effective maintenance, repair and replacement (MRR) strategies while maintaining the desired bridge performance. Furthermore, the development of a decision support methodology to select proper MRR strategies requires enhanced methods for condition assessment and deterioration modeling.

1.3. RESEARCH OBJECTIVES

The main objective of this research is to develop a decision support method for concrete bridge maintenance, repair and replacement (MRR) strategies under Performance-Based Contracting (PBC). The following sub-objectives are developed for bridge decks:

1. Develop an integrated condition assessment and rating model.
2. Develop a forecasting model to assess bridge condition reliability and predict future deterioration/improvement of the bridge deck.
3. Develop short- and long-term optimized rehabilitation plans.
4. Design a PBC-based framework for rehabilitation decisions implementation.

1.4. SUMMARY OF RESEARCH METHODOLOGY

A comprehensive research methodology of this study is described in chapter 3. Yet, the following steps summarize the entire research methodology:

Step 1: A comprehensive state-of-the-art literature review is conducted where previous efforts related to current practices and methods for bridges condition assessment, deterioration models, and decision making for bridge management are studied. Then, performance-based contracting (PBC) is reviewed.

Step 2: Bridge deck common defects are identified by examining manuals of practice adopted by different departments of transportation. Then, the correlation between the identified defects is established based on experts' opinion to discover the relationship degree between these defects.

Step 3: A condition assessment and rating model is developed based on the identified defects and a Quality Function Deployment (QFD) method where surface and subsurface defects are assessed by integrating the dominant visual inspection practice with Ground Penetrating Radar (GPR) technology to establish an integrated bridge deck condition rating system using *k*-means clustering technique.

Step 4: Deterioration curves using Weibull Distribution Function (WDF) method is developed to predict bridge deck deterioration whether any rehabilitation action is performed or not using established ideal, updated and predicted curves.

Step 5: Models for identifying types of rehabilitation, condition improvements, and associated costs are then formulated in order to build the decision-making support tool for selecting suitable maintenance, repair or replacement (MRR) action.

Step 6: An optimization model under PBC setting is developed for selecting the best iteration of MRR actions based on identified performance trigger through a pre-defined maintenance period using a genetic algorithm (GA).

Step 7: A framework based on Performance-Based Contracts (PBC) is proposed for executing long term performance-based bridge maintenance works including identifying performance indicators and their Levels of Service (LOS), establishing a payments system, bidding preparation, selecting a maintenance contractor, and monitoring performance in order to assist the transportation agencies and maintenance contractors in arriving at a fair contract value and facilitate a successful implementation of the PBC. The

Fuzzy Set Theory (FST) is applied to help in minimizing the vast amount of error due to imperfect knowledge and human subjectivity while grading performance indicators.

1.5. THESIS ORGANIZATION

The thesis structure consists of seven chapters which are summarized as follows:

Chapter 1: introduces the research endeavor where it highlights the research motivation and problem statement. Then, it states the research objectives and summarizes the proposed methodology.

Chapter 2: reviews the literature where deterioration of concrete structures (i.e., bridges) is discussed. After that, current practices and methods for bridge condition assessment are reviewed. Common deterioration models related to bridges are also studied. Then, previous works on decision making for bridge management are highlighted. Afterward, performance-based contracting for maintenance is explained. At the end of each section, the limitations are addressed and research theory methods and techniques are introduced to overcome these limitations.

Chapter 3: describes the proposed methodology in detail. First, concrete bridge deck defects are identified. Second, NDE techniques are compared to augment the visual inspection technique while assessing the condition of concrete bridge deck. Then models for i) condition assessment, ii) deterioration and iii) MRR decision-making and optimization are developed reaching to the development of the decision support tool.

Chapter 4: explains how data are collected and analyzed where it consists of three main datasets collection followed by data analysis for case studies.

Chapter 5: illustrates the implementation of the resulting model on several case studies for bridges located in Quebec, Canada. The obtained results were discussed and, in selected cases, the results were validated with the real values and further compared with other approaches from the literature.

Chapter 6: introduces a performance-based contract framework for maintaining bridges. The framework consists of six main stages to facilitate the implementation of the PBC in bridges asset.

Chapter 7: wraps up the thesis with summary and conclusions, research contributions, limitations, and future recommendations.

CHAPTER 2: LITERATURE REVIEW

2.1. INTRODUCTION

Bridges, roadways, transit, water, and sewer networks are main components of a civil infrastructure system. Billions of dollars are invested annually in infrastructure assets in Canada and the United States to cope up with growing population and to maintain serviceability and safety. Civil infrastructure systems contribute to social and economic welfare through serving a large number of population and businesses. Thus, evaluating these systems' condition and performance is a necessity. In North America, infrastructure condition is reported by a grade or percentage, similar to schooling system, which known as infrastructure report cards. Table 2.1, represents the infrastructure ratings for Canada and the US.

Table 2.1: Infrastructure Ratings for Canada and US

Description	**Canada**	**United States**
Fit for the Future	80% or Higher; Very Good	A; Exceptional
Adequate for Now	70% to 80%; Good	B; Good
Requires Attention	60% to 69%; Fair	C; Mediocre
At Risk	50% to 59%; Poor	D; Poor
Unfit for Sustained Service/ Purpose	50% or Less; Very Poor	F; Failing/Critical

In Canada, findings show that around one-third of the Canadian municipal infrastructure that includes drinking water, wastewater, stormwater, municipal roads, and bridges are assessed in between fair and very poor condition as per the Canadian Infrastructure Report Cards (2012; 2016), increasing the risk of service disruption. Besides, the total replacement value of all infrastructure assets in the 2016 year report was estimated to be around $1.1

trillion stated by the same report. In the United States, the American Society of Civil Engineers (ASCE) report card shows an overall rating for America's infrastructure is poor. Furthermore, the cost to improve infrastructures including solid waste, drinking water, wastewater, roads, bridges, and rail is estimated at $3.6 trillion (ASCE 2017).

Historical and forecasted figures indicate that 'transportation' sector forms the largest portion of the Canadian infrastructure industry value. Roads and bridges segments have the largest portion of the transport infrastructure representing the highest worth share according to the estimates by the Business Monitors International report on Canadian infrastructure (BMI 2013). However, large parts of these infrastructures were constructed during the 1960s and 1970s. Accordingly, they are facing an increasingly deteriorating problem (Dori et al. 2011). Many of the nation's bridges are approaching or exceeding their design life where more than 26% of Canada's 75,000 highway bridges fall under fair, poor and very poor categories and a value of $50 billion is required to replace all of these bridges assets (CIRC 2016). In 2016 Quebec province alone estimated $854 million asset maintenance deficit on the ministry's bridge network (Quebec Ministry of Transportation (MTQ) 2016). The aging problem of the bridge infrastructure is similar in the US as the average age of the nation's 607,380 bridges is 42 years compared to their service life of 50 years as stated in ASCE report card. The report also graded bridges' conditions as "C+" which refers to mediocre that requires attention (ASCE 2017). The growing problem of the deterioration of the bridges imposes challenges on ministries of transportation to maintain and preserve them. Statistics show that about 24% of concrete bridge decks in the US is either structurally deficient (SD) or functionally obsolete (FO) as of December 2013.

ASCE estimated that $20.5 billion of annual investment is needed to eliminate the country's bridge deficient backlog by 2028. The largest portion of this expected expenditure is allocated to bridge decks (Dinh et al. 2015). With this huge number of aging bridges in North America, Bridge Management Systems (BMSs) have been the subject of ongoing research. Extensive effort has begun to develop and use BMSs to facilitate managing bridge infrastructures as a developed decision support tool that helps in determining how and when to carry out all activities related to Maintenance, Repair, and Replacement (MRR) of bridges. Such tool aims to identify future funding needs as well. Many BMSs have been developed all over the world. It is noted that each bridge management system has its own decision-making methodology. However, all these systems are developed to address problems related to defects and deterioration of bridges. In general, BMS facilitates interconnecting three scenarios for bridges as a particular domain of asset management; these are: i) condition evaluation, ii) prediction, and iii) decision-making and optimization.

In this literature review, basic causes and effects of concrete bridge deterioration are explained. Afterward, current practices of bridge condition assessment are discussed where previous and related work on visual inspection and Non-Destructive Evaluation (NDE) techniques are enlightened. Then, common bridge deterioration modeling techniques are highlighted. After that, related bridge maintenance decision-making and optimization is emphasized. Last but not least, performance-based contracting (PBC) for maintenance has been introduced. Performance indicators are defined and PBC payments system has been underlined. At the end of each section, the research theories and techniques that are used in this research are proposed. Finally, research gaps are summarized. The literature review flowchart is illustrated in Figure 2.1.

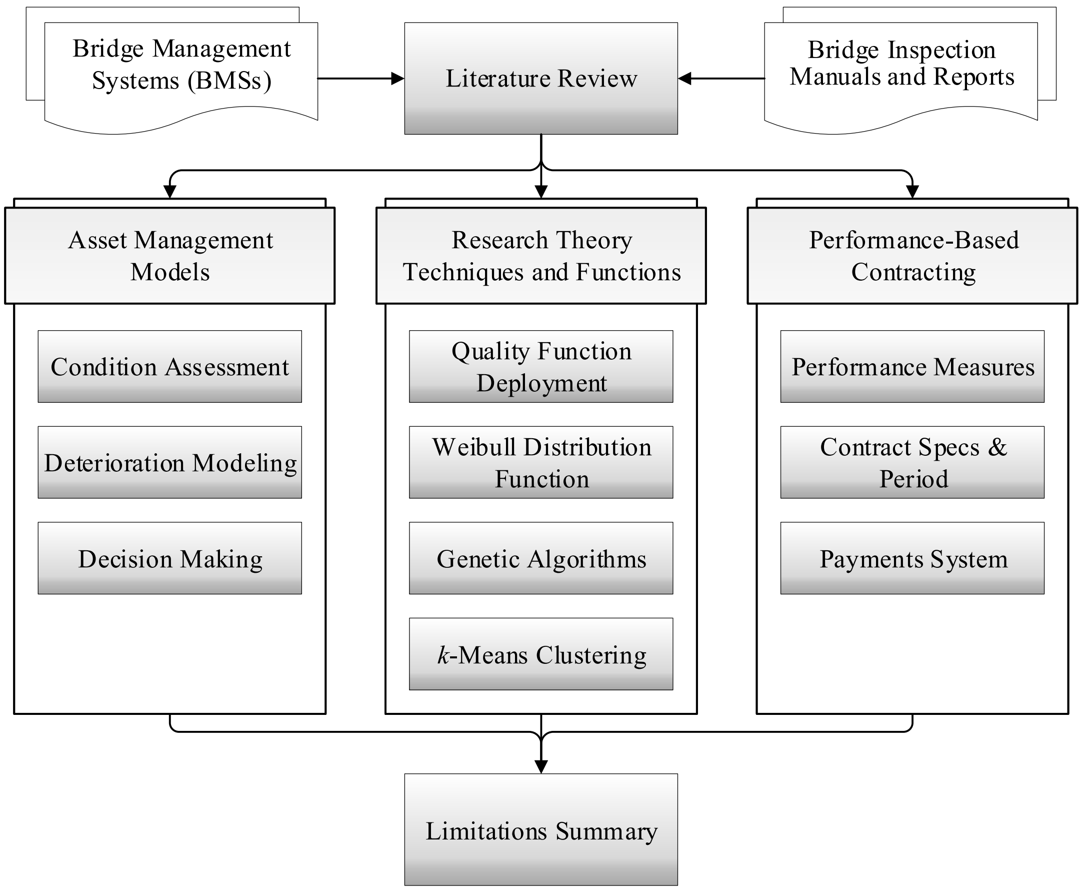


Figure 2.1: Literature review flowchart

2.2. DETERIORATION OF CONCRETE BRIDGES

Large parts of infrastructures were constructed during the 1960s and 1970s. Accordingly, they are facing an increasingly deteriorating problem (Dori et al. 2011). Age is one of many factors that decreases infrastructures' robustness. Along with the effect of other factors like poor maintenance, external environmental conditions, and extreme natural hazards, infrastructure performance is reduced for users. Such factors act as contributors to concrete bridge deterioration and cause vulnerability to any infrastructure. Similar to any reinforced concrete structures, bridges deteriorate over time due to degradation caused by chemical (alkali-silica reaction, carbonation, corrosion, crystallization, leaching, salt and acid action), physical (temperature gradient, fatigue, overloading, shrinkage, freeze-thaw cycles, etc.) and even biological (accumulation of organic matter, living organisms, etc.) mechanisms (Gucunski et al. 2010). It is important to understand different deterioration processes as each process leads to different types of defects such as cracking, scaling, spalling, concrete delamination and corrosion of reinforcing steel. Figure 2.2 shows some of the defects in bridge deck elements. It is also important to identify the basic causes of deterioration. It is evident that some mechanisms affect the reinforcement and others the concrete itself, yet all degradations mechanisms lead to structure vulnerability, and thus reduce the infrastructure reliability.

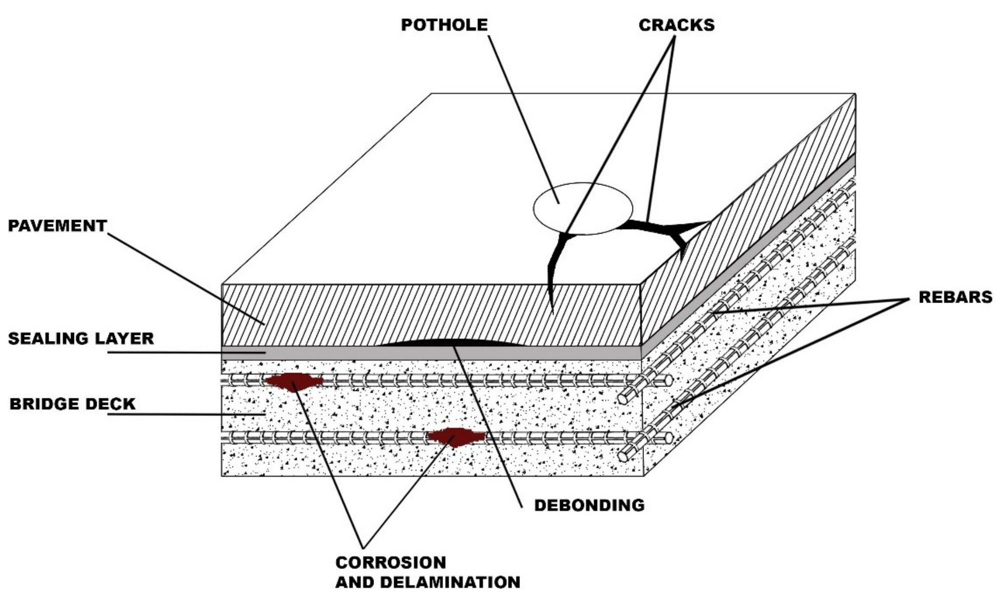


Figure 2.2: Some defects in concrete bridge deck elements

Corrosion of steel reinforcement, for instance, is considered as the most leading cause of deterioration (Bolar et al. 2013) and has always been viewed as the biggest problem that reasons in many consequent damages as the structure ages. Increased concentration of chloride ions over the top steel bars results in a higher negative charge (anode) while other less chloride-infected regions like bottom bars with the less negative charge create (cathode). Available water in the poles acts as an active electrolyte which allows electrons and OH^- ions to flow between anodes and cathodes forming an electric cell and makes the electrochemical process of corrosion to begin. Typical electrochemical corrosion process is illustrated in Figure 2.3.

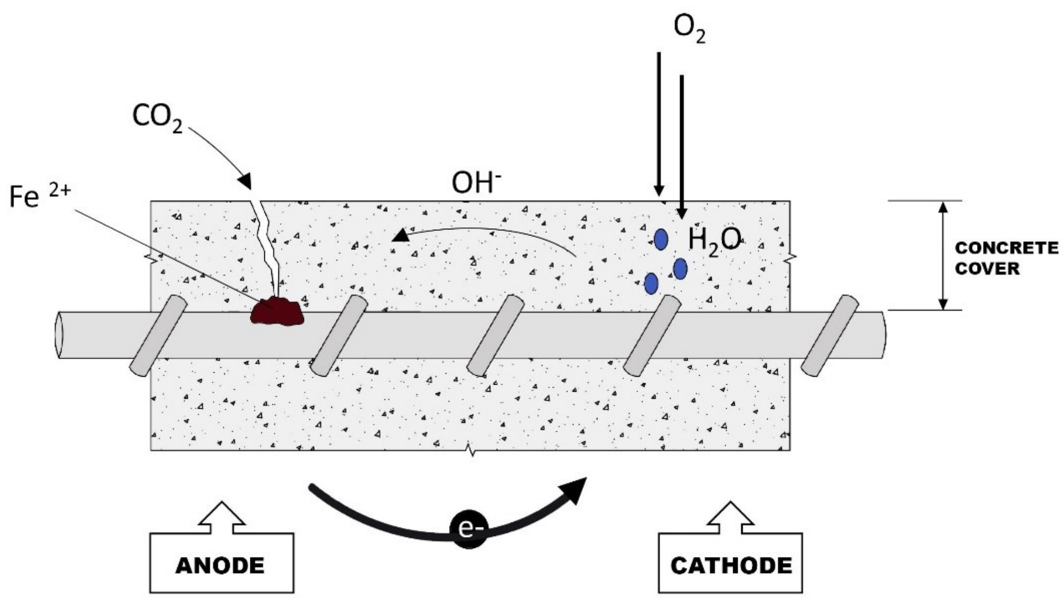


Figure 2.3: Typical electrochemical corrosion process

The final product of this repeated process is rust. However, deterioration comes from the fact that rust occupies much larger volume than the original steel over the years. This causes internal stresses in the surrounding concrete leading to cracking. As corrosion gets more severe, those internal cracks progressively cause partial separation of concrete at the level of reinforcement known as delamination. Several delaminated areas eventually form spalling of concrete which results in structural disintegration. Cracking, delamination and other discontinuities can occur due to overloading as well. Such defects in some cases could threaten public safety. Examples of cracking, spalling and rebar corrosion are shown in Figure 2.4.

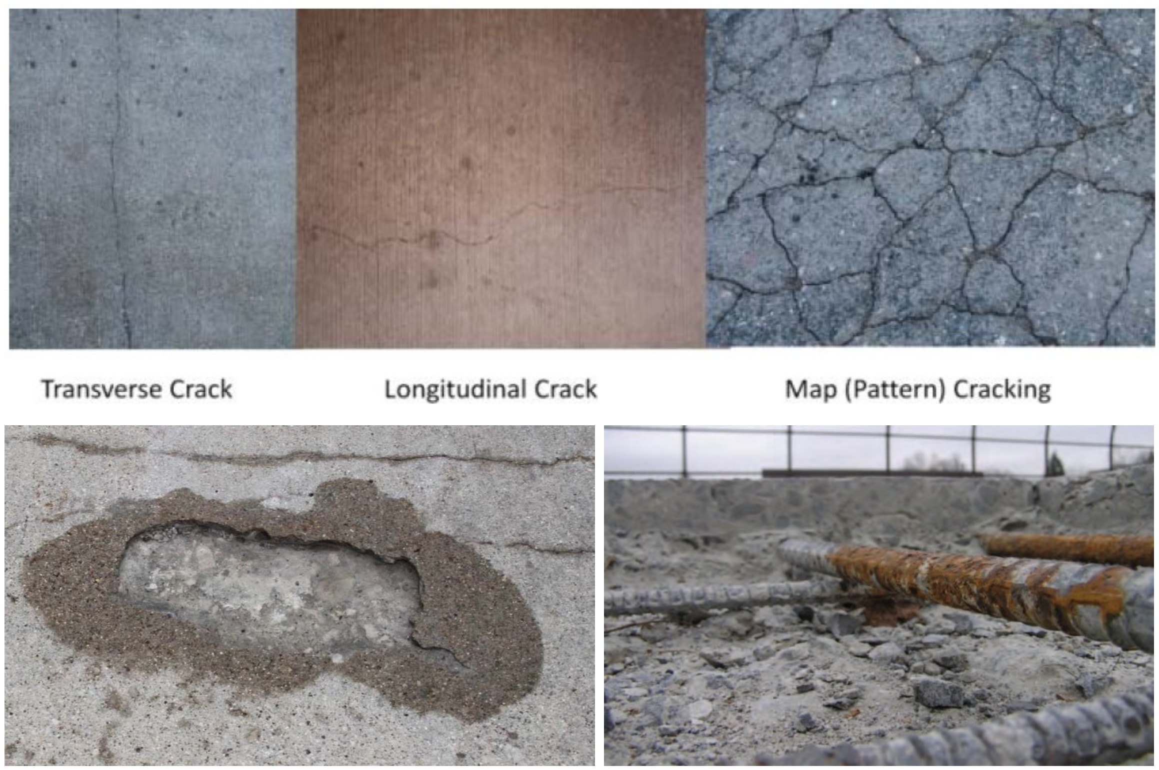


Figure 2.4: (a) Classification of concrete cracks (b) Spalling (Hoensheid 2012) (c) Rebar corrosion (Gucunski et al. 2013)

2.3. CONDITION ASSESSMENT OF BRIDGES

Bridge structures play a critical role in the transportation system as they serve millions of people on a daily basis. Any failure in these structures will result in both human life and economic loss. Consequently, condition assessment is performed on routine or scheduled basis to ensure public safety and prevent such catastrophic events. Bridge assessment is mainly an interpretation that identifies the appropriate Maintenance, Repair or Replacement (MRR) action. With the increasing number of deteriorated bridges in Canada, the US and around the globe, condition assessment techniques of concrete bridges are evolving.

2.3.1. Current Practices and Methods for Condition Assessment

Investigation of defects and determination of their severity level are the main objectives of bridge condition assessment. Detailed inspections are regularly conducted in order to discover serious defects and to evaluate the degree of bridge deterioration. Besides, emergency or ad hoc inspections due to specific defect are carried out after coincidences or natural disasters such as earthquakes. Concrete bridges are exposed to various forms of deteriorations. Some can be visible on the surface while others are hidden beneath it.

In North America, the commonly used techniques to assess bridge conditions is through visual inspection and close observation to bridge elements because it is inexpensive and requires a minimal level of experience. Visual examinations involve using specific techniques to provide valuable information on bridges physical condition. This inspection process can evaluate surface defects such as cracking and spalling. Hammer sounding and chain dragging as shown in Figure 2.5 are the most commonly used techniques to determine subsurface defects such as delamination where boundaries of delaminated areas within the concrete slab are measured. If severe damages are identified during the visual inspection, a more in-depth condition survey is conducted with the aid of destructive and non-destructive techniques. These two types of techniques are used to assess or inspect the subsurface condition of bridges as reinforcing steel corrosion. The first type is destructive techniques; coring test is the most common test belonging to this class of inspection techniques. The destructive techniques provide direct and quick results, but they are expensive, time-consuming and have a destructive nature; therefore, they cannot be applied on a regular basis. In addition, the results of these techniques assume that the rest of the material is having the same properties of the tested parts which are not always authentic. Thus, there is a need for a non-destructive type of testing (Yehia et al. 2007). The NDE techniques are less expensive; however, the results are mostly indirect and required some interpretation to get useful outputs.

Figure 2.5: Hammer sounding and chain-drag testing (Hoensheid 2012)

In order to enhance the current inspection practices, a detailed bridge element inspection system called 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. The new guide was built on the concept of element-level condition rating where an in-depth assessment of bridge elements is captured. Recently, the Federal

Highway Administration (FHWA) has started work on updating the widely used National Bridge Inventory (NBI) coding guide to incorporate the newly developed guide for the detailed bridge element inspection and to reflect the condition and performance of highway bridges accurately. In parallel, many transportation agencies supported the adoption and implementation of the detailed element inspection because of its ability to assist in providing sufficient MRR decision-making and for later analysis in deterioration curves and performance models.

By looking at the literature, previous research works on condition assessment can be categorized into two groups. The first is based on the visual inspection where researchers try to improve the current practices for condition assessment. According to Bolar et al. (2013), the condition assessment can be made more effective by determining the condition of groups of elements classified based on their resilience. In his paper, he proposed a Hierarchical Evidential Reasoning (HER) framework for the condition assessment of bridge where it is classified into primary, secondary, tertiary and life safety-critical elements. One of the major advantages of using HER is that it can deal with incomplete and conflicting evidence without making a strong assumption about missing data as required in other soft computing methods. Moufti et al. (2014) enhanced the application of HER approach by proposing an assessment tool that can handle the prescribed uncertainties in the bridge inspection process and objectively translate the real condition of a bridge through further in-depth measurements. In his model, he hierarchy structured several levels of a concrete bridge under assessment; namely: bridge components, structural elements, and most particularly, the measured defects. Other researchers try to reduce the uncertainties associated with visual inspection and evaluation subjectivity by applying the Fuzzy Set Theory (FST). Several studies on applications of FST with condition assessment of concrete bridges have been reported. Condition evaluation of existing reinforced concrete bridges using fuzzy based analytic hierarchy approach was proposed by Sasmal and Ramanjaneyulu (2008). FST-based evaluation system for existing bridges was proposed by Chen (2009) where he emphasis on the use of fuzzy sets to represent bridge condition ratings instead of precise numerical numbers. Jain and Bhattacharjee (2011) applied fuzzy concepts to visual assessment of deteriorating specific distress manifestation in reinforced concrete structures. Among the drawbacks of the above mentioned approaches are: i) the provided condition assessment depends mainly on visual inspection which can be imprecise, ii) weights of bridge elements and defects are measured by simple techniques such as Analytical Hierarchy Process (AHP) without considering interdependency, and iii) some of the studies consider specific deterioration mechanisms and certain type of defects.

The other research group utilizes technologies for condition assessment. Advanced technologies have been utilized to provide information about deteriorations of bridge elements. For surface deteriorations such as cracking, a digital image processing was developed by Adhikari et al. (2014) for crack quantification in such a way that it mimics the on-site visual inspections. Such developments will resolve the subjectivity problem, but still it is limited to certain type of defects. For subsurface deteriorations, some Non-Destructive Evaluation (NDE) technologies are used to evaluate subsurface conditions on bridge decks. For instance, Washer et al. (2010) used Infrared thermography (IR), IR mostly used to detect voids and delamination in concrete bridge decks through measuring the radiant energy emitted from the surface. Gucunski et al. (2010) used Ultrasonic Pulse-Echo (UPE) and Half-Cell Potential (HCP). UPE technology uses ultrasonic (acoustic) stress waves assess in detecting defects in concrete elements, debonding of reinforcement bars, shallow cracking, and delamination. HCP measures the potential corrosion of steel reinforcement and prestressed concrete structures. Chase (2015) used Impact Echo (IE), IE detects and characterizes delamination within concrete bridge decks. Tarussov et al. (2013), Dinh et al. (2015), Abouhamad et al. (2017) used Ground Penetrating Radar (GPR), GPR utilizes electromagnetic (EM) waves that can identify the subsurface defects locations for both corrosions of steel and also some concrete delamination. Table 2.2, summarizes the main limitations of the above mentioned technologies. In general, the main disadvantages of using NDE technologies are relatively the high cost and complex data interpretation in addition to detecting certain type of defects.

Table 2.2: Main NDE Technologies Limitations

NDE Technology	Main Potential Limitations
Infrared thermography (IR)	-Mostly detect voids and delamination in concrete
Ultrasonic Pulse-Echo (UPE)	-Detect debonding of reinforcement, shallow cracking, and delamination
Half-Cell Potential (HCP)	-Evaluate the probable steel corrosion activity in concrete structures
Impact Echo (IE)	-Detect only delamination within bridge decks
Ground Penetrating Radar (GPR)	-Limited to steel corrosion and potential delamination

It has been observed from the current practices that the commonly used technique to assess bridge conditions is through visual inspection and close observation to bridge elements because it is inexpensive and requires a minimal level of experience. Information from routine visual inspections (VI) is being used to update lifetime reliability assessments and set life-cycle MRR strategies (Estes and Frangopol 2003). The visual examination provides valuable information on bridges condition. However, this practice is not always reliable because it is limited to detect surface defects and depends mainly on the inspector judgment while completing the visual inspection, which can be imprecise (Alsharqawi et al. 2018). Subsurface defects are mostly measured with the aid of NDE techniques. Moreover, NDE technologies are involved in objectifying the inspection process and making it more reliable. These techniques are becoming popular in augmenting the visual inspections (Alampalli 2010). Among the most effective technologies, the Ground Penetrating Radar (GPR) has been considered for many years as a highly promising technique for deterioration mapping (Alsharqawi et al. 2018). Further, the combined application of the GPR with the VI can improve the identification and quantification of the deck defects (Barnes and Trottier 2004). Thus, in this research, GPR is integrated with visual inspection technique to assess the condition of concrete bridge decks using Quality Function Deployment (QFD) theory in order to enhance the reliability of bridge condition assessment.

2.3.2. Quality Function Deployment (QFD)

Quality Function Deployment (QFD) theory was invented four decades ago. It is namely a key tool for application of concurrent engineering and implementing Total Quality Management (TQM). Historically, Japanese industry began to formalize QFD main characteristics in 1966. However, it is Yoji Akao who conceptualized QFD when he utilized this method to improve quality in products which was called *hinshitsu kino tenkai* (quality function deployment) (Kahraman et al. 2006). 'Quality Function Deployment' terms can be defined within the context of QFD as (Roberts 2014):

- *Quality:* meeting customer requirements
- *Function:* what must be done; focusing the attention
- *Deployment:* who will do it, when

QFD is a systematic method to achieve higher customer satisfaction through listening to the voice of customers. The method has been successfully used in many industries. Initially, it was used in the shipbuilding, automobiles, and electronics nevertheless, now it is hard to find an industry to which QFD has not yet been applied. Among the industries that QFD is used are transportation and communication, electronic and electrical utilities, software systems, manufacturing, services, education and research, and other industries including aerospace, agriculture, and construction (Chan and Wu 2002). In bridge management, the QFD concept is relatively new. According to Bolar et al. (2014), QFD has been applied only in three references closely related to bridge design and maintenance where Söderqvist and Vesikari (2003) used it in the European life-cycle management system (LMS) tool LIFECON, Ma et al. (2009) proposed a general framework using QFD in bridge life-cycle design in China, and Malekly et al. (2010) used QFD in evaluating conceptual bridge design. In addition to Bolar's (2014) application in inspection prioritizing and maintenance decision making of bridges. QFD method has main components which form the House of Quality (HOQ) skeleton. HOQ is a term associated with the QFD theory. It is the matrix where QFD is implemented. The components are described for the purpose of assessing the bridge condition as follows:

- *Customer demands (WHATs):* are the primary input for the HOQ. They are also called "demanded quality" in which they highlight the quality characteristics that should be paid attention to. In this research, WHATs are demonstrated by severity degrees in which it describes what the bridge condition ratings are.
- *Quality characteristics (HOWs):* are the characteristics that make a set of WHATs or severity degrees to be realized. Defects represent these characteristics in this instance where for each defect a corresponding severity degree is provided. This is how defects are used to utilize the ratings in the bridge condition assessment.
- *Relationship matrix:* is the rectangular area between the rows and columns that depicts the relationship between the WHATs and HOWs. Relationships within the matrix are condition ratings collected from inspection reports in this case.
- *Correlation matrix:* is the roof of the HOQ where quantitative correlations (interdependencies) between the characteristics (HOWs) are determined. These relationships are described by means of a matrix using a rating scale in which it describes the relationship between each attribute.
- *Absolute weights of WHATs:* are the results obtained from implementing QFD. These weights are the aggregation of each HOW (defect) to each WHAT (condition rating). It represents the impact of the defects on the bridge overall condition ratings.

2.3.3. Condition Rating Systems for Bridges

Once condition assessment is executed a score is provided to interpret the bridge condition rating. National Bridge Inventory (NBI) condition rating system is the oldest rating used in the US to evaluate the bridge under three main components, namely: deck, superstructure, and substructure. Two performance measures are deduced from the NBI ratings, namely: structurally deficient (SD) and functionally obsolete (FO). Another measurement from NBI data is called the sufficiency rating (SR) which is a numeric value that is used to allocate federal funds. The less a bridge's SR is, the more it is eligible for MRR funds. Although NBI program is wildly in use the US, its ratings and condition metrics (SD, FO, and SR) are not providing detail measurement and considered to be general in identifying maintenance strategies and cost estimates for federal or state funds distribution (Moufti 2013).

Due to NBI rating system limitations, FHWA and the California Department of Transportation (CalTrans) decided to develop the first bridge management system in the US (Pontis) where Commonly Recognized (CoRe) elements level condition is described instead of dividing a bridge into several main components. Although deterioration of bridges is a continuous process, the Federal Highway Administration (FHWA) uses an ordinal bridge deck rating system (FHWA 2005). FHWA bridge deck ratings, for example, range from 9 to 0, with 9 representing an excellent condition or new condition and 0 representing a deck that has deteriorated to a failed condition. Table 2.3 illustrates the FHWA condition ratings using the National Bridge Inventory (NBI) standards. Discrete ratings are used instead of continuous condition measures mainly to simplify the computational complexity of the MRR decision-making process (Madanat et al. 1995).

Table 2.3: FHWA Bridge Condition Ratings

Rating	Condition	Description
N	Not applicable	
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 or scouring
3	Serious condition	Loss of section, deterioration, 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 structure stability. Bridge is closed to traffic but corrective action may put back in light service.
0	Failed condition	Out of service and beyond corrective action

Later, the California Department of Transportation came up with a new concept called Bridge Health Index (BHI) based on element level condition data obtained from implementing Pontis system. The BHI is basically a ranking system for bridge maintenance ranging between 0-100, with 100% indicating the best state and 0% indicating the worst. Caltrans has developed a visual representation of the BHI (Figure 2.6). The index accuracy comes from the fact of using element weighting factors aggregation where failure elements that could threaten the public safety, for example, receive more weight than the ones have a relatively little economic effect.

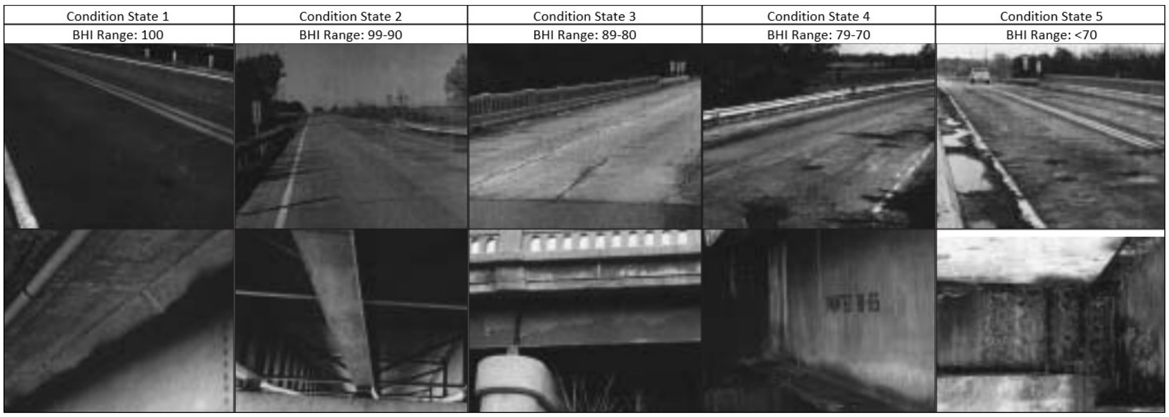


Figure 2.6: Visual representation of BHI values (Shepard and Johnson 2001)

Although the BHI enhances bridge performance measure, Dinh (2014) noticed that condition state weighting factors are simply calculated. Moreover, the values for the economic effect of element failure are difficult to obtain. Thus, it is concluded that a more appropriate method to calculate condition rates is needed in which weighting factors should take into account defects interdependencies at the element level.

In Canada, bridge management systems have been developed like most states in the US. Some Canadian transportation agencies adopt Pontis system with minor modifications to suit their inventories while others like Ontario, Alberta, Quebec and Nova Scotia provinces use their own condition ratings similar to the NBI ratings described previously. Hammad et al. (2007) reviewed Canadian BMSs in deep and compared between diverse provinces and territories management systems of bridge. In his research, he took a further step and proposed a unified National Bridge Inventory (NBI) development for Canada. Appendix A highlights the different condition ratings for the most popular BMSs based on the location where BMSs in USA, Canada, Australia, some Europe countries, Japan and South African are compared. While many condition ratings have been developed for bridges, most of these rating are computed based on visual inspections and the rating thresholds defining the severity of concrete deterioration are selected arbitrarily. *k*-means clustering, the utilized technique in this research to solve this problem, is explained below.

2.3.4. *k*-Means Clustering Technique

Among clustering formulations, *k*-means clustering is one of the most popular types of clustering algorithm because of its ease of implementation, efficiency, empirical clustering, and simplicity (A. K. Jain 2010). *k*-means clustering is a partitioning technique that is based on minimizing a formal objective function. The problem of object clustering was widely used and studied in various scientific areas such as machine learning (K. Thompson and Langley 1991), data mining and knowledge discovery (Fisher 1987; Huang 1998), and pattern recognition and pattern classification (Sung and Poggio 1998; Kanungo et al. 2002; Duda et al. 2012). The three main user-defined parameters required to perform *k*-means clustering are i) the number of clusters k, ii) cluster initialization, and iii) distance metric. In general, *k*-means clustering starts by randomly selecting k initial cluster centers c_j and adjusting them by repeating the following steps: i) calculate the Euclidean distance using Equation 1, where x_i is the data point and c_j is the centroid of the cluster of j and ii) each cluster center c_j is updated to be the mean of its constituent points. The two steps are repeated until the centroids and points no longer move where the clustering process stops (Wagstaff et al. 2001).

$$d(x_i, c_i) = \left(\sum_{d=1}^{D} |x_{id} - c_{id}|\right)^{\frac{1}{2}} \tag{1}$$

where:

x_i = data point i;

c_j = centroid of cluster j;

d = d^{th} dimension; and

D = dimension of the data needed to be classified.

2.4. BRIDGE DETERIORATION MODELING

Periodic bridge inspection results and the produced condition ratings are essential to predict the future condition of the bridge elements, which in turns enables scheduling of the maintenance, repair, and replacement decisions. In addition, the accurate modeling of the deterioration process is important to allocate the available resources efficiently (Alsharqawi et al. 2018). Bridge elements or structures prone to rapid deterioration occur due to many reasons such as natural aging, exposure to the harsh environmental conditions, increasing load spectra, traffic wear and tear, or other deteriorating factors such as freeze-thaw cycling. Such factors act as contributors to concrete bridge deterioration and cause vulnerability to any infrastructure. Concrete bridge structures are one category of civil infrastructure that deteriorate over time due to degradation caused by chemical, physical, and even biological mechanisms (Gucunski et al. 2010).

Modeling deterioration is essential for any bridge management system. Markov chain models are used in Pontis BMS element condition ratings to predict the performance of bridge elements and components. Transition probabilities are used to predict the movement from any given state to another (Saydam and Frangopol 2015). Deterioration modeling for bridges is considered to be a complex process as it is occurring at different element levels. For instance, bridge deck deteriorates faster than any other bridge elements due to its direct exposure to traffic in addition to the other deteriorating factors. In general, modeling bridge deterioration can be categorized into deterministic, stochastic, and artificial intelligence models (Morcous et al. 2002; Morcous 2006; Abu Dabous 2008). By the time in which current and future conditions are available, decision-makers have to take the suitable MRR actions in which the allocation of scarce resources on these actions is optimized.

2.4.1. Common Deterioration Models

Deterministic deterioration models are developed using straight-line extrapolation, regression, and curve-fitting methods. The deterioration rate affecting bridge condition is calculated based on a mathematical or statistical formula where deterministic values express the output of each model. Supposing that the deterioration rate will continue, future conditions can be predicted. For a bridge, given the assumption that traffic loading and maintenance history follow a straight-line, the simplest condition prediction model that can be used is the straight-line extrapolation. One of the limitations of this method is that it cannot predict the rate of deterioration of a relatively new bridge. Regression models use the best curve fit to establish an empirical relationship between more variables than straight-line extrapolation (Morcous and Hatami 2011). In general, deterministic deterioration models are limited to predict short-term conditions only as the accuracy decreases significantly for long terms. Further, deterministic deterioration models are incapable of predicting the behavior of structures that have undergone some repair or maintenance to a certain extent. Due to the above mentioned reasons and in addition to the complexity and the interaction of several deterioration mechanisms of different bridge elements, it is unrealistic to model the deterioration process using a deterministic approach.

Since deterioration process in bridges has stochastic characteristics rather than a deterministic nature due to several complex deterioration mechanisms, probabilistic or stochastic models are being used to predict the conditions of bridge elements as a probabilistic estimate. Among these models, Markovian models have been used extensively in modeling the deterioration of infrastructure facilities. Transition probabilities are used to predict the movement from any given state to another (Saydam and Frangopol 2015). Stochastic models such as Markovian treat the facility as one or more random variables that capture the uncertainty and randomness of the deterioration process. In general, stochastic models can be classified into state-based or time-based models (Mauch and Madanat 2001). In state-based models, also known as Markov chains, deterioration models predict the probability that a facility will undergo a change in condition from one state to another at a given time using a set of explanatory variables like structure type, truck traffic, environment, and maintenance history. In time-based models, deterioration models predict the probability distribution of the time taken by an infrastructure facility to change its condition-state depending on the same set of explanatory variables described above. Many bridge management systems, such as Pontis, BRIDGIT, and the Ontario Bridge Management System, have adopted Markov chain models as a stochastic approach for predicting the performance of bridge components and networks. Although these models addressed the problem in the deterministic models by capturing the uncertainty of the deterioration process and accounting for the current condition in predicting the future one, they still suffer from the requirement of updating the transition probabilities after each inspection, maintenance or rehabilitation action which is time consuming.

Artificial Intelligence (AI) models exploit computer techniques that deal with intelligent behavior. Research in AI is focused on producing machines to automate tasks that require intelligent behavior, learning, and adaptation. AI techniques that have been used in deterioration modeling contains artificial neural networks (ANN), genetic algorithm (GA), case-based reasoning (CBR) and expert systems (Setunge and Hasan 2013). Although AI techniques have automated the process of finding the polynomial that best fits a set of data points, they still have some problems. For example, ANN have been criticized for being black boxes in which the mathematical mapping between inputs and outputs and the learning process cannot be explained. CBR suffers from subjectivity in determining the attribute weights and degrees of similarity in matching cases also it needs a large size of case library.

By looking into the literature, a number of bridge deterioration models have been developed to calculate the deterioration rates and to determine the bridge life-cycle for MRR needs using bridge inspection data. Research efforts were made in the area of bridge asset management in an attempt to improve the overall quality of BMS outcomes (Jiang and Sinha 1989). Reported models include Zayed et al. (2002) who developed and used regression analysis and Markov chain to predict a defined performance function for bridge protection systems based on historical data. While Bu et al. (2013) incorporated backward prediction to model bridge deterioration, many other researchers modeled deterioration stochastically based on Markov chains to predict future condition of reinforced concrete bridge elements. Among them are Lounis and Madanat (2002), Frangopol et al. (2004), Morcous (2006) Yang et al. (2009) and Li et al. (2016). AI is also used to develop bridge deterioration models where Lee et al. (2008) used ANN to improve the reliability of a Bridge Management System (BMS) and Morcous et al. (2002) used CBR for modeling the deterioration of concrete bridge decks. The drawbacks of the current concrete bridge deterioration models and the previously discussed ones are summarized as: i) deterministic deterioration models are limited to predict short-term conditions these models are incapable of predicting the behavior of structures that have undergone some rehabilitation actions, ii) stochastic Markovian models suffer from the complexity and time consumption while developing and updating the transition probabilities after each inspection, maintenance or rehabilitation action, and iii) AI models shortcoming is mainly the requirement of huge amount of inspection data. In order to overcome these limitations, the current research introduces the reliability function approach produced based on the Weibull Distribution

Function (WDF) to model bridge deterioration, which falls under the probabilistic deterioration models' category.

2.4.2. Weibull Distribution Function (WDF)

The Weibull Distribution Function (WDF) can be used as a condition life-cycle curve model. The Weibull statistical distribution represents the probability of time to failure of a component-section in service. The mathematical condition prediction model was originally developed by Grussing et al. (2006) using the Weibull probability distribution, but the Weibull distribution is named after Waloddi Weibull (1887-1979). Weibull analysis is one of the most popular methods for analyzing and predicting failures and malfunctions of all types. It has been widely implemented in many applications of different natures and for solving a variety of problems from many different disciplines (Jardine and Tsang 2013). As a condition prediction model for concrete structures, the Weibull function can be used to predict the structure life-cycle performance while assuming the following natural boundary conditions (Grussing et al. 2006):

- The condition curve is maximum (100) at or near start of the service life;
- The condition curve approaches the minimum state (0) asymptotically;
- The element deteriorates and the condition drops unless corrective action is performed; and
- As the structure deteriorates, reliability decreases.

Thus, the mathematical condition prediction model can be transformed into the following Equation:

$$C(t) = a \times e^{-(t/\beta)^{\alpha}} \tag{2}$$

where:

$C(t)$ = component condition index as a function of time;

t = number of years since the component was constructed;

e = exponential;

a = parameter related to initial steady state component condition index;

β = parameter related to service life adjustment factor; and

α = parameter to reflect accelerated deterioration factor.

Typical life of a product or component (e.g., concrete bridges life-cycle condition trend) can be seen in Figure 2.7 and is described by the following Equation:

$$f(t) = \frac{\beta}{\eta}\left(\frac{t-\gamma}{\eta}\right)^{\beta-1} e^{-\left(\frac{t-\gamma}{\eta}\right)^{\beta}} \; for \; t > \gamma \tag{3}$$

where:

β = shape/slope parameter, greater than zero;

γ = location parameter, greater than zero;

η = scale parameter; and

t = time.

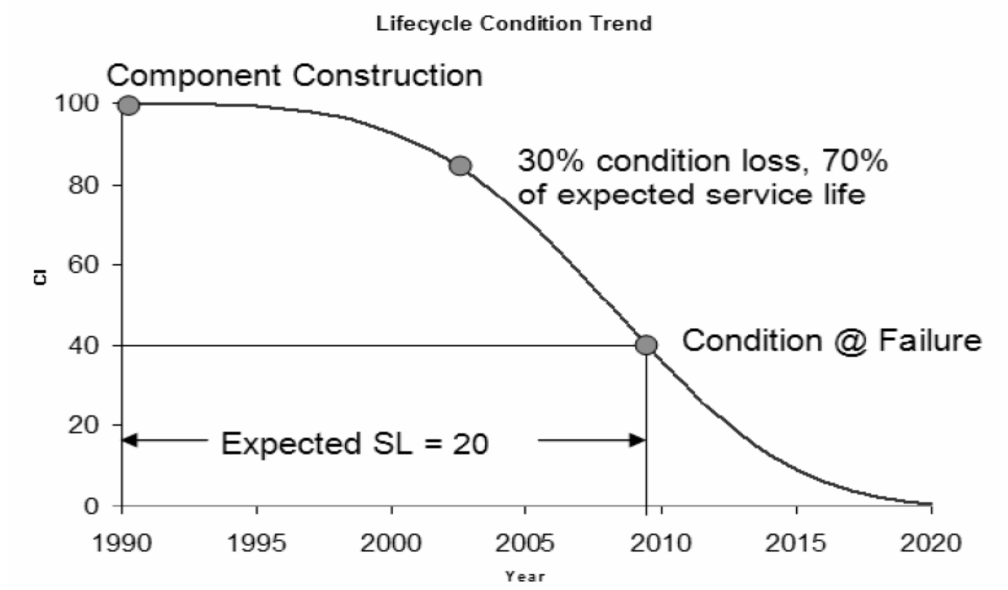


Figure 2.7: Condition prediction curve using Weibull distribution (Grussing et al. 2006)

The following Equation gives the cumulative Weibull distribution function defined by Semaan (2011):

$$F(t) = 1 - e^{-(\frac{t-\gamma}{\eta})^{\beta}} \tag{4}$$

By using appropriate reliability models, one can obtain information on the need and timing for MRR actions of bridges (Tabatabai et al. 2010). Reliability is simply one minus the cumulative distribution function. Hence, the reliability function for the Weibull distribution is given by Equation 5:

$$R(t) = 1 - F(t) = e^{-(\frac{t-\gamma}{\eta})^{\beta}} \tag{5}$$

According to Semaan (2011), the key in plotting $F(t)$ and $R(t)$ is the estimation of the parameters β, γ, and η. The shape parameter β is often referred to as the slope of the cumulative distribution function and $R(t)$:

- For $0 < \beta < 1$, $R(t)$decreases sharply and monotonically and is convex.
- For $\beta = 1$, $R(t)$decreases monotonically but less sharply than $0 < \beta < 1$ and is convex.
- For $\beta > 1$, $R(t)$decreases as time "t" increases. The curve goes through an inflection point after this point, then decreases sharply.

The location parameter γ, as the name implies, locates the distribution along the abscissa. Changing the value of γ has the effect of 'sliding' the cumulative distribution function and $R(t)$ either to the right ($\gamma > 0$), or to the left ($\gamma < 0$). When $\gamma = 0$, the distribution starts at t=0 or at the origin. Finally, the scale parameter η has the same effect on the cumulative distribution function and $R(t)$as the abscissa scale (time). Thus, η has the same units as time.

2.4.3. After-Repair Deterioration

Modeling condition improvement and predicting the condition and deterioration behavior of an asset after a rehabilitation or maintenance action is necessary. In bridge management, Hegazy et al. (2004), Elbeltagi et al. (2005) and Elbehairy et al. (2006) classified bridge rehabilitation or repair actions into light, medium, and extensive repairs. The improvement values of the bridge element (i.e., bridge deck) are graphed in Figure 2.8. For example, a medium repair (type 2) action increases the rating from 3 to 5, while extensive repair (type 3) raises the rating to 7 with an increase of 4 levels compared with the condition ratings before and after the repair actions.

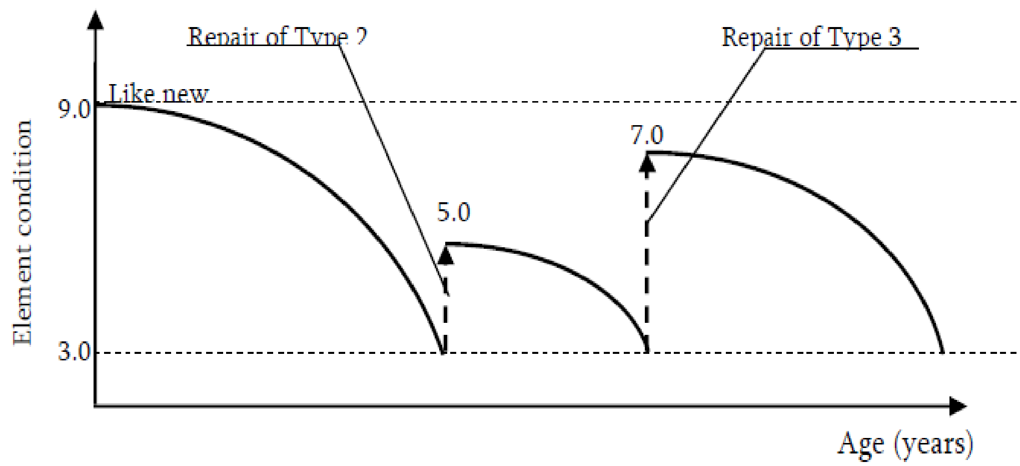


Figure 2.8: Impact of repair option on bridge deck condition (Elbehairy 2007)

The improvement condition ratings by the three types of repair actions relied on a study that was performed by Seo (1994). Seo's estimated repair improvements according to FHWA condition rates as per Table 2.4. In his study, the repair activities in the light category included patching, sealing, and clearing debris; a medium repair would be to strengthen or increase the thickness of a bridge deck; and an extensive repair would be full deck replacement.

It is noticed from Seo's study that the rehabilitated action of replacing a new deck does not revert the condition to its best condition as of newly constructed bridges. It was, also, reported in the literature that the deterioration rate of a rehabilitated asset is greater than that of a new constructed. Bolukbasi et al. (2006) compared the rates of deterioration for new bridge decks and those for reconstructed decks. In his study, he concluded that a reconstructed deck has at least a 25% shorter lifespan than new decks. However, assuming that the deterioration rate of a newly constructed and a rehabilitated asset are equal is a common practice (Elbehairy 2007).

Table 2.4: Impact of Repair Action on Bridge Deck Condition (Seo 1994)

Condition rating		**Condition rating after repair**		
		Good (8,7)	*Poor (6,5)*	*Deficient (4,3)*
Condition rating before repair	*Good (8,7)*	Light	–	–
	Poor (6,5)	Medium	Light	–
	Deficient (4,3)	Extensive	Medium	Light

2.5. BRIDGE DECISION-MAKING AND OPTIMIZATION

Decision-making in bridge management is done at two levels: project level and network level. Project level bridge management treats each bridge as an individual project where inspection, maintenance, repair and replacement actions (also referred to as "policies" in many literatures) are determined at the component level. Network level bridge management is concerned with all bridges in an agency's inventory. Its objective is to perform network analysis in order to maintain a pre-determined level of performance for all the bridges in that network and to determine the impacts of implementing or deferring action plans based on prioritizing funding allocations.

Based on the condition survey, appropriate corrective actions (i.e., maintenance, repair, and replacement) for the bridge are recommended by the decision makers. Strategic decision making for MRR activities has become a concern for many transportation agencies. The two main reasons behind that are: i) many bridges are deteriorating and aging; moreover, ii) resources are scarce and budget is limited. Most of the existing decision-making methodologies attempt to maintain or extend the bridge serviceability at a minimum total cost. These conflicting objectives have made the bridge management decision process very complex. Generally, decision-making approaches focus on one criterion which is optimizing the life-cycle cost. Abu Dabous and Alkass (2008) developed a decision-making approach for evaluating the available MRR alternatives for bridge improvement projects. The developed decision support tool considers multiple criteria for selecting maintenance actions such as agency cost, user cost, bridge safety, useful life and environmental impact.

MRR decision-making processes at the project level and the network level are very different. Selection of rehabilitation actions for an individual bridge is considered a project level decision, while prioritization of bridges for rehabilitation is considered a network level decision. Most of current BMS systems are developed to support either project or network level decisions and only lesser extent to support both (Elbehairy et al. 2006). The project level produces a set of candidate projects with costs and benefits that can readily be used in a network level priority setting and budgeting analysis (P. D. Thompson et al. 2003). By reviewing the literature, decisions regarding bridge management can be categorized into project level, network level or combined project and network levels decisions. The following reviews some of the work done in each category.

2.5.1. Decisions at Project, Network, and Combined Project & Network Levels

Decisions at the project level determine the selection of MRR actions considering how much is the cost associated with a rehabilitation action, what is the expected improvement from that action and when the best time for performing this action is. Elbehairy (2007) categorized project level decisions into Benefit/Cost (B/C) ratio and mathematical optimization techniques. B/C ratio technique is used to compare different rehabilitation strategies at the project level. Benefits gained from rehabilitation actions are estimated for bridges and then the bridge with the highest B/C ratio is selected. Another technique is the mathematical models where they provide exact/near exact set of solutions. Basically, a trade-off between the objectives and constraints are manipulated so that an optimal solution can be reached.

Selecting bridges for MRR actions are based on prioritization methods. According to Elbehairy (2007), network level decisions can be grouped into priority ranking and mathematical optimization techniques. Priority ranking techniques are based on calculating a crisp index value for each bridge and then sorting all bridges in descending order of their indices. Some decision-making processes for selecting MRR projects are made by sorting bridges based on their worst conditions. Projects will start with the worst condition index and continue until the available funds are exhausted. Alternative priority ranking techniques are based on condition- and sufficiency-rating systems. Condition-rating models sort the bridges according to their relative importance in the network while sufficiency-rating approach ranks the bridges based on a calculated numerical value that indicates whether the bridge can remain in service or not. Another type of priority ranking is the level of service deficiency rating (LOS). This approach evaluates the bridges based on different characteristics such as load capacity, clear deck width, and vertical roadway clearance. After evaluation, bridges are prioritized according to the degree in which a bridge is deficient in meeting it is intended function and the needs of public. The Benefit/Cost (B/C) ratio can be used to allocate funds for bridges in the network level. Alternatives are selected in descending order of their B/C ratios until the budget is exhausted (Elbehairy 2007). Mathematical optimization techniques are implemented at the network level to optimize the budget allocation for the bridges network while considering other constraints including but not limited to life-cycle costing, LOS, and the minimum allowable condition rating. Integer programming and evolutionary algorithms are used as optimization techniques.

Ideally, project level and network level decisions should be integrated. The output from the project level represented by rehabilitation action cost, improvement and occurrence time for various bridges can then be combined with network level decisions related to prioritizing the bridges. However, the problem with incorporating project level into network level decisions is that it complicates handling the extremely large size optimization problem. Lately, several attempts have been made to combine both levels using non-traditional optimization tools. Most of these attempts used genetic algorithms technique (Elbehairy 2007). The growing needs and limited resources to maintain the transportation infrastructure have resulted in motivating provincial and state agencies to expand the amount of contracting they do. Thus, this research is proposing a decision support tool for MRR activities under Performance-Based Contracting (PBC) arrangements. The tool provides an optimized rehabilitation program for transportation agencies and maintenance contractors using genetic algorithms technique.

2.5.2. Genetic Algorithms

Genetic algorithms are robust and practical search-based optimization technique that are highly recognized for their computational efficiency (Morcous and Lounis 2005). The original genetic algorithms were developed by Holland (1975) in the early 1970s based on the principles of natural selection and genetics. Genetic Algorithms (GAs) belong to a larger class of evolutionary algorithms (EAs) and rely on bio-inspired operators such as selection, crossover, and mutation (Mitchell 1998). GAs mimic the Darwinian principle of "survival of the fittest." Basic genetic algorithm operations are illustrated in Figure 2.9 and can be generalized to the following steps (C. Lee and Kim 2007):

1. Organize initial population $P(0)$ of solutions, which includes M number individuals as the initial generation, $g = 0$;
2. Evaluate the fitness of all individuals in $P(0)$;
3. Finish operations when the solution has been found or the designated number of generation exchanges is reached, otherwise proceed to Step 4;
4. Select the more fit individuals based on fitness from $P(g)$ and transform them to new individuals, called offspring, by means of genetic operations like crossover and mutation;
5. Increase the number of generations from the new population, $P(g+1)$, $g = g + 1$; go to Step 2.

In GAs the decision variables are encoded in a string form. The encoded solutions are called chromosomes and the elements of the chromosomes are called genes. Typically, solutions are presented as binary [0, 1] digits, but higher digits (e.g., [0, 1, 2]) or real numbers are also possible depending on the nature of the problem. Further details on GA fundamentals are available in standard references (Holland and Goldberg 1989).

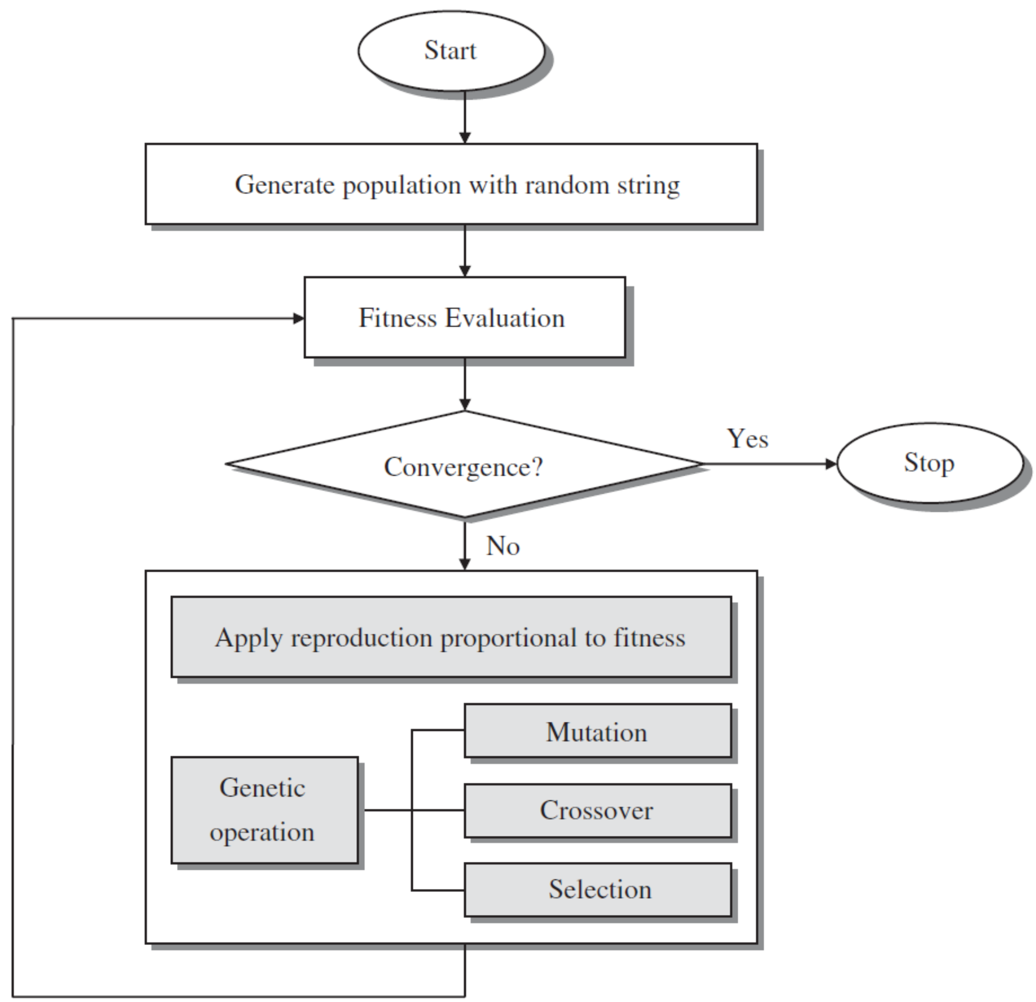


Figure 2.9: Simple genetic algorithm optimization process flowchart (Turban 1993)

Since GAs can provide a comparable level of accuracy in the area of global optimization while being more efficient than conventional optimization techniques (Morcous and Lounis 2005), they have been extensively used by many researchers in civil engineering for solving optimization problems, such as the design of structures and transportation networks (Rajeev and Krishnamoorthy 1992; Xiong and Schneider 1992). GAs have also been applied in maintenance optimization problems. Fwa et al. (1994) first introduced the use of GAs in maintenance optimization for the development of a computer model, known as PAVENET, for maintenance planning of pavement networks. In the area of bridge maintenance and management, Liu et al. (1997) used GA to find the near optimization long-term maintenance planning for a network-level bridge system. Miyamoto et al. (2000), Liu and Frangopol (2005), and Alonso et al. (2006) also developed GA-based models for existing bridges maintenance optimization. More recently, Sabatino et al. (2015), Dong et al. (2015), and Ghodoosi et al. (2017) proposed models to find optimal maintenance and rehabilitation scenarios using genetic algorithms.

2.5.3. Maintaining Bridges under Performance-Based Contracting (PBC)

Earlier practices for maintaining and managing transportation infrastructure were using separate contracts for each activity and with a labor rate or unit price. In recent years, there are innovative methods of procuring maintenance activities for all products and services under one contract and for a long term. These methods define and specify the desired end result which provides the contractor with more flexibility, innovation potential, and cost savings measures for the client organization (Pakkala 2002). In the traditional method-based contracts, the transportation agency specifies techniques, methods, materials, quantities, along with the time that the maintenance activities should be executed (Alyami and Tighe 2013). This causes a limitation on the use of new materials and innovative techniques and technologies while maintaining the infrastructure asset (Panthi et al. 2008). Moreover, this traditional way of contracting had failed in achieving the agencies' main goal of reducing maintenance expenditures and improving the transportation networks at an acceptable Level of Service (LOS) to the traveling public (Porter 2001; Piñero and de la Garza, Jesus M. 2004). The challenge of maintaining infrastructure to the highest possible condition while investing the minimal amount of money has promoted innovative contracting approaches. Contractors have better competencies of fulfilling required performance level through innovating in such new and unproven technologies (Panthi 2009). This can be achieved by extending the short maintenance period to a longer period so that the performance level can be satisfactorily measured over a longer duration. This type of agreement is called Performance-Based Contracting (PBC) which is a type of contract that pays a contractor based on the results achieved, not on the methods for performing the work. Such contracts allow the maintenance contractors to innovate and improve the efficiency of the services provided to public (de la Garza et al. 2009). Prior to explaining the methodology, PBC is reviewed in the following section.

2.6. PERFORMANCE-BASED CONTRACTING (PBC)

There has been some movement over the past decade toward a performance-based contract model for maintaining and managing transportation infrastructure. Performance-Based Contracting (PBC) is defined as "a contracting method that provides incentives and/or disincentives to the maintenance contractor to achieve desired outcomes or results; in its purest form, PBC does not detail how, when, or where to do the work." (Hyman 2009). The hallmark of PBC is that maintenance contractors are paid based on the results achieved and not on the methods utilized for performing the work. By paying contractors incentives and/or disincentives for measured performance instead of the quality of inputs and giving them the flexibility in choosing materials and methods in delivering maintenance services, contractors are encouraged to use new methods and mix quality which fosters innovation and improves the performance of the structure throughout its life. As in the case of PBC, the long term planning provides the contractor with enough time horizon to deploy new technologies. Further, the contractors are encouraged to eliminate the future need to perform costly corrective maintenance actions by taking timely preventive maintenance ones (Gupta et al. 2011). Interest in performance-based maintenance and operations management is driven by the increased focus on performance management, as well. Currently, many transportation agencies use performance-based measures to manage their highway programs (Markow 2012). For instance, 20 departments of transportation (DOT) in the US applies measures of condition or performance for bridges as one of the program assets.

Types of performance-based contracts differ according to scope and coverage. The scope refers to the service activities and the addressed assets while the coverage relates to the amount of highway covered and the geographic area (Hyman 2009). For instance, a PBC can cover either single asset such as only traffic signs, only pavement, only bridges, … etc. or all assets along a highway corridor. In terms of service activities, PBC may deal with only a single activity (e.g., sign replacement) or a set of related activities (e.g., rest area maintenance). Stankevich et al. (2009) stated that the complexity level of a PBC can range from "simple" to "comprehensive" depending on the number of assets and range of services included where a simple PBC may cover a single service and a comprehensive one typically covers all assets and comprise the full range of services needed to manage and maintain the contracted corridor. Usually, transportation agencies that are newly practicing PBC find the best approach to first gain experience is by contracting a single activity, a single asset, or one set of related activities in a single maintenance area. Once the agency has acquired experience, it is likely to expand the number of assets under contract, coverage area, and the period of performance. Performance contracts are not limited to maintenance including preventive, routine, periodic, and demand-responsive maintenance as rehabilitation of roads and bridges has been part of many performance-based contracts (Hyman 2009).

2.6.1. Background on Performance-Based Contracting

PBC for maintenance was first implemented in the province of British Columbia in Canada. Then, provinces of Ontario and Alberta followed by developing their own performance-based contracts. Later, it has become the backbone of maintenance contracting in Australia, New Zealand, some European countries as Finland and England, and to an increasing degree in other countries including the United States where states of Virginia, Texas, and Florida are leading in this area. Gradually, PBC trends spread to other developed and developing countries worldwide. Anastasopoulos et al. (2010) illustrate a map of countries (dark highlighted areas) which implemented PBCs as can be seen in Figure 2.10. Moreover, transportation agencies in North America and around the globe have developed a variety of methods for undertaking PBC, known by other names such as Performance-Specified Maintenance Contracts in Australia and New Zealand; Asset Management Contract in the US; Area Maintenance Contract in Finland and Ontario, Canada; Managing Agent Contract in the UK; and *Contrato de REcuperacion y MAntenimiento* (CREMA), which means Contract for Rehabilitation and Maintenance in Argentina (Hyman 2009).

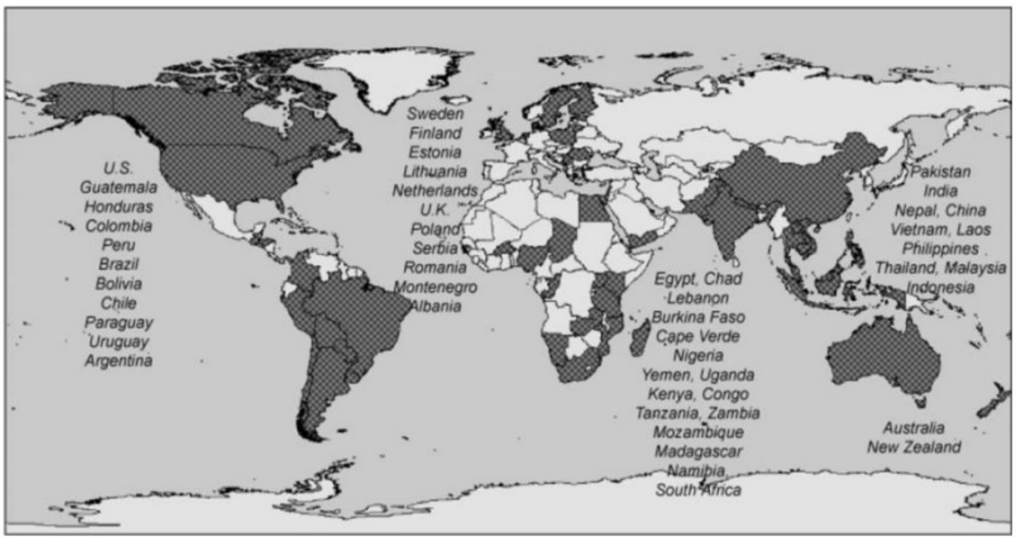


Figure 2.10: PBCs implementation worldwide (Anastasopoulos et al. 2010)

2.6.2. Performance-Based Contracting Application

As discussed above, there is a high tendency towards PBC in the area of transportation. In road maintenance, Sultana, M. et al. (2012) introduced seven main issues that should be considered by the transportation agencies before applying PBC as shown in Figure 2.11. The first issue is the transportation agency obligation to define the performance specifications and set-up a standard for these performance measures or indicators. After that, the agency should check the private sector capability of handling the maintenance to reach the desired LOS quality. Then, the implementation stage takes place where an initial project has to be decided for the performance-based contracts to be applied on. Afterward, a detailed risk analysis has to be done in order to define the events that are out of the maintenance contractors' control and share those risks with the maintenance contractor. Hence, the performance monitoring process takes place where the maintenance contractors are evaluated according to their performance within the contract period. The sixth issue is related to the agency employees as the agency requires fewer people for administrative work and supervision in PBC. The fear of losing job is considered an important issue for the employees and the transportation agency. Finally, the seventh issue is the proper definition of the payment and termination clauses in the contract to avoid any conflicts or disputes that may arise during the contract period.

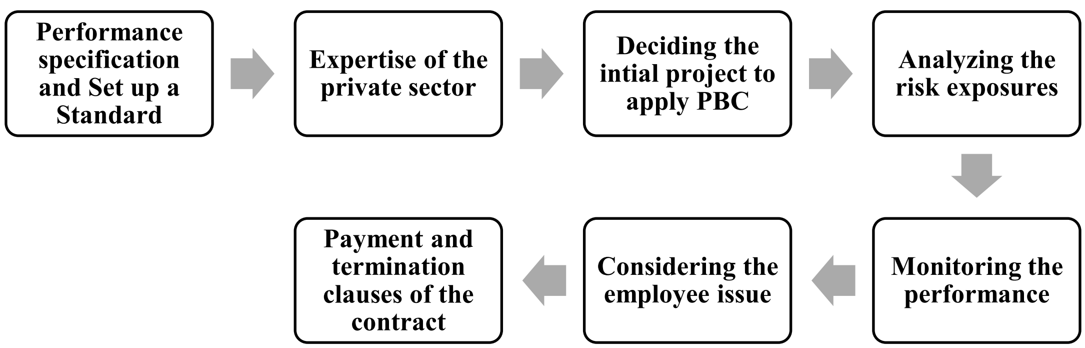


Figure 2.11: The seven main issues before applying PBC (Sultana et al. 2012)

2.6.3. Benefits of Performance-Based Contracting

Agencies that have converted to PBC claim cost saving between 10-50%.

Table 2.5 shows the cost savings of different countries under PBC over the traditional contracts (Stankevich et al. 2009). Moreover, many government agencies reported an 18% improvement in satisfaction with the maintenance contractors' performance. The agencies added that this price reduction and customer satisfaction took place in several areas from the non-technical, technical, and professional services (Office of Federal Procurement Policy (OFPP) 1998). Some other advantages found in the literature (Hyman 2009) include but not limited to:

- Improved level of service (could cost more)
- The transfer of risk to the contractor
- More innovation
- More integrated services
- Enhanced asset management
- Ability to reap the benefits of partnering
- Building a new industry
- Achieving economies of scale

Table 2.5: Cost Savings of Different Countries under PBC over the Traditional Contracts (Stankevich et al. 2009)

Country	Cost Savings (%)
Norway	About 20% - 40%
Sweden	About 30%
Finland	About 30% - 35%; about 50% less cost/km
Holland	About 30% - 40%
Estonia	20% - 40%
England	10% minimum
Australia	10% - 40%
New Zealand	About 20% - 30%
USA	10% - 15%
Ontario, Canada	About 10%
Alberta, Canada	About 20%
British Columbia, Canada	Some, but might be in the order of 10%
Source: P. Pakkala.	

2.6.4. Risks in Performance-Based Contracting

In the conventional traditional-based contracts, transportation agencies, called as a client normally, prescribe the techniques, methods, materials specifications, and the time to execute maintenance activities which reduce risks of the maintenance contractors. However, this is not the case in performance-based contracts where a contractor is generally free to make decisions of "what to do," "how," and "when" as the specified performance measures or indicators are achieved (Alyami 2012; Al-Kathairi 2014). Hence, the entire risk of any failure or shortcomings of made decisions is transferred to the contractor. Figure 2.12 depicts the distribution of risk moving from traditional contracting to different forms of contracting. As shown in the figure, agency risks tend to decrease while contractor risks increase when long term contracts are adopted (Queiroz 1999). The best way of managing the risk associated with such long term maintenance contracts is to properly i) predict deterioration rates of contracted assets, ii) determine appropriate design, specifications, and materials, iii) plan needed maintenance interventions, iv) estimate quantities (Stankevich et al. 2009; Alyami and Tighe 2013; Al-Kathairi and El Halim 2014); additionally, v) inspect and measure the current condition state of contracted assets, and vi) select suitable MRR actions.

Maintenance contracting has various risks. Types of risks, among others, are identified as follows (Hyman 2009):

- Poor quality of construction
- Unexpectedly severe weather
- Unanticipated environmental problems

– Emergencies

– Unanticipated legislative change

– Unexpected traffic growth

– A short-term focus that fails to minimize long-term life-cycle costs

– Difficulty in acquiring the resources needed to perform the work (e.g., subcontractors)

– The possibility of having to correct problems covered under a warranty.

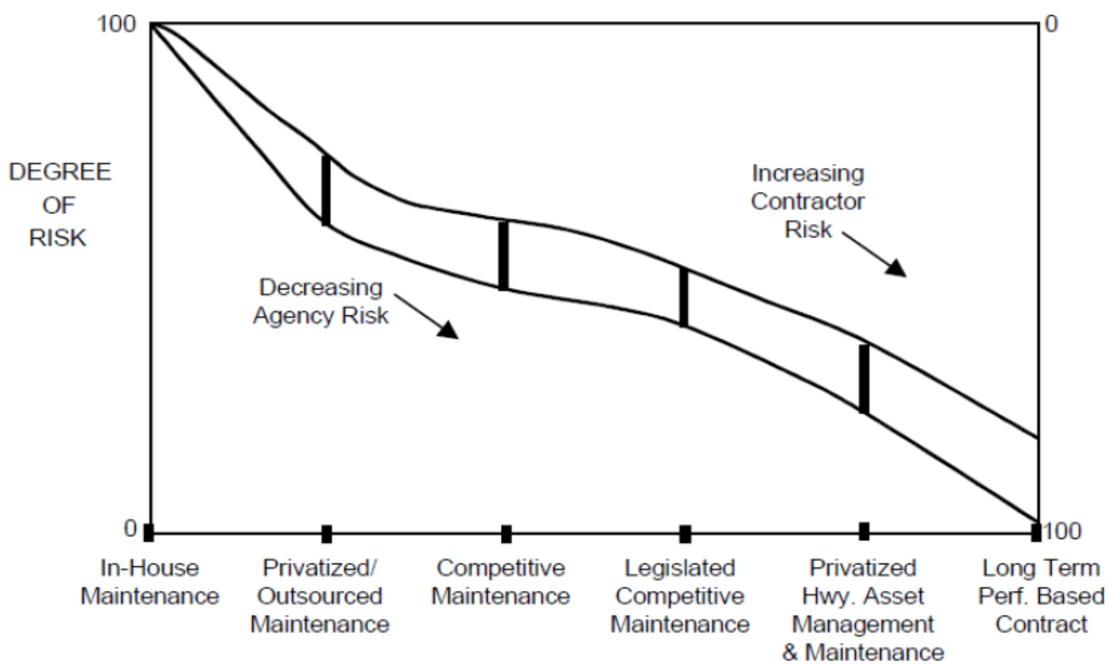


Figure 2.12: Distribution of risk with different contract approaches (Haas et al. 2001)

However, it is worth noting that the allocation of risks in PBC varies from one country to another (Segal et al. 2003). It is essential to identify all the risk variables in order to avoid or mitigate the undesirable consequences such as the first identified risk, poor quality of construction. Thus, the concept of warranty is implemented.

2.6.5. Warranties and Warranties Period

Warranty is a form of PBC and has been implemented by many agencies in order achieve objectives such as: i) improved quality, ii) augmentation of agency expertise, iii) redistribution of performance risk, and iv) reduction of agency design, testing, and inspection personnel (Queiroz 1999). During the warranty period, the maintenance contractor has to ensure that the constructed infrastructure provides the level of service specified in the contract by the agency (Panthi et al. 2008). The next section aims to provide an overview of the key aspect that is needed before applying PBC which is the performance indicators that are used to define the level of service (LOS).

2.6.6. Performance Indicators in the PBC

PBCs can address different types of assets and operations including: pavements, bridges, roadsides, traffic operations, traffic services, incident response, hazardous materials cleanup, and emergency repairs (Hyman 2009). For every type of maintenance and operations, performance indicators should be clearly defined and objectively measurable in order to avoid ambiguity and risk disputes (Zietlow 2005). In some countries, measures or indicators of performance are often expressed in terms of levels of service represented by specific rating scales corresponding to the condition of different assets. Performance indicators (PIs) and performance level of service (LOS) are the basis of a PBC which are expected to be achieved by the maintenance contractor (Alyami 2012). By looking into the literature, it is observed that many transportation agencies established and defined many performance indicators extensively for roads maintenance. However, in the area of bridge management, only a few distinct has addressed these indicators. Haas et al. (2008; 2009) suggested performance indicators for bridges to be the remaining life in years and safety.

Many experts on performance measurement argue the benefit of using a few key performance measures or indicators instead of many because of the associated simplicity and manageability of those few performance measures (Hyman 2009). Accordingly, performance indicators can be defined as a set of outcome-based levels that an agency uses to evaluate the success of the contractor and performance goals are the minimum acceptable levels to be achieved for each performance measure or indicator (Alyami 2012).

Performance measures are prescribed in the contract specifications as performance indicators or measures. Performance indicators are measures of quality; quite often the distress indicators which are monitored annually, or more frequently. It is assumed that these indicators give an accurate picture of the asset condition (Panthi 2009). In general, for a performance measure or indicator to be effective, the following questions should be addressed (SAIC 2006):

- Is the performance measure/indicator specific?
- Is the performance measure/indicator measurable?
- Is the performance measure/indicator achievable?
- Is the performance measure/indicator results- oriented?
- Is the performance measure/indicator timely?
- Does the measurement meet with the agency's objectives and desires?
- Has the performance been measured before?
- Does the measurement conflict with the agency's standard specifications?
- Does the measurement aim to improve performance?

2.6.7. Payments System for the PBC

Payments in performance-based contracts differ significantly from traditional-based contracts. In traditional contracts, payments to contractors are based on inputs such as number of working hours or quantities; for example, cubic meters of concrete (Alyami 2012). In contrast, performance-based contracting is a type of contract that pays a contractor based on the results achieved, not on the methods for performing the work. It provides incentives, disincentives, or both to the contractor to achieve desired LOS for measurable outcomes. In other words, payments for the management and maintenance of assets in this type of contract are explicitly linked to the contractor successfully meeting or exceeding these minimum performance indicators. Failing to comply with the pre-defined performance indicators or to promptly rectify revealed deficiencies affects the contractor's payment adversely through a series of clearly defined penalties. Accordingly, a proper payment system that provides incentives, disincentives, or both is needed in PBC (Alsharqawi et al. 2017).

2.7. GAPS IN THE BODY OF KNOWLEDGE VERSUS POTENTIALS

This chapter discussed the various previous work related to bridge condition assessment, deterioration modeling, decision-making and optimization, and performance-based contracting for maintaining and managing transportation infrastructure. While many condition ratings have been developed for the bridge in general and bridge deck in particular, all of them are computed based on visual inspections. Also, most of them are limited to certain defect types without integrating both surface and subsurface defects. Furthermore, the thresholds that define the severity of concrete deterioration is selected arbitrarily. The current concrete bridge deterioration models and the previously discussed ones have many drawbacks including the requirement of huge amount of inspection data and the time consumption and complexity while updating the deterioration prediction after each inspection, maintenance or rehabilitation action. Most of the existing decision-making methodologies attempt to maintain or extend the bridge serviceability at a minimum total cost. These conflicting objectives have made the bridge management decision process very complex. Besides, the growing needs and limited resources to maintain the transportation infrastructure have resulted in motivating provincial and state agencies to expand the amount of contracting they do.

CHAPTER 3: PROPOSED METHODOLOGY AND MODEL DEVELOPMENT

3.1. OVERALL RESEARCH METHODOLOGY

Management of Infrastructure systems support the tasks of facility inspection for data collection, prediction of facility deterioration through performance models, and the subsequent selection of the appropriate MRR policies over a planning horizon (Jha 2010). In this context, managing bridges include: i) condition assessment based on detection and evaluation of bridge main defects and distresses, ii) deterioration modeling to assist in forecasting bridge future condition, and iii) decision-making to select and schedule the appropriate Maintenance, Repair and Replacement (MRR) actions. The models proposed in this research analyze bridge deck in specific because it deteriorates faster than other elements due to its direct exposure to traffic in addition to the external environmental factors (e.g., freeze-thaw cycling) (Gucunski et al. 2013; Frosch et al. 2014). After reviewing the literature and identifying relevant defects that reflect the bridge deck condition, a correlation between the identified defects is established. Then a bridge deck management system is developed including, i) a Quality Function Deployment (QFD) model to provide condition assessment for the concrete bridge deck, ii) a condition reliability-based Weibull Distribution Function (WDF) to establish the ideal, updated and predicted deterioration curves based on the QFD model, and iii) a rehabilitation model to provide recommendations for the MRR needed action along with formulating condition improvement and cost models for the selected MRR action. After that, a decision-making tool is developed based on a designed Performance-Based Contracting (PBC) framework.

The tool optimizes the selection of MRR actions within a pre-defined period. The models' development process is illustrated in Figure 3.1 and discussed in the following sections.

3.2. IDENTIFICATION OF BRIDGE DECK COMMON DEFECTS

Common defects of bridge decks are reviewed from the literature and manuals of practice adopted by departments of transportation. Similar to any reinforced concrete structures, bridges experience loss of integrity over time due to degradation caused by chemical, physical and even biological mechanisms (Gucunski et al. 2010). It is evident that some mechanisms affect the reinforcement and other mechanisms impact the concrete itself, yet all degradations mechanisms lead to structure vulnerability and thus reduce the infrastructure reliability. Two inspection manuals were studied further in order to identify the defects occurring in concrete bridges in specific. The first manual is "Manuel d'Inspection des Structure" issued by the Ministry of Transportation of Quebec (MTQ) (2012) and the second is "Ontario Structure Inspection Manual (OSIM)" issued by Ministry of Transportation of Ontario (MTO) (2008). Alsharqawi et al. (2018) summarized in Table 3.1 the main defects founded in these manuals. Along with the concrete defects, the expansion joints problem was added as these joints are part of any bridge deck and they can cause local deterioration if they are not functional. Expansion joints are real problems in concrete bridges. According to a bridge expert, they have higher maintenance and repair needs and thus new bridge design tends to eliminate these joints if possible.

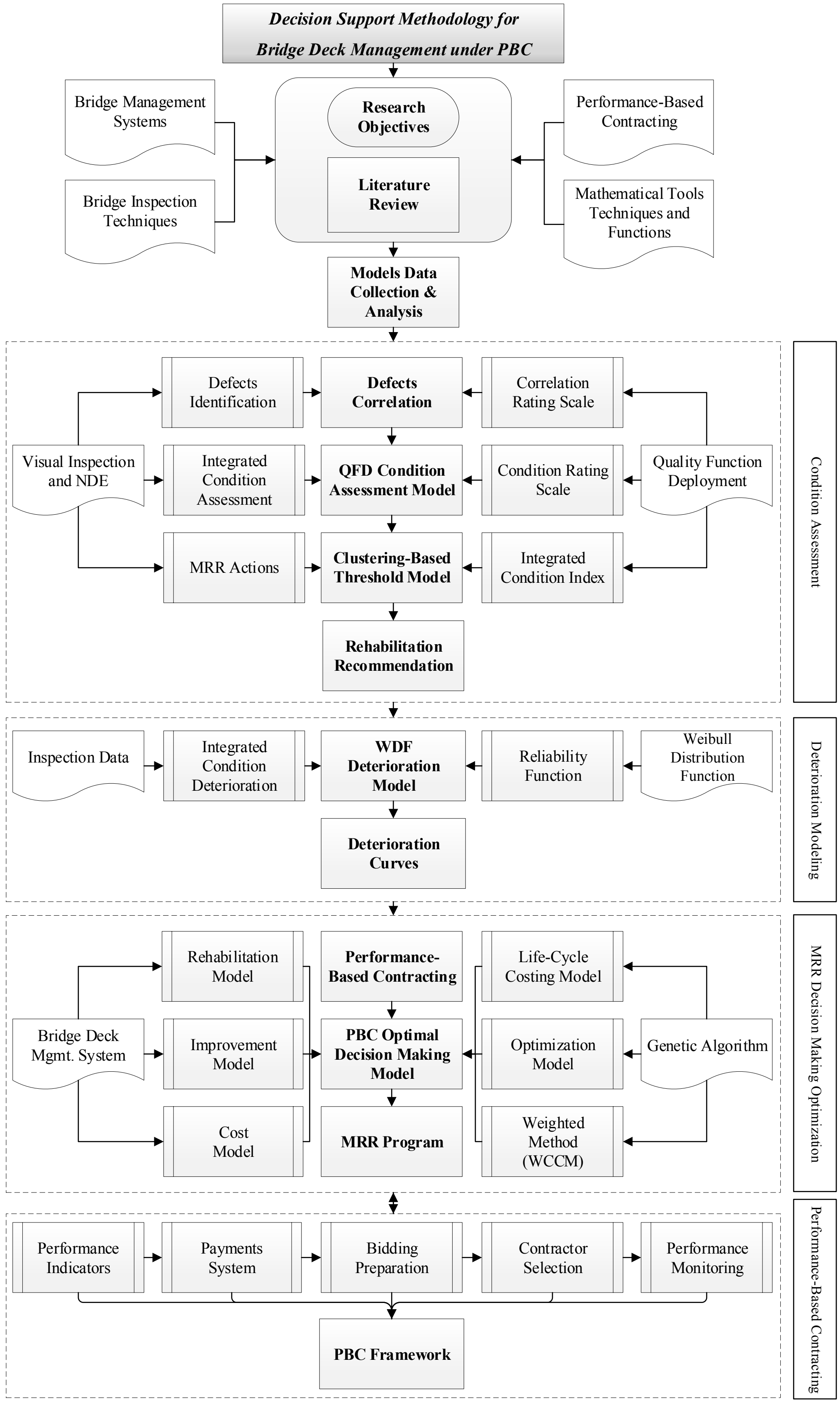


Figure 3.1: Schematic flow diagram of the proposed methodology

Table 3.1: Concrete Bridge Deck Defect Types and Descriptions

Defect Type	Description
Cracking	Linear fractures in the concrete surface caused by tensile and/or compressive stresses in concrete
Concrete Disintegration, Scaling and Erosion	Disintegration is the concrete damages due to progression of physical deterioration (freeze-thaw cycles). Scaling is local flaking or loss of surface mortar or concrete. Erosion is the detachment of concrete surface resulting from the friction of ice or water containing stones or gravel
Corrosion of Steel Reinforcements	Steel reinforcement damages due to electrochemical corrosion process. It appears as rust stain. At severe stages, surface concrete above reinforcements cracks, delaminate, and spalls
Delamination	Separation of concrete layers along a horizontal plane at or near the outermost layer of reinforcing steel
Spalling	Complete separation of delaminated area from concrete. The roughly circular or oval depression left is known as a spall
Deposits	Deposits are formed when water percolates through the concrete and dissolves leaches chemicals from it and deposits them on the surface. Deposits may appear as efflorescence or stalactite
Expansion Joints Problems	Problems related to torn or missing seals, armored plate damage or chemical leaching on the bottom of joint
Pop-outs	Pop-outs are conical, shallow depressions caused from the breaking of small portions of the concrete surface, due to the expansion of some aggregates or due to frost action

3.3. COMPARISON AND SELECTION OF NDE TECHNIQUE

Many NDE technologies can be utilized for reinforced concrete bridge inspection (Alsharqawi et al. 2017). In this task, the available NDE technologies were studied and analyzed to highlight the detection capabilities as well as investigate their limitations. The examined techniques include: Infrared Thermography, Ultrasonic, Half-Cell Potential, Impact Echo, Ground Penetrating Radar and Chain drag is added as a classical NDE technique. Theoretically, some or all of the techniques can be performed on the same bridge. Using multiple techniques provides a comprehensive understanding of the bridge condition as each technique will eliminate the drawbacks of the others. That approach may exist but would require more time and financial resources to inspect and interpret the results than the case if one single method is selected. In a research sponsored by the Federal Highway Administration (FHWA) in cooperation with the American Association of State Highway and Transportation Officials (AASHTO), Gucunski et al. (2013) performed an independent evaluation of the capabilities and limitations of the most common NDE techniques to detect and characterize typical deterioration mechanisms in concrete bridge decks. Based on several performance criteria as well as field and laboratory testings, it was reported that among tested NDE technologies, GPR ranks first for condition assessment of concrete bridge decks.

In this research, technical benefits of concrete bridge decks inspection are considered for the selection of the most appropriate NDE technique. These technical benefits, or selection criteria, are identified as follows: (1) capable of detecting corrosion-induced defects such as (1a) corrosion and (1b) delamination, (2) does not require traffic close for bridge inspection, (3) inspection result can be reproducible, (4) usable for various bridge elements, (5) works well with asphalt overlays that is commonplace in Canada, (6) objective with minimal subjective interpretation from operators, and (7) can be used as a stand-alone technique that does not require other tests.

As shown in Table 3.2, consistent with the result of Gucunski et al. (2013), Ground Penetrating Radar (GPR) also is the most appropriate technology for inspection of concrete bridge decks. Therefore, GPR is selected to augment the visual inspection technique to assess the condition of concrete bridge decks. GPR technology is described in detail in Appendix B.

Table 3.2: Comparison of NDE Techniques to Selection Criteria

NDE Techniques	**Selection Criteria** **(1a): capable of detecting corrosion** **(1b): capable of detecting delamination** **(2) : does not require traffic close** **(3) : inspection result can be reproducible** **(4) : usable for various bridge elements** **(5) : works well with asphalt overlays** **(6) : objective with minimal subjective interpretation** **(7) : can be used as a stand-alone technique**							
	(1a)	(1b)	(2)	(3)	(4)	(5)	(6)	(7)
Infrared Thermography		x	x				x	x
Ultrasonic				x	x		x	x
Half-Cell Potential	x			x	x		x	x
Impact Echo		x		x	x	x	x	x
Ground Penetrating Radar	x	x	x	x	x	x	x	x
Chain Drag & Hammer Sounding		x			x			x

3.4. DEVELOPMENT OF CONDITION ASSESSMENT MODEL

Inspection data from defects are gathered and collected from two sources: i) visual inspection reports and ii) GPR scans. Visual examination process can evaluate surface defects. Moreover, GPR technology evaluates subsurface defects such as chloride contamination and corrosion of steel reinforcements. This can be obtained from the condition map generated from performing the amplitude analysis to the GPR profiles. Once inspection data are analyzed, quantified surface and subsurface defects are fed in a QFD condition assessment model. The QFD model output is an integrated condition crisp value. Using *k*-means technique, integrated condition values from different case studies are divided into clusters. Based on the result of clustering, the threshold value for each condition category will be determined. The final output of this step is a clustering-based rating model represented by an integrated condition index. The condition assessment model is then implemented into real case studies. Furthermore, statistical analyses are made to validate the model using various indicators. The model's development process is illustrated in Figure 3.2 and discussed in the following sections.

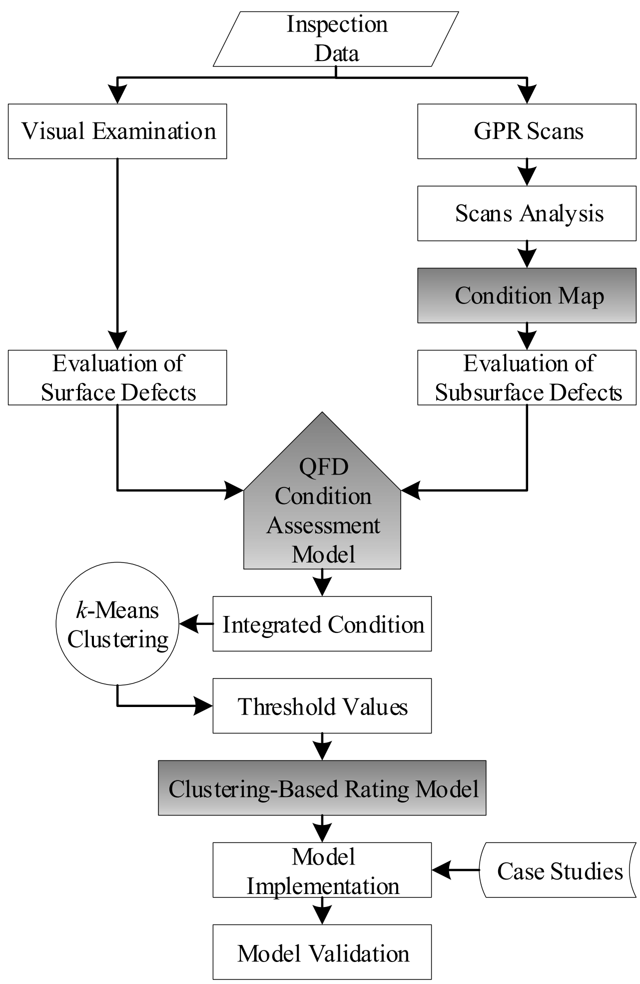


Figure 3.2: Methodology for development of condition assessment model

3.4.1. QFD Condition Assessment Model

QFD condition assessment model inputs and outputs are demonstrated in Figure 3.3. Defects from visual inspection and GPR evaluation are the inputs to the QFD model along with the defects correlation and the condition rating scale. The outputs are condition ratings and an integrated condition crisp value that recommends a MRR action.

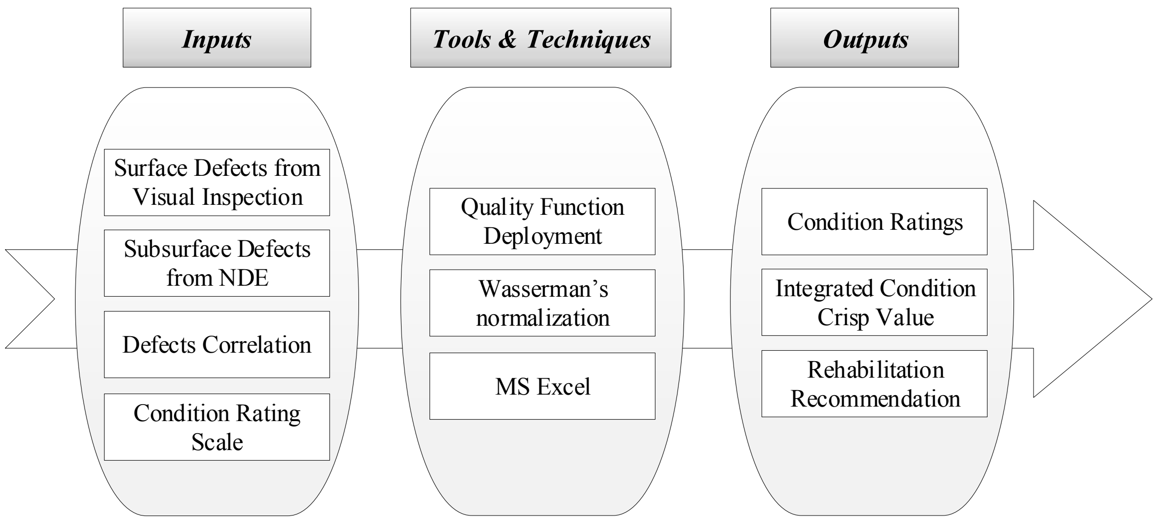


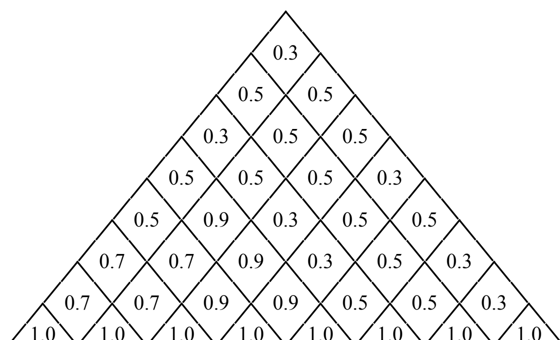
Figure 3.3: QFD condition assessment model inputs and outputs

The QFD model includes five main components which form the HOQ skeleton needed for the analysis. The research extended the traditional QFD guidelines from their typical use (extracting the "voice of the customer" to produce a product that meets the customer needs) and applied the approach to perform bridge condition rating (meeting the need of infrastructure facility based on its condition assessment). Thus, the components are customized to be suitable for implementation in bridge deck condition assessment as follows:

i. ***Customer demands (WHATs)*** are represented in the current QFD model as the bridge condition scale following to the MTQ inspection manual and reports. The linguistic scale is described as ("none or light", "medium", "severe", "very severe"). This scale should be translated to numerical values in order to quantify the bridge condition assessment. The model scale rating had been calibrated using 11 different numerical scales; eventually, the scale of (1, 3, 6, 9) as shown in Figure 3.4 was chosen after performing a close analysis. The proposed numerical rating scale has been found to provide a better representation of results since it follows a geometric progression where severity is scored proportionally (Alsharqawi et al. 2016b).

Quality Function Deployment (QFD) Model

(1) Cracking
(2) Disintegration, Scalling and Erosion
(3) Corrosion of Steel Reinforcements
(4) Delamination
(5) Spalling
(6) Deposits
(7) Joints Problems
(8) Pop-outs



		4.5	5.5	5.5	5.1	5.7	3.7	4.3	3.7	
Description	Rating	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	
None or Light	**1**	64%	87%	57%	40%	4%	62%	89%	35%	**25.27%**
Medium	**3**	32%	12%	38%	53%	75%	35%	2%	54%	**50.04%**
Severe	**6**	3%	1%	5%	7%	20%	3%	9%	10%	**20.54%**
Very Severe	**9**	0%	0%	1%	1%	1%	0%	0%	1%	**4.15%**
		2.50	2.79	2.66	2.51	2.77	2.01	2.46	1.52	19.21
		1.44	1.86	1.97	1.91	2.04	1.17	1.25	1.03	38.04
		0.27	0.38	0.39	0.38	0.41	0.24	0.30	0.23	15.61
		0.03	0.05	0.05	0.04	0.05	0.03	0.03	0.07	3.16

Figure 3.4: Quality Function Deployment (QFD) model

ii. ***Quality characteristics (HOWs)*** are the defects where eight main defects are identified in the concrete bridge deck following to ministries of transportation inspection manuals. These defects were described already in Table 3.1. It is not a must that each bridge has all the defect types as it has been observed that some cases were free of certain type of defects.

iii. ***Relationship matrix*** describes each defect with its associated condition rating in percentages. For instance, the inspector can evaluate the bridge cracking severity degrees on reinforced and pre-stressed concrete as medium if crack opening extends less than 0.8 mm through the bridge member, severe if the linear fracture is between 0.8 and 3.0 mm, and very severe if the crack opening is greater than 3.0 mm according to MTQ (2012) inspection manual. The first column in the relationship matrix, Figure 3.4, illustrates the bridge condition in terms of cracking defect where it was quantified in percentages based on the defect areas.

iv. ***Correlation matrix*** is the top roof of the HOQ in Figure 3.4 where defects are correlated. After identifying the defects, every single defect was correlated with other defects to discover the relationship degree between them. Structured interviews were conducted to get these relationships where a 5 point ordinary scale ("very weak", "weak", "moderate", "strong", "very strong") is used. Degrees obtained with this scale are scalarized with numerical series (0.1, 0.3, 0.5, 0.7, 0.9) with the larger number indicating a stronger relationship. Figure 3.5 shows the correlation scale that is used to transform these linguistic presentations of relationships into numerical values.

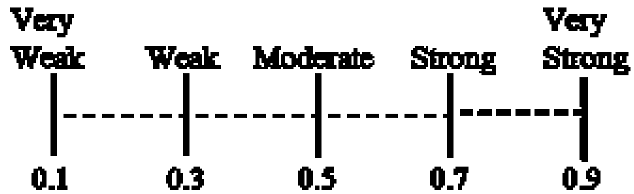


Figure 3.5: Correlation linguistic-numerical scale

In the conducted literature review on QFD, it has been found a use of 3 point ratings on 1 to 5 (1, 3, 5) or 1 to 9 (1, 3, 9) scales, where the first series is derived from a linear interval scale and the second can be considered as being derived from a logarithmic interval with 3 as a basis (Wasserman 1993). Akao (1990) who conceptualized QFD stated that there is no established theory in attaching rating numbers to relationships scale. Accordingly, the rating on 0.1 to 0.9 scale is used where it represents an arithmetic progression with even discriminations. Also, it is regarded that this scale is consistent with Saaty's fundamental scale of absolute numbers for pairwise comparisons. At the end, the correlation is established using the roof of the House of Quality (HOQ) in the QFD model. For example, it has been said that the relationship between defect 3 (corrosion of steel reinforcements) and defect 4 (delamination) is very strong; thus, it is given a rate of 0.9 (90% correlated). However, the correlation between defect 8 (pop-outs) and 6 (deposits) is weak, and therefore it is rated as 0.3 (30% correlated). A rate of 1.0 is given for each defect as it represents itself.

The weights in the QFD correlation matrix are designed as weighted averages in order to provide flexibility to the user to change the correlation values and the weights of the defined defects. Hence, the proposed method has an advantage over the traditional pairwise comparison techniques such as the Analytical Hierarchy Process (AHP) and Analytical Network Process (ANP). AHP cannot take into consideration the correlation between different defects. ANP considers the correlation but provides fixed weights. Embedding the weighted average technique in the QFD model overcomes this limitation. The proposed model takes into consideration the correlation between the different defects and provides flexible weights based on the number of the existing defects since the type and number of defects can be different from one bridge to another. Therefore, the one defect that strongly correlates with the other defects will have the highest weight.

v. ***Absolute weights of WHATs*** are the final results of the QFD model where these weights demonstrate the rate for each condition scale in percentages (represented in green, yellow, orange and red colors in Figure 3.4). The coefficients r_{ij} in the relationship matrix are normalized using Wasserman's (1993) normalization method where the expression of the normalized coefficient $r_{i,j}^{norm}$ is described in Equation 6 as follows:

$$r_{i,j}^{norm} = \sum_{k=1}^{m}\left(r_{i,k} \cdot \gamma_{k,j}\right) \Big/ \sum_{j=1}^{m}\sum_{k=1}^{m}\left(r_{i,j} \cdot \gamma_{j,k}\right) \qquad (6)$$

where:

i–th is the condition scale;

j–th is the defect type, $j = 1, 2, \ldots, m$; m is the number of defects; and

γ_{jk} is the intensity of the correlation between two defects j and k. Values of γ_{jk} must be between 0 and 1.

The weights of each condition rating w_i are then calculated using Equation 7:

$$w_i = \sum_{j=1}^{8} d_j \, . r_{i,j}^{norm} \tag{7}$$

where:

d_j is the degree of relative severity score (condition rating scale) of the j-th defect type.

The last step is to convert the calculated weights into percentages by dividing each one over the weights summation. These weights are represented by the last colored column in the matrix in percentages. By applying the QFD model, a final crisp integrated condition value C_I is calculated representing the bridge deck overall condition using Equation 8:

$$C_I = (1 \times Good + 3 \times medium \; + \; 6 \times Severe \; + \; 9 \times V.Severe)/9 \tag{8}$$

where:

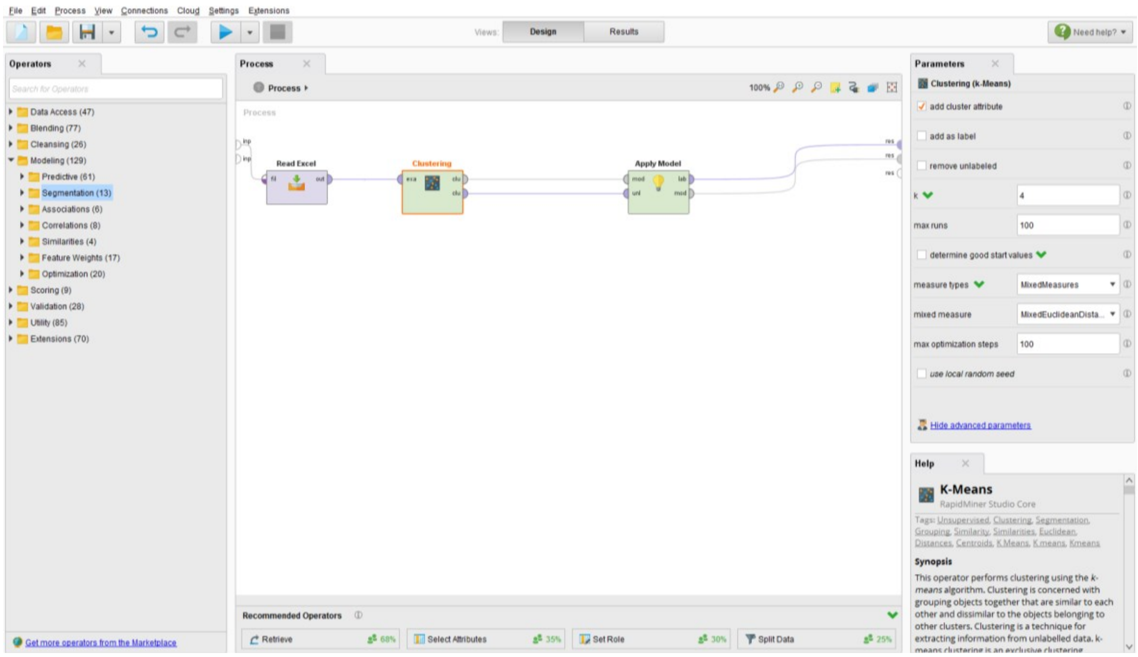
C_I = integrated component condition; and $1, 3, 6, 9$ = numerical rating scale following a geometric progression where severity (condition ratings) is scored proportionally.

The crisp value calculation is similar to a material condition rating used by MTQ (2012) inspection manual “Cote de Matériau Intégré (CMI)”.

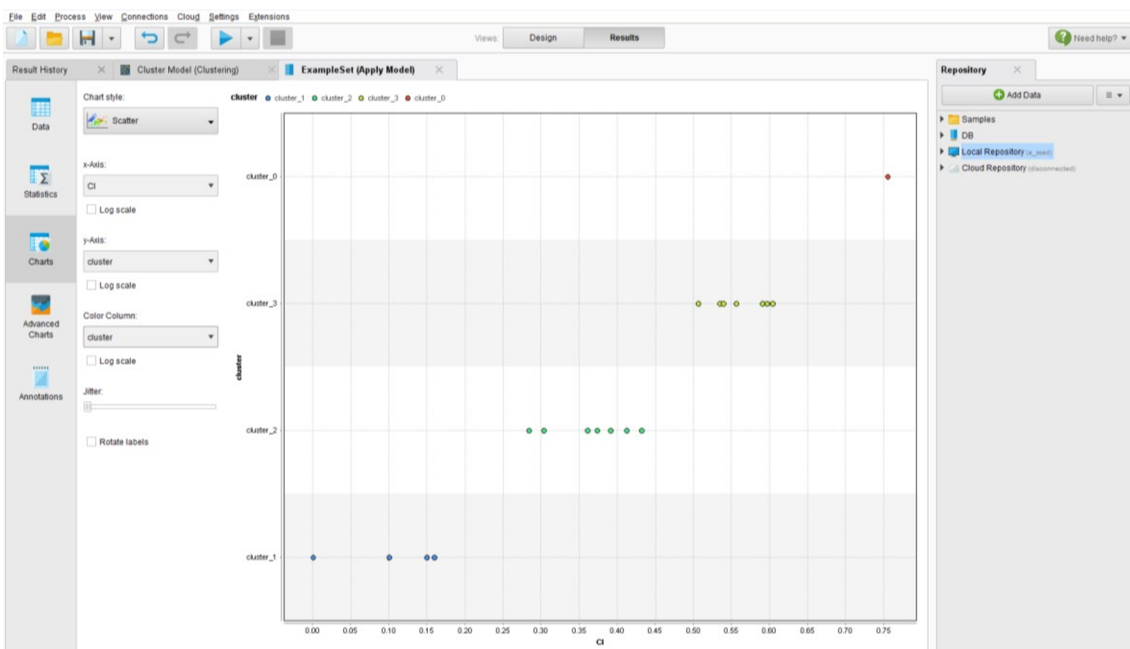
3.4.2. Clustering-Based Threshold Model

Commonly, the thresholds that define the severity of bridge deterioration are subjectively assigned based on the experience and judgment of the inspector or expert. This can misrepresent the obtained assessment result and may cause wrong intervention decision. In this research, a robust method based on *k*-means clustering technique is proposed to solve this issue. In order to perform clustering using the *k*-means algorithm, a software named RapidMiner, formerly known as YALE, (Mierswa et al. 2006) is employed. Figure 3.6 (a) displays the software interface showing the design view. For the proposed method of clustering, the value of *k* is chosen to be 4 indicating the number of clusters we would like to form from our data. The 'Read Excel' dataset is loaded from a database using the retrieve operator. The data set contains twenty crisp integrated condition values after analyzing real case studies using the QFD model. Then the *k*-means 'Clustering' operator is applied to this dataset. Finally, 'Apply Model' operator is added to generate the clusters. Since parameter *k* was set to 4, four clusters are generated after running the process. Results presented in Figure 3.6 (b) to show how the algorithm has created separate groups in the plot view, where each crisp value is assigned to either 'cluster_0', 'cluster_1', 'cluster_2', or 'cluster_3'. The centroid of each cluster is also delivered through the cluster model output.

The next step is to determine the condition threshold values. This is achieved by calculating the mean between two consecutive centroids. For example, the mean of 'cluster_1' and 'cluster_2' centroids. The final output of this process is a rating model represented by an integrated condition index (Table 3.3). The developed index recommends suitable MRR intervention actions which are translations of integrated condition states in each cluster.



(a)



(b)

Figure 3.6: Clustering model (a) design view (b) results plot view

Table 3.3: Bridge Deck Integrated Condition Index Interpretation

Integrated Condition		Action
Linguistic	Numeric	
- Excellent	0.00-0.23	No action is needed
- Good	0.23-0.46	Bridge needs routine maintenance
- Poor	0.46-0.65	Bridge needs repair
- Critical	0.65-0.76	Bridge needs replacement
- Failed	0.76-1.00	Close bridge until the deck is replaced

3.5. DEVELOPMENT OF DETERIORATION MODEL

The most widely used approach for calculating deterioration rates is the Markovian-based models. However, they assume that the history of the deteriorated element is irrelevant and suffer from other limitations discussed in the literature (Morcous et al. 2002; Agrawal et al. 2010). Weibull-based approach performs better for developing deterioration curves for different bridge elements (Agrawal et al. 2010). Generally, Weibull distribution is suitable to describe the distribution of the time-in-condition rating (Nasrollahi and Washer 2014), to model uncertainties (Sobanjo et al. 2010), and to provide the best overall fit for infrastructure deterioration data (DeLisle et al. 2004). The current research introduces the reliability function for the Weibull distribution to model bridge deterioration because of its particular advantages, including: i) the WDF has proven to be one of the best functions to represent concrete deterioration (Semaan 2011; Shami 2015). The function starts at the maximum performance level (100%) and remains steady for certain time such that slope equals zero. Then, it decreases quite rapidly with negative slope and towards the end, the rate slows down and the function passes through an inflection point. This trend is similar to concrete structures where a structure component typically has an excellent condition at the early stages of its service life. However, after some time (i.e., deterioration occurs), concrete structural performance starts decreasing slightly at first, faster later and near the end of component's service life, the concrete deterioration develops with a much reduced speed until complete failure, ii) Unlike other deterioration models, the Weibull function does not require large amount of historical inspection data to model deterioration. In fact, the Weibull approach requires two pieces of information to be solved, namely the age and the current condition of the component. Thus, it functions when inspection data is scarce or missing, and iii) the Weibull reliability function parameters are easily calculated.

The model inputs and outputs are described in Figure 3.7. As the flow diagram shows, the main input for the WDF is an integrated condition rate retrieved from the quality function deployment (QFD) condition assessment model that provides an integrated condition index at the element level. Other inputs are the Weibull function parameters, age of the component (i.e., bridge deck), and deterioration thresholds. The outputs are the ideal, updated, and predicted deterioration curves. The following sections describe the development of these deterioration curves in details.

To model deterioration, the current condition is defined as a basis and then the future condition is forecasted and it takes into account any scheduled MRR activities (Alsharqawi et al. 2017). The following sub-sections describe the development of the ideal, updated, and predicted deterioration curves in details.

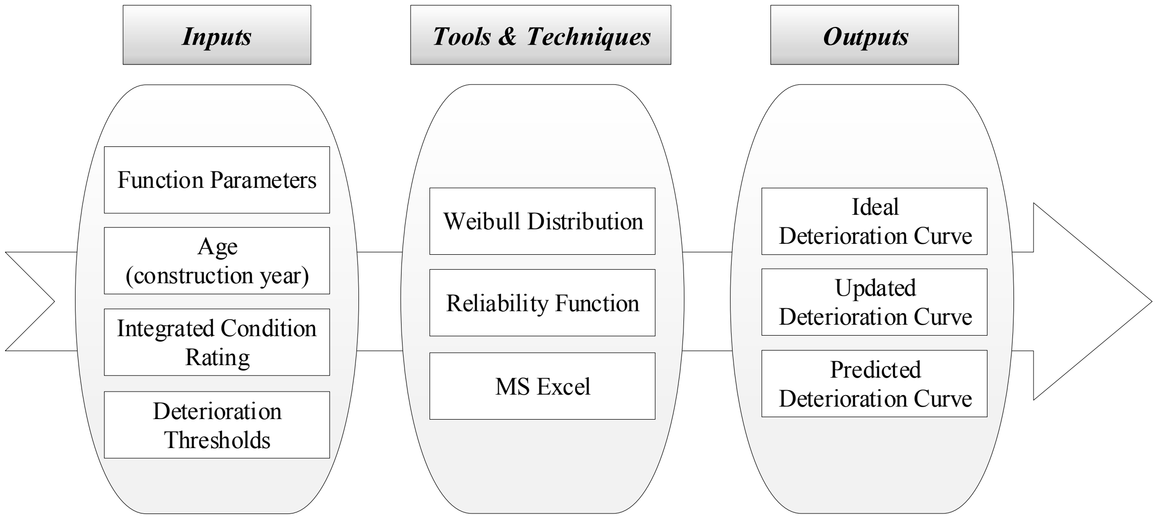


Figure 3.7: WDF deterioration model inputs and outputs

3.5.1. WDF Deterioration Curves

i. Ideal Deterioration Curve

To model deterioration over time, the literature recommends using the reliability which is the good condition state of the structural component (i.e., bridge deck). Therefore, the Weibull reliability function is plotted, shaping the component ideal deterioration curve first. The Ideal Deterioration Curve (IDC) has a similar shape of Equation 5 and is defined in Equation 9:

$$CR_I^{IDC}(t) = a \times e^{-(t/\beta)^{\alpha}} \tag{9}$$

where:

CR_I is the component integrated condition reliability of the IDC.

The condition reliability relates to the physical condition fitness of the structure or the element, indicating its structural performance state. Besides that, the IDC has the following characteristics:

- At time = 0 (initial time), the slope of the deterioration curve is equal to zero as shown in the following Equation:

$$\frac{\partial(CR_I^{IDC})}{\partial t} = CR_I^{IDC'}(t) = 0 \qquad (10)$$

- The ideal Service Life (SL) is equal to 75 years for bridges based on the Canadian Highway Bridge Design Code and AASHTO LRFD Bridge Design Specifications (Canadian Standards Association 2014; AASHTO 2014).
- The Useful Service Life (USL) is consistent with the MTQ's inspection scale and can be assumed 40% (poor state) (Semaan 2011), where the life of the structure is at the minimum acceptable level.
- The Minimum Service Life (MSL) is assumed 20% (critical state) (Semaan 2011).
- The failure level is defined as $\frac{1}{\beta}$ or the inverse of the service life adjustment parameter.

The ideal deterioration curve must go through an inflection point and then decrease sharply such that it matches the concrete deterioration trend. Thus, the deterioration parameter α should be more than 1 ($\alpha > 1$) and an integer. $\alpha = 3$ according to Semaan (2011) where 1, 2, 4 etc…will not bring the desired shape of the curve similar to the concrete deterioration trend. As the time passes, the deterioration increases and accordingly the ideal curve is constructed as follows:

- At time t = 0, the CR_I^{IDC} = 100% or 1 (maximum condition), thus:

$1 = a \times e^{-(0/\beta)^{\alpha}} = a$, so $a = 1$

- At time t = 0, the slope or the tangent equals zero, so:

$$\frac{\partial(CR_I^{IDC})}{\partial t} = CR_I^{IDC'}(t) = 0$$

- At time = SL = 75 years, CR_I^{IDC} = 20% (minimum service life), so:

If $0.2 = 1 \times e^{-(75/\beta)^{\alpha}}$, then

$\ln(0.2) = \ln(1) - (75/\beta)^{\alpha}$, and

$\ln(1) - \ln(0.2) = 0 - \ln(0.2) = -\ln(0.2) = (75/\beta)^{\alpha}$, hence

β is defined in Equation 11 as:

$$\beta = \frac{75}{[-ln(0.2)]^{1/\alpha}} = \frac{75}{\sqrt[\alpha]{[-ln(0.2)]}} \text{ , and} \tag{11}$$

β = 64 years (by taking α = 3)

In addition to the above, at t = USL, the CR_I^{IDC} = 40% (useful service life). Substituting in Eq. 9, the CR_I of the IDC is calculated using Equation 12 and the ideal curve is illustrated in Figure 3.8:

$$CR_I^{IDC}(t) = 1 \times e^{-(t/_{64})^3} \tag{12}$$

where:

CR_I = component integrated condition reliability of the IDC at time t.

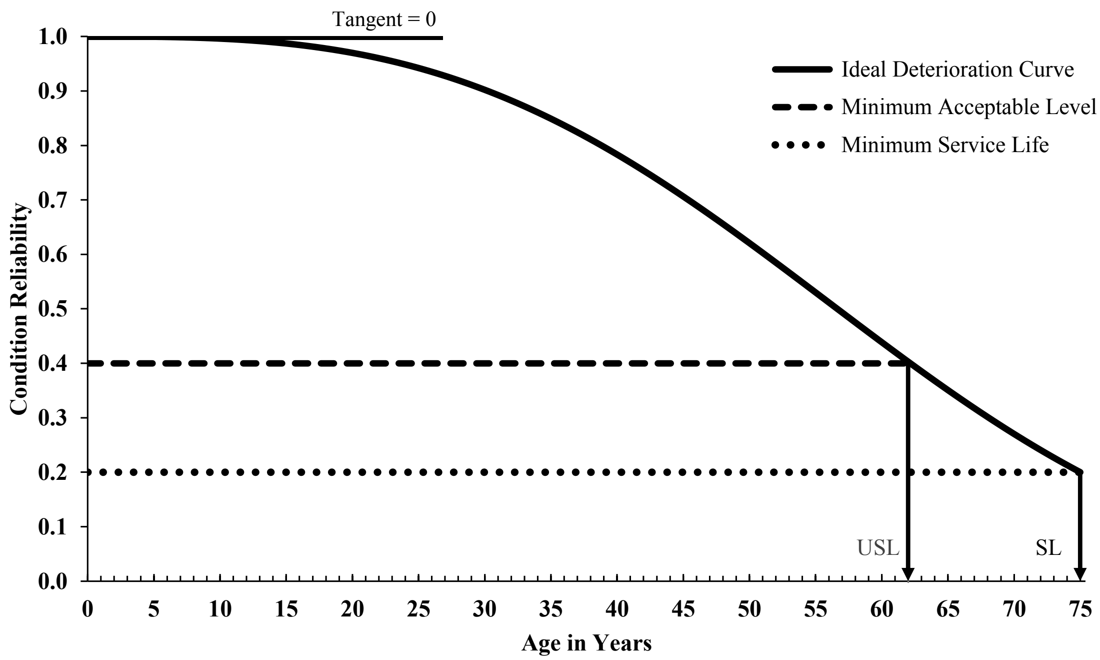


Figure 3.8: Ideal deterioration curve

ii. Updated Deterioration Curve

After evaluating the component's integrated condition C_I at the inspection time, the ideal deterioration curve needs to be updated where a new curve named the Updated Deterioration Curve (UDC) is plotted. IDC draws a theoretical 'desired' deterioration curve; however, the UDC is a real representation of component deterioration over time. Since the proposed method is concerned with modeling the change in the condition reliability of the bridge deck, the reliability value is calculated which is one minus the component's integrated condition. Hence, after performing each inspection, a new UDC is created based on new Weibull function and parameter calculations. The following characteristics are met in order to construct the UDC:

- At time t = 0 (construction year), CR_I = 1 (maximum condition), thus:

$1 = \; a \times e^{-(0/\beta)^{\alpha}} = a$, so a = 1

- At time t = 0, the slope or the tangent equals zero, so:

$$\frac{\partial(CR_I^{UDC})}{\partial t} = \; CR_I^{UDC'}(t) = 0$$

- At the time of inspection t_i, the integrated condition reliability CR_{Ii} is between 1 and 0.4, so if

$CR_{Ii} = 1 \times e^{-({t_i}/{\beta})^{\alpha}}$, thus

$ln(CR_{Ii}) = ln(1) - \; (t_i/\beta)^{\alpha}$, so

$ln(1) - ln(CR_{Ii}) = 0 - ln(CR_{Ii}) = - ln(CR_{Ii}) = (t_i/\beta)^{\alpha}$, hence

β is defined in Equation 13 as:

$$\beta = \frac{t_i}{[-\,ln(CR_{Ii})]^{1/\alpha}} = \frac{t_i}{\sqrt[\alpha]{[-\,ln(CR_{Ii})]}} \qquad (13)$$

where:

CR_{Ii} = integrated condition reliability at time t_i ; and

t_i = inspection time.

By solving Equation 13 where α = 3 and substituting in Equation 9, CR_I^{UDC} is defined in Equation 14:

$$CR_I^{UDC}(t) = 1 \times e^{\left[\frac{ln(CR_{Ii}) \times t^3}{{t_i}^3}\right]} \qquad (14)$$

Shuffling the previous equation, the CR_I of the UDC after performing inspection at t_i is illustrated in Figure 3.9 and defined in Equation 15 as:

$$CR_I^{UDC}(t) = 1 \times e^{ln(CR_{Ii})(t/t_i)^3} \qquad (15)$$

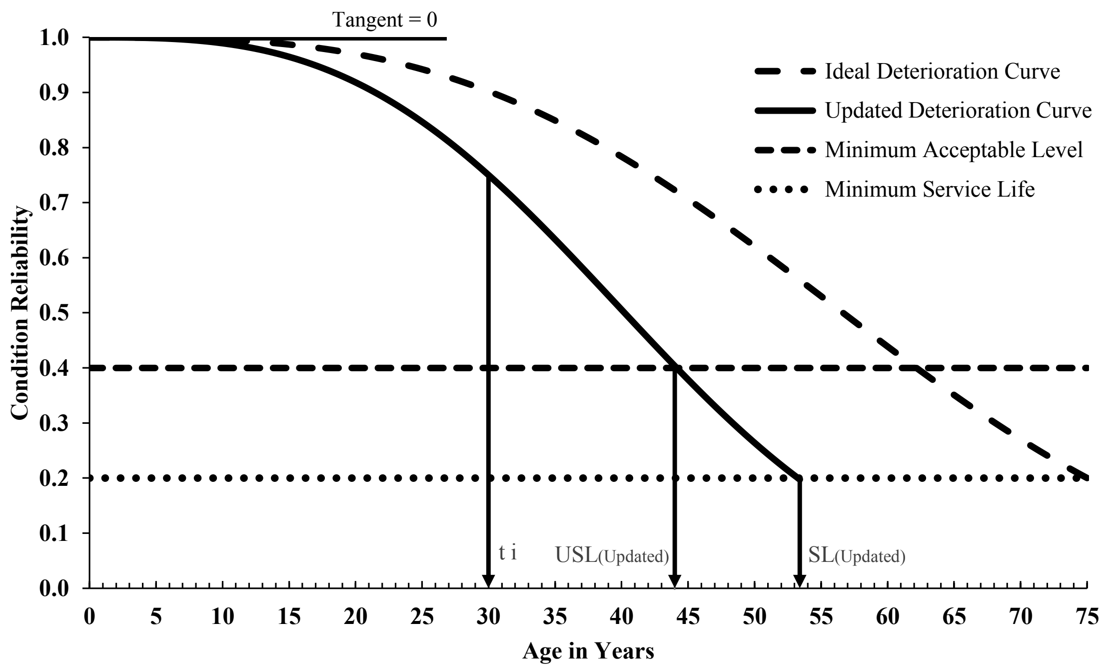


Figure 3.9: Updated deterioration curve

iii. Predicted Deterioration Curve

PDC is the latest updated deterioration curve based on the latest inspection data. The PDC predicts the future integrated condition CR_I^{PDC} and the new useful service life ($USL_{predicted}$) by considering the inspection history of the component. If no MRR action is done, the PDC follows Equation 14 and Equation 15 (i.e., same as the latest UDC evaluation). However, if rehabilitation action is done at time t_r, the PDC is divided into two parts: before the MRR action and after. In the part just before the MRR action (at t_r), PDC is identical to the UDC, and thus it follows the same derivation of the UDC equation. In this case, the integrated condition reliability before the MRR action CR_I^{PDC} is defined in Equation 16:

$$CR_I^{PDC}(t) = 1 \times e^{ln(CR_{Ir})(t/t_r)^3} \tag{16}$$

where:

CR_{Ir} = integrated condition reliability directly before the rehabilitation action; and

t_r = time of the rehabilitation action.

In the part after the MRR action (at t_r+1), PDC shape remains the same as the UPC function; however, the only difference is that at t_r+1 the integrated condition reliability is equal to the improved condition value as defined in Equation 17:

$$CR_{IR} = CR_{Ir} + \Delta R \tag{17}$$

where:

CR_{IR} = improved integrated condition reliability after the rehabilitation action; and

ΔR = rehabilitation action or MRR action improvement.

By substituting in Equation 14, the PDC for the integrated condition reliability after the MRR action CR_I^{PDC} is defined using Equation 18:

$$CR_I^{PDC}(t) = CR_{IR} \times e^{\left[\frac{ln(CR_{Ii}) \times (t - t_r + 1)^3}{{t_i}^3}\right]} \tag{18}$$

where:

CR_{IR} = improved integrated condition reliability directly after the rehabilitation action;

CR_{Ii} = integrated condition reliability after the rehabilitation action;

t_r = time of the rehabilitation action; and

t_i = inspection time after the rehabilitation action.

The predicted deterioration curve without rehabilitation action or with a specific MRR action implemented at time t_r is illustrated in Figure 3.10 and Figure 3.11, respectively.

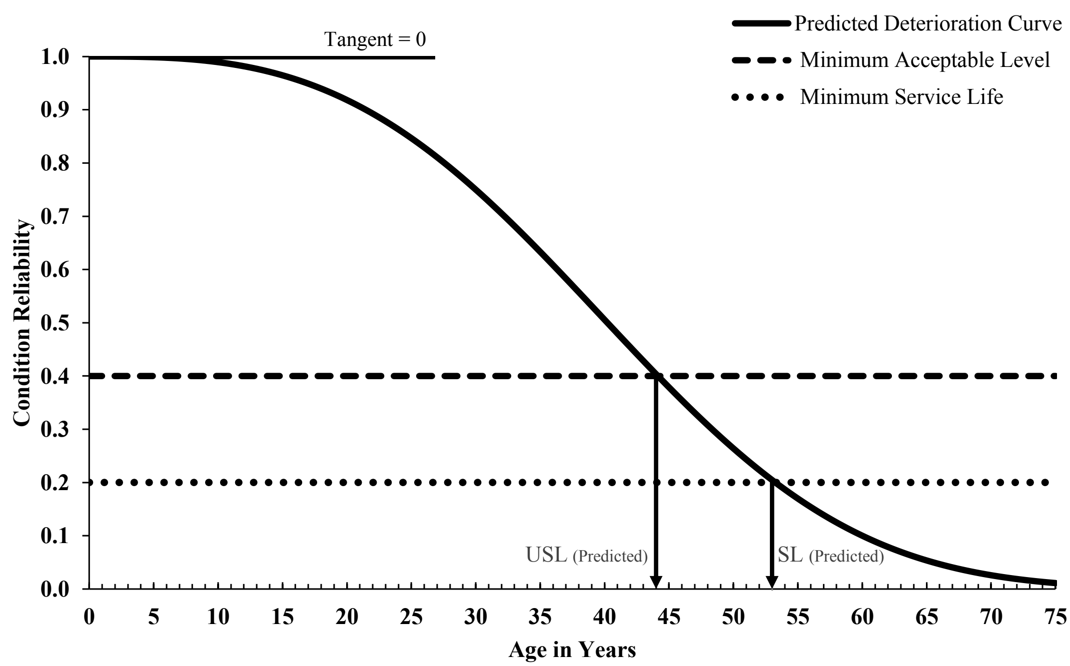


Figure 3.10: Predicted deterioration curve without rehabilitation action

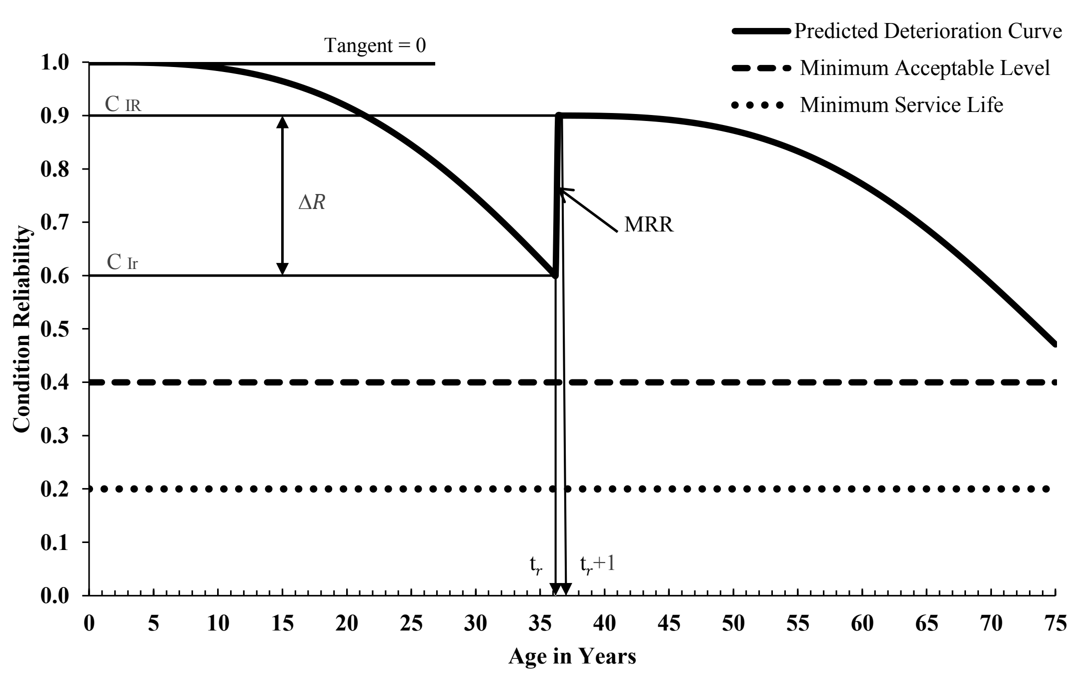


Figure 3.11: Predicted deterioration curve with MRR action

3.6. DEVELOPMENT OF MRR DECISION-MAKING OPTIMIZATION MODEL UNDER PBC SETTING

The development process of the MRR decision-making model is illustrated in Figure 3.12 and discussed in the following sections. Initially, the previously developed models are combined forming a bridge deck management system. The first model is a condition assessment model to rate current condition. The second is deterioration modeling to predict future ratings. Moreover, the third is a rehabilitation model to determine suitable MRR interventions. Then, the decision-making support model is developed based on an optimized selection of MRR actions while minimizing the total rehabilitation actions cost, maximizing condition reliability and considering PBC setting of maintaining the specified performance LOS during predefined duration.

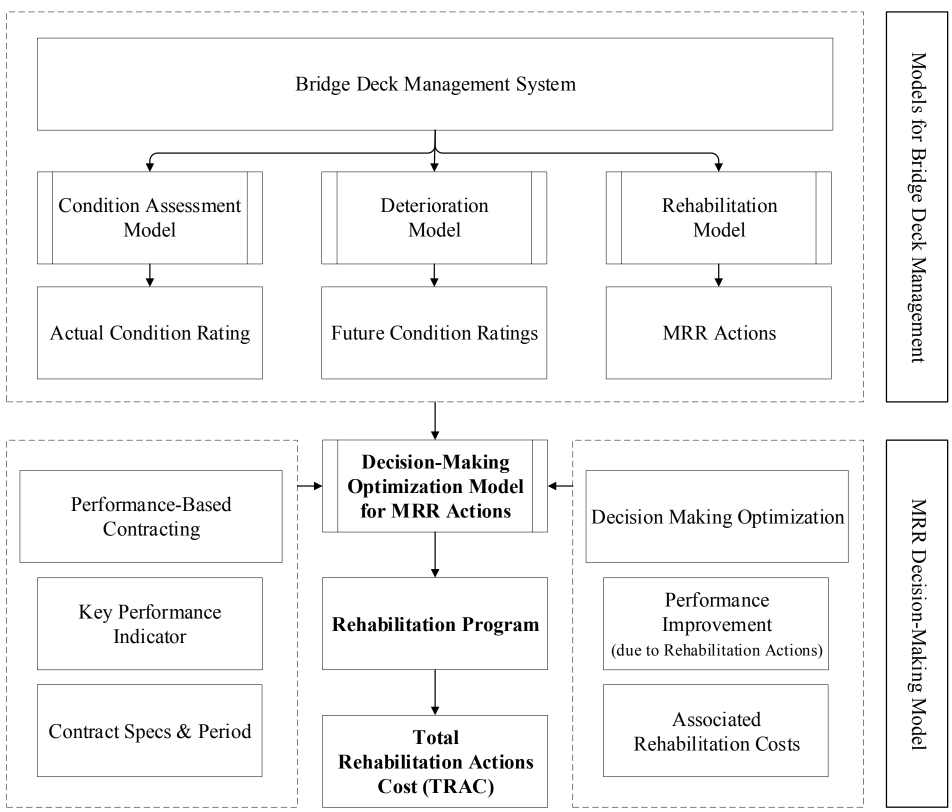


Figure 3.12: Methodology for development of decision-making optimization model

3.6.1. Formulation of Rehabilitation, Condition Improvement, and Cost Models

Rehabilitation modeling is the process of determining the suitable Maintenance, Repair and Replacement (MRR) actions. Condition improvement modeling is about predicting the conditions after performing MRR actions, and cost modeling is for estimating the associated costs of these actions.

i. Rehabilitation Model

Based on the developed QFD condition assessment model, suitable MRR types are formulated and categorized into three types of rehabilitation actions. Table 3.4 proposes an interpretation of each MRR action corresponding to the performance of the bridge deck. For bridge decks under good performances, rehabilitation actions such as regular routine maintenance could be performed. Bridge decks with poor performances need repair actions such as strengthen the thickness of these bridge decks by doing local repairs to the concrete structure. For critical performance, the action would be full deck replacement.

Table 3.4: Performance Assessment Interpretation for Bridge Decks

Severity	Performance Assessment	Action
None or Light	Excellent	No Action is needed.
Medium	Good	Bridge needs routine maintenance
Severe	Poor	Bridge needs repair
Very Severe	Critical	Bridge needs replacement

ii. Improvement Model

Modeling performance improvement and predicting the deterioration behavior of an asset after a rehabilitation action is necessary. This research classifies bridge rehabilitation actions into maintenance, repair or replacement (MRR) strategy. In reference to the literature review, condition reliability is estimated to have an improvement of 20% when repair actions are implemented on the bridge deck. In the case of bridge deck replacement, the condition reliability is raised to be equal to 90%. For maintenance action, no improvement occurs, but this action reduces the deterioration of the bridge deck represented by a steady state line where the bridge deck deteriorates quite slowly. The improvement values by the three types of rehabilitation actions relied on a study that was performed by Seo (1994). The improvements of the bridge component (i.e., bridge deck) are graphed in Figure 3.13.

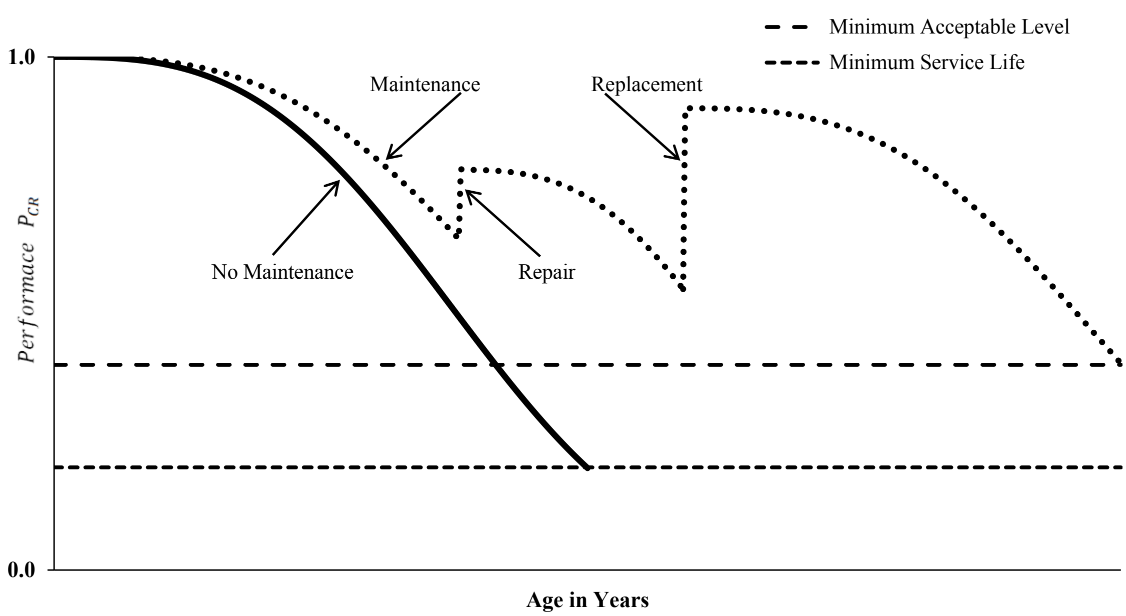


Figure 3.13: Conceptual deterioration curve with and without MRR interventions

iii. Cost Model

Calculating a component's rehabilitation cost in BMSs is often performed neither by assigning a fixed cost as a percentage of initial construction or replacement cost nor as a unit cost. Many researchers have discussed the importance of defining the cost of the available repair options; however, the cost calculations were discussed without much detail. In practice, repair costs are associated through soliciting quotes to contractors/suppliers or by consulting published data references. As an example of percentage MRR costs, Seo (1994) proposed predefined estimates for the after-repair condition for repair types (i.e., light, medium, or extensive repair). Abu Dabous and Alkass (2011) developed three unit cost models for each rehabilitation type. The first model for maintenance cost is developed based on a model from the literature. The second and third models for bridge deck repair and replacement are developed based on data and collected reports. The unit cost of each rehabilitation action includes the direct cost, indirect cost and the markup added to cover the contractor's profit and contingency. Table 3.5 summarizes the rehabilitation actions costs as a percentage of initial construction (Seo 1994) and as a fixed unit cost (Abu Dabous and Alkass 2011). The unit costs are in Canadian dollars, adjusted for inflation and based on the 2017 dollar value.

Table 3.5: Rehabilitation Strategies Improvements and Costs per Type

Rehabilitation Actions	Improvement	Rehabilitation Cost	
		% of initial construction	**unit cost ($/m^2)**
Maintenance	0%	28.5%	$26.43
Repair	20%	65.0%	$789.40
Replacement	90%	100.0%	$1033.20

3.6.2. Performance-Based Contracting Setting

Performance indicators are the basis of implementing and monitoring long term performance-based maintenance contracts. These performance criteria and their thresholds and targets are prescribed on contract specifications as performance measures or indicators.

Performance indicators are measures of quality; quite often the distress indicators which are monitored annually, or more frequently. It is assumed that these indicators which are visible distresses, and measured easily, give an accurate picture of the asset condition (Panthi 2009).

In this research, the reliability of the bridge condition is suggested to be the key performance indicator (KPI) to express a bridge's level of service (LOS). Reliability is the base measure for the bridge remaining service life, an indicator that is suggested by Haas et al. (2008; 2009). After defining the performance indicators, it is equally important to define performance thresholds such that the LOS never exceeds the threshold limit. The client (transportation) agency specifies these performance thresholds to be met or exceeded during the contract period. A threshold of 0.65 or 65% is suggested representing a good performance state. Therefore, if the key performance indicator (i.e., bridge reliability) fails to meet performance threshold (e.g., CR_I < 0.65) within the warranty period, the maintenance contractor must repair or rehabilitate the bridge and may also has to pay a penalty for failing to maintain the pre-defined bridge LOS. The client agency can define performance targets or standards where a LOS rating of 0.80 or 80% for primary systems and 0.75 or 75% for the secondary systems is desired targets (Smith et al. 1997). The threshold and target values can be adjusted by user's feedback and experience in practical situations.

3.6.3. MRR Decision-Making Optimization Model

For developing MRR decision-making model, good quality data are important for identifying feasible rehabilitation actions. These data including current condition, historical performance, and improvements of bridge decks are obtained from previous research methodology steps. The decision-making model inputs and outputs for optimizing MRR actions are shown in Figure 3.14 and described in the following section. Inputs to the decision-making optimization model are current condition and future deterioration rates, rehabilitation and condition improvement modeling with associated MRR costs represented by the bridge deck management system. In addition, a key performance indicator, named condition reliability and associated LOS threshold and target are specified as discussed in the previous section. Along with that, a pre-defined contract period and payments system including incentives and disincentives (penalties) are outlined. Outputs include a MRR decision-making optimization tool where a rehabilitation program during the defined period is generated accompanied by total rehabilitation actions cost.

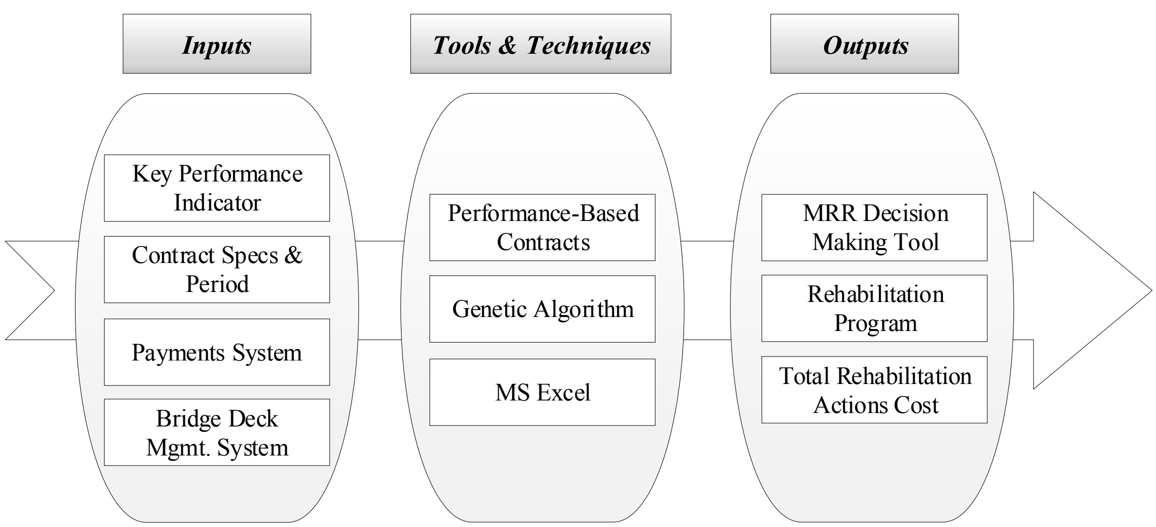


Figure 3.14: MRR decision making optimization model inputs and outputs

i. Life-Cycle Costing Model Formulation

Prediction models are the basis of life-cycle cost analysis (LCCA). This is because the maintenance costs are basically dependent on the bridge performance which varies over the analysis period. The setting of a PBC is accomplished in the traditional life-cycle cost model through adding the administration costs of the performance-based contract. These costs are the ones associated with the changing roles of the contractor assuming more administrative and maintenance works are performed in the performance-based environment. Therefore, Total Rehabilitation Actions Costs ($TRAC$) for PBC is calculated for all expected future expenses, including Rehabilitation Actions Costs (RAC) costs and PBC ADMinistrative costs (ADM). ADM cost definitions and estimates are further explained in Chapter 6. The calculations for each cost element and $TRAC$ are evaluated using net present value (NPV) and shown in the following Equations:

$$RAC_n = \sum_{n=1}^{N} \sum_{m=1}^{M} \frac{1}{(1+i)^n} (RAC_m . X_{nm}) \tag{19}$$

$$\sum_{m=1}^{M} X_{nm} = 1 \;\; \forall \, n = 1, \dots, N \tag{20}$$

$$ADM_n = \sum_{n=1}^{N} \frac{1}{(1+i)^n} (ADM) \tag{21}$$

$$TRAC = \sum_{n=1}^{N} RAC_n + ADM_n \tag{22}$$

where:

RAC_n = rehabilitation action cost at year n;

RAC_m = rehabilitation action cost associated with the selected MRR strategy m;

$X_{nm} \in \{0,1\}$ binary constrain ensures that one MRR strategy is chosen at year n of the contract period. X_{nm} = 1 if MRR action m is applied, and 0 otherwise;

ADM_n = PBC administrative costs at year n;

$TRAC$ = total costs associated with complying the contract specifications;

N = total number of contractual years (contract duration);

M = total number of applied MRR strategies m; and

i = discount rate.

ii. Optimization Model Formulation: Target Program

The outputs of the prediction model and life-cycle costing model are integrated as inputs to the optimization model in order to obtain a set of tradeoff lists of rehabilitation actions applied over the predefined period of the contracted bridge asset. The optimization model is developed to meet the needs that arise within a PBC setting. Thus, the bi-objective functions and constraints are formed as:

Objective functions:

$$minimize\ TRAC = \sum_{n=1}^{N} RAC_n + ADM_n\ \forall n, 1 \leq n \leq N \tag{23}$$

$$maximize\ P = AVG\ P_n\ \ \forall n, 1 \leq n \leq N \tag{24}$$

Subject to (constraints):

$$TRAC \leq B \tag{25}$$

$$P_{limit} < \ P_n \leq P_{max} \tag{26}$$

where:

B= budget constraint;

P_{limit} = minimum allowable performance that could be reached even after applying disincentives (penalties);

P_n = predicted performance after MRR action at year n; and

P_{max} = maximum performance.

In addition, the optimization model should account for the constraint of the contract specified LOS of the key performance indicator, condition reliability. In other words, the condition reliability state should satisfy the specified LOS threshold of the performance indicator:

$$P_{CR} > PI \tag{27}$$

where:

P_{CR} is the condition reliability performance as a result of the latest rehabilitation action applied; and

PI is the specified LOS (predefined threshold value) of the performance indicator.

The process of selection of the combination of rehabilitation actions that will maintain the specified LOS for the key performance indicator is repeated to arrive at the optimum rehabilitation program.

iii. Re-optimization Model Formulation: Updated Program

Since deterioration process in bridges follows stochastic behavior, probabilistic or stochastic modeling is being used to predict the bridge performance as a probabilistic estimate. Moreover, the deterioration process and the improvement due MRR actions vary based on many factors such as aggressive environments, traffic wear and tear, material properties, in addition to other influencing factors (e.g., freeze-thaw cycling). Furthermore, performance-based contracting involves repetition and could be extended to 30 years (McCullouch et al. 2009). In addition, in case of failing to achieve the specified performance LOS, disincentives (penalties) will be applied. In contrary, if applicable, incentives defined in the contract are applied once the performance exceeds the target LOS. The updated program is offered to update the bridge performance using real time data after performing inspections and to re-optimize the rehabilitation program with the constraint of new budget. The new budget constraint is basically the remaining of the total cost estimated from the target program in addition to any incentives (INC) or disincentives (DIS), if applied. Thus, Total Rehabilitation Actions Costs ($TRAC$) for PBC is calculated for all expected future expenses, including any incentives (INC) or disincentives (DIS) as follows:

$$TRAC = \sum_{n=1}^{N} RAC_n + ADM_n - INC_n + DIS_n \tag{28}$$

$$INC_n = \sum_{n=1}^{N} \frac{1}{(1+i)^n} (INC) \tag{29}$$

$$DIS_n = \sum_{n=1}^{N} \frac{1}{(1+i)^n} (DIS) \tag{30}$$

where:

INC_n = incentive for achieving performance indicator target as defined in the contract; and

DIS_n = disincentive (penalty) failing to maintain performance indicator threshold as defined in the contract in year n.

Therefore, the objective from the re-optimization process is to maintain the specified performance indicator such that its LOS never exceed the threshold limits for the remaining period of the contract with the remaining budget. This can be achieved by reconstructing the objective function and constraint Equations as follows while keeping the other optimization model formulation (Equations 22, 26, and 27) as it is:

Objective function:

$$minimize\ TRAC = \sum_{n=y}^{N} RAC_n + ADM_n - INC_n + DIS_n \tag{31}$$

Subject to (constraint):

$$TRAC \leq Remaining\ Budget \tag{32}$$

where:

y = the year at which the re-optimization is applied; and the remaining budget is the amount of money remaining from the total cost submitted in the bid.

iv. Solution Approach

The multi-objective optimization problem usually has no unique, perfect solution, but a set of non-dominated, alternative solutions, known as the Pareto optimal set (Fonseca and Fleming 1998). In order to obtain all the Pareto optimal solutions, the Weighted Comprehensive Criterion Method (WCCM) is used to solve the bi-objective model described previously. WCCM is a scalarization approach that uses normalization to allow combining objective functions with different units in one objective function. The use of weighted methods is a common means of providing the Pareto optimal set and has been used in many multi-objective optimization fields (Alsyouf and Hamdan 2017; Hamdan and Cheaitou 2017).

The procedure to employ the WCCM in the current research can be described in the following steps:

1. The bi-objective model defined previously is divided into two single objective functions. Each function is solved separately, subject to its associated constraint(s), to obtain the two single objective function optimal solutions, $TRAC_{min}$ and P_{max}.
2. Each objective function is normalized using the weighted method, where the relative variation (normalization) is calculated in Equation 33 for the minimization problem and Equation 34 for the maximization problem, respectively as follows:

$$f_1 = \frac{TRAC - TRAC_{limit}}{TRAC_{limit}} \tag{33}$$

$$f_2 = \frac{P_{limit} - P}{P_{limit}} \tag{34}$$

3. Both normalized functions are multiplied by relative weights and then combined into a single objective function that aims to minimize the total relative variation of each objective function as follows:

$$\min f = \alpha_1 f_1 + \alpha_2 f_2 \tag{35}$$

where: α_1 and α_2 are the relative weights that the decision maker can set and $\alpha_1 + \alpha_2 = 1$.

For given values of the relative weights, only one Pareto optimal solution is obtained while changing the values of α_1 and α_2 may result in different Pareto optimal solutions. Each solution is non-dominated by any other solution.

CHAPTER 4: DATA COLLECTION AND ANALYSIS

This research aims to develop an integrated decision support methodology for bridge deck management under performance-based contracting setting. Given the scope of the research, data solicited for the proposed methodology consist of three main types (Figure 4.1):

1. Case studies: information related to the extents and severities of the detected defects in studied cases collected from detailed inspection reports and condition surveys of concrete bridges.
2. Defects correlation: the proposed condition assessment model is set to correlate the different identified defects. Defects correlation forms the top roof of the HOQ.
3. PBC payment system: this covers all costs including administrative costs of the performance-based contract and disincentives (penalties).

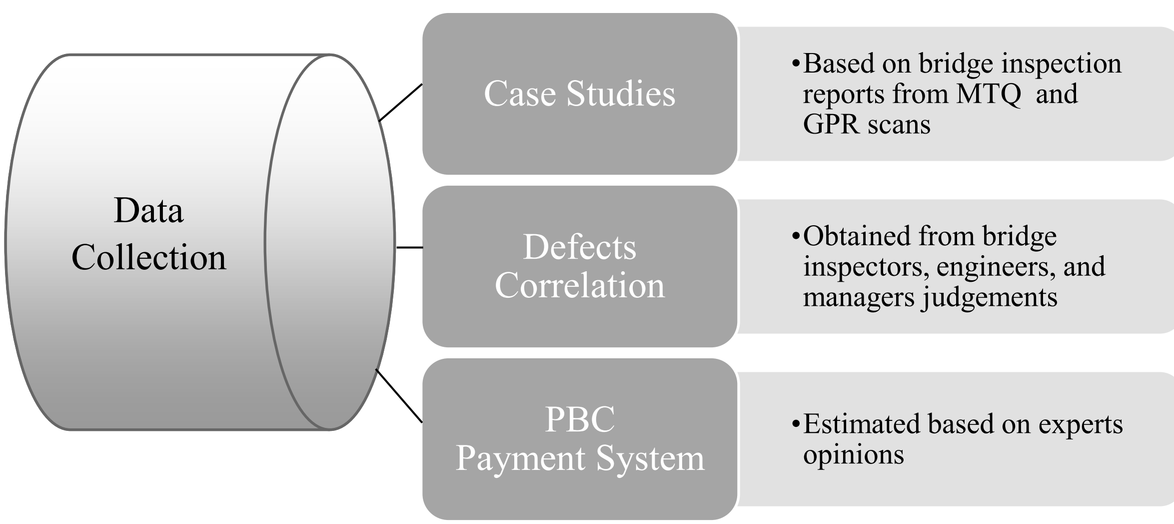


Figure 4.1: Model datasets collection

4.1. CASE STUDIES

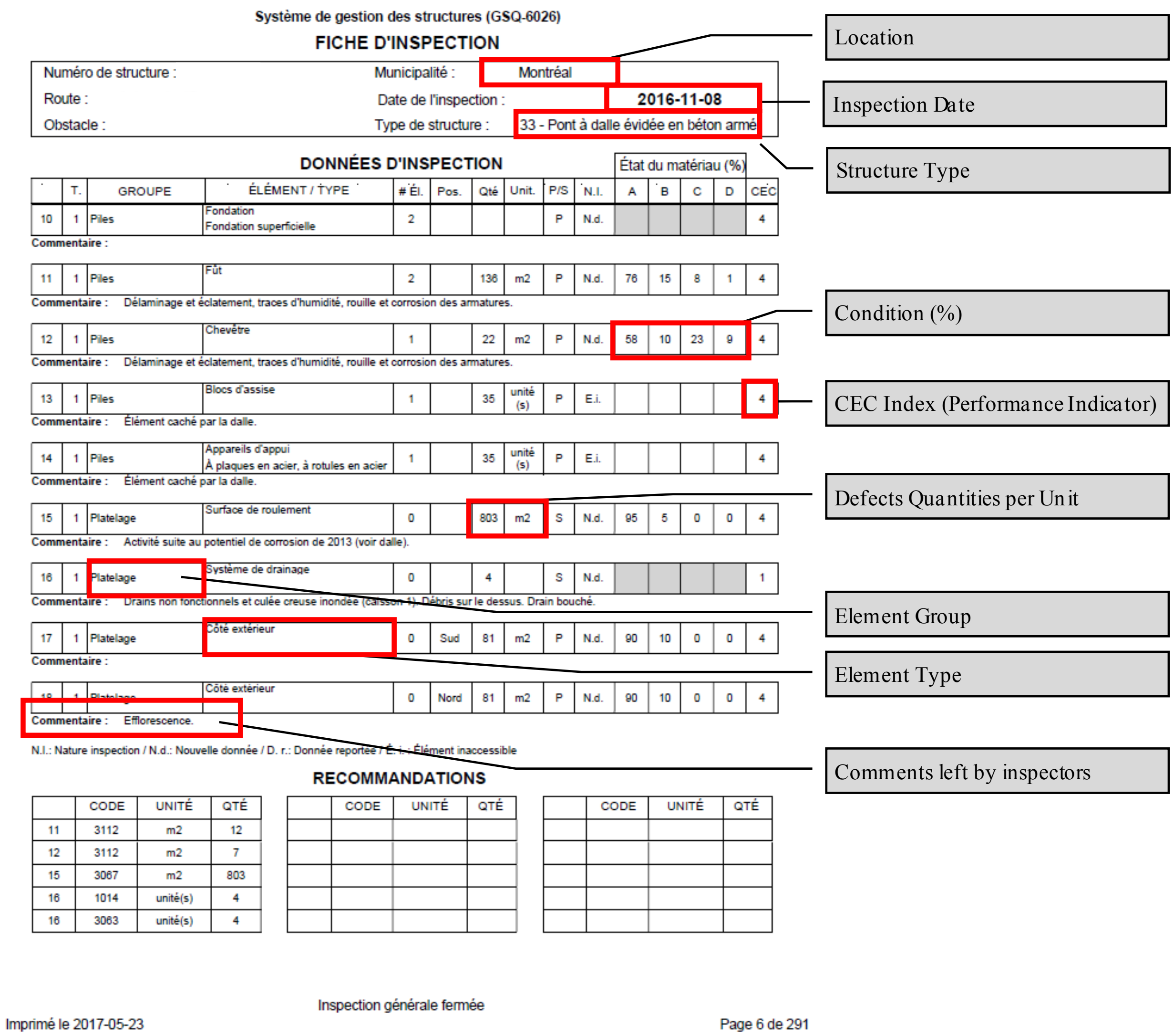
This type of data includes detailed inspection reports and condition surveys of twenty case studies of existing concrete bridges. Two approaches of data collection are employed in this research. The first approach of data collection is by gathering inspection reports from the Ministère des Transports du Québec (MTQ) in order to get info related to surface defects detected from visual inspections. The second approach of collecting data is done by the research team where the same inspected bridge decks are scanned using GSSI ground-coupled radar system. This approach is important in order to assess and evaluate corrosion potentials of steel reinforcements, which is a subsurface defect.

4.1.1. Visual Inspection

Data for case studies related to extents and severities of detected surface defects is collected through careful reviewing of detailed bridge inspection/condition survey reports. Information about the studied bridge is also extracted from the inspection report such as: bridge location, structure type, construction year, inspection date, next inspection year, description of defects, defects quantities, condition in percentages (%s), general condition indicators, … etc. A sample of inspection report provided by MTQ is shown in Figure 4.2.

4.1.2. GPR Surveys

GPR surveys for the bridge decks were performed following two simple procedures: establishing the scanning paths and scanning the deck.



Système de gestion des structures (GSQ-6026)

FICHE D'INSPECTION

Numéro de structure :	Municipalité : Montréal
Route :	Date de l'inspection : 2016-11-08
Obstacle :	Type de structure : 33 - Pont à dalle évidée en béton armé

DONNÉES D'INSPECTION

	T.	GROUPE	ÉLÉMENT / TYPE	# Él.	Pos.	Qté	Unit.	P/S	N.I.	A	B	C	D	CEC
10	1	Piles	Fondation Fondation superficielle	2				P	N.d.					4
Commentaire :														
11	1	Piles	Fût	2		136	m2	P	N.d.	76	15	8	1	4
Commentaire :	Délaminage et éclatement, traces d'humidité, rouille et corrosion des armatures.													
12	1	Piles	Chevêtre	1		22	m2	P	N.d.	58	10	23	9	4
Commentaire :	Délaminage et éclatement, traces d'humidité, rouille et corrosion des armatures.													
13	1	Piles	Blocs d'assise	1		35	unité (s)	P	E.i.					4
Commentaire :	Élément caché par la dalle.													
14	1	Piles	Appareils d'appui À plaques en acier, à rotules en acier	1		35	unité (s)	P	E.i.					4
Commentaire :	Élément caché par la dalle.													
15	1	Platelage	Surface de roulement	0		803	m2	S	N.d.	95	5	0	0	4
Commentaire :	Activité suite au potentiel de corrosion de 2013 (voir dalle).													
16	1	Platelage	Système de drainage	0		4		S	N.d.					1
Commentaire :	Drains non fonctionnels et culée creuse inondée (caisson 1). Débris sur le dessus. Drain bouché.													
17	1	Platelage	Côté extérieur	0	Sud	81	m2	P	N.d.	90	10	0	0	4
Commentaire :														
18	1	Platelage	Côté extérieur	0	Nord	81	m2	P	N.d.	90	10	0	0	4
Commentaire :	Efflorescence.													

État du matériau (%)

N.I.: Nature inspection / N.d.: Nouvelle donnée / D. r.: Donnée reportée / É. i.: Élément inaccessible

RECOMMANDATIONS

	CODE	UNITÉ	QTÉ
11	3112	m2	12
12	3112	m2	7
15	3067	m2	803
16	1014	unité(s)	4
16	3063	unité(s)	4

Inspection générale fermée

Imprimé le 2017-05-23

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Figure 4.2: Sample of inspection report provided by MTQ

Establish the Scanning Paths

After studying the bridge deck geometry, a grid of scanning paths with 0.6 meters spacing was established as shown in Figure 4.3 to cover roadway passes over the deck. According to ASTM D6087 for standard GPR test requirements, this spacing is acceptable for GPR inspection. For each path, its two ending points were determined by a survey tape, measuring from curb to curb. Then, these points were marked. Before scanning each path, a survey string (pink color) was used to make a straight line between the two ending points. The purpose of establishing the scanning path is to assist and guide the inspector while moving the GPR tool in straight lines.

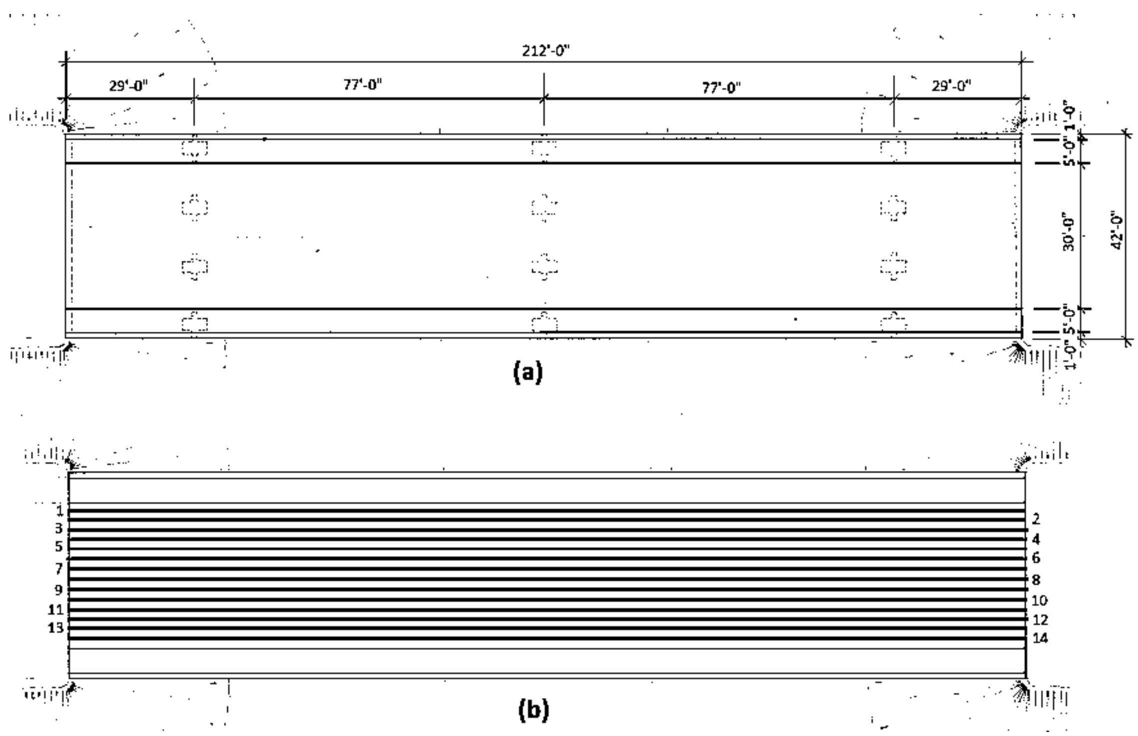


Figure 4.3: Bridge A deck plan and established scanning paths

Scan the Bridge Deck

A pushing cart carrying GSSI GPR antenna was then pushed by an operator, following the pink line as shown in Figure 4.4. For this case study (Bridge A), the scanning took less than one hour to finish all fourteen paths, where each line was scanned twice by going forward and backward.

Figure 4.4: Bridge A deck scanning with GPR tool

Data analyses are divided into two parts. The first part related to data sets from the collected inspection reports. These reports were carefully studied to extract information related to the extents and severities of the detected defects on the studied bridges. The second part is analyzing data extracted from GPR profiles. Each of the following sections explains the procedures to analyze GPR data.

Rebar Picking

After scanning bridge decks, the GPR profiles are imported to GSSI RADAN 7® software in order to find the amplitude of the reflected wave of the top reinforcing bars by picking the peak of the parabolic shapes that represent the location of the reinforcing bars as shown in Figure 4.5. This step had been repeated for all profiles of bridge decks, then each profile with its corresponding amplitude values is exported into an excel sheet. These excel sheets are used in the process of depth-correction.

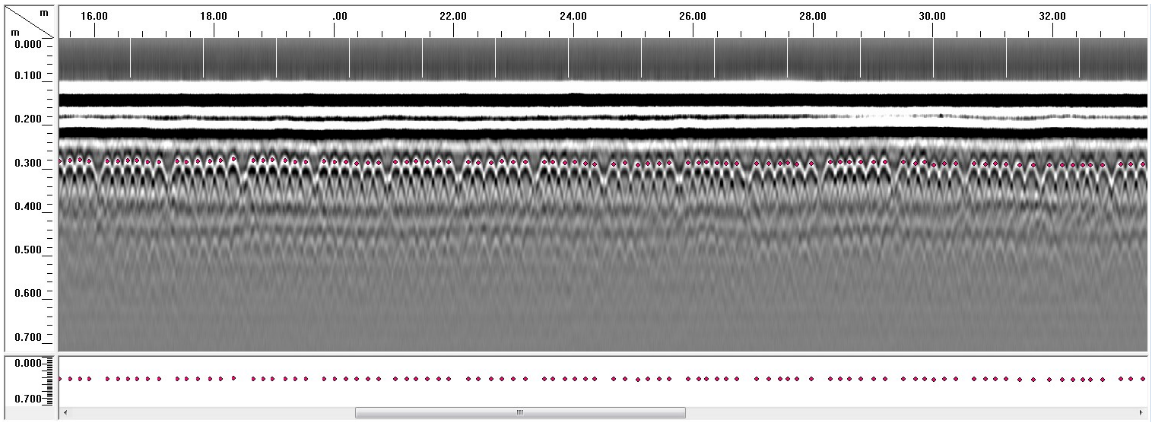


Figure 4.5: Example of picking the top reinforcing bars

Depth Correction

In practice, even in the newly constructed bridge decks, there is a variation in the depth of the reinforcing bars due to construction issue and waves of GPR are so sensitive to the variation in depth; therefore, any small difference in depth could lead to errors in the final result (Gucunski et al. 2011; Romero et al. 2015). In order to eliminate the effect of this difference, depth-correction is done by plotting the two-way travel time on the x-axis and the amplitude on the y-axis. The process of depth-correction is performed by using an automated tool developed by Dinh et al. (2015). This tool provides new values of amplitude after performing the depth-correction.

Mapping Corrosion

For visualization and mapping corrosion, the corrected values obtained in the previous step are imported to Surfer®, a popular mapping software, to create an attenuation map. This attenuation (decibel) output map can then be used for evaluating the bridge deck corrosion condition, where thresholds of different categories of corrosion are defined by Shami

(2015) and Alsharqawi et al. (2016a). Corrosion maps for the case studies are presented in chapter 5, where model implementation is detailed.

4.2. DEFECTS CORRELATION

As mentioned earlier, it is intended for the QFD condition assessment model to discover the relationship degree between the detected defects. Degrees are obtained through a questionnaire that was distributed and circulated to bridge inspectors, engineers, and managers. Questions were formulated to guide respondents through a pairwise comparison procedure. Two defects were correlated at a time by participants using a simply defined rating scale displayed in Table 4.1. Additionally, a brief definition table (Table 3.1) with all identified bridge deck defects was appended at the end of the questionnaire for reference.

Table 4.1: Correlation Linguistic-Numerical Scale

Degree of relationship				
V. Weak	Weak	Moderate	Strong	V. Strong
0.1	0.3	0.5	0.7	0.9

At every step, respondents were advised to carry out the pairwise comparison between two defects (D_1) and (D_2) as follows:

1. If the comparison indicates that defect (D_1) has a strong relationship with defect (D_2), please indicate so by choosing a value from the middle to the right using ascending degree of relationship scale.

2. If the comparison indicates that defect (D_1) has a weak relationship with defect (D_2), please indicate so by choosing a value from the middle to the left using descending degree of relationship scale.
3. If the comparison indicates that defect (D_1) has a moderate relationship with defect (D_2), please mark "0.5".

The following figure summarizes the resultant correlation values, which were obtained from averaging the received responses. These values are taken as default correlations however the user/expert can adjust them based on his/her judgment.

(1) Cracking
(2) Disintegration, Scalling and Erosion
(3) Corrosion of Steel Reinforcements
(4) Delamination
(5) Spalling
(6) Deposits
(7) Joints Problems
(8) Pop-outs

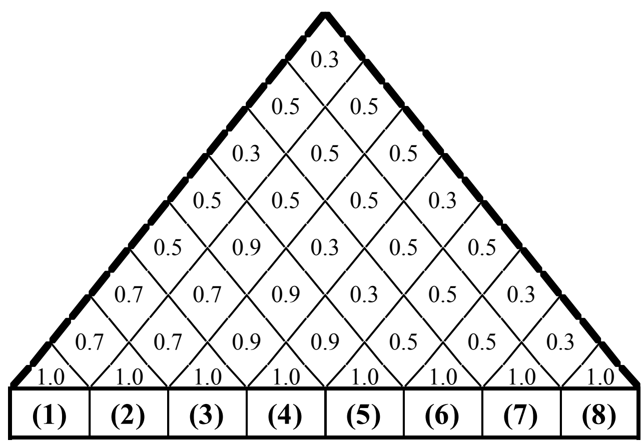


Figure 4.6: Bridge deck defects' correlation values

4.3. PBC PAYMENT SYSTEM

The third type of data had drawn from the population of experts and practitioners in performance-based contracting. Initially, actual performance-based contracts have been reviewed for performed projects. Costs have been studied and analyzed prior to establishing a payments system for such type of contract (i.e., PBC). This data type is explained in chapter 6, where the PBC framework is detailed. Experts' opinions of the targeted population are grasped through a group decision-making technique. Unanimity as one of the methods of reaching a group decision is used in this research. PMI defines unanimity as "a decision that is reached whereby everyone agrees on a single course of action" (Project Management Institute 2013). In this instance, a group of experts in PBC agreed on estimated administrative costs of the performance-based contract and disincentives (penalties). The estimate values have been materialized by structured interviews, and consecutively processing responses from the interviewed experts. However, these estimates can be adjusted by user feedback and experience in practical situations.

4.4. QUESTIONNAIRE AND INTERVIEW RESPONDENTS

The questionnaire was distributed and circulated in an effort to procure responses from field-related experts. Structured interviews were undertaken throughout the research with experts and practitioners in performance-based contracting. The purpose was to ensure practicality for real life analysis and credibility of proposed attributes. 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. Answers were obtained from different countries including the United States, Canada, Spain, United Arab Emirates, and Australia. Responses were successfully collected and the replies were thoroughly examined.

Based on the provided feedback, the respondents' years of experience varied. Figure 4.7 shows the respondents' classification based on the years of experience. The highest participation rate belongs to youngest professionals with 35% responses. Almost one-quarter of the respondents had between 5 to 10 years of experience at 23% followed by respondents ranging from 10 to 20 years of experience constituting 18% of the respondents. The senior group represented by experts with years of experience more than or equal to 20 years accounted for 24% of the overall respondents.

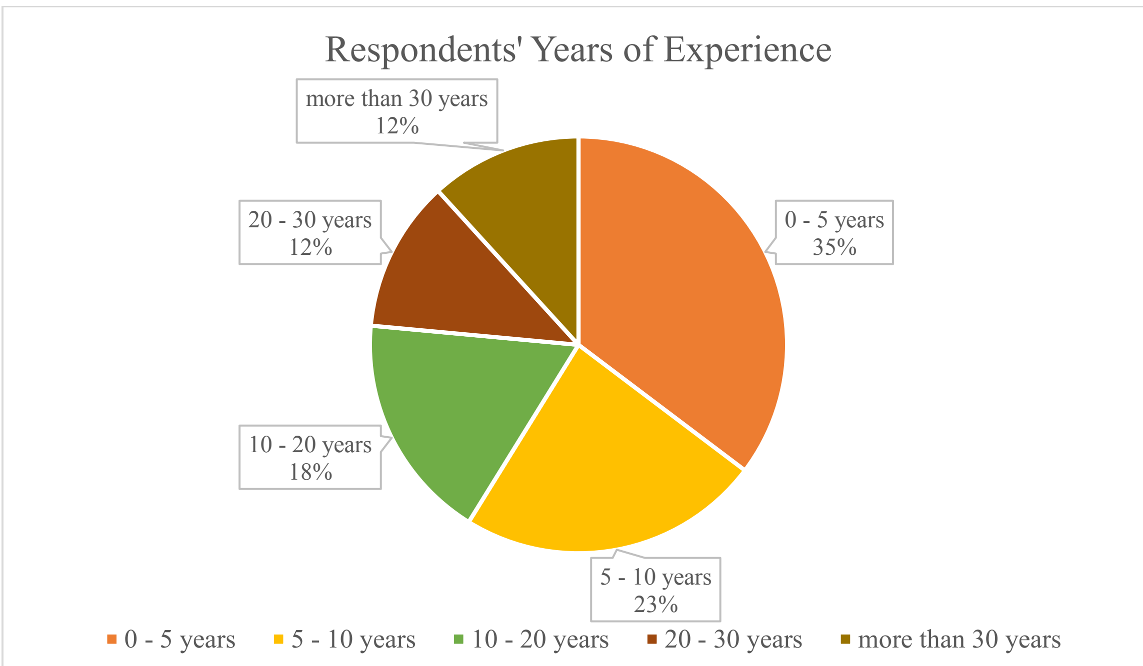


Figure 4.7: Respondents' classification with years of experience

CHAPTER 5: MODEL IMPLEMENTATION

Case studies are discussed here as a proof of concept and illustration of the presented models. These models are applied to bridges located in Québec, Canada. Names and exact locations of the bridges are not given for confidentiality purposes as per agreements with data providers.

5.1. IMPLEMENTATION OF CONDITION ASSESSMENT MODEL

To implement the developed model, four case studies have been studied and investigated from the collected data. Each of them is described in turn as follows.

5.1.1. Bridge A

The bridge is located in the municipality of Les Cèdres, Québec. The reinforced concrete bridge was built in 1965. It consists of four spans in the North-South direction. The deck was formed by a thick slab (varied in thickness between 0.61 and 1.07 meters) resting directly on piers and abutments. It has a total length of 64.7 meters and a total width of the deck is 12.8 meters with 9.1 meters of roadway. As shown in Figure 5.1 (a), the output provided by the corrosion map using GPR is area percentages of each condition category subject to rebar corrosion. The percentages values are then combined with other defect measurements (obtained from the inspection report) through the QFD assessment model to obtain an integrated condition assessment of the bridge deck as is shown by Figure 5.1 (b).

The coefficients r_{ij} in the relationship matrix are normalized considering the correlation values and the weights of the detected defects (the roof of the HOQ). Using Equation 6, normalized coefficients $r_{i,j}^{norm}$ of condition scale and defects are computed. For example, Wasserman's normalized correlations $r_{2,1}^{norm}$ and $r_{2,2}^{norm}$ (between condition scale "medium" and defect D_1 (Cracking) and between condition scale "medium" and defect D_2 are computed as follows:

$$r_{2,1}^{norm} = \frac{r_{2,1} \cdot \gamma_{1,1} + r_{2,2} \cdot \gamma_{2,1} + r_{2,3} \cdot \gamma_{3,1} + r_{2,4} \cdot \gamma_{4,1} + r_{2,5} \cdot \gamma_{5,1}}{r_{2,1}(\gamma_{1,1} + \gamma_{1,2} + \cdots + \gamma_{1,5}) + r_{2,2}(\gamma_{2,1} + \gamma_{2,2} + \cdots + \gamma_{2,5}) + \cdots + r_{2,5}(\gamma_{5,1} + \gamma_{5,2} + \cdots + \gamma_{5,5})} \text{ and}$$

$$r_{2,1}^{norm} = \frac{0.5\times1.0 + 0.4\times0.7 + 0.2\times0.7 + 1\times0.5 + 0.83\times0.5}{0.5\times3.4 + 0.4\times4.0 + 0.2\times4.2 + 1\times4.0 + 0.83\times4.2} = 1.83$$

$$r_{2,2}^{norm} = \frac{r_{2,1} \cdot \gamma_{1,2} + r_{2,2} \cdot \gamma_{2,2} + r_{2,3} \cdot \gamma_{3,2} + r_{2,4} \cdot \gamma_{4,2} + r_{2,5} \cdot \gamma_{5,2}}{r_{2,1}(\gamma_{1,1} + \gamma_{1,2} + \cdots + \gamma_{1,5}) + r_{2,2}(\gamma_{2,1} + \gamma_{2,2} + \cdots + \gamma_{2,5}) + \cdots + r_{2,5}(\gamma_{5,1} + \gamma_{5,2} + \cdots + \gamma_{5,5})} \text{ and}$$

$$r_{2,2}^{norm} = \frac{0.5\times0.7 + 0.4\times1.0 + 0.2\times0.7 + 1\times0.7 + 0.83\times0.9}{0.5\times3.4 + 0.4\times4.0 + 0.2\times4.2 + 1\times4.0 + 0.83\times4.2} = 2.34$$

The weight of the condition scale "medium" is computed using Equation 7 as follows:

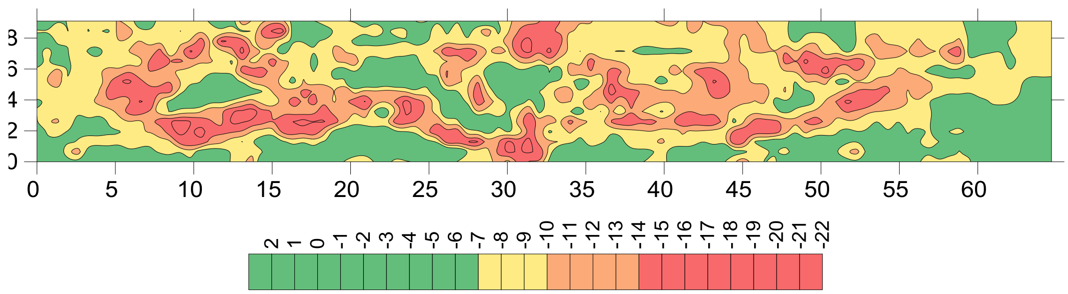
$$w = \sum_{j=1}^{8} d_j \,.\, r_{i,j}^{norm} = 3\ (1.83 + 2.34 + 2.47 + 2.46 + 2.52 + 0 + 0 + 0) = 34.85$$

The last step is to convert the calculated weights (of all condition scale "none or light", "medium", "severe", "very severe") into percentages by dividing each one over the weights summation. A final crisp integrated condition value C_I is calculated representing the bridge deck overall condition using Equation 8:

$C_I = (1 \times Good + 3 \times medium + 6 \times Severe + 9 \times V.Severe)/9$ and

$C_I = (1 \times 0.0657 + 3 \times 0.5381 \; + \; 6 \times 0.3102 \; + \; 9 \times 0.0860)/9$

By applying the QFD model, an integrated condition value C_I of 0.48 is estimated, indicating that the bridge deck condition is poor and needs repair.

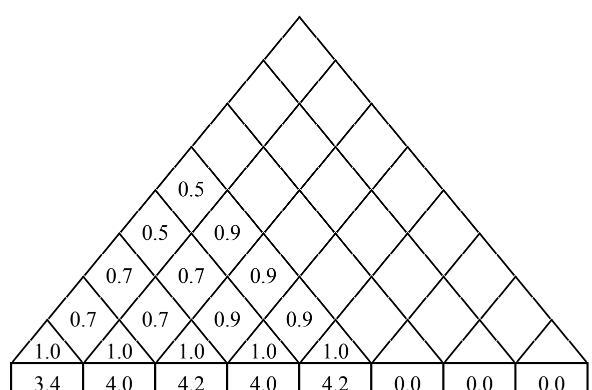


Good	Fair	Poor	Critical
48.90%	19.79%	17.58%	13.73%

(a)

Quality Function Deployment (QFD) Model

(1) Cracking
(2) Disintegration, Scalling and Erosion
(3) Corrosion of Steel Reinforcements
(4) Delamination
(5) Spalling
(6) Deposits
(7) Joints Problems
(8) Pop-outs



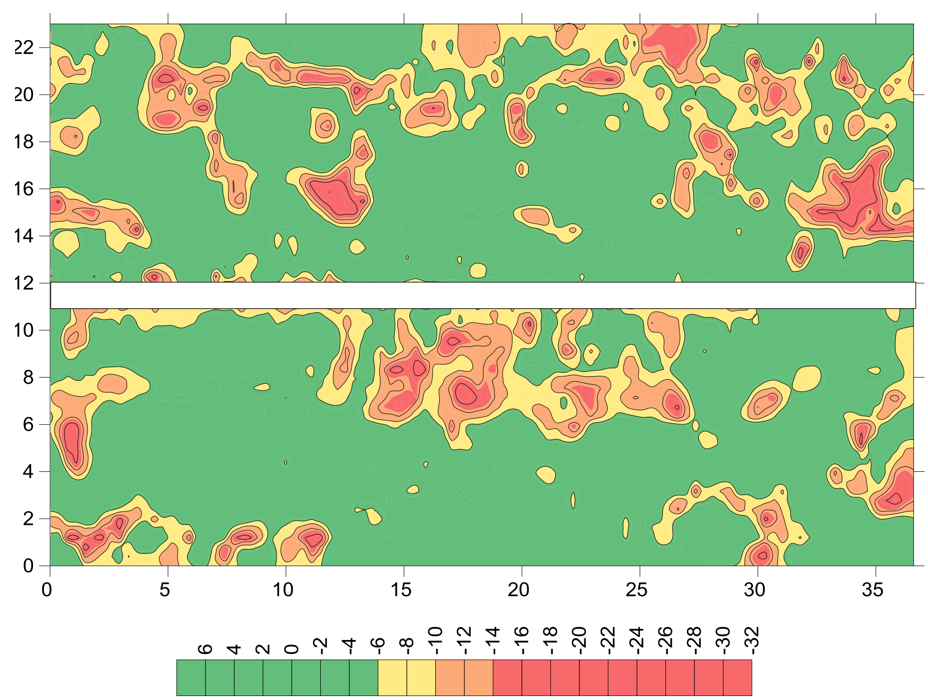
Description	Rating	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	
None or Light	1	50%	0%	49%	0%	12%				6.57%
Medium	3	50%	40%	20%	100%	83%				53.81%
Severe	6	0%	60%	18%	0%	5%				31.02%
Very Severe	9	0%	0%	14%	0%	1%				8.60%

(b)

Figure 5.1: (a) Corrosion map (b) QFD model application for Bridge A

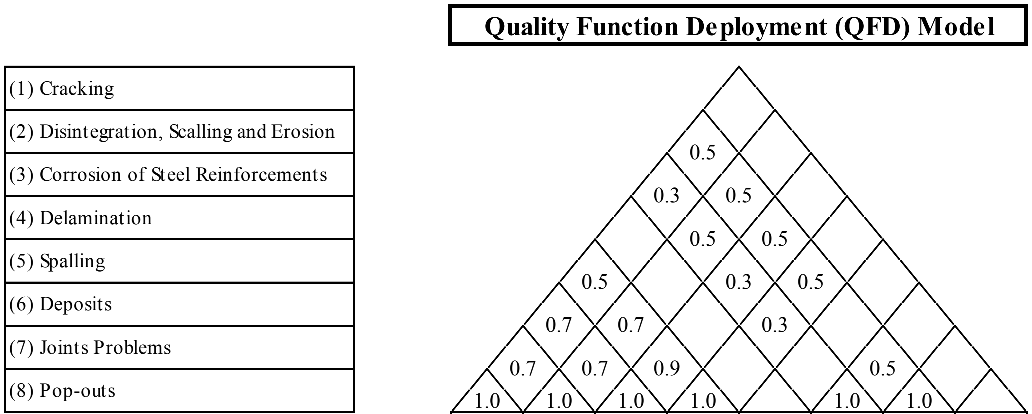
5.1.2. Bridge B

The bridge is located in the city of Montréal with high density traffic. The reinforced concrete hollow core bridge facilitates two-way traffic with three lanes in each direction. The bridge was built in 1960. It has a total length of 443.322 meters and a width of 24.384 meters with 10.9 meters of roadway for each direction. The bridge deck is 36.6 meters long and is supported by four longitudinal concrete walls. As is shown, the result in Figure 5.2, represents the C_I value of 0.49. Based on Table 3.3, intervention for the bridge owner is doing repair action over precast deck panels (PCP) using available techniques such as patching or a full-depth deck repair. The selection of which technique should depend on level of damage extends into the PCP portion of the deck.



Good	Fair	Poor	Critical
73.14%	9.86%	9.95%	7.06%

(a)



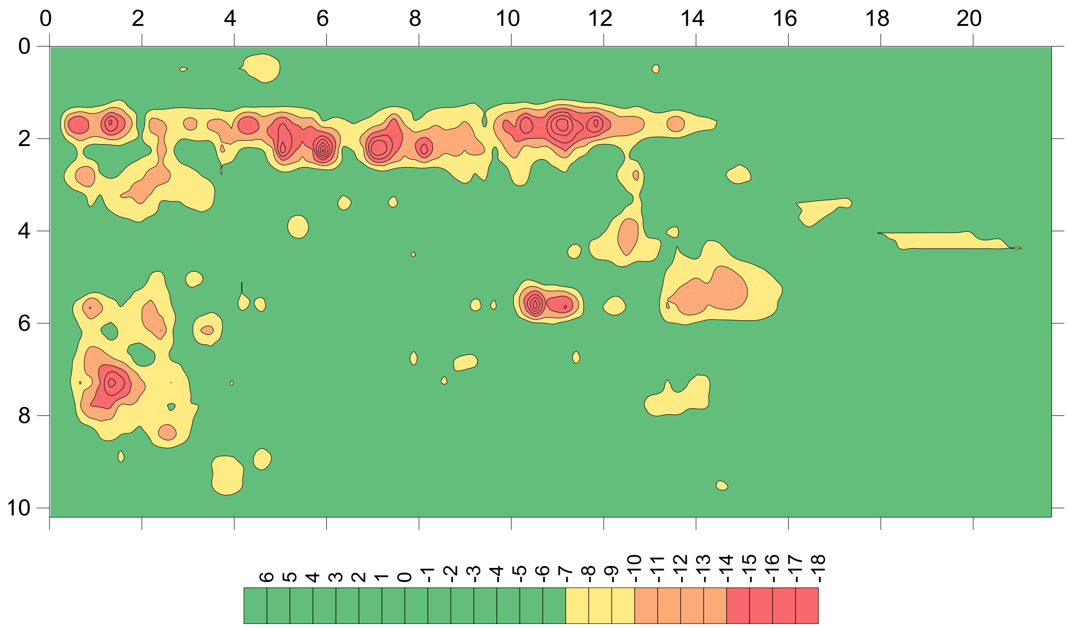
Description	Rating	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	
		3.7	4.1	4.1	3.9	0.0	2.9	3.5	0.0	
None or Light	1	94%	46%	73%	59%		49%	73%		27.60%
Medium	3	6%	30%	10%	9%		27%	1%		17.02%
Severe	6	0%	22%	10%	30%		21%	25%		44.67%
Very Severe	9	0%	3%	7%	2%		3%	1%		10.72%

(b)

Figure 5.2: (a) Corrosion map (b) QFD model application for Bridge B

5.1.3. Bridge C

The Bridge C is a small concrete bridge located in the municipality of Frelighsburg, Québec. The bridge was built in 1960. The bridge deck has a total length of 21.7 meters and a width of 11.11 meters with 10.2 meters of roadway. The C_I value of 0.37, calculated from the result in Figure 5.3 (b), suggests that the deck of Bridge C is in good condition. The recommendation for this deck is to perform routine maintenance.

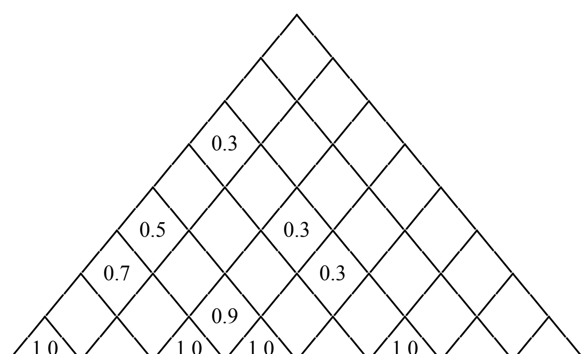


Good	Fair	Poor	Critical
84.68%	6.70%	5.39%	3.23%

(a)

Quality Function Deployment (QFD) Model

(1) Cracking
(2) Disintegration, Scalling and Erosion
(3) Corrosion of Steel Reinforcements
(4) Delamination
(5) Spalling
(6) Deposits
(7) Joints Problems
(8) Pop-outs



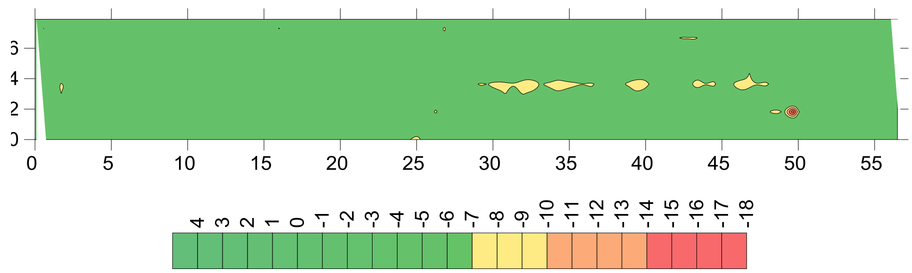
		2.5	0.0	2.9	2.7	0.0	1.9	0.0	0.0	
Description	Rating	**(1)**	**(2)**	**(3)**	**(4)**	**(5)**	**(6)**	**(7)**	**(8)**	
None or Light	**1**	90%		85%	25%		95%			**38.46%**
Medium	**3**	10%		7%	50%		5%			**30.33%**
Severe	**6**	0%		5%	25%		0%			**26.69%**
Very Severe	**9**	0%		3%	0%		0%			**4.51%**

(b)

Figure 5.3: (a) Corrosion map (b) QFD model application for Bridge C

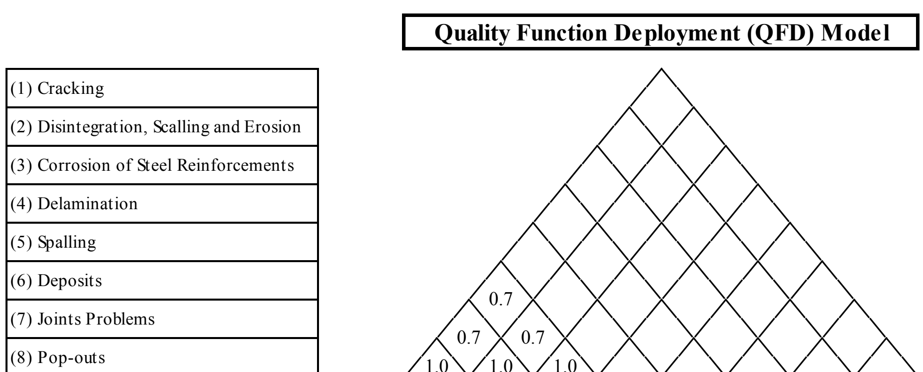
5.1.4. Bridge D

The steel crutch bridge in Laval, Québec was built in 1966 with asphalt overlay that rests on five I-shaped steel girders. It is a little skewed with an angle of 3°08’. The bridge deck has a total length of 56.5 meters and a width of 11.52 meters with 7.9 meters of roadway. As shown in Figure 5.4 (a), the GPR corrosion map for Bridge D shows a healthy deck with sound concrete. Based on Figure 5.4 (b) output, the C_I value is calculated to be 0.15. According to Table 3.3, the bridge is in excellent condition. Therefore, no action is required in order to improve the condition of the bridge deck.



Good	Fair	Poor	Critical
99.13%	0.78%	0.04%	0.04%

(a)



		2.4	2.4	2.4	0.0	0.0	0.0	0.0	0.0	
Description	Rating	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	
None or Light	1	87%	97%	99%						84.77%
Medium	3	13%	3%	1%						15.06%
Severe	6	0%	0%	0%						0.07%
Very Severe	9	0%	0%	0%						0.11%

(b)

Figure 5.4: (a) Corrosion map (b) QFD model application for Bridge D

Discussion

The detailed analysis of one of the cases is discussed here to illustrate the steps needed to apply the proposed models. Information related to the bridge deck structural defects is retrieved from the detailed inspection records completed by the MTQ inspectors. The inspections took place in 2012 and 2015. Measurements of defects are extracted from notes and comments on the reports added by bridge inspectors that reported the defects' severities and extents. Other information is retrieved from in-depth condition surveys based on VI and supplemented by NDE completed on the bridge. In this case, the surface defects were detected and measured by VI. Further investigation was completed by the research group on this bridge, using GPR technique in order to detect and evaluate subsurface defects. The collected scan data from the bridge deck, using the GPR tool (GSSI 1.0 GHz ground-coupled radar system), were used to assess the condition of the reinforcing steel. Typically, high frequency antenna can produce higher resolution B-scans but at the shallow depth. The lower frequency antennas can penetrate to more depth but at a lower resolution. In the conducted experiment for the case study, 1.0 GHz antenna was utilized in favor of having more depth penetration of the thick slab. Other subsurface defects like delamination were not measured precisely, where hammer sounding and chain dragging techniques are utilized to determine boundaries of delaminated areas within the concrete slab. Typically, inspectors include comments about the defects existence and approximate severity. The deck was reported free of other defects such as deposits, joint problems or pop-outs.

Based on results of the inspection that was performed in 2012, it can be noticed that the cracking existed with medium severity. Severe scaling, steel reinforcement corrosion at various severity levels, and spalling were detected. The bridge deck suffered from medium delamination as well. As per the 2015 inspection report, light cracking was observed, medium scaling was noticed and the same medium delamination persisted in the bridge deck. No steel reinforcement corrosion was noticed or reported. The obtained evaluations for the studied case are summarized in Figure 5.5.

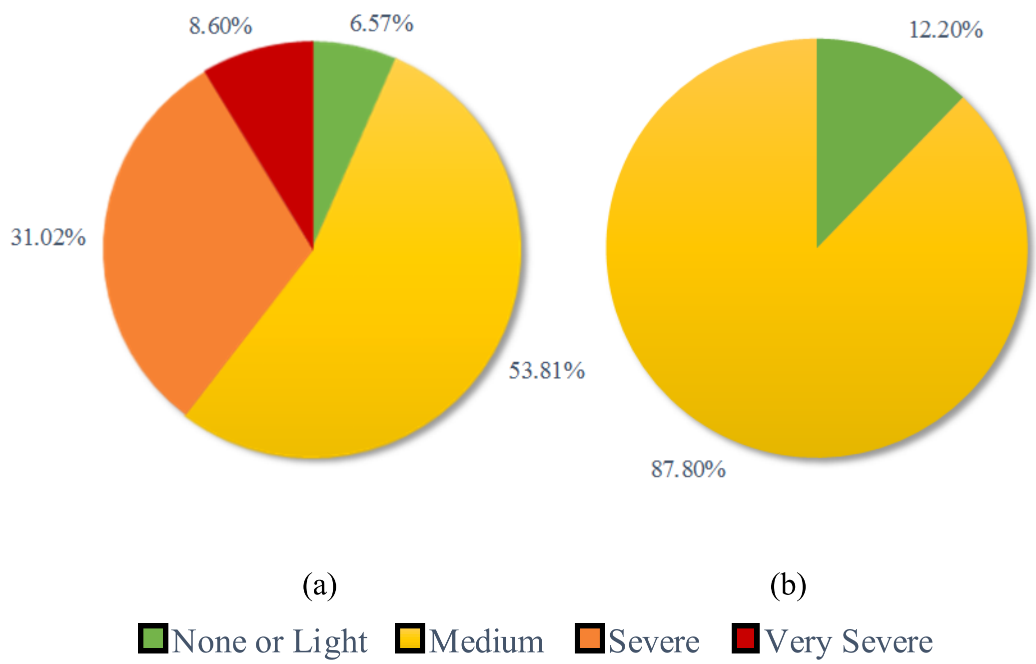


Figure 5.5: Assessment output for the case study based on (a) 2012 inspection (b) 2015 inspection results

The results obtained by implementing the QFD model (Figure 5.1 a) show that the medium condition state attained the highest percentage of 53.81%. The severe condition state got a relatively high percentage of 31.02%. It is concluded that the bridge deck condition based on the 2012 inspection was in poor condition represented by 30.84% index value. Based on the proposed bridge deck Integrated Condition Index (ICI), the bridge deck needs repair as per the information provided in Table 3.3. Similarly, the MTQ inspection team recommended that the bridge needs repair work (i.e., local repairs to the concrete structure). This recommendation represents the assessment of the overall bridge condition with all elements including bridge deck. Based on the 2015 inspection, the condition index improved to 10.98% indicating that bridge deck is in good condition where medium condition attained 87.80% of the overall bridge deck condition and the remaining percentages go to none or light severities (Figure 5.1 b). As per the inspection team and the MTQ condition index, the bridge does not require any MRR action.

Additionally, the obtained result from the 2012 inspection is compared with other approaches. First, the results are compared to the existing QFD model proposed in this research where data for steel reinforcement corrosion are measured using Half-Cell Potential (HCP) test. In this case, the results reflected a poor condition for the bridge deck as well, represented by 22.77% index value. A comparison of corrosion defect severities using GPR and HCP tests is provided in Table 5.1. The HCP is time-consuming and requires the electrical access to the reinforcement and the electrical connectivity between the rebars and therefore can be considered as a semi-nondestructive technique.

Table 5.1: Corrosion Defect Detected on the Studied Bridge Deck

Corrosion of Steel Reinforcements Defect	**Defect Severity**			
	None or Light	**Medium**	**Severe**	**Very Severe**
Ground Penetrating Radar (GPR) test	48.90%	19.79%	17.58%	13.73%
Half-Cell Potential (HCP) test	62.00%	33.60%	3.70%	0.14%

The second approach is proposed by Moufti et al. (2014) which implemented a defect-based condition assessment of concrete bridges using a fuzzy hierarchical evidential reasoning. In his approach, the same bridge deck was given a grade C (or “poor”) with crisp value evaluation that was equal to 2.43 out of 4 based on the same inspection report. According to this HER model, the condition assessment distributions were equal to A=21.00%, B=15.83%, C=53.92% and D=3.63%. The model developed by Moufti et al. (2014) did not consider the defects interdependencies (or correlations). Instead, the model used the AHP to determine the relative importance weights for surface defects mainly. Also, Moufti’s model does not recommend any MRR action based on the condition assessment. In general, the developed HER model seems to be a little pessimistic by giving a lower grade to the bridge deck condition. The last approach is based on NDE technology where Dinh et al. (2015) proposed a clustering-based threshold model for condition assessment of concrete bridge decks using GPR. In his model and for the same case study, the bridge deck condition was evaluated as sound concrete=33.62%, moderate corrosion=47.80%, and severe corrosion=18.57%. Further, Dinh et al. (2015) developed a bridge deck index where it uses a scale from 0 to 100 to represent the overall bridge condition. According to this health index, a value of 60.26 was given to the bridge with a D classification, meaning it is a very unhealthy deck and intervention is strongly recommended. This recommendation is consistent with the proposed ICI and the MTQ inspection team recommendations. The drawback of Dinh’s approach is that it is only appropriate for detecting steel reinforcement corrosion without considering any other defects and its intervention recommendation is based on that. Thus, it does not provide an accurate condition of the bridge deck.

The implementation of the developed clustering-based model is illustrated through twenty case studies. Similar to the idea of the BHI, the integrated condition index provides ratings and recommendations for intervention actions. However, this index has some distinguished features as follows. First, the index is based on assessing concrete bridge decks while integrating surface and subsurface defects. Therefore, an enhancement to the condition rating is achieved at the defect level. Second, the condition threshold values are selected using *k*-means technique, which eliminates the subjectivity associated with the traditional method for threshold selection.

Model Test

In order to validate the developed model, the recommendations for intervention actions were compared against the twenty bridge projects, which were assessed by bridge inspection teams. The efficiency was also measured through various indicators as computed in Equations 36 - 39:

$$AIP = \left\{\sum_{i=1}^{n} \left|1 - \left(\frac{R_i}{A_i}\right)\right|\right\} \times \frac{100}{n} \quad (36)$$

$$AVP = 100 - AIP \quad (37)$$

$$RMSE = \sqrt{\sum_{i=1}^{n} \frac{(A_i - R_i)^2}{n}} \quad (38)$$

$$MAE = \frac{\sum_{i=1}^{n} |A_i - R_i|}{n} \quad (39)$$

where:

AIP = Average Invalidity Percent;

AVP = Average Validity Percent;

$RMSE$ = Root Mean Squared Error;

MAE = Mean Absolute Error;

R_i = recommended action;

A_i = actual action; and

n = number of events.

The validation results are shown in Table 5.2. The Average Invalidity Percentage (AIP) value was less than 20%, which implies the validity of the developed model. The Root Mean Squared Error (RMSE) was 0.74, which was satisfactory and showed the reliability of the produced model. The achieved Mean Absolute Error (MAE) value was less than 0.50, which also proved the validity of the developed model. This may be considered acceptable, but an additional measure was used which is the Chi-squared test.

P-value from Chi-squared test is calculated in order to have the highest confidence level of validity. A Chi-square test compares two variables in a contingency table to see if they are related. A low value for Chi-square means there is a high correlation between two sets of data. The Chi-square value is calculated using Equation 40 as follows:

$$x^2 = \sum_{i=1}^{k} (O_i - E_i)^2/E_i \tag{40}$$

where:

O_i is the observed value (number of recommended action) of action i; and

E_i is the expected value (number of actual action).

A Chi-square test will give a p-value. The p-value will tell if the test results are significant or not. A small p-value (≤ 0.01) means strong evidence of dependence between the variables, which indicates in this case that the difference between recommended and actual actions decisions is small enough.

Table 5.2: Validation Results of the Clustering-Based Model

Measure of error method	**AIP**	**AVP**	**RMSE**	**MAE**	**Chi-Squared Test**	**P-Value**
Calculated index	18.3333	81.6667	0.7416	0.4500	18.6111	0.0001

5.2. IMPLEMENTATION OF DETERIORATION MODEL

Following the condition assessment, WDF model is applied on Bridge A and deterioration curves are mapped where the ideal deterioration curve (IDC) is constructed using Equation 9, knowing that the bridge was constructed back in 1965. To construct the updated deterioration curve (UDC) based on data retrieved from the inspection survey conducted in 2012, first, a final crisp integrated condition C_I is assessed to be 0.48, representing the bridge deck overall condition calculated. Then, by calculating the reliability value CR_{Ii} which is one minus the integrated component condition C_{Ii} at the inspection time (the good condition of the bridge deck), the UDC is plotted using Equation 15. In this case, t_i is equal to the t_i value in year 2012. Having the latest update represented by the inspection performed in 2015, the predicted deterioration curve (PDC) is constructed using Equation

16 and Equation 18. The first equation is implemented for the period before 2015 and the second equation for the period after 2015. In this case, it is assumed that the t_r is equal to the t_r value in year 2015. Similarly, a final crisp integrated condition C_I is calculated to be 0.31, representing the bridge deck overall condition based on the 2015 inspection survey and the assessment result. Then, the reliability value is calculated in order to plot the PDC. Figure 5.6 illustrates the deterioration curves using reliability values CR_I of 0.52 and 0.69, representing year 2012 and 2015, respectively.

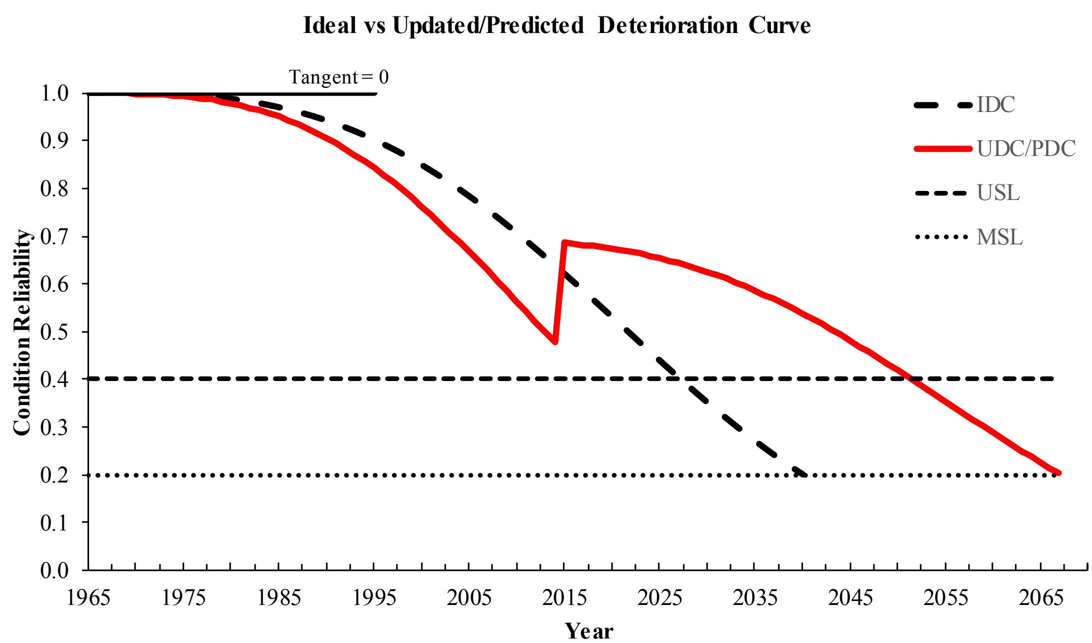


Figure 5.6: Case study ideal vs. updated/predicted deterioration curve

Discussion

After implementing the Weibull deterioration model, it is concluded that it needs almost 62 years for a concrete bridge deck to reach the minimum acceptable level. This is concluded by inspecting the ideal deterioration curve (IDC) shown in Figure 3.8, where the

USL is a threshold for functional and safety levels. This deterioration curve is plotted based on a 75 years service life (SL) as per the American and Canadian Bridge Design Codes (AASHTO 2014; Canadian Standards Association 2014). The IDC for the presented case showed that the concrete element (i.e., bridge deck) will last until the year 2027 before reaching its minimum acceptable level threshold (USL at condition = 0.4) and the minimum service life (SL at condition = 0.2) is forecasted to happen in the year 2040 (Figure 5.6). This prediction is based on normal circumstances and an assumption that no MRR actions are performed during the component life-cycle. Bridge decks, in particular, sustain several stressors including, exposure to aggressive environments, traffic loads excessive use, poor maintenance, and other environmental factors (e.g., freeze-thaw cycling). As a result, regular condition assessment is needed to be conducted on the bridge decks in order to maintain acceptable functional and safety levels and to enhance the accuracy of the forecast especially if MRR actions are applied.

Once the more recent inspection data are received, the Weibull deterioration curves can be updated to provide enhanced forecasts. After performing a condition inspection in 2012, the updated deterioration curve (UDC) is constructed and a reduction in the service life is noticed. The influencing factors discussed previously prone the concrete bridge deck to rapid deterioration and thus reduced the service life as shown in the updated deterioration curve UDC (Figure 5.6). The difference in reliability value between IDC and UDC at the year 2012 is 15%, where the reliability value is equal to 0.67 at the ideal deterioration curve and equal to 0.52 at the updated deterioration curve. The outcome from the 2012 inspection after implementing the WDF model delivers an indication to the ministry of transportation to interfere and to do a rehabilitation action no later than 2017 in order to maintain the minimum acceptable level that keeps the bridge functional and safe for users. It was suggested to do a repair action following the Dinh's GPR-based bridge deck index (2015), Alsharqawi's QFD-based condition assessment model (2016b), and the MTQ inspection team and general condition index recommendations. Although the discussed models and indices give a similar recommendation, the WDF model is better as it specifies a particular year to do the maintenance, repair or replacement action.

This repair action actually happened as an improvement can be observed in the bridge deck condition index based on the 2015 inspection data. This rehabilitation action increased the useful service life as shown in the predicted deterioration curve PDC (Figure 5.6). This increase is due to a decision taken by the ministry of transportation to do 'repair action' to the bridge deck which leads to an improvement of approximately 21% (ΔR=0.21). Eventually, this repair action rises the condition reliability of the bridge deck CR_{IR} to almost 0.70. The deterioration curve delivers an indication to the ministry of transportation to interfere and to do a rehabilitation action no later than 2051 in order to maintain the minimum acceptable level (Performance = 0.40) that keeps the bridge functional and safe for users as forecasted.

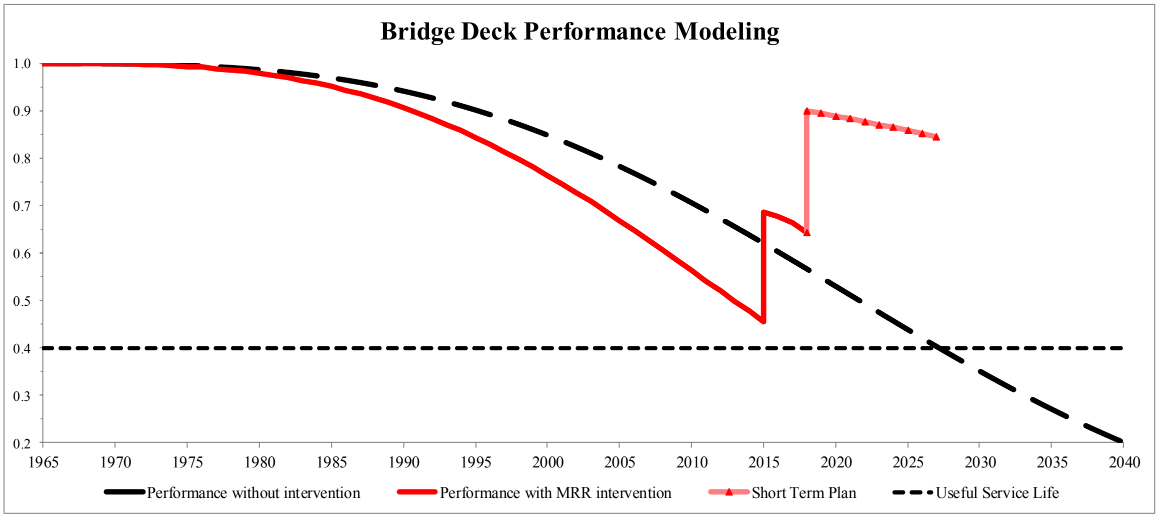
5.3. IMPLEMENTATION OF DECISION-MAKING OPTIMIZATION MODEL UNDER PBC

For this case study, two optimization scenarios were developed, based on the following parameters: performance LOS threshold = 0.65; ADM = \$28,733; discount rate i = 4%;

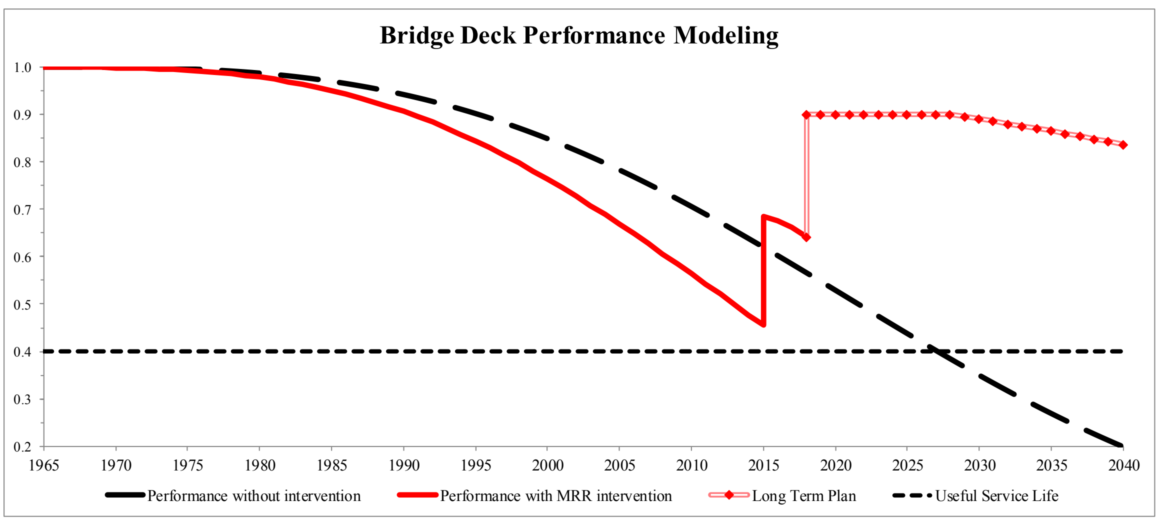
max number of iteration = 100000; population size = 20; α_1 = 0.95; and α_2 = 0.05. The first scenario was carried out over a 10 years predefined contract period and the second was carried out over the remaining life-cycle time of the contracted bridge asset. According to Haas (2008), the use of PBC for periods of 10 years or more has proven to be an effective means for sustained preservation of pavement networks. Moreover, the Australian experience in the New South Wales (NSW) 10-year contract has shown that realistic PBC's criteria/targets are achievable for network wide (Haas et al. 2009). For the short-term plan, a $TRAC_{limit}$ = \$1,000,000 is set allowing for a possibility to cover the cost of replacing the bridge deck. In the long-term plan, the $TRAC_{limit}$ is doubled to B = \$2,000,000. A set of tradeoff lists of rehabilitation actions applied over the two scenarios will be discussed in the next section.

Discussion

The implementation of the optimization model presented earlier provides a variation of rehabilitation strategies applied at various years throughout the time horizon of the two described scenarios. Figure 5.7 shows the bridge deck performance with (a) short-term and (b) long-term MRR interventions plans after applying the optimization model. The present worth of each scenario for the case study is also calculated. For scenario 1, the short-term plan, the total rehabilitation actions costs $TRAC$ = \$817,970.72. While in scenario 2, the long-term plan, the total rehabilitation actions costs $TRAC$ = \$1,229,452.40. Table 5.3 presents the program developed from implementing this model where the type of rehabilitation and corresponding performance for both scenarios are shown.



(a)



(b)

Figure 5.7: Case study performance curve with MRR interventions (a) short-term (b) long-term plans

Table 5.3: Case Study Optimal Rehabilitation Program

Year	Short-Term Plan			Long-Term Plan		
	Rehabilitation Strategy	$TRAC$	$Performace$	Rehabilitation Strategy	$TRAC$	$Performace$
0			0.66			0.66
1	Replace	$612,548.23	0.90	Replace	$612,548.23	0.90
2	No Action	$26,565.27	0.89	Maintenance	$52,367.51	0.90
3	No Action	$25,543.53	0.89	Maintenance	$50,353.37	0.90
4	No Action	$24,561.09	0.88	Maintenance	$48,416.71	0.90
5	No Action	$23,616.43	0.88	Maintenance	$46,554.53	0.90
6	No Action	$22,708.11	0.87	Maintenance	$44,763.97	0.90
7	No Action	$21,834.72	0.86	Maintenance	$43,042.28	0.90
8	No Action	$20,994.92	0.86	Maintenance	$41,386.80	0.90
9	No Action	$20,187.42	0.85	Maintenance	$39,795.00	0.90
10	No Action	$19,410.99	0.84	Maintenance	$38,264.43	0.90
11				Maintenance	$36,792.72	0.90
12				No Action	$17,946.55	0.90
13				No Action	$17,256.30	0.89
14				No Action	$16,592.59	0.89
15				No Action	$15,954.41	0.88
16				No Action	$15,340.78	0.88
17				No Action	$14,750.75	0.87
18				No Action	$14,183.42	0.87
19				No Action	$13,637.90	0.86
20				No Action	$13,113.37	0.85
21				No Action	$12,609.01	0.85
22				No Action	$12,124.04	0.84
23				No Action	$11,657.73	0.84

By comparing the first ten years in each plan, it is noticed that the long-term plan has a better performance where the average performance P_{avg} is equal to 0.90 versus 0.87 in the short-term plan. Even for the whole period in the long-term plan, the performance is better, where P_{avg} is equal to 0.88. In terms of total rehabilitation actions costs, an increase about 50% has been noticed in the long-term plan; yet this plan extends the study period by additional 13 years comparing to the short-term plan (more than double the period). In this case, it is suggested that the contracting agency tendering the contract can allocate a budget of no more than B = \$817,970.72 for a short-term plan and B = \$1,229,452.40 for a long-term plan, respectively. The budget assignment is based on the developed optimization model which minimizing the total cost while maximizing the specified performance LOS.

Based on the performance monitoring data, actual performance may vary compared to the predicted from the deterioration model. Using real time data after performing inspections, the rehabilitation program must be updated by running the re-optimization model. The significance of this stage abridged in two points. First, the contractor risk of failing to meet the specified performance LOS along the contract period is mitigated by validating the deterioration model using the performance monitoring data. Therefore, adjustment of the rehabilitation program may be necessary in case of underestimating the deterioration rate. Second, re-optimization of the rehabilitation program using real time data and continuous validation of deterioration model may result in cost saving and higher profit to the contractor in case of overestimating the deterioration rate and consequently scheduling the MRR action earlier than needed.

5.4. SENSITIVITY ANALYSIS

As discussed in Chapter 2, risks in performance-based contracting are usually much more than that of traditional contracts. In addition, the proposed methodology concluded by the MRR decision-making optimization model is based on various variables. Regardless of the modeling method, infrastructure deterioration cannot be predicted with certainty due to the stochastic nature of the deterioration process (Madanat et al. 1995). Moreover, the selection of MRR actions is subject to the specified performance level of service. Also, changing the relative weights may result in different Pareto optimal solutions. Therefore, a series of what-if scenarios is performed to study and evaluate the sensitivity to variability in the performance deterioration, specified performance level of service, and objective function importance weights. The sensitivity analysis variables and the ranges considered are presented in Table 5.4. The proposed analysis is evaluated for measuring the financial effect, represented by Total Rehabilitation Actions Costs ($TRAC$). For this purpose, the value of one variable is changed while keeping the other variables fixed. This has been repeated for all scenarios, where the optimization model is initiated, considering the change in each trail, to solve and obtain the new $TRAC$. The output from each trial is then recorded.

Table 5.4: Variables and Ranges for Sensitivity Analysis

Variable	Sensitivity Range
Performance deterioration modeling	-20% to +20%
Performance level of service threshold	-15% to +15%
α_1 importance weight	100% to 0%
α_2 importance weight	0% to 100%

i. Performance Deterioration Modeling Sensitivity Analysis

In the case study, performance was modeled using the developed WDF updated deterioration curve. The updated curve is a real representation of component deterioration over time and developed based on evaluating the component's integrated condition after each inspection. The input to the deterioration curve, integrated condition reliability CR_I, is used for the sensitivity analysis by changing its value by percentage range from -20% (overestimated) to +20% (underestimated). Figure 5.8 graphically presents an example of varying the base deterioration curve by changing the performance (i.e., integrated condition reliability) value.

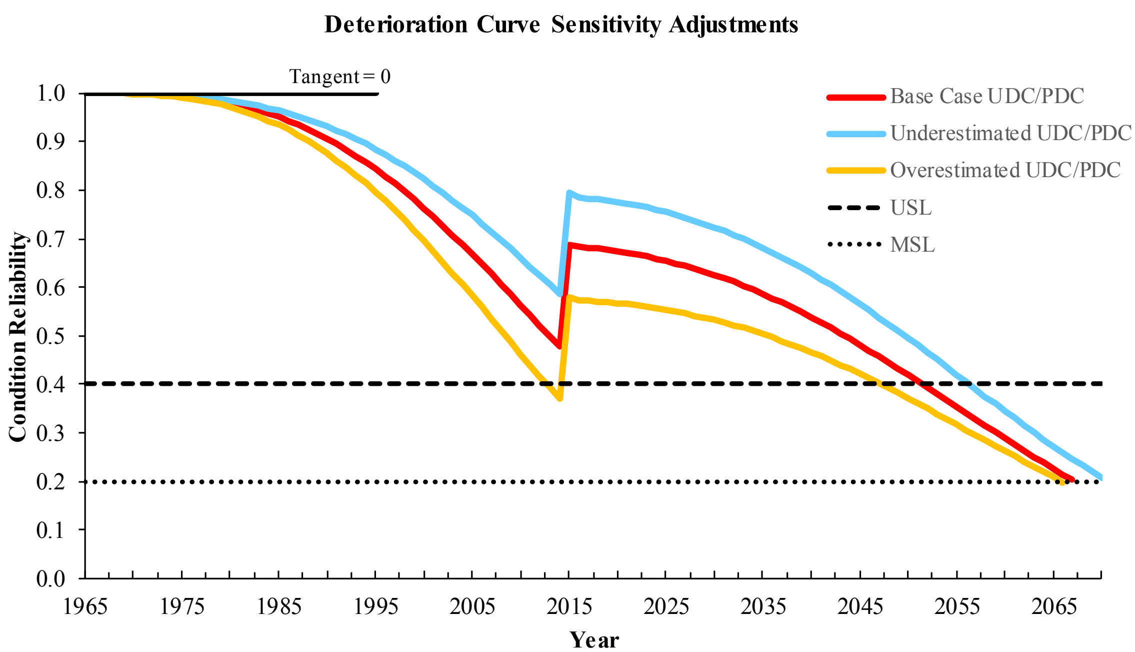


Figure 5.8: Case study deterioration variation scenarios

Based on the results, it is noted that the decrease in performance increases the total cost of the rehabilitation program (short-term plan) as shown in Figure 5.9. In other words, if the bridge deck deteriorates faster, the number of MRR actions to be applied throughout the contract period will increase causing the $TRAC$ to increase. In contrast, an increase in performance resulted in a remarkable decrease in the total cost. Figure 5.9 presents the total rehabilitation actions cost for the performance modeling sensitivity analysis, ranging between -20% and +20%. Accordingly, performance modeling has a high effect on the rehabilitation program total cost. Consequently, maintenance contactors should perform similar analysis while estimating program cost as means to quantify the risk accepted in this type of contract.

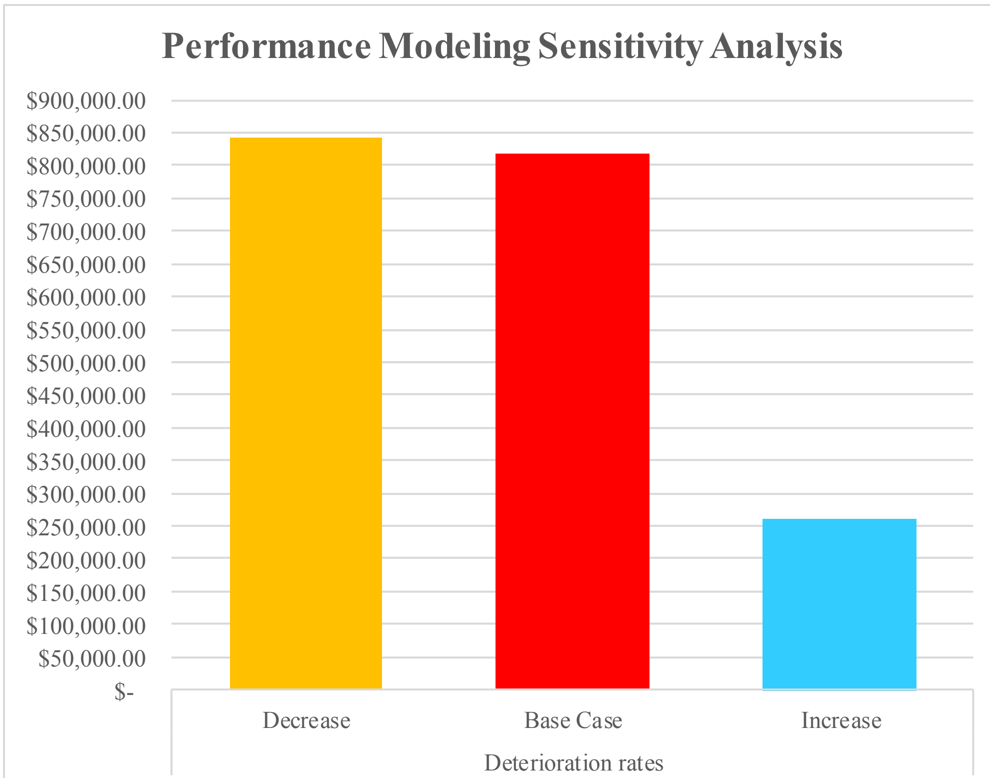


Figure 5.9: Result of performance modeling sensitivity analysis

ii. Performance LOS Threshold Sensitivity Analysis

To develop a rehabilitation program, a specified performance level of service of 0.65 is used for the presented case study. The specified performance LOS is associated with the key performance indicator, condition reliability, such that $P_{CR} \geq 65\%$. In practice, the performance LOS is specified by the contracting agency tendering the contract. Yet, it is valued to study the effect of this constraint on the total rehabilitation cost. For the sensitivity analysis, the performance specification is changed ranging from -15% (relaxed) to +15% (tightened). Figure 5.10 shows the final cost for the performance LOS sensitivity analysis for short term plan rehabilitation program. The LOS is relaxed by decreasing the threshold to 0.55 then tightened by increasing it to 0.75.

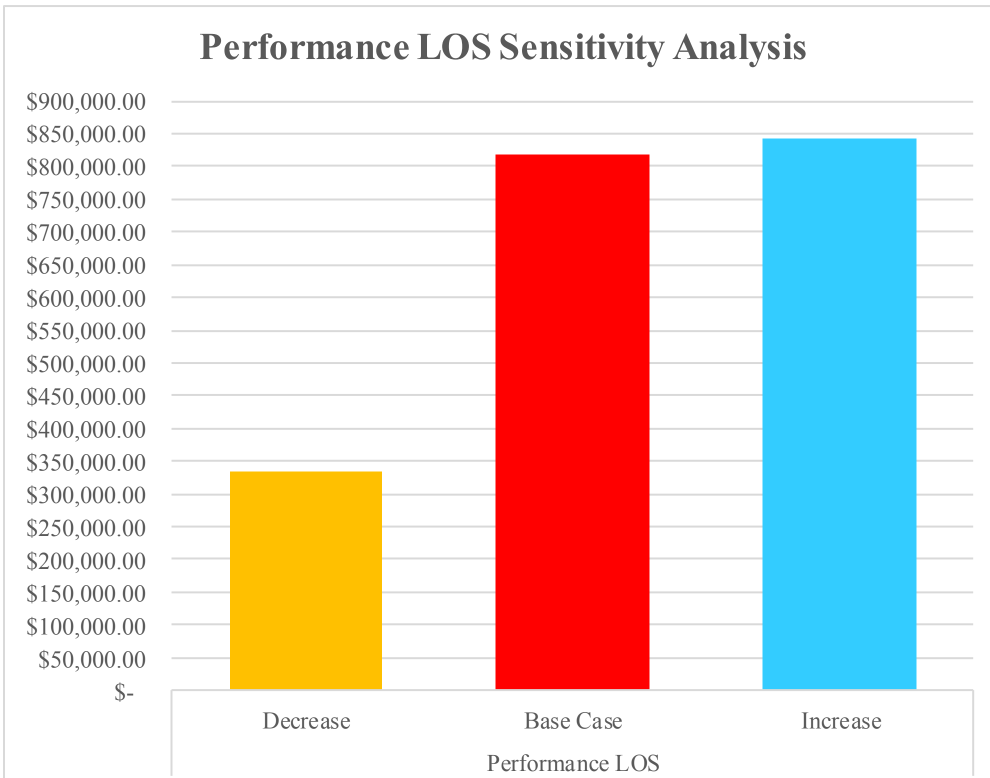


Figure 5.10: Result of performance LOS sensitivity analysis

Based on the results, it is apparent that relaxing the specified LOS allows for more deterioration; hence, reduces the total rehabilitation cost (Figure 5.10). On the other hand, a slight increase in the total cost was noticed as a result of restricting the specification. This is due to the replacement strategy that increased the total cost significantly in the base case.

It is concluded that the level of service specified in the performance-based contract has a high impact on the total cost. Thus, agencies implementing PBC should take that into account and carefully select the appropriate level of service. Figure 5.11 illustrates the result of sensitivity analysis for considered variables on total rehabilitation actions cost $TRAC$.

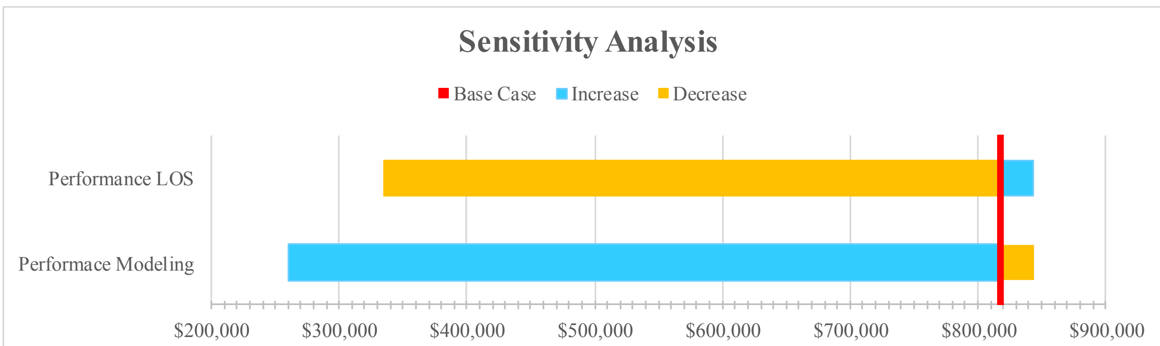


Figure 5.11: Sensitivity analysis of variables on cost

iii. Objective Function Importance Weights Sensitivity Analysis

Solving the bi-objective model defined previously using WCCM can provide different Pareto optimal solutions by changing α_1 (importance weight of $TRAC_{min}$ objective function) and α_2 (importance weight of P_{max} objective function). Table 5.5 displays optimal solutions obtained by solving the defined problem by decreasing α_1 and increasing α_2 with increments of 0.10.

Table 5.5: Case Study Pareto Optimal Solutions

Objective function importance weights		$TRAC$	$-P_{avg}$
α_1	α_2		
1	0	$459,406.80	-0.66
0.90	0.10	$892,438.41	-0.89
0.80	0.20	$958,639.92	-0.90
0.70	0.30	$998,639.38	-0.90
0.60	0.40	$998,639.38	-0.90
0.50	0.50	$998,639.38	-0.90
0.40	0.60	$998,639.38	-0.90
0.30	0.70	$998,639.38	-0.90
0.20	0.80	$998,639.38	-0.90
0.10	0.90	$998,639.38	-0.90
0	1	$998,639.38	-0.90

Figure 5.12 depicts the Pareto set for the case study by plotting $TRAC$ versus $-P_{avg}$ values. As the figure shows, three Pareto optimal solutions were obtained. An improvement in the performance P_{avg} occurs with an increase in the total cost $TRAC$. The results show that an improvement in performance, which is around 37% (from the least value to the greatest value), causes a significant increase in the total cost of more than double. Therefore, it is concluded that the importance weights should be selected carefully by the decision maker as per the importance needs whether to reduce the cost or improve the performance.

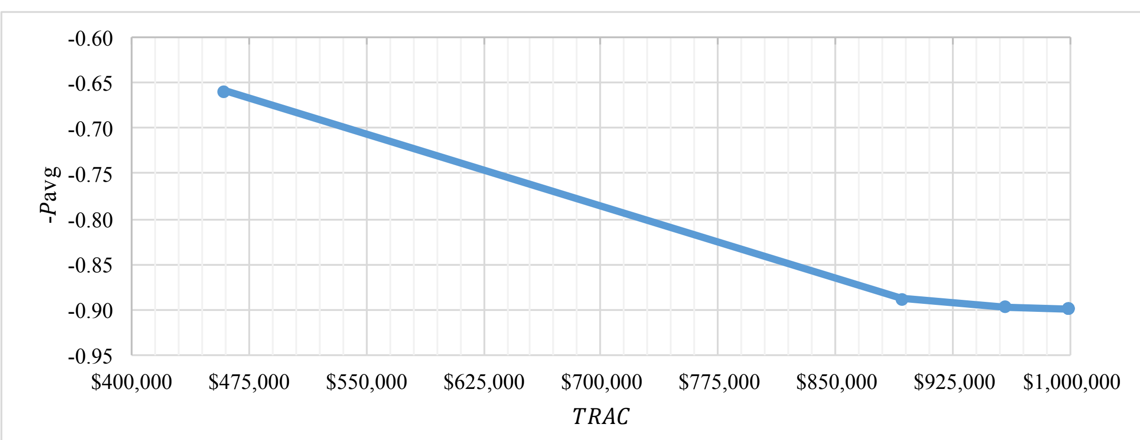


Figure 5.12: Result of Pareto optimal set

5.5. AUTOMATION OF THE DEVELOPED MODEL

After demonstrating the applicability of the proposed methodology, the developed integrated condition-based decision-making optimization model is semi-automated. The first part of the model that belongs to the condition assessment is automated in Microsoft® Excel worksheet. All of the required functions and equations to normalize the coefficients in the relationship matrix and to calculate weights of each condition rating and integrated bridge deck condition rating are incorporated in Microsoft® Excel workbook. Figure 5.13 demonstrates a sample excel sheet that is developed to automate the computation of integrated condition rate based on the QFD condition assessment model. To use the model, it is only required to insert each defect severity degree(s) and the integrated condition rating will be computed automatically since the deterioration model is embedded in the automation tool.

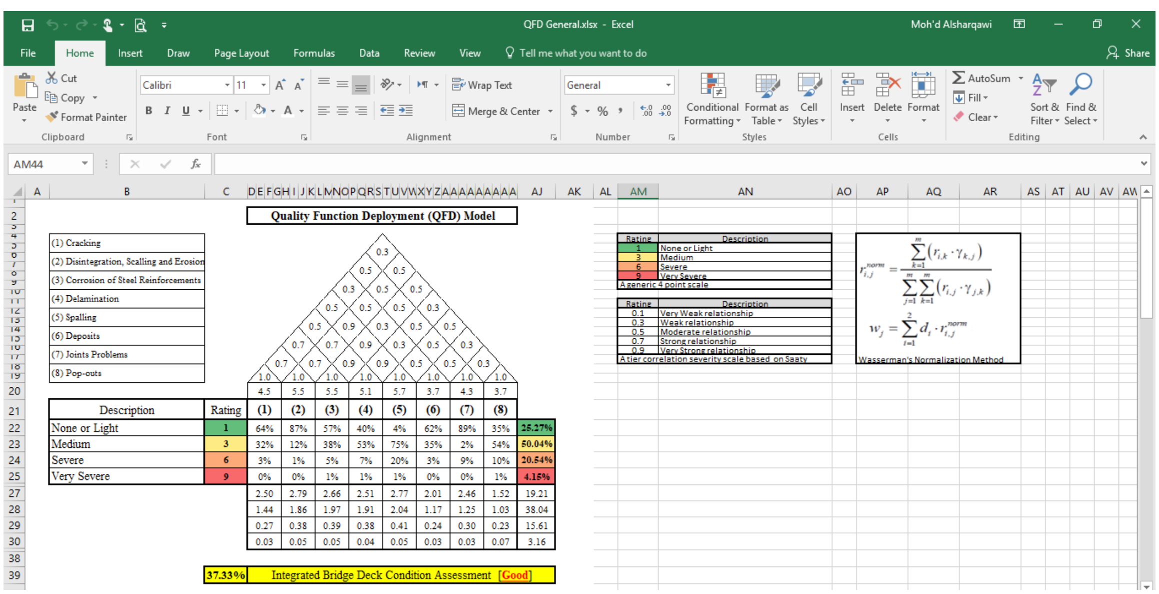
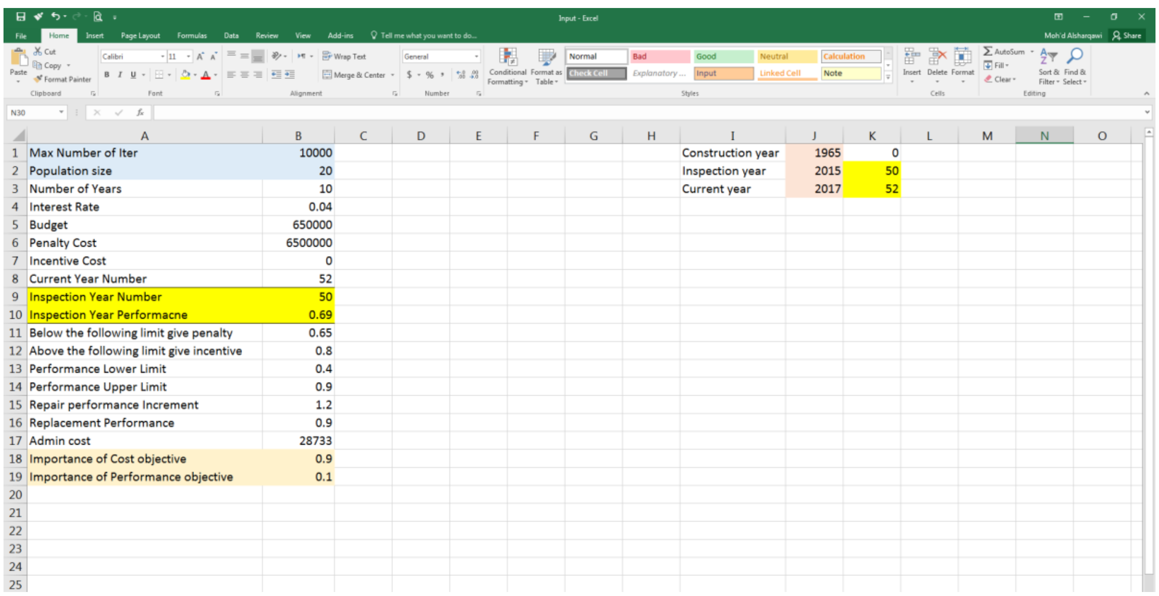


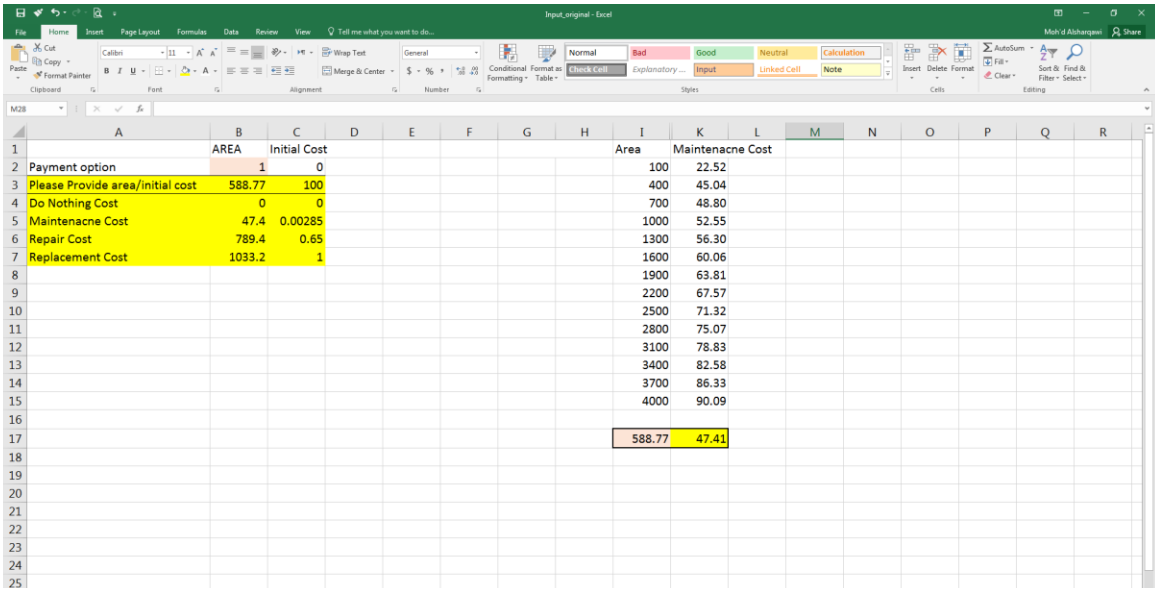
Figure 5.13: Sample Excel sheet for computing integrated condition rating

Once the integrated condition rating is calculated, it is inserted into the decision-making optimization model using an Excel sheet. Figure 5.14 (a) shows a sample "Input" sheet. In addition to the rating, the values of other inputs are needed to run the decision-making optimization model. The variables include rehabilitation plan number of years, budget, and performance threshold, target, lower and upper limits, and objective function importance weights. The values of the performance threshold and target might be fixed if the specified LOS does not change. Moreover, MRR costs are added following to the cost model formulation as shown in Figure 5.14 (b).



	A	B	I	J	K
1	Max Number of Iter	10000	Construction year	1965	0
2	Population size	20	Inspection year	2015	50
3	Number of Years	10	Current year	2017	52
4	Interest Rate	0.04			
5	Budget	650000			
6	Penalty Cost	6500000			
7	Incentive Cost	0			
8	Current Year Number	52			
9	Inspection Year Number	50			
10	Inspection Year Performacne	0.69			
11	Below the following limit give penalty	0.65			
12	Above the following limit give incentive	0.8			
13	Performance Lower Limit	0.4			
14	Performance Upper Limit	0.9			
15	Repair performance Increment	1.2			
16	Replacement Performance	0.9			
17	Admin cost	28733			
18	Importance of Cost objective	0.9			
19	Importance of Performance objective	0.1			

(a)



(b)

Figure 5.14: Sample input Excel sheets for decision-making optimization model

In order to solve the bi-objective optimization model, a genetic algorithm (GA) code is developed using MATLAB® software language and optimization toolbox. Figure 5.15 shows a screenshot of this software while running the optimization code. Once the specified number of iterations is reached, the optimization model creates a new Excel file named “Output”.

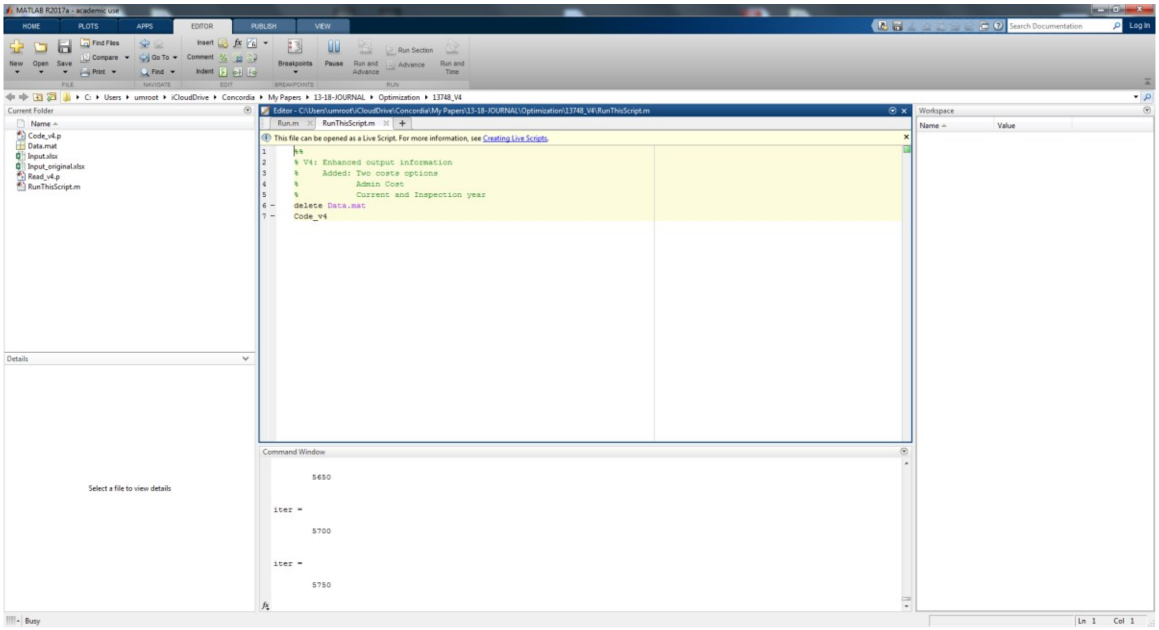
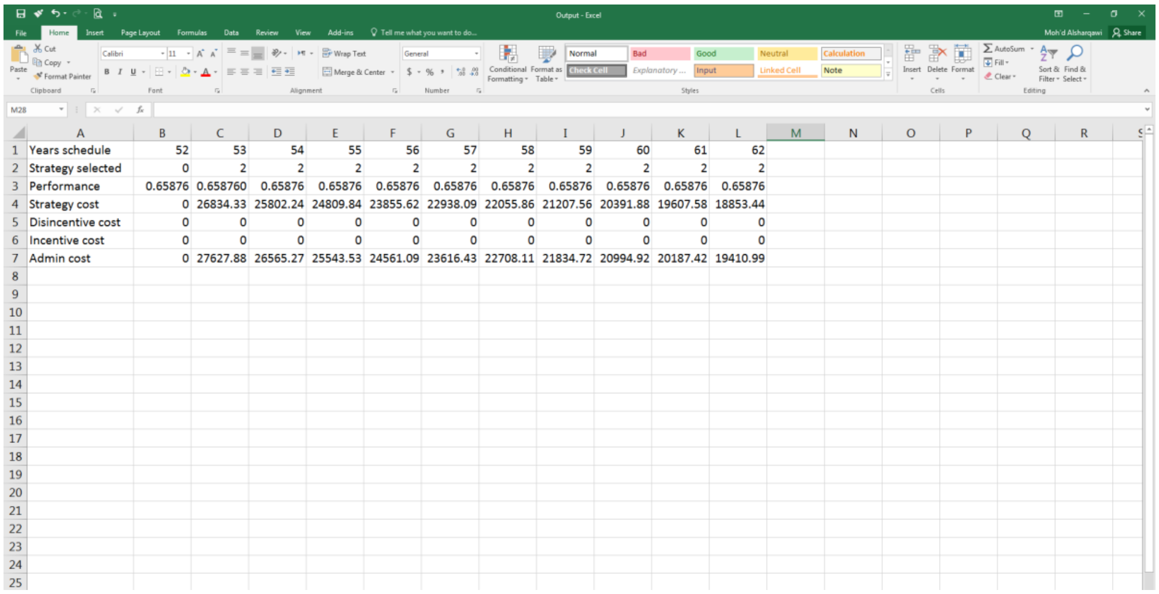


Figure 5.15: Running genetic algorithm (GA) code in MATLAB

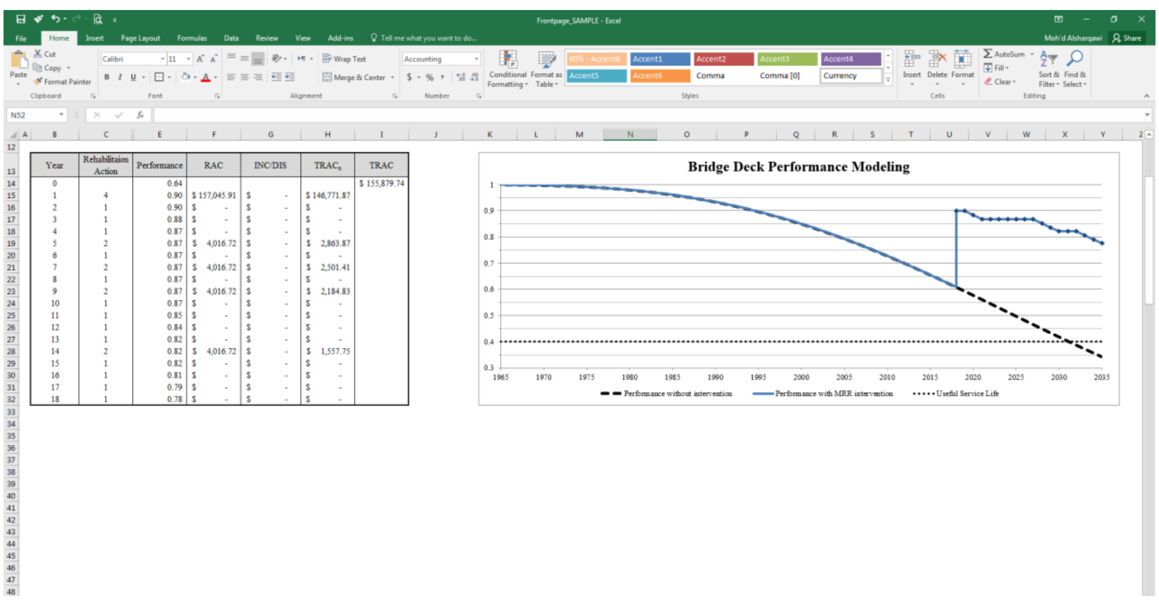
Figure 5.16 shows a sample "Output" sheet in Excel software in which the rehabilitation actions should be executed are determined, the performance prediction in each year is forecasted, and the calculations of the net present value (NPV) during the specified years of rehabilitation plan is estimated.

Figure 5.17 shows a sample calculation sheet of $TRAC$ that computes the total costs of associated rehabilitation scenarios while complying the with contract specifications. The developed model provides the incentives and disincentives amounts during total number of contractual years (contract duration), if any. The deterioration curve forecasts the performance for the same duration. The performance of the bridge deck before and after rehabilitation is computed, and performance values are determined based on functions developed in the WDF deterioration model. The performance improvement is defined based on the type of the applied rehabilitation action.



	B	C	D	E	F	G	H	I	J	K	L
Years schedule	52	53	54	55	56	57	58	59	60	61	62
Strategy selected	0	2	2	2	2	2	2	2	2	2	2
Performance	0.65876	0.658760	0.65876	0.65876	0.65876	0.65876	0.65876	0.65876	0.65876	0.65876	0.65876
Strategy cost	0	26834.33	25802.24	24809.84	23855.62	22938.09	22055.86	21207.56	20391.88	19607.58	18853.44
Disincentive cost	0	0	0	0	0	0	0	0	0	0	0
Incentive cost	0	0	0	0	0	0	0	0	0	0	0
Admin cost	0	27627.88	26565.27	25543.53	24561.09	23616.43	22708.11	21834.72	20994.92	20187.42	19410.99

Figure 5.16: Sample output Excel sheet for decision-making optimization model



Year	Rehabilitaion Action	Performance	RAC	INC/DIS	$TRAC_n$	TRAC
0		0.64				$ 155,879.74
1	4	0.90	$ 157,045.91	$ -	$ 146,771.87	
2	1	0.90	$ -	$ -	$ -	
3	1	0.88	$ -	$ -	$ -	
4	1	0.87	$ -	$ -	$ -	
5	2	0.87	$ 4,016.72	$ -	$ 2,863.87	
6	1	0.87	$ -	$ -	$ -	
7	2	0.87	$ 4,016.72	$ -	$ 2,501.41	
8	1	0.87	$ -	$ -	$ -	
9	2	0.87	$ 4,016.72	$ -	$ 2,184.83	
10	1	0.87	$ -	$ -	$ -	
11	1	0.85	$ -	$ -	$ -	
12	1	0.84	$ -	$ -	$ -	
13	1	0.82	$ -	$ -	$ -	
14	2	0.82	$ 4,016.72	$ -	$ 1,557.75	
15	1	0.82	$ -	$ -	$ -	
16	1	0.81	$ -	$ -	$ -	
17	1	0.79	$ -	$ -	$ -	
18	1	0.78	$ -	$ -	$ -	

Figure 5.17: Sample calculation sheet for TRAC of rehabilitation scenarios and deterioration curve developed based on the rehabilitation actions

CHAPTER 6: DESIGN OF PBC IMPLEMENTATION FRAMEWORK

A framework is designed in order to facilitate a successful implementation of the PBC and to assist the transportation agencies and maintenance contractors in arriving at fair contract value. Figure 6.1 depicts the stages of the proposed framework. The framework consists of six main stages. Stage 1 is identifying performance indicators for the bridge deck components and elements. Stage 2 is defining the required LOS for the identified performance indicators. Stage 3 is establishing a payments system for maintenance contractors. A proper definition of the payments and disincentives (penalties) are identified to avoid any conflicts or disputes. Stage 4 is bidding preparation and stage 5 is maintenance contractor selection reaching to awarding the contract. Finally, stage 6 is monitoring performance. A resource guide for performance-based contracting used in various countries is provided in Appendix D.

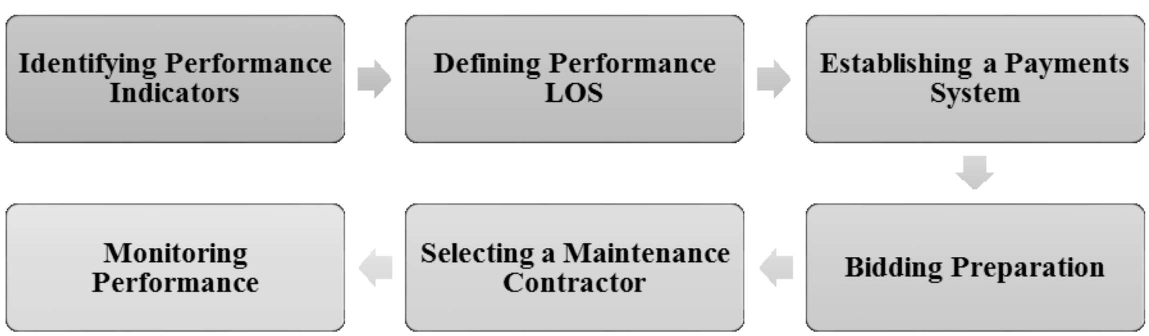


Figure 6.1: Proposed framework for implementing PBC

6.1. IDENTIFYING PERFORMANCE INDICATORS

As an integral part of the PBC framework design process, this research attempts to realize a generic hierarchy of bridge deck elements. This is achieved by breaking down the concrete bridge structure into its fundamental components and elements. Further, this step expands to identify key performance measures or indicators based on which a concrete bridge deck is going to be evaluated. In the National Bridge Inventory (NBI) system, bridge deck elements are defined as wearing surface, deck topside, deck bottom side, deck underside, SIP forms, curbs, medians, sidewalks, parapets, railing, expansion joints, drainage system, lighting, utilities. Similarly, the Bridge Inspection Manual (BIM) issued by Alberta infrastructure and transportation (Alberta Transportation 2008) recommends that the inspection of a bridge must be done through breaking it down to a set of defined elements. Different transportation agencies have slightly varying definitions regarding the breakdown of concrete bridges, with many of them having their own agency-specific definitions. It is essential to have a standard to breakdown and classify the different bridge components and elements. In this research, elements of the bridge deck component are defined and categorized into two groups as displayed in Figure 6.2. Eight elements have been defined after reviewing several bridge inspection manuals published by different agencies/departments of transportation. Those elements are grouped into a structural or non-structural category.

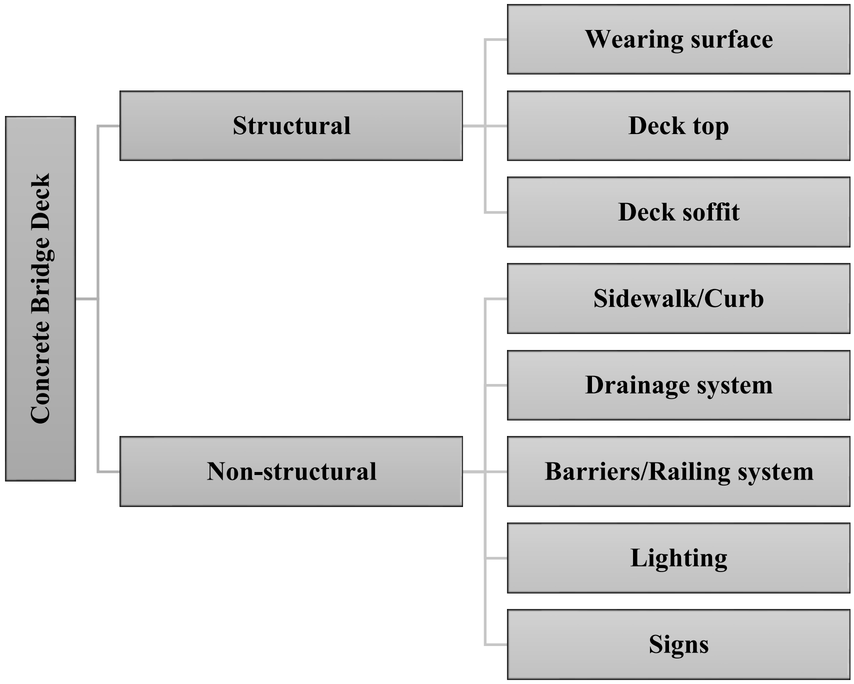


Figure 6.2: Hierarchy of concrete bridge deck elements

Many experts on performance measurement argue the benefit of using a few key performance indicators instead of many because of the associated simplicity and manageability of those few performance measures or indicators (Hyman 2009). Performance indicators can be defined as a set of outcome-based levels that an agency uses to evaluate the success of the contractor. Accordingly, bridge deck performance indicators could be categorized into two groups: structural performance and non-structural performance indicators. Structural performance indicators are these attributes that indicate the condition or performance of the bridge deck elements, such as: cracking, corrosion of steel reinforcements, spalling, and expansion joint problems. Non-structural performance indicators assess these non-structural elements. Table 6.1 identifies the entire performance indicators of the bridge deck components.

Table 6.1: Concrete Bridge Deck Performance Indicators

Bridge Deck Component	Element	Performance Indicator
Structural	Wearing surface	– Cracking – Loss of Bond – Rippling – Potholes – Local Protrusions (Delaminations) – Ravelling – Flushing – Rutting
	Deck top	– Cracking – Scaling – Corrosion of Steel Reinforcements – Delamination – Spalling – Expansion Joints Problems – Pop-outs
	Deck soffit	– Cracking – Scaling – Corrosion of Steel Reinforcements – Delamination – Spalling – Deposits
Non-structural	Sidewalk/Curb	– Cracking – Scaling – Corrosion of Steel Reinforcements – Delamination – Spalling – Pop-outs – Expansion Joints Problems
	Drainage system	– Pipe Breakage
	Barriers/Railing system	– Performance defects of barrier walls and railings
	Lighting	– Lighting Faults
	Signs	– Signs Faults

6.2. DEFINING PERFORMANCE LEVELS OF SERVICE (LOS)

After identifying the performance indicators, it is equally important to define required LOS. Desired performance targets can be defined too. Required LOS are the minimum acceptable levels to be achieved for each performance indicator such that the LOS never exceeds the defined threshold limit. The Ontario Structures Inspection Manual (OSIM) (Ontario Ministry of Transportation (MTO) 2008) defines a list of the most common concrete bridge defects, used as performance indicators in this study, along with respective measurement extents defining their levels of severity. Appendix C lists these common defects in bridge elements with severity definitions. Corrosion of steel reinforcements' levels of severity has been added to the list as per Shami (2015) and Alsharqawi et al. (2016a) definition. Verbal or numerical defect extents are defined in some cases to aid in classifying a defect in the right levels of severity (none, light, medium, severe, very severe). For example, a light concrete cracking severity is less than 0.1mm in depth, medium severity is from 0.1mm to 0.3mm in width, and so on. Similarly, severity levels are defined to other performance indicators. Structural concrete performance indicators including deck top and deck soffit are discussed in this research.

Since the translation of defects' levels of severity is imprecise by nature and is subject to a vast amount of error due to imperfect knowledge and human subjectivity, application of fuzzy based techniques is suggested to help in minimizing this uncertainty. Fuzzy Set Theory (FST) has proven its ability to effectively model uncertain variables using the concept of fuzzy membership (Zadeh 1965). 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 G_n is a fuzzy set, representing the evaluation grade of an element, then the general form of the rating membership function R_n can be formed as follows (Sasmal et al. 2012):

$$R_n = (G_n) | G ; 0 < \mu_n < 1 , n = 1,...,N \tag{41}$$

where:

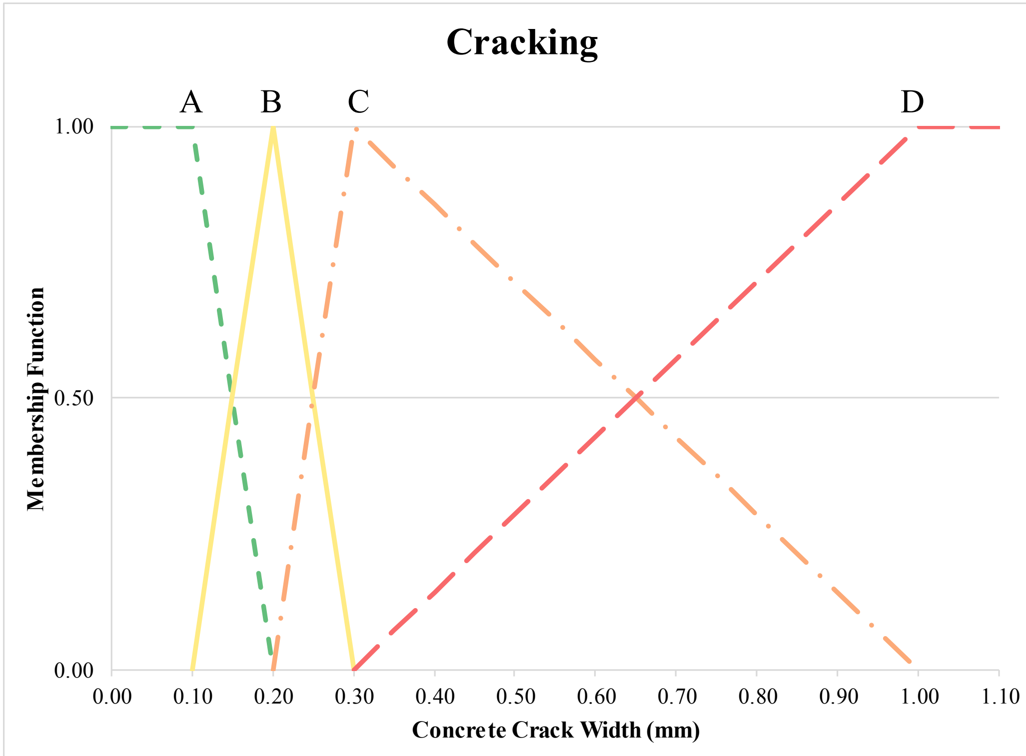
μ_n is a membership value representing the degree of membership to that grade/fuzzy set G_n.

The function as described in Equation 41 quantifies the ambiguity in the performance LOS rating, such that partial membership can describe the rating of two or more adjacent fuzzy grades. The methodology of implementing the FST can be described in the following steps:

1. Performance assessment grades are defined as fuzzy sets.
2. Severity levels are deduced from literature review to be used as thresholds.
3. The severities are distributed over four linguistic graded performance assessment scale.
4. The thresholds are fuzzified with respect to their common property.
5. Triangular shape is chosen to represent membership function since only the upper and lower boundaries of each severity level or LOS are known.
6. Fuzzy membership functions are created for all performance indicators' LOS.
7. The developed performance assessment scale is also fuzzified into the four linguistic grades.

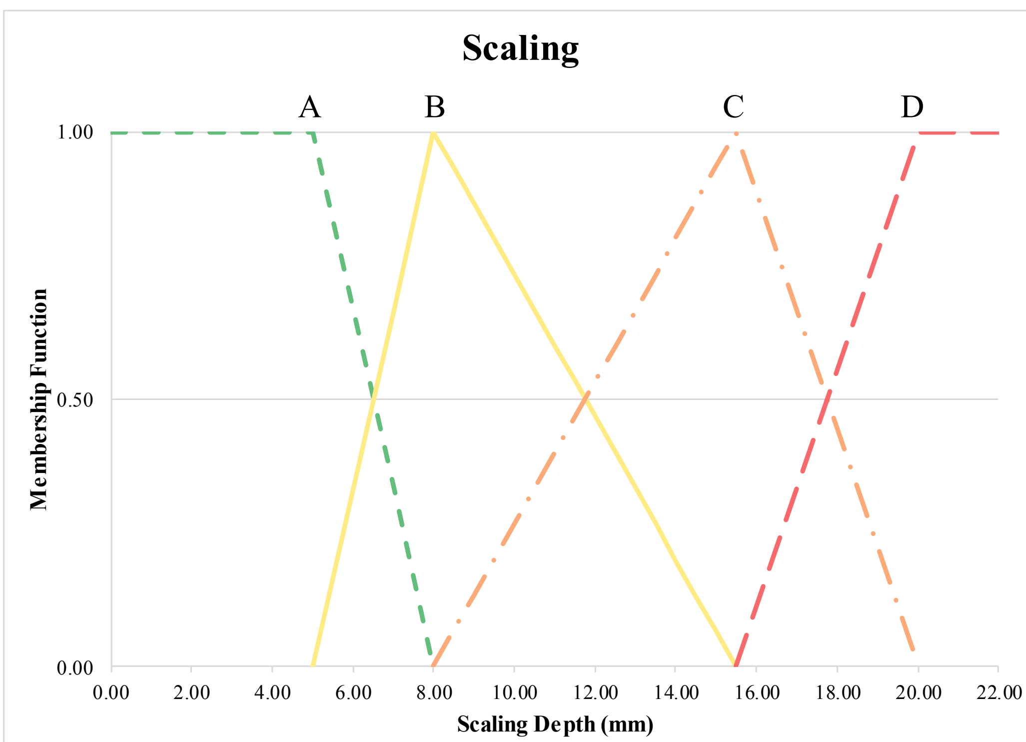
8. Defuzzification method is applied to convert all these fuzzy membership functions into a crisp value.

After determining the thresholds and performance grades, the membership functions are developed as shown in Figure 6.3. In general, the y-axis in the developed functions represents membership function with a value from [0-1] and the x-axis represents the levels of severity. The severity value is used as an input to enter the membership function in order to determine the percentages to which the severity supports the hypothesis which is the performance grades in this case (Excellent, Good, Poor, and Critical). The midpoint of each level of severity corresponding to each grade is taken as the point that corresponds to full membership of a certain performance grade. In Figure 6.3 (b) for example, points (5, 8, 16, and 20) correspond to a 100% of their related grades. If the user enters the graph with an input of 12mm scaling depth, the membership value will be 0.5 Good, 0.5 Poor. The transition of an element in a classical set is well defined. However, the transition of an element in a fuzzy set is through membership with a defined function that would portray the ambiguity in evaluating performance LOS. In a fuzzy set, the same LOS may be a member of another fuzzy set in the same universe, unlike the classical set in which LOS would have a complete membership i.e., [0 or 1].



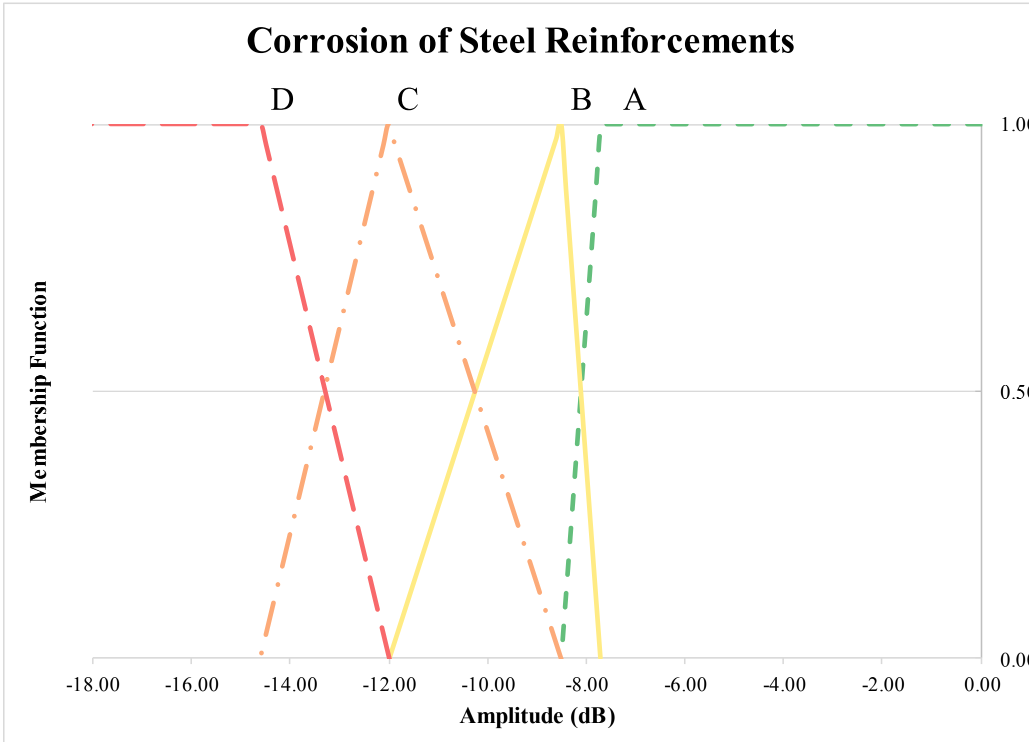
Cracking
A
B
C
D
1.00
0.50
0.00
Membership Function
0.00
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80
0.90
1.00
1.10
Concrete Crack Width (mm)

(a)

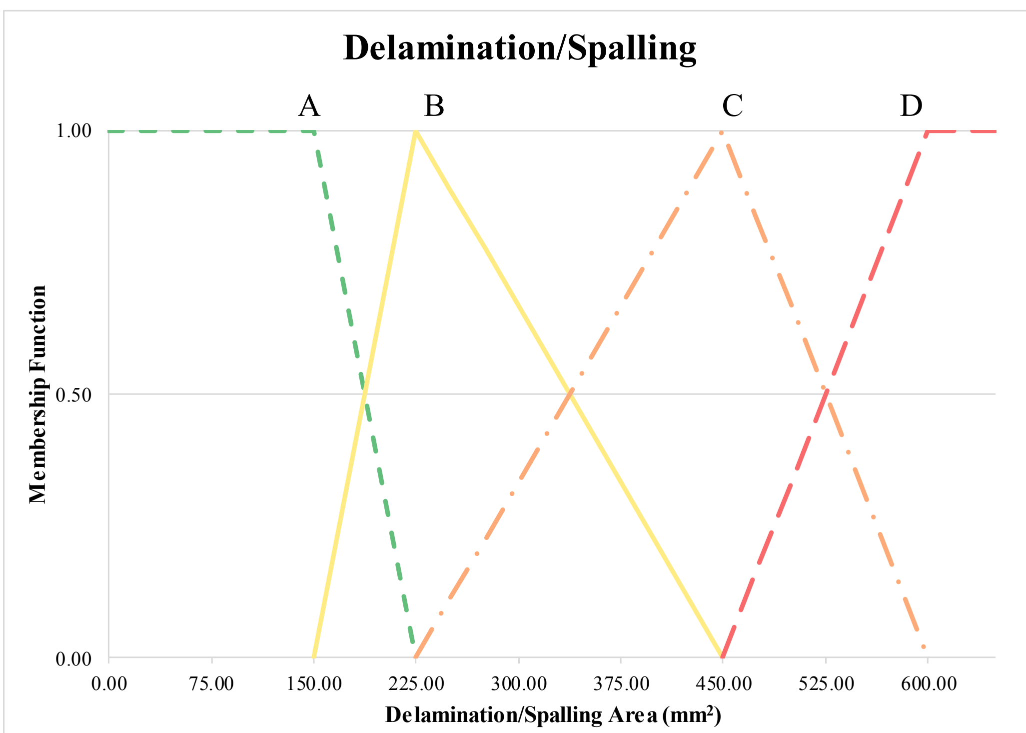


Scaling
A
B
C
D
1.00
0.50
0.00
Membership Function
0.00
2.00
4.00
6.00
8.00
10.00
12.00
14.00
16.00
18.00
20.00
22.00
Scaling Depth (mm)

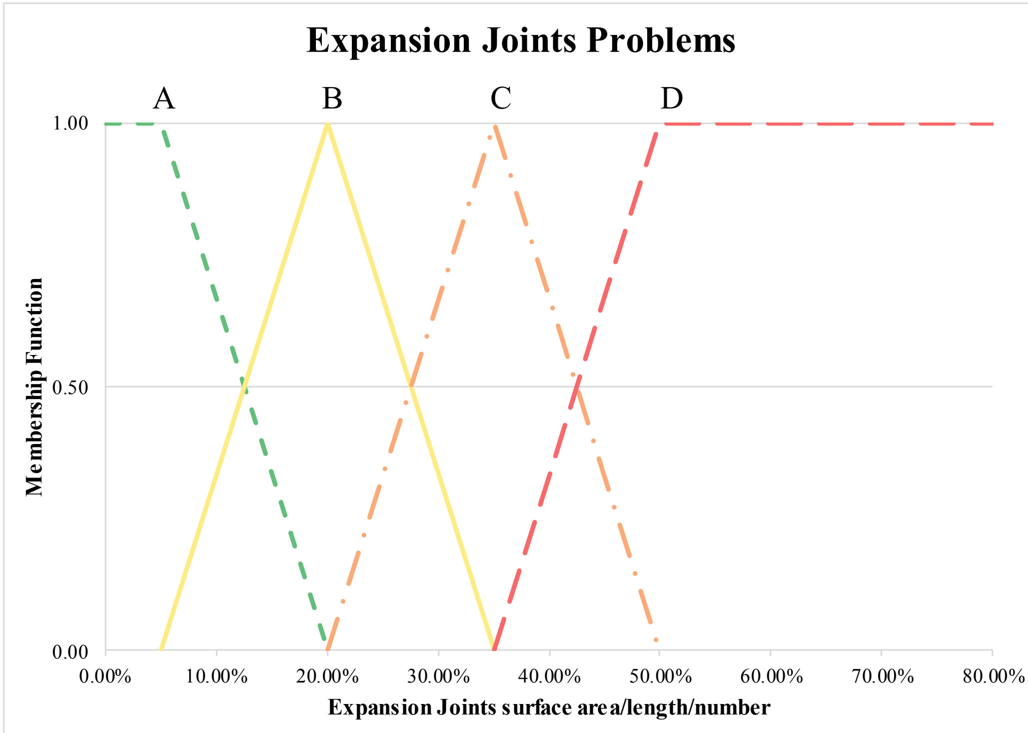
(b)


Corrosion of Steel Reinforcements
D
C
B
A
1.00
0.50
0.00
Membership Function
-18.00
-16.00
-14.00
-12.00
-10.00
-8.00
-6.00
-4.00
-2.00
0.00
Amplitude (dB)

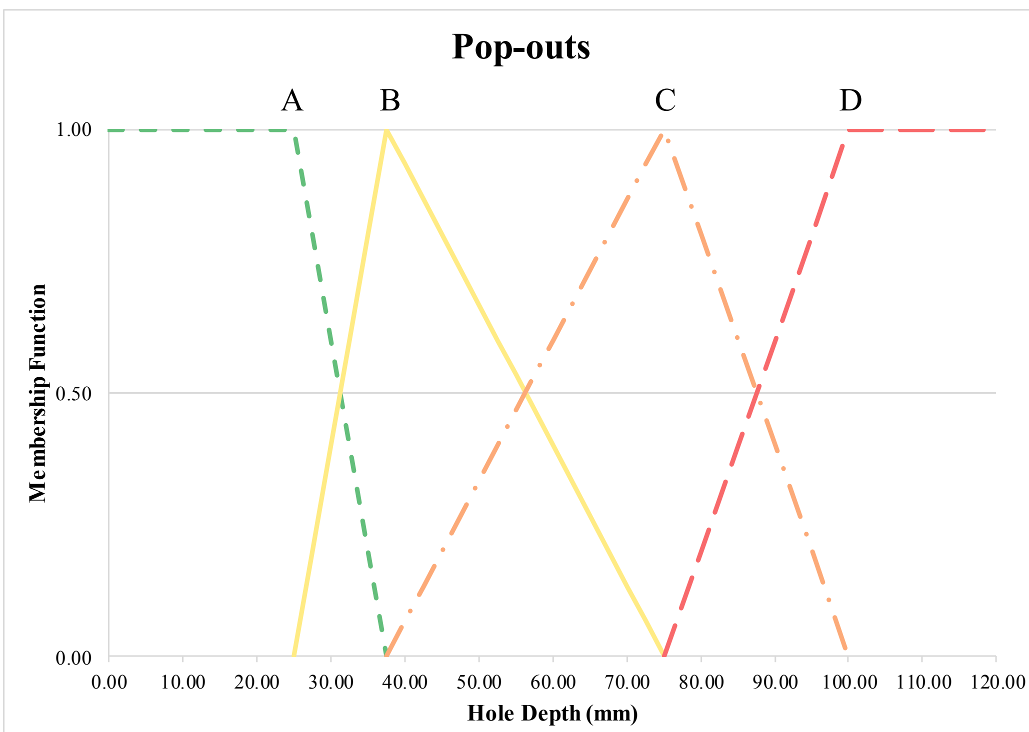
(c)


Delamination/Spalling
A
B
C
D
1.00
0.50
0.00
Membership Function
0.00
75.00
150.00
225.00
300.00
375.00
450.00
525.00
600.00
Delamination/Spalling Area (mm²)

(d)



(e)



(f)

Excellent {A} Good {B} Poor {C} Critical {D}

Figure 6.3: Structural concrete performance indicators' LOS membership functions

The next step is the development of a fuzzy-based performance grading scale for the designed framework in order to treat the subjective and judgmental nature of the assessment and represent the proposed evaluation grades. Table 6.2 presents the developed scale where the performance grade is reflected from a fuzzy output variable of the newly created scale. The 4-grades scale has a grading scheme with A and D being best and worst grades, respectively. The scale will evaluate performance to an order of descending fuzzy grades for an objective assessment of all performance indicators. In this application, it is assumed that every performance indicator is associated with a fuzzy set G which is defined by 4 fuzzy linguistic grades ranging over the performance LOS. Also, Figure 6.4 represents the fuzzy output membership function of the proposed performance grading scale. This function is used to obtain a crisp value through the defuzzification process.

$$\mathrm{G} = \{A, B, C, D\} \tag{42}$$

Table 6.2: Proposed Performance Grading Scale

Severity	**Performance Assessment**		**Performance Grade**	**Performance Grade**
	Linguistic	**Numeric**		
None or Light	Excellent	0-2	1	**A**
Medium	Good	2-4	3	**B**
Severe	Poor	4-8	6	**C**
Very Severe	Critical	8-10	9	**D**

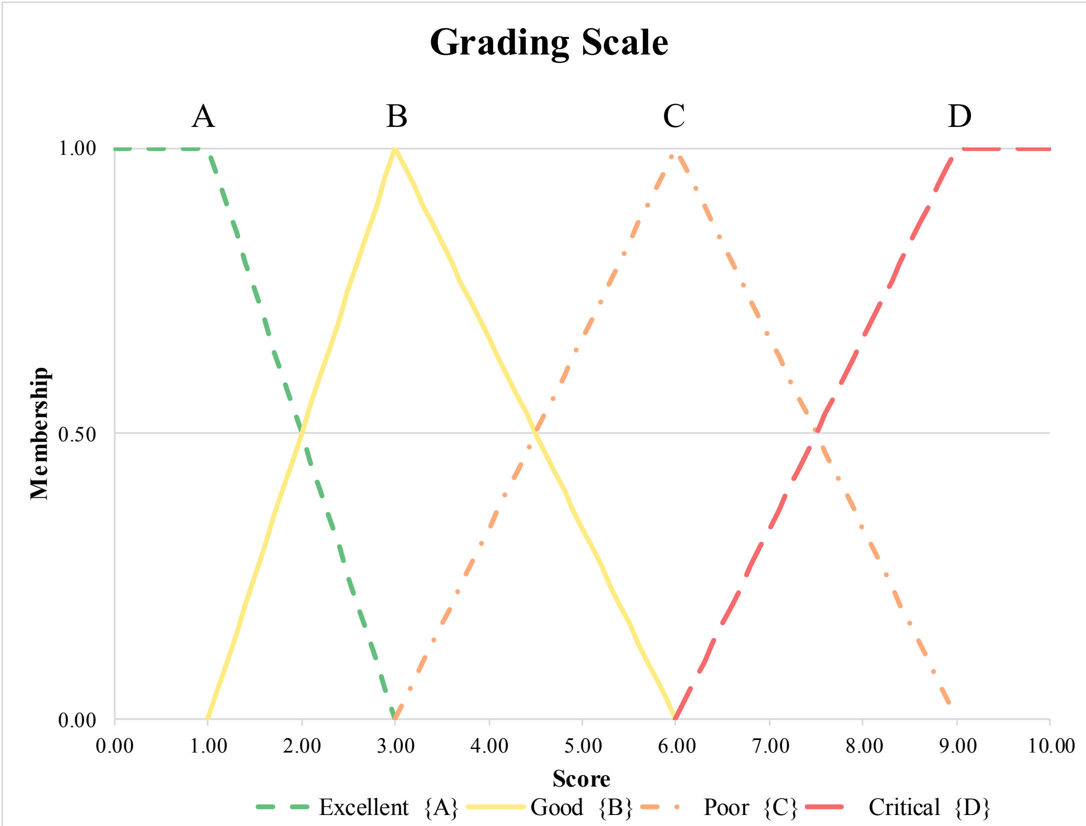


Figure 6.4: Fuzzy output variable of the proposed performance grading scale

The developed membership function, in Figure 6.4, will provide a framework for agencies/departments of transportation to decide on the required LOS to maintain their bridges. For example, if a contracted bridge asset is required to be maintained at a "Good" cracking performance indicator, the required LOS will be to maintain a {Grade B} performance by the maintenance contractor.

The final step is the defuzzification in order to encode the fuzzy output variable into a single crisp value. Several methods are used to defuzzify a fuzzy set. The weighted average method is considered to be computationally efficient (Ross 2010) and thus it is used in this research. In order to obtain the crisp value Equation 43 adopted from (Ross 2010) is used:

$$Crisp\,Value = {\sum \mu_i z . z}\Big/{\sum \mu_i z} \tag{43}$$

where:

z is the centroid of each symmetric membership function.

For example, if the cracking performance indicator was represented by the following severity degrees Cracking (G_1) = {(A,0.35), (B,0.55), (C,0.10), (D,0.00)}, then the defuzzification process will be calculated using the centroids of the scale fuzzy diagram (Figure 6.4) to give a crisp value of 2.29 {Grade B}. The resulting grade will be used by decision makers to decide on the incentive or disincentive (penalty) amount, if any.

6.3. ESTABLISHING A PBC-BASED PAYMENT SYSTEM

PBC pays the maintenance contractor based on the results achieved. It provides incentives, disincentives, or both to the contractor to achieve required or desired LOS. In other words, payments for the management and maintenance of assets in this type of contract are explicitly linked to the contractor successfully meeting or exceeding the performance indicators. Failing to comply with the performance indicators or to promptly rectify revealed deficiencies affects the contractor's payment adversely through penalties. Accordingly, a proper payment system is needed in PBC.

Maintenance type projects typically have been procured under yearly or multi-year agreements for maintenance activities via negotiated contracts. With the changing roles of the contractor assuming more maintenance work in the performance-based environment, a need has been created to estimate the value of these maintenance contracts in the viewpoint of the contractors (Panthi et al. 2008). Thus, in this research, the economic valuation to carry out maintenance work is estimated on annuity basis. These estimates are per bridge structure where structures are categorized according to their performance assessment. Estimated administrative costs (ADM), as shown in Table 6.3, have been verified by experts and consultants who experienced such type of contract (i.e., PBC). The objective here is to obtain a benchmark price for the contract against which bids will be compared later.

Table 6.3: Estimated performance-based contract cost per year

Bridge Performance Assessment	**Description**		**C$**
Good	Structural and functional diagnostic report	$	9,398
	Maintenance and replacement plan	$	20,885
	Technical assistance	$	22,100
	Total	$	52,383
Poor	Structural and functional diagnostic report	$	14,097
	Maintenance and replacement plan	$	31,327
	Technical assistance	$	33,150
	Total	$	78,575
Critical	Structural and functional diagnostic report	$	18,796
	Maintenance and replacement plan	$	41,769
	Technical assistance	$	44,200
	Total	$	104,766
	Report of proposed actions	$	28,733

In this type of contract, disincentives or penalties are introduced to the maintenance contractors as a decrease of payments failing to comply with performance indicators or to rectify revealed deficiencies promptly. It is assumed that the contractor is required to maintain a "Good" performance {Grade B} for all performance indicators. In addition to the discussed performance indicators, integrated condition reliability is added as a key performance indicator, where a threshold of 0.65 or 65% is representing a good performance state (e.g., $CR_I \geq 0.65$). As stated earlier, the condition reliability relates to the physical condition fitness of the structure, indicating its structural performance state. A Fine Unit (FU) will be applied in case of failing to meet the required LOS as shown in Table 6.4. FU = 1/500 of PBC contract amount. The unit was determined after reviewing the literature and consulting experts in PBC. For example, if contract value = C$150,000, then FU = C$300 and the penalty for 1 pop-out = C$1,200/day. Another example is having a cracking width more than 0.2mm over 0.5m for 2 weeks = 2×300×2×0.5 = C$600 penalties. Moreover, building in a reward mechanism in the contract is recommended to reward the contractor if he manages to retain or exceed the desired LOS for a sustained period. Such a mechanism provides an incentive to the contractor to innovate and deliver high standards.

Table 6.4: Suggested Penalties for Noncompliance with Performance Criteria

Performance Indicator	Disincentive in Fine Unit	Equivalence in C$
Cracking	2/week/m	600/week/m
Scaling	2/week/m	600/week/m
Corrosion of Steel Reinforcements	3/week/area	900/week/area
Delamination/Spalling	4/week/area	1200/week/area
Deposits	0.5/week/location	150/week/location
Expansion Joints Problems	4/week/length	1200/week/length
Pop-outs	4/day/hole	1200/day/hole
Integrated Condition Reliability	4/week	1200/week

Fine Unit = 1/500 contract amount

6.4. BIDDING PREPARATION

Since performance contracts are new for agencies administration and contractors' alike, close cooperation between both parties is vital for success. Both sides should be comfortable with the contractual arrangement and understand the risks involved. In all performance contracts that have been let until now, administrations and contractors have closely worked together in preparing the bidding documents (Zietlow 2005). Al-Kathairi et al. (Al-Kathairi and El Halim 2014) proposed some steps to be taken into consideration prior to the preparation of the bidding documents, including but not limited to:

1. Clearly, define the assets to be contracted out.
2. Make an inventory of the assets involved in contracting and assess their condition.
3. Select and define appropriate performance LOS.

4. Select and define the methods of measuring defined performance LOS.
5. Define the likely maintenance and possibly rehabilitation works.
6. Prepare preliminary cost estimates.

The contracted asset should be clearly defined along with the scope of work and the objectives of introducing PBC. The objectives, for example, could be to lower the costs, to implement higher level government directives, to manage the network with fewer staff, to improve user satisfaction (Al-Kathairi and El Halim 2014). Besides, the agency shall arrange the inventory and collection of data before the bidding stage. It needs to accurately determine the conditions of the assets to be contracted out. The data on the inventory and the conditions of the assets shall be given to the potential contractor as reference only. However, it is the responsibility of the contractor to make sure that the information is correct. During bridge field inspection, several defects may be detected on the surface and/or subsurface of bridge elements indicating different distress and deterioration mechanisms. In the proposed framework, condition assessment can be made by visual inspection except for corrosion of steel reinforcements to be evaluated by Ground Penetrating Radar (GPR). Performance indicators shall be identified for each asset to be contracted out. The proposed framework, identify performance indicators in various elements of bridge deck (Table 6.1). The selected indicators shall be based on the user needs, expectation of the agency to have assets back on contract completion at the same level as they were contracted out or better, and affordability, or the level of funding available (Stankevich et al. 2009). The methodology (i.e., methods and tools) which will be applied to measure the selected performance indicators shall be determined by the agency, too. The methodology should be clearly and accurately spelled out in the contract to prevent any misunderstanding from the contractor's side and avoid potential disputes (Stankevich et al. 2009). Maintenance and rehabilitation works can be defined according to the structural performance. For bridge decks under good performance, rehabilitation actions such as regular routine maintenance could be performed. Bridge decks with poor performance need repair actions such as strengthen the thickness of these bridge decks by doing local repairs to the concrete structure. For critical performance, the action would be full deck replacement. Agencies shall prepare estimates for services under a PBC (Stankevich et al. 2009). In this research, estimated PBC contact cost per year is evaluated (Table 6.4). However, it is the maintenance contractor responsibility to estimate and provide unite prices for executing maintenance and rehabilitation works.

6.5. MAINTENANCE CONTRACTOR SELECTION

Prior to selecting a maintenance contractor, assessing the bids is performed through evaluation criteria to select the best bidder, where the agency should check the maintenance contractors' capabilities. The evaluation criteria are the sole responsibility of the client. It is widely recognized that under PBC, the value-based method is applied to select the bidder rather than the low-cost method. For example, in Finland, the selection criteria are weighted 75% to price and 25% to the technical aspect (Al-Kathairi and El Halim 2014). In this research, award criteria are established as displayed in Table 6.5 after consulting an expert firm which employed PBC. The selection is based on the price (cost) of the contract and other technical criteria so that the lowest bidder is not always the winner.

Table 6.5: Criteria for Evaluating Maintenance Contractors

Award Criterion	Description	Weight
1. Price (cost) of the contract	The extent to which proposed costs are realistic and reflect the likely overall cost over the terms of the contract.	20 %
2. Technical report		
2.1. Work methodology	The soundness of the technical methodology reflecting knowledge and understanding of issues related to maintenance of the assets, coordination of the different maintenance activities of the contract and the supervision of the same.	30 %
2.2. Innovation	Improvements proposed by the bidder with respect to what is required in the bidding document.	10 %
3. Adequacy of the assigned personnel		
3.1. Staffing and Management	Suitability of the proposed technical team and management in terms of qualification, training and professional experience.	20 %
3.2. Equipment	Capacity and availability of equipment utilized by the bidder.	10 %
4. Past performances (if any)	The extent to which the contractor or members of the technical team successfully performed similar work.	10 %

6.6. MONITORING PERFORMANCE

In order to assess the maintenance contractors' performance, five key components for monitoring PBC for maintenance and their direct relationship with the overall performance are presented. The identification of these five key components is based on de la Garza et al. (2009) study to existing approaches commonly used in the public and private sector for measuring and monitoring performance including the following six approaches: i) ISO 9001:2000 Criteria for Performance Excellence, ii) Malcolm Baldrige National Quality Program, iii) Kaplan and Norton's Balance Scorecard Approach, iv) Mark Graham Brown's Scorecard Approach, v) Department of Energy Performance Measurement Program, and vi) NCHRP 14-12: Highway Maintenance Quality Assurance Program.

Figure 6.5: The five components for monitoring the PBC

As shown from the figure, the five main components for monitoring the PBC are the quality of service, timeless of response, safety procedures, cost-efficiency, and LOS effectiveness (de la Garza et al. 2009). A brief description of each component is as follows:

1. Quality of service evaluates the customers' perception concerning the condition of the asset and the performance of the maintenance contractor. Customers are the ultimate evaluators of the quality of service provided; therefore, it is extremely important to assess their satisfaction.
2. Timeliness of response assesses the response time of the maintenance contractor to service events or deficient elements as requested by users and need to be attended to promptly.

3. Safety procedures ensure that the maintenance contractor is properly implementing a safety program. It also checks that the maintenance crews performing the work are exposed to minimum risk of accidents.
4. Cost efficiency assesses the cost savings, if any, accrued by the government because of engaging a contractor to perform performance-based maintenance services.
5. Level of service effectiveness indicates how far the maintenance contractor is meeting the defined performance criteria throughout the contract period.

CHAPTER 7: CONCLUSIONS AND RECOMMENDATIONS

7.1. SUMMARY AND CONCLUSIONS

The large number of bridges built during the 19th century has aged and produced a complex decision-making problem. Moreover, the growing problem of bridge deterioration globally has imposed prominent challenges on transportation agencies to deal with. In order to maintain these deteriorated bridges, a novel method for bridge condition assessment using the QFD theory is introduced as a decision-making tool in the area of bridge management. The model functions at the element level and provides an enhanced condition assessment for the bridge deck. The developed model can integrate surface defects detected by visual inspection and subsurface defects using NDE techniques in one framework. Data for surface defects are retrieved from MTQ visual inspection reports. For subsurface defects, mainly corrosion of steel reinforcement, defects are measured using the GPR technology as a NDE technique. The advantages of the QFD model include its capability in correlating different types of defects based on their correlation factors assessed by Wasserman's normalization technique. These correlations can help in determining priorities and directions for improving bridge condition as well as providing an objective means of quality assurance for the end-users and the transportation agencies.

Beside accurate assessment of bridge condition, agencies/departments of transportation require a rating system to interpret their bridge condition. Subjective determination of threshold values between condition categories may lead to selecting improper maintenance, repair and replacement decisions. This research developed a robust method for resolving that issue using *k*-means clustering technique. Twenty case studies provided by the MTQ are analyzed using the developed condition assessment model. The results from the analyzed case studies show that the proposed model produces robust MRR recommendations consistent with decisions and recommendations made by bridge managers on these projects. Moreover, the AIP, AVP, RMSE, MAE, Chi-squared test and p-value were calculated to validate the produced model. Results showed effectiveness of the model with high validity.

To model the deterioration process, the reliability function based on the Weibull distribution is used to produce deterioration curves as a probabilistic forecasting approach. The model estimates the ideal performance of the concrete component (i.e., bridge deck) and is able to predict the future performances accurately based on the available inspection data. The QFD and the WDF models are integrated to provide consistent condition ratings and performance predictions. Data represented by the identified concrete defects are assessed using the QFD model which is used to calculate a component's integrated condition reliability, a performance indicator. The updated condition assessment is used to update the deterioration curves for an enhanced condition reliability assessment based on the latest bridge inspection reports available. Among all the advantages discussed previously, the WDF deterioration model can function with the relatively limited amount of historical inspection data and stochastically capture the uncertainty and randomness of the deterioration process. Having few inspection records, deterioration curves can be created to forecast performance and assess the remaining services life for each component. This provides basis for doing MRR projects or delaying actions till specific time that keeps the bridge functional and safe for users as forecasted. Future predictions can be updated with the deterioration model as more data become available.

There has been some movement over the past decade toward a performance-based contract model for maintaining and managing transportation infrastructure. However, there are limited computational tools to support optimal management and decision making under this innovative method of contracting. This research presents an overall methodology and introduces a decision support tool to optimize rehabilitation strategies under PBC setting. The availability of such tool will allow transportation agencies and maintenance contractors that are unfamiliar with PBC to make more informed decisions on their approach to better allocate the contractual risks. Implementing this tool can help contractors to establish optimum rehabilitation program with the lowest cost and appropriate time of MRR application while maximizing the specified performance LOS along the contract period. Also, agencies can benefit from this tool to benchmark prices against bids during the bid evaluation process. Furthermore, it can be used in estimating the budget needed for contracted bridge assets. For assistance with the optimization process, a genetic algorithm (GA) code for optimization is developed using MATLAB® software. As discussed earlier, contractual risks in performance-based contracting are usually much more than that of traditional contracts. Hence, a sensitivity analysis is conducted to study and evaluate the influence of variability in the performance deterioration, specified performance level of service, and objective function importance weights. It was evident that variables have a significant effect on total rehabilitation cost. Accordingly, maintenance contractors should consider quantifying the risk accepted in this type of contract while estimating program cost. Besides, agencies implementing PBC should rationally select the appropriate level of service.

Finally, a framework is designed to implement PBC in maintaining bridges. Agencies can benefit from the framework in evaluating the benefits of using the PBC model in comparison to the traditional model. The proposed framework suggests structural and non-structural performance indicators and defines their performance LOS through fuzzy membership functions. Further, a developed fuzzy-based performance scale will provide a basis for agencies/departments of transportation to decide on the required LOS to maintain their bridges. Agencies may also use the framework to establish a cost baseline and a general idea during contractors' bid evaluation process reaching to awarding the contract and further, performance monitoring of the contracted asset. Defects are primary attributes that indicate the performance of any structure. In this research, defects have been utilized as performance indicators of which a concrete bridge deck is going to be evaluated. After identifying the performance indicators, their required and desired LOS are defined, too. In order to minimize the subjectivity and uncertainty while defining those LOS, fuzzy membership functions are developed. Further, a fuzzy-based performance grading scale is developed to represent the proposed LOS evaluation grades. Also, the performance-based contract payments per year are estimated per the bridge performance. If the maintenance contractor fails to comply with performance criteria, penalties will be applied resulting in a decrease in payments. In such type of contract, the agencies administration and contractors should closely work together in preparing the bidding documents. Six steps are listed and detailed such that these steps will provide basis for bidding preparation between both parties. In PBC, the selection is not always for the lowest bidder where some technical aspects are involved in the evaluation criteria. This research proposed price (cost) of the contract, technicality, adequacy of the assigned personnel, and past performances (if any) as criteria for evaluating maintenance contractors. The last stage in the proposed framework is monitoring performance where five main components are presented for assessing the maintenance contractor performance and monitoring the contracted asset overall performance.

7.2. RESEARCH CONTRIBUTIONS

The performance-based maintenance contracting decision support methodology proposed in this research when realized would expect to provide a management tool for bridges asset and advance the knowledge in the area of infrastructure management. The research contributions would be beneficial to transportation agencies, local municipalities, engineering consultants, and maintenance contractors involved in the rehabilitation of bridge infrastructure by developing an effective decision support system. Key contributions of the presented research are outlined as follows:

- Reviewed main defects of concrete bridges and then identified and correlated concrete bridge deck common defects.
- Developed a condition assessment model based on QFD method which integrates data collected from visual inspection and GPR technology and thus, assess both surface and subsurface defects.

- Developed a rating system based on *k*-means clustering technique that categorizes rehabilitation actions into three types.
- Developed a Weibull-based forecasting model to assess bridge condition reliability, where ideal, updated and predicted condition-based probabilistic deterioration curves are modeled.
- Built a bridge deck management system that includes an improvement model that predicts the behavior after implementing any of the three different rehabilitation actions and estimates the associated cost for each rehabilitation type.
- Optimized the selection of the best iteration of MRR actions under PBC settings by coding a GA algorithm using MATLAB®.
- Developed a decision support tool for concrete bridge MRR strategies that provides a rehabilitation program for transportation agencies and maintenance contractors using the mathematical optimization tool.
- Designed a framework based on Performance-Based Contracts (PBC) for executing long term performance-based bridge maintenance works to facilitate a successful implementation of the PBC.

7.3. RESEARCH LIMITATIONS

Despite all findings during the course of this study, it has some limitations identified as follows:

- The developed clustering-based threshold model is based only on twenty case studies. A more precise bridge deck integrated condition index can be developed.

- The considered deterioration model is based on a study that was presented in the literature. More research is needed to determine the accelerated deterioration factor parameter (shape/slope parameter of the deterioration curve).
- The improvement model considered is based on the assumption that the condition reliability rating will improve according to the applied rehabilitation type. More research is needed in order to better estimate of the improvement rate due to a specific rehabilitation action.
- In the decision-making model, the rehabilitation types in its current format consider three types of strategies with their associated costs. A more defined rehabilitation model can be developed through consulting transportation agencies and maintenance contractors involved in bridge maintenance and rehabilitation.
- The designed PBC framework considers structural and nonstructural performance indicators. Broader non-technical indicators or measures can be added.

7.4. FUTURE WORK AND RECOMMENDATIONS

The developed methodology was able to achieve the proposed objectives, but potential improvements and extensions to this research are recommended as follows:

7.4.1. Enhancement Areas

- More data collection is recommended. Extensive time-series data collection and analysis should be performed in order to build a database that can serve in enhancing the integrated condition index precision and determining the parameters of the deterioration curve. Further, a second round of data collection from experts in the field would allow to review and refine results of estimating administration costs of the performance-based contract and disincentives (penalties) after collecting and analyzing data from the first round data collection.

- The recommended MRR actions were compared to the MRR actions were actually implemented by Ministère des Transports du Québec (MTQ). Yet, to successfully test and validate the model, it is worth to examine them against different transportation agencies.
- Although the developed automated tool provided required output for the case studies, it still requires the user to enter some inputs manually. In order to fully automate the tool, an enhanced user-friendly software can be developed.

7.4.2. Extension Areas

- While the current research focused only on reinforced concrete bridge decks, the developed methodology can be extended to cover other concrete bridge elements such as girders, piers, abutments, and so on. Since that may be the case, there should be an approach to aggregate and produce an overall condition assessment for the entire bridge structure. Moreover, the model can be expanded for other types of concrete structures and even other types of transportation networks.
- Since the Genetic Algorithm (GA) was proposed by the author to assist with the optimization process, comparison between other optimization algorithms is needed to be aware of each algorithm applicability.

- A wider application of the same PBC framework could be extended to accommodate other performance indicators, including compliance with inspection plans, traffic management and disruption, smoothness of ride, social and environmental impact. The framework could be improved through placing weights that constitute the importance of each indicator.
- The monitoring performance stage in the presented framework has potential for future research and development work, where a complete procedure can be established to assist the transportation agencies in monitoring maintenance contractors' performance.

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APPENDIX A: CONDITION RATINGS IN POPULAR BMSs

Country/ Province		BMS Name	Condition Rating	Remarks
USA		Pontis	1-5 where 1 represents good (low damage) and 5 represents bad (deteriorated) state	-Pontis uses a National Bridge Inventory (NBI) rating for bridge elements -Two distinct maintenance actions are identified in Pontis: preservation actions and functional improvements -Pontis can evaluate scenarios among a network of bridges and rank them by maximizing benefits with limited costs -The deterioration modeling required for evaluating future condition is performed using Markovian deterioration process -No uncertainties are considered within the condition assessment process which is input to the predictive modeling and analysis
Canada	Quebec	QBMS	1-4	-Rehabilitation scenarios are based on MTQ manual -Has treatment model for deterioration and cost that specifies a list of treatments -Inspection is stored in time-series -Element alternatives and Project alternatives are generated by performing life-cycle cost analysis
	Ontario	OBMS	1-4	-Knowledge-based model for choosing rehabilitation scenarios based on MTO manual -Comprehensive cost database with tender items covering 12 Ontario Districts
	Manitoba	Pontis	1-5	Pontis is used for Manitoba Bridges
	Alberta	BIMS	1-9, N, X where 1 represents immediate action (poor) and 9 represents very good state, N represents not accessible, X represents not applicable	-Various Modules for condition state and functionality -Results from above Modules used in Substructure and Replacement Modules -Cost Estimate and Timing of each activity is generated

Country/ Province		BMS Name	Condition Rating	Remarks
Canada	British Columbia	BIMS	1-5	-Map-based interface for recording inspection data -For each component in each condition state a percentage of inspection records is provided -No module for budget forecasting and what-if scenarios
Australia	New South Wales and Victoria	Pontis	1-5	Pontis is adopted with minor modifications to suit their inventory
	Queensland	WHICH-BRIDGE	1-5	-Advanced BMS that can facilitate risk management based on probability of failure along with cost estimates for maintenance -Agencies depend on inspection guidelines provided by their own bridge experts for maintenance and rehabilitation
Germany		SIB-Bauwerke	1-4	-Defects are weighted with respect to structural safety (stability) 'S' traffic safety 'V', and durability 'D'. This scheme is applied both to either an element or the whole bridge -A defect catalog in the system tries to cover all possible defects. Overall the conditions of bridges are stored. No cost analysis is carried out -3 year interval for general inspection and 6 year interval for major inspections
Sweden		BaTMan	0-4	-Condition Class (CC) is used rather than state. CC is defined as the extent to which functional properties are satisfied. OCC is defined as overall condition class for the bridge -No deterioration models; physical and functional conditions are based on inspectors judgment. For example, comparison between previous inspection state and current state is made to identify deterioration process
Finland		FBMS	0-4	-Results from network level are used to evaluate individual bridge condition -Damages are grouped into surface, structural and water leakage. Interdependency between the groups is considered -Cost minimization is done based on repair costs and bridge user costs

Country/ Province	BMS Name	Condition Rating	Remarks
Japan	JBMS	0-100 where 100 represents excellent condition	-JBMS stores large number of technical specifications, inspection and other data related to bridge maintenance -JBMS evaluates the condition of each element of the bridge at project level only -The evaluation is done using a program called Bridge Expert Rating System (BREX) that is dealing with uncertainty -The program has prediction deterioration curves that are soundness vs. deterioration curves established using experimental work -Remaining life of the bridge is estimated along with a rehabilitation strategy -The system cannot keep track of a network of bridges and allocate funds among the bridge inventory
South African	STRUMAN	0-4	-It has four modules - inventory, inspection, condition and budget -In the inspection module, the worst defect per element is rated for Degree (D), Extent (E), Relevancy (R), and Urgency (U) of repair -In the condition module, three indices are evaluated - a condition index (for ranking the bridges requiring repairs), priority index (priority among overall bridge elements) and functional index (importance depending on location of bridge)

APPENDIX B: GROUND PENETRATING RADAR

Radar is an object-detection technology that was developed before and during World War II for military purpose. The term “radar” stands for **ra**dio **d**etection **a**nd **r**anging and as this full name implies, the technology uses electromagnetic (radio) waves as a means to detect the presence and location of concerned objects. The earliest civil engineering application of radar, according to ACI 288.2R-98, was for probing into the soil to detect buried pipelines and tanks. Many studies have been performed in this area such as for detecting cavities below airfield pavements, determining concrete thickness, locating voids or reinforcing bars, and identifying deterioration. Later, an extensive number of studies have been performed investigating Ground Penetrating Radar (GPR) technology for reinforced concrete bridge inspection, especially for bridge decks. Based on the proposals of some of these studies and considering the results of others, a standard test method – ASTM D6087 – has been adopted and issued by American Society of Testings and Materials (ASTM), using GPR for evaluating asphalt-covered concrete bridge decks. Although the standard was originally intended to be used for asphalt-covered concrete bridge, it was also recommended for bridge decks overlaid with Portland cement concrete or bridge decks without overlays.

GPR, also referred to as Ground Probing Radar, is an emerging and powerful non-destructive geophysical tool that uses that uses pulsed electromagnetic radiation to provide a high resolution image of subsurface features in the form of a cross-section view. The short pulses are radiated into the ground from a transmitting antenna placed either on the ground or in close proximity. With respect to this research, the pulses of electromagnetic radiation that are emitted are partially reflected by the top of the bridge deck, the base of the deck, and from features such as embedded reinforcing steel bar (rebar) and delaminations. Analysis of the reflected signal (magnitude and arrival time) enables the operator to estimate the depth to each reflector and to assess the overall condition of the bridge deck. The principles of the GPR tool operation are shown in Figure A.1. Based on the operation methods, there are two types of GPR systems: air-coupled and ground-coupled. Most GPR antennas can be easily manhandled and the usual method of operation is to drag the antennas slowly across the ground surface in a straight-line traverse, transmitting and receiving continuously, so that a GPR profile picture builds up (referred to as linescan or B-scan).

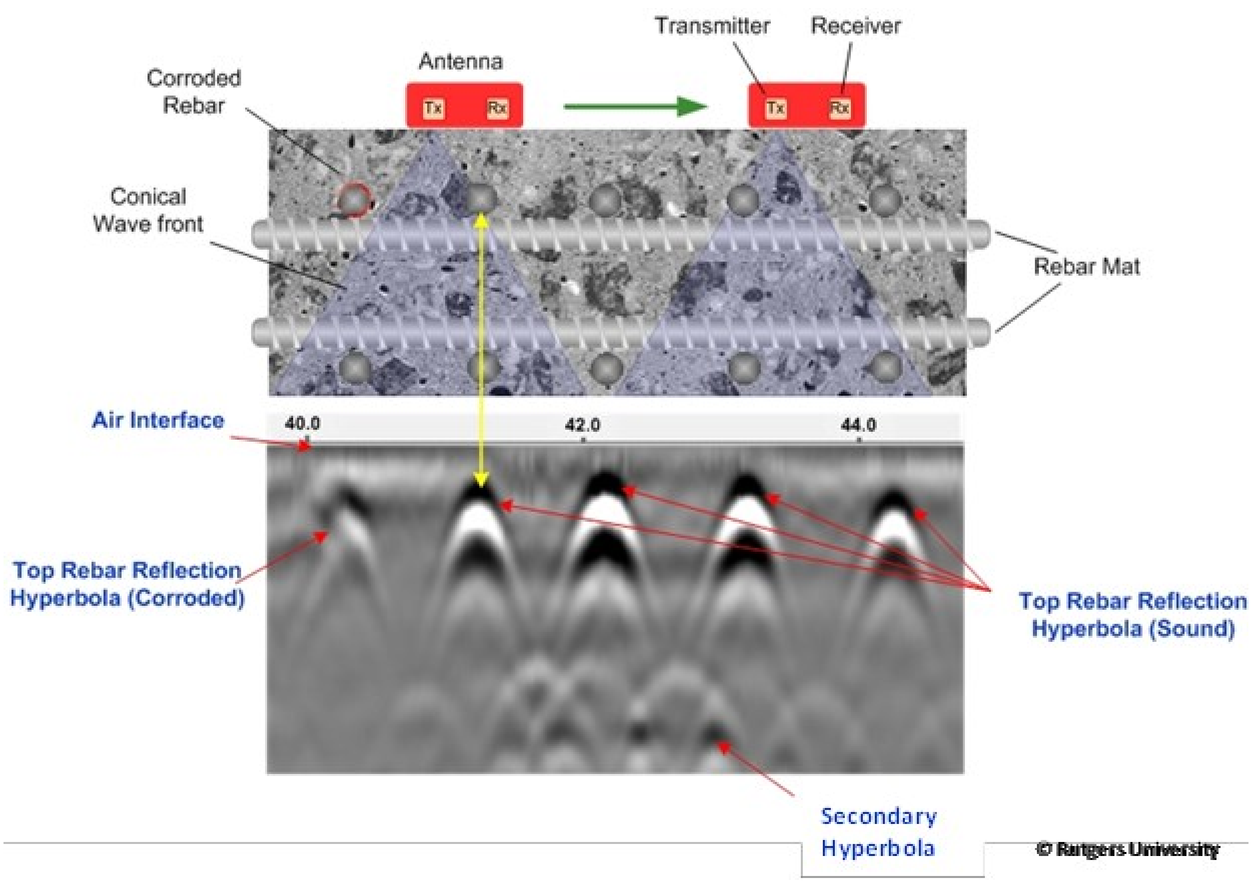

Figure A.1: Illustration of physical principle of GPR operation

A GPR system for bridge deck inspections typically consists of an antenna unit (for transmitting and receiving the electromagnetic signal), a signal recorder, a control unit, and a power supply/converter unit. As the system moves along the bridge deck, received radar signals will be recorded for later processing and analysis. To inspect a bridge deck, the GPR system could either be manually dragged over the surface, or attached to a 3-wheeled vehicle (Figure A.2). Advanced GPR systems can be mounted on traffic vehicles, allowing for faster scans and excluding the need to for traffic interruptions.

Figure A.2: Ground-coupled (wheel mounted) GPR system

One of the great advantages of the GPR method is the fact that raw data acquired in a manner that allows it to be easily viewed in real time using a computer screen. Often very little processing is required for initial interpretation of the data, with most of the effort directed towards data visualization. As stated earlier, the basic form of the data is a profile or section view of subsurface features beneath a straight-line traverse. GPR profiles can be represented as greyscale that use the different shades of grey to represent the variation in the signal amplitude. A typical GPR profile for a concrete bridge with asphalt overlay is shown in Figure A.3. The ultimate output of the GPR evaluation is a map depicting variations in the amplitude of the reflection from the top of the transverse layer of rebar. Figure A.4 shows a GPR amplitude map based on top rebar reflection for a bridge deck. Based on the interpretation of the amplitude map, the interpreter is able to identify areas of the bridge deck where there is evidence of deterioration.

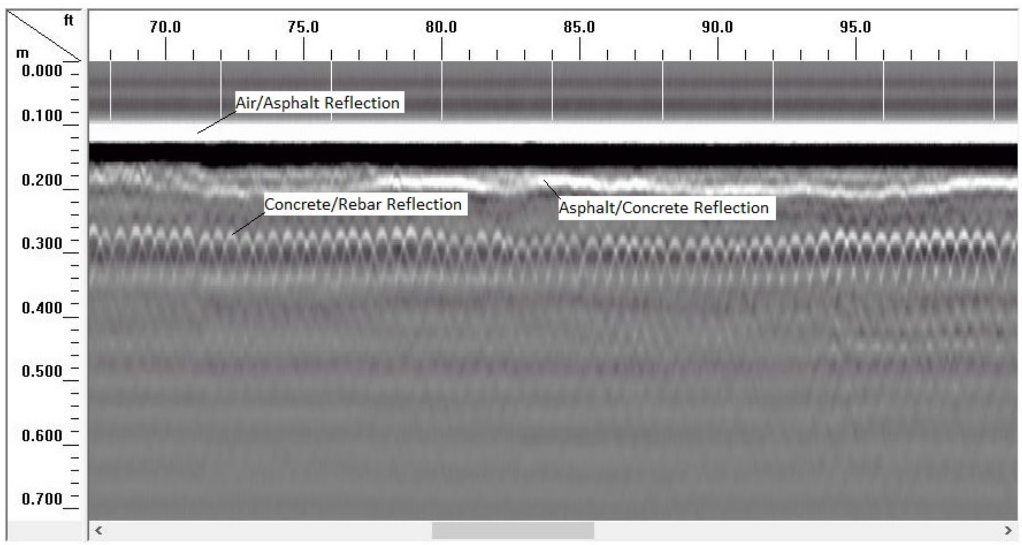

Figure A.3: Typical GPR profile for a concrete bridge with asphalt overlay

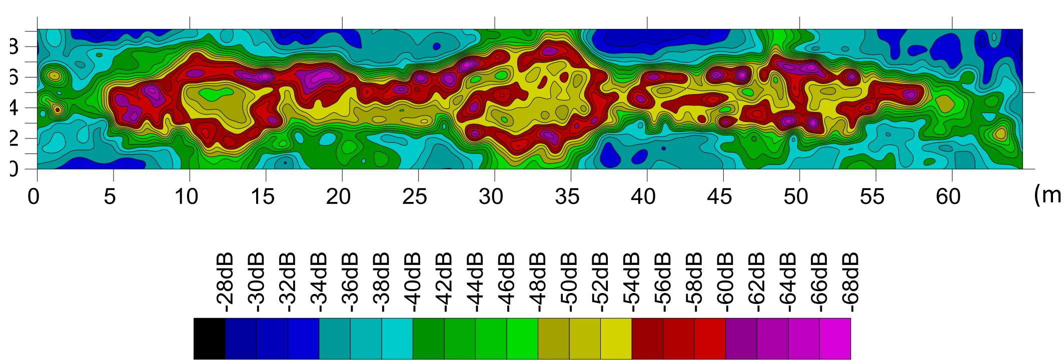

Figure A.4: GPR amplitude map based on top rebar reflection for a bridge deck

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APPENDIX C: COMMON DEFECTS IN BRIDGE ELEMENTS WITH RESPECTIVE LEVELS OF SEVERITY (Ontario Ministry of Transportation (MTO) 2008; Shami 2015; Alsharqawi et al. 2016a)

Performance Measure Group	**Performance Indicator**	**Severity (none, light, medium, severe, very severe)**
Wearing Surface	Cracking	(none, 1mm < width < 5mm, 6mm < width < 10mm, 10mm < width < 20mm, width > 20mm)
	Loss of Bond	(none, loss of bond over area < 150mm, 150mm < loss of bond over area < 300mm, 300mm < loss of bond over area < 600mm, loss of bond over area > 600mm)
	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)
	Local Protrusions (Delaminations)	(none, height < 10mm, 10mm < height < 20mm, 20mm < height < 40mm, depth > 40mm)
	Ravelling	(none, noticeable loss of pavement material, shallow disintegration of the pavement surface with an open textured appearance, shallow disintegration of the pavement surface with small potholes; very open textured appearance with loose material over the surface, deep disintegration of the pavement surface with numerous potholes; very open textured appearance with loose material over the surface)
	Flushing	(none, visible coloring of the pavement surface occurring in localized areas, distinctive coloring of the pavement surface with excess asphalt free on the pavement surface, free asphalt gives the pavement surface a 'wet' look; vehicle traffic leaves visible tire marks and impressions on the pavement surface, excessive free asphalt on the pavement surface with a 'wet' look; footprints leave visible impressions in the pavement surface)
	Rutting	(none, depth < 10mm, 10mm < depth < 20mm, 20mm < depth < 40mm, depth > 40mm)

Performance Measure Group	Performance Indicator	Severity (none, light, medium, severe, very severe)
Concrete	Cracking	(none, width < 0.1mm, 0.1 mm < width < 0.3mm, 0.3 mm < width < 1.0mm, width > 1.0mm)
	Scaling	(none, depth < 5mm, 6mm < depth < 10mm, 11mm < depth < 20mm, depth > 20mm)
	Corrosion of Steel Reinforcements	(amplitude > -7.71 dB, -7.71 dB > amplitude > -10.04 dB, -10.04 dB > amplitude > -14.63 dB, amplitude < -14.63 dB)
	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)
	Deposits	(none, exist without cracks, exist with some cracks, exist with many cracks, exist with severe cracks)
	Expansion Joints Problems	(none, < 5% of surface area/length/number, 20% of surface area/length/number, 20% - 50% of surface area/length/number, > 50% of surface area/length/number)
	Pop-outs	(none, hole depth < 25mm, 25mm < hole depth < 50mm, 50 mm < hole depth < 100 mm, hole depth > 100 mm)
Drainage system	Pipe Breakage	(none, exist)
Barriers/Railing system	Performance defects of barrier walls and railings	The performance of the top rail on parapet walls, barrier walls and railings shall be based upon their present condition with regards to their ability to meet the safety standards, guide vehicular traffic and pedestrians along the structure, and other requirements in effect at the time they were originally installed
Lighting	Lighting Faults	(none, exist)
Signs	Signs Faults	(none, exist)

APPENDIX D: RESOURCE GUIDE FOR PERFORMANCE-BASED CONTRACTING

This appendix provides practical documentation for various stages of the Performance-Based Contracting (PBC) process. These documents were developed for PBC of road works in specific countries. A number of sample performance specifications are provided as part of either bidding or contract documents. Users considering applying these materials in their own context should only treat them as good starting points, seek appropriate professional advice, and contact directly the source entities for clarifications and additional information.

BIDDING DOCUMENTS

This section contains sample bidding documents issued by agencies/departments of transportation to procure road management and maintenance under PBC. Moreover, this section provides samples of some particularly interesting elements of a bid package, e.g. a request for contractor qualifications, maintenance standards, and others. World Bank sample bidding documents are provided as a starting point. In addition, bidding documents for Cambodia, British Columbia, Canada, Madagascar, New Zealand, Serbia and Montenegro, Tanzania, and Washington, D.C, USA are included.

- **World Bank**: Sample Bidding Documents for Procurement of Works and Services under Performance-Based Contracts.

  These documents were issued to provide its clients with an alternative to the traditional methods of procuring road reconstruction, rehabilitation and maintenance.

- **Cambodia**: Bidding Document.

  The document was developed by the Ministry of Public Works and Transport of the Kingdom of Cambodia in 2004 to invite bid through the National Bidding Competition to execute routine maintenance on 331 km of national roads under a performance-based contract.

- **British Columbia, Canada**: Request for Qualifications and Maintenance Specifications.

  The request for qualifications, issued by the British Columbia Ministry of Transportation (BC MOT), is for the purpose of eliciting responses from firms wishing to be qualified by the Ministry to submit proposals for highway maintenance contracts. Another useful document from British Columbia is also included, the maintenance specifications.

- **Madagascar**: Bidding Documents and Prequalification Document for Pilot Projects for Performance-Based Management and Maintenance of Roads.

  These documents were prepared to launch pilot performance-based contracts for management and maintenance of national roads. The documents are similar to the ones prepared for Tanzania, shown below, except for the language used: they are in French for Madagascar and in English for Tanzania. The enclosed technical report presents the summary of findings and recommendations generated during the preparation of this pilot.

  - Prequalification Document for Procurement

- Bidding Document – Volume 1
- Bidding Document – Volume 2
- Technical Report

- **Tanzania**: Bidding Documents and Prequalification Document for Pilot Projects for Performance-Based Management and Maintenance of Roads.

  These documents were prepared to launch on a pilot basis six packages of performance-based contracts for management and maintenance of roads covering about 1,100 km of Tanzania's national road network. The documents are similar to the ones prepared for Madagascar (shown above). The technical report presents the summary of findings and recommendations generated during the tender preparation and bid evaluation process.
  - Prequalification Document for Procurement
  - Entire Bidding Document – Volume 1
  - Entire Bidding Document – Volume 2
  - Technical Report

- **New Zealand**: State Highway Maintenance Contract Proforma Manual and Prequalification Procedure Manual.

  In 2006, Transit New Zealand released the sixth edition of the state highway maintenance contract proforma manual and the fourth edition of prequalification - procedure manual for trial. It contains all of Transit New Zealand's tender documents relevant for performance-based road maintenance contracting, in a standard format. Part of the first-edition tender document that New Zealand developed for its pilot

Performance Specified Maintenance Contract is enclosed for those readers who are interested in following the evolution of the PBC principles and practices.

- **Serbia and Montenegro**: Bidding Document for Routine and Winter Maintenance of Main and Regional Roads.

  The tender documents for international competitive bidding were developed in 2004 by the Road Directorate of Serbia and Montenegro, to invite bidders to execute routine and winter maintenance on 660 km of main and regional roads, under a three-year hybrid performance-based contract.

  - Invitation for Bids and Instruction to Bidders
  - Conditions of Contract and Contract Data
  - Specifications
  - Security Forms

- **Washington, D.C., USA**: Request for Proposals.

  This document was developed in 2000 by the District of Columbia Department of Public Works (DCDPW), in cooperation with the US Federal Highway Administration (FHWA) to request proposals from the private sector for management of roadway assets of the National Highway System along approximately 75 miles of streets and urban roads for the period of five years.

CONTRACT DOCUMENTS

This section provides several model contract documents used by a sample of developed and developing countries. Readers should review first the respective country's information shown in the country matrix, described in the next section, to become familiar with the contexts in which specific contracts were applied.

- **Argentina**: Contract for Rehabilitation and Maintenance.

  This three-page contract is supplemented by sample performance specifications. Both documents are in Spanish.

  - Contract for Rehabilitation and Maintenance of Roads
  - Performance Specifications

- **Queensland, Australia**: Road Maintenance Performance Contracts (RMPC) for Main Roads.

  - RMPC: Volume 1 - Sole Invitee
  - RMPC: Volume 2 - Open Competition
  - RMPC: Volume 3 - Guidelines for Undertaking Routine Maintenance

- **Western Bay of Plenty District, New Zealand**: Performance-Based Road Maintenance Contract for Low Volume Roads.

  In 2002, Western Bay of Plenty District Council in association with Transit New Zealand launched a PBC to maintain low volume roads. The contract was designed to obtain better service delivery value through a single contract for road maintenance, renewal and capital expenditure, covering the 122 km of state highway and 1,040 km of local authority roads in the Western Bay of Plenty district. It included specified levels of service with pre-determined response times. A set of operational, management and key performance measures were established to monitor compliance. The contract was let for ten years to a contracting alliance of Opus Consultants, McBreen Jenkins and Works Infrastructure.

- Conditions of Contract
- Maintenance Specification
- Data Collection Contract
- Bay Roads Governance and Management Structure
- Appendices (Performance Measures, etc.)

- **Peru**: Service Contract for Maintenance of Rural Roads.

The first PBC document that was piloted to maintain rural roads in Peru was translated into English from its original version in Spanish. Similar contracts have been used on multiple rural road sections that have been maintained by micro-enterprises in Peru since 1996-97. That contract has been revised and improved. Its most recent version (2nd edition) is enclosed and is accompanied by Performance Specifications. The two latter documents are in Spanish.

- Service Contract for Maintenance of Rural Roads (1st edition)
- Service Contract for Maintenance of Rural Roads (2nd edition)
- Performance Standards

COUNTRY MATRIX: SCALE AND FORMATS OF PERFORMANCE-BASED PRACTICES IN SEVERAL COUNTRIES

The country matrix provides an overview of PBCs as used across the world, in both developed and developing countries. The data shown were drawn from the World Bank publications and archives, and from country-based information available in published form, including both printed and web-based sources. PBC formats vary from country to country, having been tailored to accommodate the specific needs of each country, match the capacities of its public and private sectors, and fit into the relevant regulatory and administrative frameworks.

To learn about the experience of countries in different regions please follow the given below links:

- Africa and Middle East
- Europe
- Latin America and the Caribbean
- North America
- South-East Asia and the Pacific

The list of countries that are interested in applying a PBC approach is growing. The following countries have, as of the beginning of 2006, started developing PBC programs, arranging for training, designing bidding packages, etc.: Albania, Burkina Faso, China, Democratic Republic of Congo, Egypt, Indonesia, Kenya, Lebanon, Nigeria, Pakistan, Paraguay, Poland, Romania, United Arab Emirates, Vietnam, and Yemen.

Source:

Resource Guide. (2009). *Performance-Based Contracts for Preservation and Improvement*. [online] Available at: http://www-esd.worldbank.org/pbc_resource_guide/index.html [Accessed 18 Jul. 2018].