INTEGRATION OF STRUCTURAL HEALTH MONITORING INFORMATION TO RELIABILITY BASED CONDITION ASSESSMENT AND LIFE CYCLE COSTING OF BRIDGES

by

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ABSTARCT

INTEGRATION OF STRUCTURAL HEALTH MONITORING INFORMATION TO RELIABILITY BASED CONDITION ASSESSMENT AND LIFE CYCLE COSTING OF BRIDGES

Bhasker Dubey

According to Transportation Association of Canada (TAC), the rough estimate of number of bridges in Canada is 80,000 with the replacement value of $35 billion. A large number of bridges will need replacement during 2005 to 2015 which will result in 50% annual increase in replacement cost. Recent alarming incidents of the Laval De la Concorde Overpass collapse (2006), Canada and the I-35W Mississippi River bridge collapse (2007), USA show the gravity of the situation. One of the main factors responsible for this situation is the present available techniques of the bridge condition monitoring and rehabilitation are not able to cope up with the drastic deterioration and ageing of the bridges. The widely employed method for bridge inspection is visual inspection, and it lacks the reliability-based assessment of bridge and its components. The instrumentation of the bridge with Structural Health Monitoring (SHM) systems and assessment of the bridge condition and behaviour based on the information obtained from SHM systems is one of the promising solutions of the present problem. The main focus of the current research is to integrate SHM data with traditional information (e.g. visual inspection), develop a reliability based structural condition index using the updated information on a structures operational performance, and assessing the value of information for SHM in regard to the overall lifecycle cost of a structure. This study

develops a methodology for a reliability based assessment of the bridge components using SHM system information, and information updating by fusing SHM data with traditional information for precise evaluation of expected life cycle cost.

The methods developed herein have been demonstrated through a case study based on an existing bridge namely, the Crowchild Bridge in Calgary, Alberta. A finite element model of the bridge has been developed and validated against the field data. This validated model has been used to simulate the static load test on the bridge, deterioration in the bridge and to study the bridge response under the different loading conditions. The artificial neural network (ANN) technique has been used for the diagnosis of the SHM data, and then the reliability index of the bridge deck has been calculated using the Monte Carlo Simulation technique.

A method for updating the bridge deck repair strategy is introduced based on the reliability index calculation. The maintenance and rehabilitation strategy is updated based on the hypothetical results. The results of updated strategy are compared with un-updated one using the Bayesian Theorem.

The expected life cycle cost is evaluated considering the capital cost, maintenance and rehabilitation cost, user cost, and failure cost. Capital cost is treated as deterministic while maintenance and rehabilitation cost, and user cost are considered probabilistic. Each individual cost and then total cost is calculated per m^2. The value of information is also discussed.

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NOMENCLATURE AND ABREVATIONS

A	Minimum cross sectional area of steel strap
ACI	American Concrete Institute
AASHTO	American Association of State Highway and Transportation Officials
A_t	Area of tension steel in a one-foot section of slab
BMS	Bridge Management System
$B_{lifecycle}$	Benefit which can be gained from the existence of the bridge after rehabilitation
cc	Unit concrete cost ($\$/m^2$)
CET	Total Expected Cost
CHBDC	Canadian Highway Bridge Design Code
CF	Expected Cost of Failure
c_F	Cost of failure set at the time of decision making
C_{ins}	Real inspection method
CINS	Cost of Performing the Inspection
$C_{lifecycle}$	Cost associated with the bridge during its whole life
CPM	Expected Cost of Routine Maintenance
CREP	Cost of Repair
C_{r_i}	Undisclosed cost of the i^{th} rehabilitation
CT	Cost of Structure
DC	Decommissioning cost ($)
$\overline{DCF_x}$	Expected discounted cash flow of series x
d_{eff}	Effective depth of the slab
D_n	Design cost ($)
d(η)	Detect ability function
DOTS	Department of Transportation
E	Modulus of elasticity of the straps
$EC_{j,i}$	Expected cost of the j^{th} rehabilitation actions when SHM test output is Z_i
$E(T)$	Expected cost of rehabilitation after updating the probabilities
EMC	Essential maintenance cost
EMC_d	Discounted essential maintenance cost
e_{rep}	Repair activity
f_c'	28 days yield strength of concrete
FEM	Finite Element Model

f_{R_0}	Probability density function
$f(\underline{x}/\underline{\theta})$	Conditional pdf of $\underline{x}$ given $\underline{\theta}$
F_R	The probability distribution of R
f_y	Yield stress of steel reinforcing in concrete deck
FWHA	Federal Highway Administration
GFRP	Glass Fibre Reinforced Polymer
$g(.)$	Limit state equation of bridge deck
$g(\underline{\theta})$	pdf of $\underline{\theta}$
$g(\underline{\theta}/\underline{x})$	Posterior pdf of $\underline{\theta}$ given $\underline{x}$
$g(t)$	Resistance degradation function
i	Discount rate
ic_n	Install rebar cost ($\$/m^2$)
J	Performance and deformability factor
JP_x	Joint probability of series x
k	Stiffness of bridge deck at any given point of time
k_0	Initial stiffness of bridge deck
L_n	Service life (years)
MC_n	Concrete repair cost ($)
M_C	Ultimate Moment Capacity
M_c	Moment corresponding to a maximum compressive strain in the concrete
M_{dl}	Dead Load Moment Capacity
MLP	Multi Layer Perceptron
M_{r0}	Original mean moment capacity
$M_{r,a}$	Residual moment capacity after
$M_{r,age}$	Mean residual moment capacity due to aging
$M_{r,b}$	Residual moment capacity before
MR	Resurface cost ($)
MT	M&R traffic control cost ($)
M_U	Live Load Moment
NEFMAC	New Fibre Composite Material for Advanced Concrete
NCHRP	National Cooperative Highway Research Program
N_c	Number of preventive maintenance cycles before essential maintenance is performed
NDE	Non Destructive Evaluation
NPV	Net Present Value
$\overline{NPV}$	Expected Net Present Value

NPV_x	Net present value for the series x
N_{pt}	Number of preventive maintenance cycle performed after the essential maintenance
PMC_j	Preventive maintenance cost corresponding to cycle having time period of *j* year for each individual path
P_f	Probability of failure
R	Flexural strength
r	Number of cycles
rc_n	Unit rebar cost ($/m^2)
$\mathscr{R}$	Reliability
$R(t)$	Time-variant resistance
R_0	Initial resistance
SHM	Structural Health Monitoring
S	Spacing of the steel girders
S_l	Spacing of the steel
S	Moment induced at cantilever
S_t	Time-variant (live) load
SC_n	Concrete cycle (years)
SR	Resurface cycle (years)
SPR	Superposed Probability of Rehabilitation
t	Thickness of the deck
T	Traffic control cost ($)
t_p	Preventive maintenance cycle period
t_e	Year of essential maintenance is performed
$TPMC$	Total preventive maintenance cost for each individual path
$TUPMC$	Total user cost for preventive maintenance (discounted)
$UPMC_j$	User cost for preventive maintenance corresponding to cycle having time period of *j* year for each individual path
$UCEM_d$	Discounted essential maintenance user cost
$UCEM$	Essential maintenance user cost
γ_{mfc}	Model uncertainty factor: concrete flexure, deck
λ_{trk}	Uncertainty factor: HS-20 truck in analysis of deck
λ_{asph}	Uncertainty factor: weight of asphalt on deck
λ_{conc}	Uncertainty factor: weight of concrete on deck
Φ	Probability density function for normal distribution

Chapter 1

Introduction

1. 1 Overview

Recent alarming incidents of the de la Concorde overpass collapsed, Canada and the I-35W Mississippi River bridge collapse, USA show the gravity of the situation produced by deteriorating infrastructure. The Johnson Commission, which was setup to inquire de la Concorde collapse, has stated that one of the main reasons of collapse was inadequate maintenance and monitoring measures (Reference??). A bridge is subjected to various types of loads during its life cycle which makes it more vulnerable compare to other civil structures. This vulnerability brings more attention to the need for appropriate and timely maintenance and rehabilitation. Before and during the 1960s and into 1970s, bridge maintenance, repair, rehabilitation, and replacement activities were performed on an as-needed basis employing the best existing practice of the time (Thompson et al. 1998). But due to aging infrastructure such actions are increasing exponentially. In the US approximately 50% of bridges are over 50 years old and over 125,000 bridges are rated as structurally deficient. This amounts of 20% of the roughly 600,000 bridges in the federal inventory (Kong 2001). It has been estimated that approximately $90 billion is needed to rectify these problems. This is in addition to the $140 billion currently spent by road authorities to maintain this infrastructure at its existing level (Kong 2001).

The exact number of Canadian bridges and their value is unknown but is estimated to consist of roughly 80,000 crossings with a replacement value of $35 billion

(TAC, 1999). About 50% of existing Canadian bridges were constructed between the great expansion periods of the middle 1950's to the late 1960's. These structures are now between 30 to 45 years old, near the ends of their service lives. A large number of Canadian bridges will require replacement between years 2005 and 2020. This will create the need for an increase in the annual bridge replacement budget of about 50% during this 15-year time period (TAC, 1999). These figures show the enormity of bridge deterioration problem in the United States and Canada. Therefore, knowing current condition of bridge is essential to engineers because it assists them to predict their performance and to optimize their replacement, maintenance, or rehabilitation activities.

1. 2 Problem statement and research objectives

The present infrastructure around the world is deteriorating rapidly because of extensive usage, ageing and negligence through the decades. Current bridge management systems, including both PONTIS (Thompson *et al.*, 1998) and BRIDGIT (Hawk and Small, 1998), are based on these subjective condition assessment and empirical models of future condition (Aktan et al. 1996, Kong 2001). The one of the main limitations of the current approach is that it doesn't address the bridge element performance from a reliability viewpoint (Frangopol and Das, 1999).

Researchers believe that the main cause of this problem lies with the inspection and monitoring methods. People are proposing new methods for inspection and monitoring, and availability of advanced technologies made it possible to adopt these new methods. The visual inspection has been a very common method for inspection and monitoring for bridges because it's easy to perform and cost effective. But the reliability of this method has always been a question as human being is always prone to error.

Therefore, the objective of current research is to provide bridge professionals with effective and practical methods in order to assess the condition of existing bridges in terms of reliability and subjective condition. And, using this condition the Life Cycle Cost Analysis is performed for the bridges.

The objectives of this study can be summarized as follows

- Develop a method to use SHM information to assess the reliability of the bridge element.
- Develop a method to incorporate new information obtained using SHM with previous information available based on historical data or visual inspection (information updating).
- Life Cycle Cost Analysis of bridge/ bridge element based on updated information.

1. 3 Research methodology

1.3. 1 Literature review

A comprehensive literature review is carried out in different areas using different sources including books, journals and the internet. The literature includes Structural Health Monitoring (SHM) techniques and their types. Further, it talks about SHM systems and smart materials. In later half it includes the reliability analysis and life cycle cost analysis. In addition, artificial neural network (ANN) and analytical hierarchy process (AHP) techniques are presented.

1.3. 2 Assessment of reliability of a bridge element

The reliability assessment passes through the following three steps

- Development of a Finite Element Model of the structure.
- Data collection and model validation.
- Modeling bridge deck deterioration.
- Reliability analysis

1.3. 3 Information updating

The previous available information is updated using the new information. A probabilistic method based on Bayesian updating has been used in this process.

1.3. 4 Life Cycle Cost Analysis

The expected life cycle cost has been calculated using the updated information.

1. 4 Theses organization

To accomplish the objectives of this research, literature survey and the synthesis on bridge condition, bridge deterioration model and bridge monitoring systems has been performed as described in Chapter 2. Literature review covers the types of monitoring techniques, types of monitoring systems and their level of sophistication, reliability analysis of bridge element and life cycle cost analysis. Moreover, a detailed description of artificial neural network (ANN) and Bayesian theorem and their application are reported.

Chapter 3 provides an overview of the proposed research methodology.

In Chapter 4 a detailed discussion of the Structural Health Monitoring system for Crowchild Bridge is done. Later, the finite element model of Crowchild bridge is developed, which is validated using real test data. Further, method to model deterioration for bridge deck is proposed.

Chapter 5 presents an overview of the reliability analysis of a bridge element (in this case bridge deck). It explains the method of calculating the structure reliability for a bridge element.

Chapter 6 presents the decision analysis methods and the proposed methodology for updating the previous information using the structural health monitoring information. Discussion and analysis of the results are presented.

Chapter 7 provides detailed life cycle cost analysis of the bridges. It compares expected life cycle cost based on updated and un-updated information. An application example of methodology implementation is shown in order to demonstrate the possible usage of the proposed methodology. Finally, it presents discussion and analysis of results in addition to limitations of the proposed method.

Chapter 8 presents conclusions, limitations of the proposed methods, and main research contributions, and recommendations for future research work.

Chapter 2

Literature review

2. 1 Overview

This chapter consists of three sections as shown in Figure 2-1. Section 2-2 covers detailed study of Structural Health Monitoring System which includes types of health monitoring; usages; advantages and disadvantages etc.

Section 2-3 presents an extensive literature review for structure reliability and its calculation for concrete bridges. It includes available structural degradation models and live load models. Structure's probability of failure and its evaluation techniques have been demonstrated. The concept of Time Variant Reliability has been discussed. The influence of load, resistance, and resistance degradation random variables on the time-variant failure probability of parallel systems is illustrated.

In section 2-4 current bridge management practices and systems have been presented. Bridge management systems like Pontis, BRIDGIT have several limitations and drawbacks; most important drawback is they don't take reliability of structure in account. Methods of evaluation of life cycle maintenance cost for highway bridges have been discussed.

2. 2 Structural Health monitoring (SHM)

The idea of SHM is not new. For thousands of years engineers have been examining the ongoing performance of their structures in an effort to prolong structures' service lives and ensure public safety (ISIS Canada, 2004). However, only recently has SHM become a more essential component of a civil engineer's education. Infrastructure sustainability is an issue that needs an immediate attention, and a general awareness of the necessity for, and implementation of, detailed SHM programs is vital to the success of the next generation of engineers. The current rapid evolution and advancement of SHM technologies can be attributed to several compounding factors, many of which are due, in part, to the efforts of organizations such as ISIS Canada. The current trend toward increased use of SHM in civil engineering and be attributed to:

- the need for long-term monitoring of innovative designs using new materials (i.e. To monitor and ensure the safety of as yet unproven materials and systems);

- the need for long-term monitoring for better management of existing structures;

- the recent advancements in the development of new, functional, and economical sensors (e.g. *Fibre optic sensors (FOSs)* and *smart materials*);

- ongoing developments in the field of digital *data acquisition systems (DASs)*;

- ongoing developments in communication technologies, including internet-based and wireless technologies;

- developments of powerful data transmission and collection systems, and data archiving and retrieval systems; and

- advances in data processing, including damage detection models and artificial intelligence algorithms.

2.2.1 Definition of SHM

SHM is defined as a non-destructive *in-situ* structural evaluation method that uses any of several types of sensors which are attached to, or embedded in, a structure (ISIS Canada, 2004). The various types of data are obtained either continuously or periodically, for future analysis and reference the data are collected, analyzed and stored. The data can be used to assess the condition (i.e. safety, integrity, strength) and performance of the structure, and to identify damage at its early stages.

The definition of SHM given above does not cover all technologies used in the evaluation and assessment of structures. The broader field would also include the use of many devices, techniques and systems that are traditionally designated as Non-Destructive Testing (NDT) and Non- Destructive Evaluation (NDE) tools (ISIS Canada, 2004). Common to all is the objective of learning about the in-service condition of the structure. There is no formal delineation between each approach, so the following distinction is adopted by ISIS. Generally NDT/NDE refers to a one-time assessment of the condition of materials in the structure using equipment external to the structure. SHM normally refers to activities focused on assessing the condition of the structure or its key components based on response to various types of loads.

It generally involves on-going or repeated assessment of this response. Some parts of the sensory system are usually embedded in or attached to the structure for the complete monitoring period.

2.2.2 SHM System Components

As noted earlier, SHM refers to the continuous or periodic monitoring of a structure using sensors. All types of civil engineering structures, including bridges, buildings, tunnels, pipes, highways and railways can be instrumented with SHM systems.

The specific details of SHM systems depend on the type of structure but a modern SHM system will typically consist of six common components, namely:

- Acquisition of data (a sensory system);
- Communication of information.
- Intelligent processing and analyzing of data.
- Storage of processed data.
- Diagnostics (i.e. damage detection and modeling algorithms) and

- Retrieval of information as required.

Figure 2-1 shows a typical flow pattern among the six components of a SHM system; however, other flow patterns are also possible, and the flow of information between system components can certainly take more than one path (ISIS Canada, 2004).

2.2.3 Acquisition of data

As name suggests this component involves the collection of raw data such as strains, deformations, accelerations, temperatures, moisture levels, acoustic emissions, and loads (ISIS Canada, 2004). Various conventional sensors may be used to record data including: *load cells, electrical resistance strain gauges, vibrating wire strain gauges, displacement transducers, accelerometers, anemometers, thermocouples* and *fibre optic sensors.*

2.2.4 Selection of sensors

It's needless to say that the selection of appropriate and robust sensors is very essential to the effectiveness of an SHM system. The specific types of sensors selected for a project depend on several considerations. In addition to the ability of measuring the desired response parameter such as strain or vibration, the selection criteria should also consider accuracy, reliability, sensor installation limitations, power requirements, signal transmission limitations, durability and cost. For cost, consideration must be given to the cost of the whole sensory system including the sensor, associated cables or wiring and the signal conditioning/data acquisition system (ISIS Canada, 2004). The type of sensors in a SHM system depends on the requirements of the project. It is critically important to have reasonable idea of the long term performance of the various types of sensors available in beginning. For instance, certain sensors are not appropriate for long term monitoring due to deterioration in sensor performance with time. The satisfactory performance can only be ensured by proper selection of sensors and their locations.

2.2.5 Sensor Installation and Placement

Recent field applications of SHM systems in real structures have demonstrated that care should be taken during the design of the SHM system to ensure that sensors can be easily installed within a structure without substantially changing the behaviour of the structure (ISIS Canada, 2004). During the design process consideration of sensor wiring, conduit, junction boxes, and other accessories required to house the SHM system on site. The Experience gained in sensor installation shows that poor durability or installation of the cable network and poor design of the data acquisition equipment for field environments can significantly reduce the functionality of the SHM system though the embedded sensors themselves can be quite durable. The various installation issues are addressed in detail in the recently published Civionics Specifications, available from ISIS Canada (ISIS Canada, 2004).

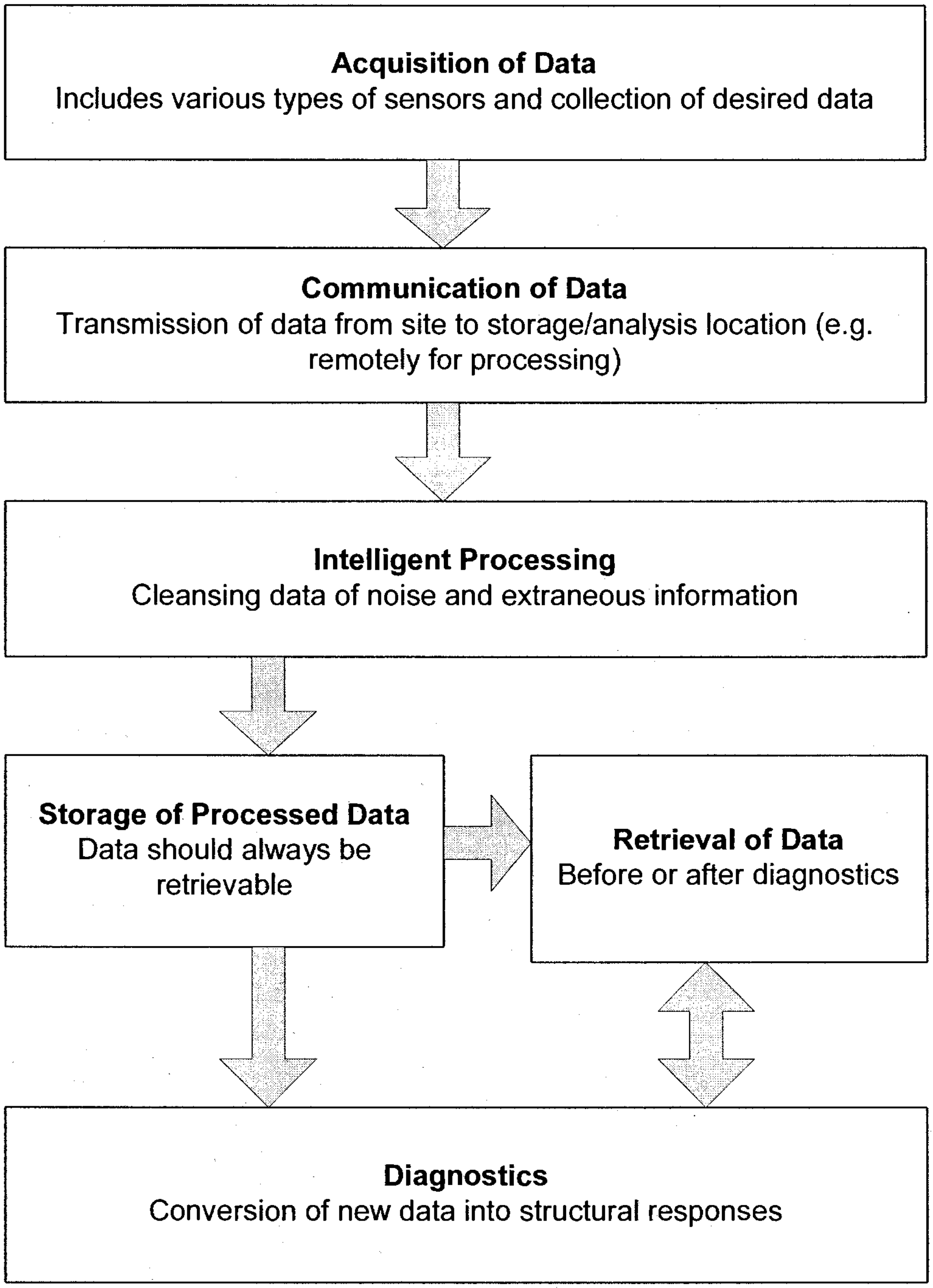


Figure 2- 1 Component of typical SHM system (ISIS Canada, 2004)

2.2.6 Transfer to Data Acquisition System (DAS)

Data acquisition is the sampling of the real world to generate data that can be manipulated to obtain desired information, and the onsite system required for this task is known as data acquisition system. The signal reception, conditioning and storage of measured data are conducted using DAS prior to being transferred to an offsite location for analysis (the data-logger). To understand the sensor output the interpretation of the output signal must be conducted to convert the analog sensor response into engineering terms. For example, for fibre optic sensors, an input light source must be supplied and the reflected light from the sensor must be measured and converted into strain. All sensors must communicate with the DAS in order to store the response information in a temporary buffer or in long-term memory. Generally physical link, *lead cable or wire,* is used to transfer the sensor signal directly to the DAS. The main advantage of this method is less cost. But in few cases very long lead wires can lead to errors resulting from electromagnetic interference (EMI), particularly in the presence of high-voltage power lines or radio transmitters. The use of differential signaling techniques and properly shielded cables can sometimes mitigate the effects of EMI. The FOS technologies are not normally affected by EMI (ISIS Canada, 2004). In any case, extreme care must be taken during the construction process to ensure that sensor cables are not accidentally sheared off or otherwise damaged. Lead cable connections are appropriate in most situations and in cases where structures are not so large as to make physical connections problematic. However, for very large structures in which lead cable transmitted sensor signals might be corrupted by excessive noise, or where long lead cables are otherwise impractical, emerging wireless communications technologies can be used to transfer sensor signals to the DAS. Wireless data transfer is currently more expensive than direct connections, data is typically transferred much more slowly, and the signals are not completely secure (ISIS Canada, 2004). However, it is expected that wireless communications will be increasingly used for SHM of very large structures in the future. For some sensory systems, a combination of the two transmission techniques may be employed. For example, many sensors will require that the sensor be connected to the signal source/demodulation system by a physical link. The communication from demodulation equipment nodes to the main data logging system for the structure can be wireless. Another solution which has been used successfully, on the Golden Boy SHM project in Manitoba (ISIS Canada 2004), is to convert voltage signal (the standard output of sensors) to current. The reason is that the current signal can be transmitted much further without corruption. Many types of DAS can read current directly, or current can be converted back to voltage at the DAS. This has proven to be a reliable and inexpensive solution.

2.2.7 Data Sampling and Collection

The online storage of sensor signals is very crucial. Once signals arrive at the DAS, capturing an adequate amount of data is an essential task, and a well thought out *data acquisition algorithm*, eventually, becomes a very important component of a successful SHM system. In the case of extensively instrumented structures the amount of data generated may be unmanageable, and to avoid this situation an efficient system set up is necessary. A general rule is that the amount of data should not be so scanty as to jeopardize its usefulness, nor should it be so voluminous as to overwhelm interpretation

(ISIS Canada, 2004). A low sampling rate leads to the former, and an unnecessarily high rate to the latter. Of course, in some cases, as in the case of *dynamic testing* (discussed later), high sampling rates are required to accurately measure the structure's response to transient loads. The decision about sampling rate depends on the type of test is being performed or conducted, and hence experience plays an important role in data sampling.

Table 2- 1 What is monitored, how and why? (ISIS Canada, 2004)

No.	Types of Information	Measured using	Useful in
1	Load	Load cells	Design load
2	Deformation	Displacement Transducers	In design
3	Strain	Electric resistance strain gauge, vibrating wire strain gauge, fiber optic strain gauge	Sudden changes in strain give info about something happening in structure
4	Temperature	Thermocouples, thermistors	How temp changes effect structure
5	Acceleration	Accelerometers	How structure resisting acceleration and resulting loads
6	Winds speed & pressure	Anemometers	Useful in long span bridges and tall bldgs
7	Displacement	GPS	Useful in long span bridges and tall bridges

2.2.8 Communication of Data

The communication of data deals with the data transfer from the onsite location (the DAS) to the location where they will be processed and analyzed (normally some remote location). This is an important aspect of an effective SHM system, since it allows remote monitoring, and reduces the frequency of site visits and inspections by engineers considerably. In this way, engineers/owners can monitor the performance of their structures from the comfort of their own offices. Modern SHM systems transmit field data remotely, either through telephone lines or the internet, or using wireless technologies such as radio or cellular transmission. Examples of communication systems used in ISIS projects can be found in Han et al. (2004).

2.2.9 Intelligent Processing and Management of data

The intelligent processing, as its name suggests, is a technique to extract useful information from the obtained data. In general, various sensors in a structure generate a large amount of data which are likely to contain extraneous information and *noise* that may not serve the purposes of structural health monitoring. Hence, intelligent processing of data is required before it can be stored for later interpretation and analysis. The main objective of intelligent processing is to make data interpretation easier, faster, and more accurate by removing this unwanted information. In many cases, intelligent processing is also required to remove the influence of thermal or other unwanted effects in the data. In addition, to deal with the sometimes overwhelming amounts of data generated by SHM systems, various data management strategies have been developed to eliminate unnecessary data without sacrificing the integrity of the overall system (ISIS Canada, 2004). One simple technique is to record only changes in readings and times corresponding to those changes. In this way, long periods in which nothing changes are omitted from the data. Alternatively, an SHM system may record readings only above a certain threshold value, or perhaps only the peak readings measured over a designated length of time.

In more sophisticated systems, neural computing and artificial neural network techniques may be employed (McNeill 2004). Algorithms are designed to learn the characteristic patterns of the signals and identify only those patterns which can be classified as 'novel'. For example, on bridges with low to medium traffic volumes, particularly with respect to heavy trucks, the majority of signals produced by a continuous monitoring program will be small compared to the signals generated by heavy trucks. The latter is of more interest. Neural computing can be used to isolate the truck response as novel compared to all other responses and only this section of the data will be tagged for storage or further analysis (ISIS Canada, 2004). This can be conducted in an unsupervised mode by the monitoring computer such that no human input is required and the data management becomes automatic and efficient. Sometimes a combination of data acquisition algorithms may be required depending on the situation. The volume and the type of diagnostic information can be obtained from the stored data depend on the data acquisition algorithms so it's very crucial component of SHM system.

2.2.10 Storage of Processed data

After intelligent processing of the data, they need to be stored for later diagnostics. Two very important points should be considered, first one is data should be stored in a way that once retrieved they are apprehensible, and other is longevity of data without susceptibility to corruption. Need less to say that amount of memory required for data storage, especially in the case of continuous health monitoring, can be very large. So care must be taken to ensure the availability of sufficient memory as it is crucial that data files have enough information about the data so that it's easy to interpret. The amount of memory space for storage can be achieved by discarding the raw data, but this takes away the flexibility of later interpretation of data (ISIS Canada, 2004).

2.2.11 Diagnostics

Diagnostics deals with further interpretation of the collected, cleansed, and intelligently processed data. The main objective of diagnostics is to convert the abstract data signals to produce useful information about the condition and behaviour of the structure. The structural behaviour always gives information about damage, deterioration and condition of the structure. So, the people concerned with the diagnostics should have an adequate knowledge and understanding of the structures. The degree of complexity of the analysis depends on the needs of the monitoring program and the SHM system components. It can be as simple as converting strain readings into stresses for assessment against critical limits, and as complex as using artificial neural network and numerical models to determine the probability that a measured change in response reading indicated a specific damage and location (ISIS Canada, 2004). The appropriate numerical model of the structure calibrated against baseline field measurements is normally required irrespective of level of sophistication.

2.2.12 Retrieval of Data

During selection of the data to store for retrieval, both the significance of the data and the confidence in its analysis should be considered (ISIS Canada, 2004). For example, for a *static field test* (discussed later), the volume of data generated is relatively small; therefore, both the raw data and the diagnostic information can be easily stored for retrieval. Conversely, for a *dynamic field test*, the volume of data generated is quite large, and therefore only the diagnostic information is stored. Of course, the overarching goal of structural health monitoring is to provide detailed physical data which can be used to enable rational, knowledge-based engineering decisions (ISIS Canada, 2004).

2.2.13 SHM Categories

In addition to the various components of SHM systems, structural health monitoring can be classified into one of at least four overall types or categories, each consisting of several smaller sub-categories (ISIS Canada, 2004). These categories are distinguished by the type of testing undertaken, both in terms of how data are physically collected, and with respect to the timescales over which data are obtained. The main categories are listed below:

1. Static Field Testing
2. Dynamic Field Testing
3. Periodic Monitoring
4. Continuous Monitoring

The details of these methods and their relative advantages and disadvantages are given in Table 2-2 and 2-3, respectively.

Table 2- 2 Types of structure monitoring and their characteristics (ISIS Canada, 2004)

S.No	Test	Types	Procedure	Measure
1	Static Field Test	Behaviour test	The test is carried out using loads that are less than or equal to the maximum allowed service load on the structure.	Test shows how a load is distributed throughout a structure.
		Diagnostic test	Same as Behaviour test	Interactions between structural components
		Proof load test	loads are gradually increased until the limit of linear elastic behaviour is reached	Maximum load that a structure has withstood without suffering any damage.
2	Dynamic Field Test	Stress History Test	The strain profiles are analysed to determine the strain ranges experienced by the components	Determine the range of stresses experienced by parts which are prone to failure by *fatigue loading*
		Dynamic Load Allowance (DLA) Test	No single standard for DLA.	Determination Dynamic Amplification Factor.
		Ambient Vibration Test	Using accelerometers vibration response of the structure is measured.	Identify the damage and deterioration using vibration characteristics
		Pull-Back Test	By pulling the structure laterally by means of cables anchored in the ground	To measure vibration characteristics.
3	Periodic Monitoring	Monitoring through ambient vibrations	Same as Ambient Vibration test	Detailed identification the damage and deterioration using vibration characteristics
		Bridge monitoring through testing under moving traffic	Periodic records of a structure's response under moving traffic.	Measuring the progression of strains observed in various bridge components over time,
		Monitoring through static field testing	Same as static field test	Periodic check for changes in bridge behaviour
		Monitoring crack growth	Manually bridge cracks are measured	Assess the bridge condition on the basis of cracks
		Periodic monitoring repairs	Static field test done before and after the repair	Quantify the effectiveness of repair
4	Continuous Monitoring		Monitoring of a structure for an extended period of time	Monitor behaviour of structure continuously.

Table 2- 3 The advantages and disadvantages associated with each test (ISIS Canada, 2004)

S.No	Testing Category	Advantage	Disadvantage
1	Static Field Test	-Interpretation data is less complex. -Easily calibrated against theoretical models.	Tests don't capture the full load response actually experienced by structure.
2	Dynamic Field Test	-More precise than static load tests. -Measure the vibration characteristics too.	-Still lacks the desired accuracy. -Sophisticated analysis techniques are required for damage identification. - Sometimes very risky to perform.
3	Periodic Monitoring	-Very accurate as it is performed several times. - More advanced.	- As it is periodic it might be possible to miss very important event during the service life of bridge. - Requires more resources
4	Continuous Monitoring	-Structured is monitored continuously so information is very precise.	-Very costly.

2. 3 FRP steel free bridge deck

One of the main causes of the deterioration of the civil engineering structures is corrosion in iron and steel material used during construction. This situation has led people to develop new techniques to increase useful life of the structures. Several countries are working on to construct structures that are lighter, stronger, and non-corrosive. ISIS Canada is playing an important role for advancement of these sorts of techniques, and use fibre reinforced polymers (FRP) and fibre optic sensing (FOS) devices are latest example of this.

The two perceived disadvantages FRPs have compared to steel are ductility and low thermal compatibility between FRP reinforcement and concrete (Mufti 2005). But reinforced concrete structures, whether reinforced with steel bars or FRPs at ultimate loads give large deformation. The research is in progress to show that if properly designed, the FRP concrete structure can also dissipate the energy. Further it says that the design of proper cover eliminates low thermal compatibility between FRP reinforcement and concrete. The glass fibre reinforced polymer (GFRP) material has a same modulus of elasticity comparable to concrete. Therefore, concrete doesn't feel any intrusion into it and performs well in resisting fatigue under dynamic loading (Mufti 2005). A list of Canadian bridges constructed using the steel-free deck system are listed in Table 2-4.

Table 2- 4 FRP bridge decks all over the Canada

Project	Details	Achievements	Reference
Salmon River highway bridge, Nova Scotia	- two 31 m spans - cost of steel free side was	This concept has won six national and one international including NOVA	Newhook et al. 2000. Mufti 2005

	6% more than the steel side	from CIF of the US.	
Crowchild Trail Bridge, Alberta	- three spans of length 30 m each - NEFMAC used for side barriers - A total of 103 strain gauges, two fibre optic strain sensors, and five thermistors were used for monitoring system	- First continuous span steel free bridge deck in the world.	Afhami and Cheng 1999 Mufti 2005
Taylor Bridge, Manitoba	- 2 lane 165.1 m long structure - 4 FRP girders out total 40 girders - CFRP reinforcements were used.	This bridge has both types of materials, so the monitoring of this bridge will allow engineers to compare both	Shehata and Rizkalla 2000
Joffre Bridge, Quebec	- built in 1950,and rehabilitated in 1997 - NEFMAC C19-R2 grid was used for the deck slab - Bridge was extensively instrumented with 180 critical locations		Benmokrane et al 2000

Hall's Harbour Wharf, Nova Scotia	- 96 years old - the concrete beam are designed with a hybrid reinforcement scheme	It is designed to last 80 years, and it received the "Award of Excellence" from the Canadian Consulting Engineer Association.	Newhook and Mufti 2000
Red River Bridge, Manitoba (Winnipeg)	- ten span bridge 347 m long - constructed in 1964 - 11% costlier than the conventional one because of unfamiliarity with FRP installation.	First application of second-generation steel free bridge deck.	Memon et al. 2003, Mufti et al. 2003,

2.3. 1 Design fundamentals of the second-generation steel-free deck slab

To design second generation steel free bridge deck slab, two parameters must be investigated (Mufti et al 2003). The first one is size and spacing of external steel strap, and second is the allowable stress and strain levels in the GFRP reinforcement under service load conditions. The CHBDC (2000) states the each steel strap must have a minimum cross-sectional area, in millimeters squared, given by

$$A = \frac{F_s S^2 S_l}{Et} 10^9 \qquad 2.\ 1$$

where the factor F_s is 6.0 for outer panels and 5.0 for internal panels, S is the spacing of the steel girders that must not be exceeded 3.0 m, S_l is the spacing of the steel and must not be more than 1.25 m, E is the modulus of elasticity of the straps, and t is the thickness of the deck in millimeters. Once area and spacing are known the failure load can be calculated using PUNCH (Mufti and Bakht 1996) software.

According to CHBDC (2000) the stress and strain levels can be determined based on a performance and deformability factor, J, greater than 4.0 where

$$J = \frac{M_{ult}\Psi_{ult}}{M_c\Psi_c} \qquad 2.2$$

in which M_{ult} is the ultimate moment capacity of the slab, M_c us the moment corresponding to a maximum compressive strain in the concrete of 0.001, Ψ_{ult} is the curvature at the moment M_{ult} , Ψ_c is the curvature at the moment M_c .

In the case of steel reinforcing bars, ACI 318 (1999) allows a crack width of 0.3 mm for exterior exposure. But when GFRP bars are used CHBDC (2000) allows the crack width up to 0.5 mm as there is no risk of corrosion (Mufti 2005).

2.4 Structural Reliability

According to Thoft-Christensen and Baker (1982), structural reliability should be considered as having two meanings- a general and mathematical one.

1) In most general sense, the reliability of a structure is its reliability to fulfill its design purpose for some specified reference period.
2) In a narrow sense it is probability that a structure will not attain each specified limit state (ultimate or serviceability) during a specified reference period.

Here we are more concerned about the narrow sense. To understand the reliability in terms of probability, a simple example is taken form Thoft-Christensen and Baker (1982).

If a cantilever has flexural strength R and the moment induced at cantilever as S then probability that the structure will collapse during any reference period of duration T years will be

$$P_f = P(M \leq 0) = \int_{-\infty}^{+\infty} F_R(x) f_S(x) dx \qquad 2.\ 3$$

where M = R-S, and F_R is the probability distribution function of R and f_s the probability density function of S. *In this case, distribution of R and S are both assumed to be stationary with time. Similarly the reliability $\mathscr{R}$, defined as*

$$\mathscr{R} = 1 - P_f \qquad 2.\ 4$$

If r is the fixed value of random variables R then probability of failure

$$P_f = P(r - s \leq 0) = 1 - F_S(r) \qquad 2.\ 5$$

2.3. 2 Fundamental of structural reliability theory

Reliability function: According to Thoft-Christensen and Baker (1982), the probability of failure of a system or component is a function of operating or exposure time; so that the reliability may be expressed in terms of the distribution FT of the variable T, random time to failure. The reliability function $\mathscr{R}$T which is the probability that the system will still be operational at time t is given by

$$\mathscr{R}_T(t) = 1 - F_T(t) = 1 - P(T \leq t) = P(T > t) \quad 2.\ 6$$

Failure rates and hazard functions: The probability of failure within any given interval [t, t + δt] is the probability that the actual life T lies in the range t to t + δt and is given by

$$P(t < T < t + \delta t) = F_T(t + \delta t) - F_T(t) = \Re_T(t) - \Re_T(t + \delta t) \quad 2.\ 7$$

The average rate at which failure occurs in any time interval [t, t+δt] is defined as the failure rate and is the probability per unit time that failure occurs within interval, given that it has not already occurred prior to time t, namely

$$\frac{\Re_T(t) - \Re(t + \delta t)}{\delta t \Re_T(t)} \quad 2.\ 8$$

The hazard function is defined as the instantaneous failure rate as the interval δt approaches zero.

$$\mathrm{h(t)} = \lim_{\delta t \to 0} \frac{\Re_T(t) - \Re(t + \delta t)}{\delta t \Re_T(t)} = \frac{f_T(t)}{\Re_T(t)} \quad 2.\ 9$$

The use of hazard function is in indicating whether a system or component becomes progressively more or less likely to fail per unit time as time progresses. If it becomes progressively more likely to fail the clearly action should be taken replace the system or at some stage or to minimize the consequences of failure.

2.3.3 Structural reliability analysis

According to Thoft-Christensen and Baker (1982), electronic/mechanical systems, structural systems tend not to deteriorate, except by the mechanisms of corrosion and fatigue, and in some cases may even get stronger for example: the increases in the strength of concrete with time, and increase in the strength of soils as a result of consolidation. What basic data are available for the time to failure of electronic and mechanical components, no such information is available for structural components, because in general they do not fail in service (this problem can be reduced using SHM).

Structure or structural components fail when they encounter an extreme load, or when a combination of loads causes an extreme load effect of sufficient magnitude for the structure to attain a failure state; this may be ultimate or a serviceability condition.

The calculated reliability or failure probability for a particular structure is not a unique property of that structure but a fraction of the reliability analyst's lack of knowledge of the properties of the structure is not a unique property of that structure and uncertain nature of loading to which it will be subjected in the future.

The reliability of a reinforced concrete bridge is a time-variant property which is dependent on the history of both the applied loads and the remaining strength of the structural elements. The reliability of bridges with nondegrading resistance can be accurately predicted using established time variant vehicle live load models (Ghosn and Moses 1986, Nowak 1993, Bailey 1996) and structural reliability methods (Ang and Tang

1984, Melchers 1987). Reliability-based design and evaluation of deteriorating bridge structures may be found elsewhere (Lin 1995, Estes and Frangopol 1996, Estes 1997, Frangopol et al. 1997b, Frangopol and Estes 1997a). For bridges subjected to environmental attack, the resistance can decrease with time. The rate of strength loss is dependent on the degradation mechanism (e.g. sulfate attack, alkali-silica reaction, freeze-thaw cycle attack, corrosion), the aggressiveness of the environment, the properties of the reinforced concrete, the degree of protection of the bridge against environmental attack, the geometry of the section, and the failure limit state under consideration, among others (Enright et al. 1996).

2.3.4 Time-Variant Reliability of Reinforced Concrete Bridges

The need for the application of time-variant reliability methods to bridge life-cycle cost prediction is becoming increasingly recognized in the North America (Chang and Shinozuka 1996, Structural 1996). The reliability of a reinforced concrete bridge is a time-variant property which is dependent on the history of both the applied loads and the remaining strength of the structural elements. For bridges subjected to environmental attack, the resistance may decrease with time. A reliability analysis of a bridge subjected to environmental attack should therefore consider both time-variant load and resistance. Bridges are exposed structures that are continuously subjected to attack from the surrounding environment. In contrast with vehicular collision damage, environmental damage occurs gradually over time, and often goes undetected until significant damage has occurred (Kong 2001). For reinforced concrete bridges, environmental attack causes minor to significant damage, including cracks and reduction in cross section of concrete and corrosion of embedded steel reinforcement (Stratfull 1973, Crumpton and Bukovatz 1974, Cady and Weyers 1984, Rabbat 1984, Tork 1985, Coggins and French 1990, Vaysburd 1990, Murray and Frantz 1991, Ohta et *al.* 1992, Dickson et *al* 1993, Whiting et *al* 1993, Schupack 1994). Some researchers have identified the original source and location of environmental damage, but few have proposed probabilistic models or predictions for future damage to concrete bridges (Kong 2001). Most studies on the reliability of reinforced concrete bridges do not consider the time dependence of the resistance of bridge elements. In these studies, it is assumed that the concrete elements are nondegrading and, consequently, the resistance does not decrease over the service-life of the structure. In several recent studies (Lin 1995, Estes 1997, Frangopol and Estes 1997a. Frangopol et al. 1997b) the reliability of deteriorating bridge structures has been estimated using an approximate time-variant reliability approach. Although this approach requires fewer computations as compared with exact time-variant methods, it tends to predict failure probabilities which might be significantly higher than actual and serves only as a crude approximation to the actual time-variant failure probability (Kong 2001). Efforts to solve time-variant reliability problems have been concentrated on weakest-link systems where failure of any member causes global failure (Mori and Ellingwood 1993). This system failure criterion can be successfully used for predicting the service-life of structural systems based on any-first component failure. However, since most buildings and bridges are, in general, redundant structures, failure of an individual component does not imply system failure. When allowance must be made for redundancy (i.e. system ability to continue to carry loads after the damage or the failure of one or more members), reliability of fail-safe systems has to be predicted. Although reliability methods are well established for time-invariant fail-safe systems (Ang and Tang 1984, Guenard 1984, Karamchandani 1987, De 1990), relatively few researchers have proposed reliability analysis methods for time-variant fail-safe (parallel) systems under time-dependent random loads and strengths.

System reliability analysis is gaining popularity for the design and evaluation of highway bridges. In the United States, AASHTO bridge design code (AASHTO 1994) includes provisions which are based on system reliability requirements. A wide variety of system models (e-g., series systems, various series-parallel systems) have also been proposed for the reliability analysis of girder bridges (Kong 2001). The selection of the system model can have a significant influence on the reliability estimate for the bridge, particularly when features such as post-failure load redistribution and correlation among strengths of the girders are considered.

2.3.2.1 Time Variant Resistance

Several strength degradation mechanisms are possible for concrete structures (including sulfate attack, alkali-silica reaction, freeze-thaw cycle attack (Enright et al. 1996)), strength loss due to corrosion of steel reinforcement. The time-variant resistance of an element can be expressed as the product of the initial resistance and a resistance degradation function (Mori and Ellingwood, 1993):

$$R(t) = R_0 . g(t) \qquad 2.\ 10$$

where R(t) = time-variant resistance, & R_0 = initial resistance, and g(t) = resistance degradation function. Resistance degradation functions can be divided in to two categories: (1) Degradation function for concrete and (2) Degradation function for steel. Table 4 and Table 5 show the degradation mechanisms for both categories. The corrosion of reinforcing steel occurs as a two stage process (Tuutti 1980). During the first stage (corrosion initiation) no metal loss occurs. The protective layer (passivation) of gamma iron oxide (formed by the alkaline environment provided by the surrounding concrete) is dissolved during this stage. Metal loss occurs during the second stage of corrosion, the propagation phase. Various degradation models as reported in Estes (1997) are listed in Table 2-5.

Table 2- 5 Degradation Mechanism for concrete (Estes 1997)

Name (1)	Equation (2)	Symbols (3)	Reference (3)
Sulfate Attack Models	Kinetic Model: $x(t) = kt^{\alpha}$ $\alpha \geq 1$	$x(t)$ = depth of deterioration k = rate parameter (dependent on environment and in situ concrete) t = elapsed time α = parameter	Mori and Ellingwood [1993] Jones and Ellingwood [1992]
	Shrinking Core Model: $x(t) = (2D_i C_0 t / C_s)^{0.5}$	x = depth of deterioration (cm) D_i = intrinsic diffusion coefficient (cm^2/s)	Walton *et al* (1990)
Alkali-Silica Reaction Model	$x(t) = t_0 + kt_1^{\alpha}$ $\alpha > 1$	$x(t)$ = penetration depth k = rate parameter t1 = elapsed time a = parameter	Clifton and Knab (1989)
Freeze-Thaw Cycle Attack Models	$N = t_0 + K_0 R$	N = number of freeze-thaw cycles to failure K_0 and R represent environmental and resistance factors, respectively	Clifton and Knab (1989)

	$R_{ft} \approx$ $(N/T_C)\left(\frac{0.05}{\sqrt{\theta - 0.21T_r}}\right)$	R_{ft} = annual rate of degradation N = number of freeze-thaw cycles T_c = time to reach damage θ = water content T_r = residual water content	Walton et al (1990)

Active corrosion is usually initiated by one of two processes: carbonation or chloride ion penetration. Carbonation is the process by which atmospheric carbon dioxide diffuses in to the concrete and reacts with the calcium hydroxide in the cement which results in a more acidic environment. The steel becomes depassivated and active corrosion is initiated. According to Clifton and Knab (1989), chloride ions are the primary cause of corrosion of concrete structures. For bridge structures, deicing salts (applied to bridge decks) are the major source of chloride ions (Whiting *et al* 1993).

2.3.2.2 Varying Load Moment

Dead load moment and resistance can be calculated from bridge plan. The mean and coefficient of the resistance and dead load effect can be based on information presented MacGregor (et al. 1983) and Nowak (et al. 1994). The mean and coefficient of variation of live load effect can be obtained from linear regression analysis of load effects due to heavily loaded trucks (Nowak 1993) and AASHTO girder distribution factors for interior bridge girders (AASHTO 1994).

A live load model which predicts the maximum truck moments and shears for different length spans was developed by Nowak (1993).The study covered 9,250 selected trucks from the Ontario Ministry of Transportation data base. The data base included number of axles, axle spacing, axle loads, and gross weight of the vehicles. The bending moments and shears were calculated for each truck in the survey for a wide range of spans. The cumulative distribution functions (CDF) of the span moments and shears were plotted on normal probability paper for spans ranging from 10 feet (3.05 m) to 200 feet (60.96 m). The maximum moments and shears for different time periods were extrapolated from these distributions. These CDFs were transformed to a standard normal distribution and the coefficients of variation for the maximum shears and moments were determined from the slope of the transformation.

The end result was a series of graphs which provide a ratio of the mean shear and moment for the live load model to the shear and moment resulting from the standard HS-20 truck. This quantity is the bias factor needed for the random variable. The coefficients of variation for the maximum moment and shear are provided on other graphs. To read the graphs, one must know only the bridge span and the desired life of the bridge. The Nowak graphs were based on a measured two week traffic flow which equates to approximately 1,000 trucks per day. It is estimated that 1.5 million trucks will pass over the bridge in five years, 15 million trucks in 50 years, and 20 million trucks in 75 years. The Nowak graphs are based on the statistics of extreme values where the probability of encountering a large truck at the extreme tail of the distribution increases as the number of trucks passing over the bridge increases. As a result, the mean values of the maximum moment and shear increase over time and the coefficients of variation decrease. The Nowak graphs can be applied to a specific bridge where the daily traffic is known by reading the data for a single truck from the Nowak study and applying extreme value statistics to the actual traffic of the bridge under consideration.

Table 2- 6 Degradation models (Estes 1997)

Name (1)	Equation (2)	Symbols (3)	Reference (4)
Carbonation Penetration Models	$dx/dt = kt^{0.5}$	dx/dt = penetration rate k = proportionality constant t = elapsed time	Clifton and Knab (1989)
	$x(t) = kt$	$x(t)$ = penetration depth k = rate constant t = elapsed time	Clifton and Knab (1989)
	$x(t) = kt^{0.5}$	$x(t)$ = depth of carbonation k = rate constant (depends on concrete and carbon dioxide concentration) t = elapsed time	Mori and Ellingwood (1993) Jones and Ellingwood (1992)
	$x(t) = \left(\frac{2D_i C_{gw} t}{C_s}\right)^{0.5}$	$x(t)$ = distance (cm) D_i = intrinsic diffusion coefficient of calcium ions in concrete (cm^2/s) C_{gw} = concentration of inorganic carbon in groundwater or soil moisture $(moles/cm^3)$ t = time (s) C_s = bulk concentration of calcium hydroxide in concrete solid $(moles/cm^3)$	Walton *et al* (1990)

Chloride Ion Penetration and corrosion Initiation Models	$C/C_s = 1 - erf\left(\frac{x}{2\sqrt{Dt}}\right)$	C = chloride concentration for initiation of corrosion C_s = surface chloride concentration x = depth erf = error function D = chloride diffusion coefficient t = elapsed time	Jones and Ellingwood (1992)
	$x(t) = k\sqrt{t}$	$x(t)$ = depth of chloride ion penetration at time t k = constant t = elapsed time	Mori and Ellingwood (1993)
	$t_c = \frac{129x_c^{1.22}}{WCR[Ct]^{0.42}}$	t_c = time to onset of corrosion (yr) x_c = thickness of concrete over rebar (in) WCR = water to cement ratio (by mass) Cl = chloride ion concentration in ground water (ppm)	
Chloride Ion Penetration and corrosion Initiation Models	$d = 1.426t^{0.5} + 1.27$	d = depth of cover (cm) t = time (*years*)	Purvis *et al* (1990)
	$R = \frac{129S_i^{1.22}}{K^{0.42}(W/C)}$	R = time to cracking of substructure pile (*years*) S_i = depth of concrete cover reinforcing steel (*inches*) K = chloride concentration of water in contact with concrete (*ppm*) W/C = water to cement ration by weight	Thoft-Christensen *et al (1997)*
	$d = 0.644t^{0.82} + 1.27$	d= depth of cover (cm) t = time (years)	Purvis et al (1990)
	$T_I = \frac{(d_1 - D_1/2)^2}{4D_C} \cdot \left(erf^{-1}\left(\frac{C_{cr} - C_0}{C_i - C_0}\right)\right)^{-2}$	T_I = time to initiation of reinforcement corrosion C_i = initial chloride concentration C_{cr} = critical chloride concentration at which corrosion starts C_0 = equilibirium chloride concentration on the concrete surface (percent weight of cement) D_C = chloride diffusion coeff. (cm^2/s) $d_1 - \frac{D_1}{2}$ = concrete cover	Thoft-Christensen *et al* (1997)

Table 2- 7 Degradation models for corrosion (Estes 1997)

Name (1)	Equation (2)	Symbols (3)	Reference (4)
Corrosion Propagation Model	$x(t) = r_c t$	$x(t) =$ depth of penetration of active corrosion $r_c =$ corrosion rate t = elapsed time	Mori and Ellingwood(1993)
	$D_i(t) = D_1 - C_{corr} i_{corr} t$	$D_i(t) =$ diameter of reinforcement bars at time t $D_1 =$ initial diameter $C_{corr} =$ corrosion coefficient $i_{corr} =$ corrosion rate t = time	Thoft-Christensen et al. (1997)
	% Area Remaining $= 100\left[1 - \frac{4asD_i C_{gw} t}{\Pi d^2 \Delta x}\right]$	$D_i =$ diffusion coefficient of oxygen s = spacing between reinforcement bars $C_{gw} =$ concentration of oxygen in surrounding groundwater t = elapsed time d = diameter of reinforcement $\Delta x =$ depth of reinforcement below surface $a =$ constant $= 9.2 \frac{cm^3}{mole}$	Walton et al. (1990)

The cumulative-time failure probability of a deteriorating element subjected to two statistically independent load processes with intensities S_l and S_2 can be expressed as (Mori and Ellingwood 1993)

$$P_f(t_L) = 1 - \int_0^\infty \int_0^\infty \exp\{-\lambda_{S_1} t_L [1 - \frac{1}{t_L} \int_0^{t_L} F_{S_1}(r.g(t) - s_2) dt]\} f_{S_2}(s_2) f_{R_0}(r) ds_2 dr \quad 2.\ 11$$

where $P_f(t_L)$ represents the probability of failure over a duration $(0,t_L)$. As mentioned, this is also called the cumulative-time failure probability or, in short, failure probability. S_l is time-variant (live) load, As, and *Fs,* are the load occurrence rate (also called mean occurrence rate) and the cumulative distribution function of time-variant (live) load, respectively, $g(t)$ is the resistance degradation function, S_2 is time-invariant (dead) load, f_{S2}, is the probability density function of S_2, and f_{R0}, is the probability density function of the initial resistance. The resistance and loads are assumed to be statistically independent. It is also assumed that the live load process S_i is modeled as a sequence of randomly occurring load events (i.e., pulses) with random intensities S_i $(i = 1, 2,...., n)$ and duration. Additionally, the random intensities are assumed to be statistically independent and identically distributed (i.e., cumulative distribution function *Fs,)*. As mentioned by Mori and Ellingwood (1993), this stochastic load model (i.e. Poisson point process) allows the temporal variation of live load to be described in simple terms. The cumulative time failure probability of a series system of m deteriorating elements subjected to the aforementioned live load process with intensity S_i can be expressed as (Mori and Ellingwood 1993):

$$P_f(t_L) = \int_0^{\infty} \underbrace{\cdots\cdots}_{m-fold} \int_0^{\infty} \{1 - \exp(-\lambda_{S_1} t_L . \{1 - \frac{1}{t_L} \int_0^{t_L} F_{S_1}[\min_{i=1}^{m}(r_i . g_i(t)] dt\})\} . f_{R_0}(r) dr \quad 2.\ 12$$

where $g_i(t)$ is the resistance degradation function for element i (i.e., fraction of initial strength of member i remaining at time t), q is the structural action coefficient for element i, and $f_{R0}(r)$ is the joint probability density function of the initial strength of the elements in the system. Equations 4.1 and 4.2 can be solved using Monte Carlo simulation method.

2. 5 Evaluation of Expected Life-Cycle Maintenance Cost

Kong and Frangopol (2003) proposed a methodology for the evaluation of expected life-cycle maintenance cost of deteriorating structures by considering uncertainties associated with the application of cyclic maintenance actions. The methodology can be used to determine the expected number of maintenance interventions on a deteriorating structure, or a group of deteriorating structures, during a specified time horizon and the associated expected maintenance costs. The method is suitable for application to both new and existing civil infrastructures under various maintenance strategies.

During their service life, structural systems can experience various types of inspections and/or maintenance actions at different times. The associated costs of these actions can only be predicted by using conditional joint distribution functions. Since multiple integral steps are required, the solution process is usually computationally inefficient. To increase computational efficiency, the prescribed probability distributions of the times of various maintenance interventions are converted to probability mass functions. The classical event tree model (Ang and Tang 1984) was modified to consider not only available event paths but also lengths (i.e., durations) of these paths. In this manner, multiple integrals are replaced by summations. This is very effective for the evaluation of the expected annual probability of maintenance when cyclic interventions are applied and the expected annual costs associated with these interventions have to be evaluated. The time-dependent effect of expenditures can be represented by the discount rate.

To calculate the probability of maintenance for each intervention cycle, PDF for that intervention cycle has to be represented by PMF. Strictly speaking, the PDFs have to be broken in a number of intervals of equal length, let's say t_u, *and* the probabilities of random variables falling in each interval t_u have to be calculated. Summation of all probabilities associated with this point in time gives the superposed probability of (any) rehabilitation at that particular time.

To calculate the cost of maintenance, the starting year of service life of a new structure is assumed as the base year of discounting. The cost of the *i*th rehabilitation occurring at time t can be calculated by taking into account the discount rate v.

$$C_{r_i}(t) = \frac{C_{r_i}}{(1+v)^t} \qquad 2.\ 13$$

where C_{r_i} =undisclosed cost of the i^{th} rehabilitation. If n rehabilitations occur at different times then total rehabilitation cost associated with this case can be calculated by adding all rehabilitations costs. Then, the expected rehabilitation cost at particular time t can be calculated by multiplying probabilities with their corresponding cost, and then summing them up.

Frangopol (1997) proposes a method to optimize the lifetime inspection/repair strategy of corrosion-critical concrete structures based on the reliability of the structure and cost effectiveness. For the bridges there two types of maintenance are performed; Preventive maintenance and Repair maintenance. Preventive or routine maintenance includes replacing small parts, patching concrete, repairing cracks, changing lubricants, and cleaning and painting exposed parts. Repair maintenance might include replacing a bearing, resurfacing a deck, or modifying a girder. Repair maintenance tends to be less frequent, requires more effort, is usually more costly, and results in a measurable increase in reliability. While guidance for routine maintenance exists, many repair maintenance strategies are based on experience and local practice rather than on sound theoretical investigation. The optimal policy has to be chosen based on minimal expected total life-cycle cost and structural reliability.

Preventive maintenance cost is, in general, estimated as an engineering cost associates with the routine maintenance expenditure. Such estimates are obtained by summing the products of input and their unit rates (McNeil and Hendrickson 1982). For a given bridge, the cost of routine maintenance at any time t, may be assumed a linear function of multiplication of cost of preventive maintenance at year one and age of the bridge in years (McNeil and Hendrickson 1982). The future maintenance costs are converted to their present cost using discount rate. For inspection, in this paper it is assumed that all inspection and repair work is for the corrosion of steel reinforcement in concrete and thus requires a nondestructive evaluation (NDE). To represent the degree of existing damage due to corrosion at time t, the damage intensity (η) is defined which is ration of difference between initial diameter and diameter at time t of bending reinforcement bar to initial diameter of a bending reinforcement bar. The impact of corrosion on the bending capacity of a concrete bridge girder is generally greater than on its shear capacity (Lin 1995). The η can range from a value 0 to 1. If T is corrosion initiation time in years then η has zero value before this time as there is no corrosion induced damage. A detect ability function $d(\eta)$ is defined which is the probability of detecting damage given η. In this paper $d(\eta)$ is modeled as a cumulative density function for each NDE method. In general, the cost of inspection is dependent on the quality of NDE method; a higher quality inspection is usually more expensive. Assuming that the cost for the ideal inspection [i.e., $d(\eta)=1$ for $\eta>0$] is αins, the cost associated with a real inspection method C_{ins}, can be estimated on the quality detectability as follows (Mori and Ellingwood 1994b):

$$C_{ins} = \alpha(1-\eta_{\min})^{20} \qquad 2.\ 14$$

where, $\eta_{\min} > 0$ is the minimum detectable damage intensity.

Inspection themselves do not affect the probability of failure of a structure. Following an inspection, a decision must be made regarding repair if damage is found.

Higher quality of inspection may lead to higher quality of repair which brings the reliability of the structure closer to its original condition (Mori and Ellingwood 1984a). In reality, however, the inspection methods are not perfect. Some items that require repair may be overlooked. When the damage intensity is less than ηmin for the inspection method being used, the probability of detection is zero and the structure will not be repaired. Consider a repair following an inspection method with median detectability $\eta_{0.5}$ at time T_i. The structure has a damage intensity η_f (i.e., $\eta_{rep} < \eta_f < \eta_{max}$). Due to the uncertainties associated with detectability, some of the damage will not be detected. After repair, the damage intensity will be reduced from η_f to η_{rep}. It is assumed that the damage intensity after repair, η_{rep}, is expressed as

$$\eta_{rep} = (\eta_{min} + \eta_{max})/2 = \eta_{0.5} \tag{2. 15}$$

In this study aging, which also represents the factors like internal degradation, accidental collisions, is assumed linear function of time. The mean residual moment capacity due to aging

$$M_{r,age}(t) = (1 - 0.004t)M_{r0} \tag{2. 16}$$

where, M_{r0} = original mean moment capacity; and t = age of the structure in years. The repair activity (e_{rep}), is defined as amount by which this activity improves the condition of structure, can be quantified as

$$e_{rep} = \frac{M_{r,a} - M_{r,b}}{M_{r0}} \tag{2. 17}$$

where $M_{r,a}$ and $M_{r,b}$ are residual moment capacity after and before repair, respectively. The repair cost can be expressed in terms of the repair effect as follows (Mori and Ellingwood 1994b):

$$C_{rep} = \alpha_{rep}(\frac{M_{r,a} - M_{r,b}}{M_{r0}})^{\gamma} = \alpha_{rep} e^{\gamma}_{rep} \quad 2.18$$

where, γ = a model parameter; and α = replacement cost.

The total expected cost (CET) is sum of cost of structure (CT), the expected cost of routine maintenance (CPM), the expected cost of inspection and maintenance, which includes the cost of performing the inspection (CINS) and the cost of repair (CREP), and expected cost of failure (CF). For optimal solution, cost has to be minimized while probability of failure shouldn't go below maximum allowable. The optimum design solution has been selected using uniform interval inspection strategy and non-uniform inspection strategy. For uniform interval inspection only number of inspection was optimized while for non- uniform interval inspection both, the number of inspection and time intervals themselves, were optimized). It is found that non-uniform time interval inspection/repair strategy is more economic and requires fewer life time inspections/repairs than that based on uniform time interval inspections. CET was most sensitive to the corrosion rate and the cost of failure. Also, CET was relatively insensitive to the quality of inspection and the number of lifetime inspections above the optimum number.

Chapter 3

Methodology

3. 1 Introduction

The methodology of current research is illustrated in Figure 3- 1. Current research employs the following steps: literature review, finite element model of Crowchild bridge deck, model validation, data generation and stiffness calculation using ANN, limit state equation for Crowchild bridge deck, reliability index calculation, decision making, and expected life cycle cost evaluation. A brief description of the intended methodology is provided below. Literature review the structural health monitoring systems and techniques, current bridge management practices and their limitation, and finally available methods of life cycle cost analysis for bridge maintenance and rehabilitation. For data generation and validation a finite element model of Crowchild Bridge is developed, and then it's used to simulate the degradation in the bridge deck. The expected life cycle cost analysis has been performed to compare different maintenance and rehabilitation strategies.

3. 2 Literature review

This part summarizes relevant literature and presents it in different sections. Section 2.2 in chapter 2 includes literature review for types and characteristics structural health monitoring including its component, data acquisition, selection, installation and placement of sensors, and data processing. This section also includes a review of data diagnosis techniques.

Section 3-3 illustrates the concept of structural reliability analysis. It also includes the time-variant reliability of reinforced concrete bridges, time variant resistance and varying load moment. These steps are important for reliability index calculation.

In addition, an extensive literature review for the available bridge management systems and their limitations are presented in section 2.4. This section also gives an historical overview of bridge management in North America.

Section 2.5, shows the literature review for the evaluation of expected life cycle cost. It includes current practices of life cycle cost estimation.

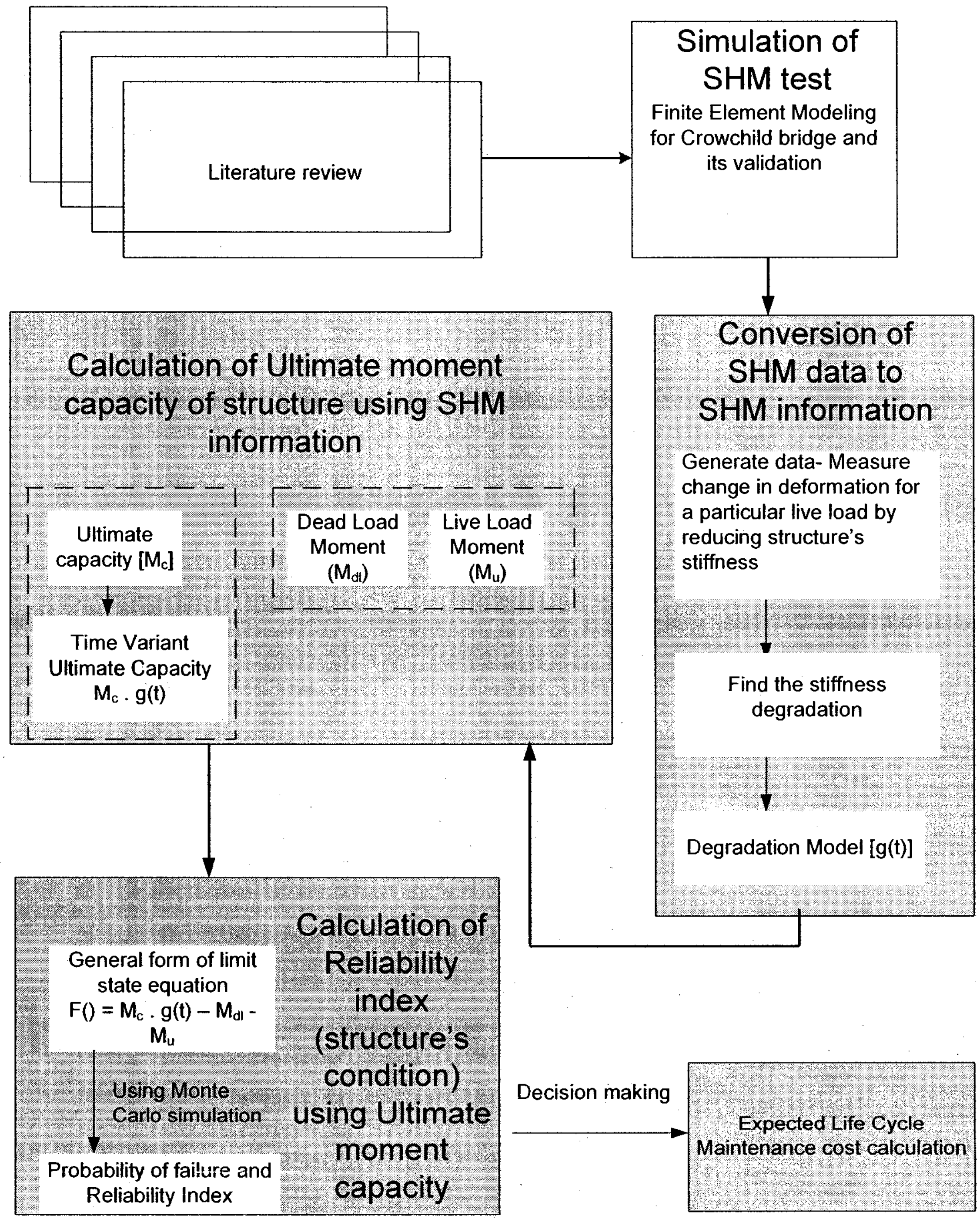


Figure 3- 1 Research methodology

3. 3 Finite Element Model of the Crowchild Bridge

An analytical model of the Crowchild Bridge is constructed using three dimensional beam elements for the piers, girders, diaphragms and the cross frames including the steel straps, and shell elements for the deck and side barriers. The deck elements are connected to the girder elements by rigid beam elements. The piers are assumed to be fixed at their base, while roller and pin supports are assumed to exist at the north and south abutments, respectively. The FE model contains 351 elements, 247 nodes and 1399 active degrees of freedom. The density of steel and concrete is assumed to be 76 and 24 kN/m3, respectively. The concrete compressive strength is taken as 35 MPa. The modulus of elasticity for concrete is assumed to be 30 GPa for the deck and 27 GPa for the barrier and pier; for steel it is assumed to be 200 GPa. Later this model is used to generate the data. Later, this model is validated against static test data (Bagchi, 2005).

3. 4 Stiffness calculation using artificial neural network

The degradation is a general function of stiffness reduction. In this study it is assumed that both are linearly correlated. The stiffness of bridge deck is gradually reduced (by 5%), and deformations at certain nodes have been measured for respective stiffness. In this way a data set has been generated which consists of stiffness values of deck and the deformations values for an applied load. Using this data set an ANN is trained which has inputs as deformations values and output as stiffness. Back propagation neural network theory is employed to design the network architecture. The 75% of data from data set selected randomly to train the network, and rest 25% is used to validate it.

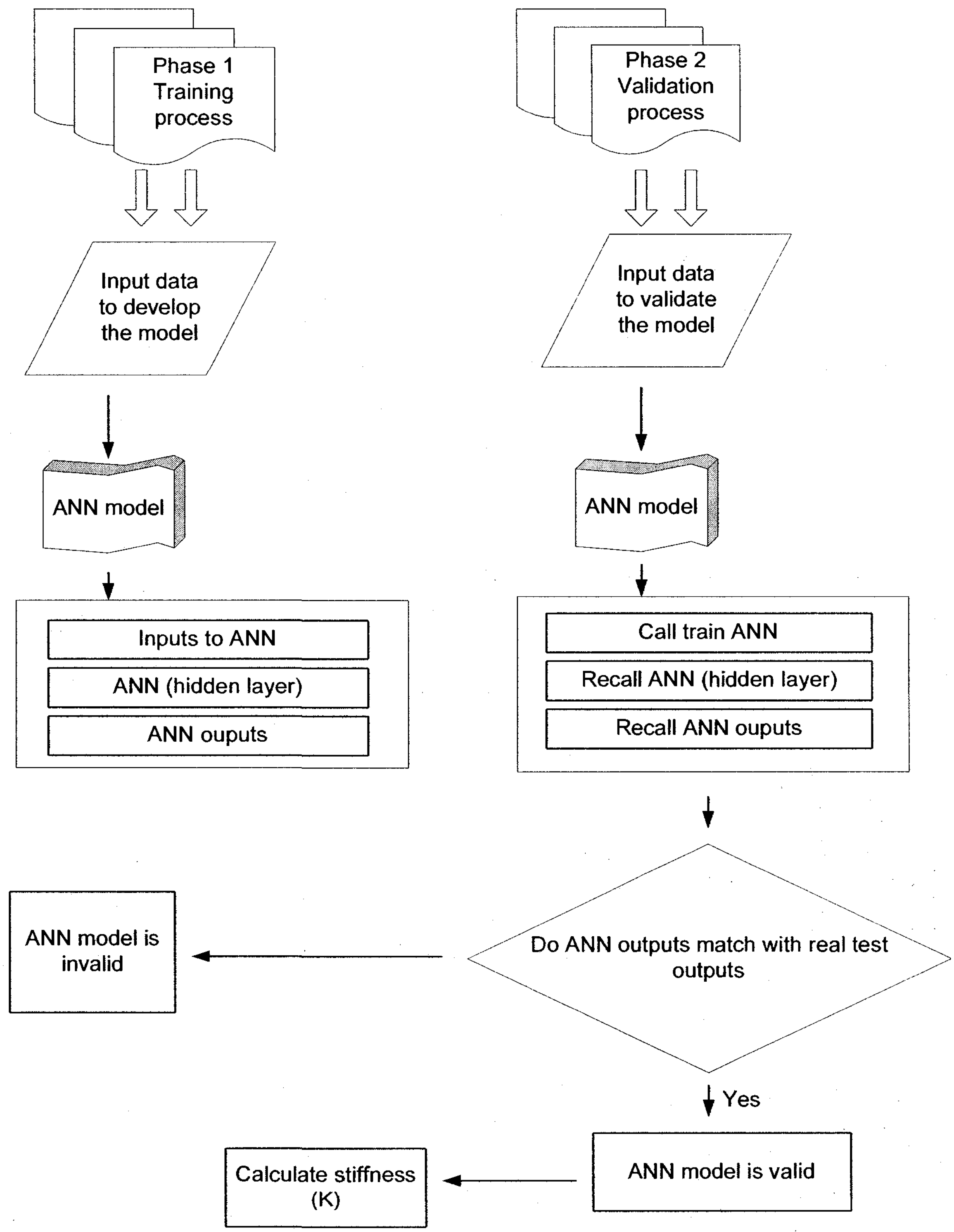


Figure 3- 2 Stiffness calculation using ANN.

Once model is validated it is used to calculate the stiffness of the deck using real test data. Using this method SHM information can be used to assess the condition of the bridge or its elements.

3. 5 Reliability index calculation

Estes (1997) proposed a method to develop limit state equation for the bridge elements. In this method the ultimate moment capacity, live load moment and dead load moment need to be calculated for a bridge element for a certain period of time. The ultimate moment capacity depends on the design of the element. The dead load moment can be calculated once dimensions and materials density are known. The calculation of live load moment is complex. In this study Nowak live load model (Nowak, 1993) has been used to calculate the live load moment. To use Nowak live load one has to know the length of bridge span and the time period for which live moment is being calculated. Using these three parameters, the limit state equation for bridge deck has been formed. Each parameter has uncertainties associated with it, so none of these parameters is deterministic in nature. All parameters have certain ranges and probability density functions.

Once limit state equation is formed the reliability index (β) is calculated for bridge element (in this case bridge deck). This index is an indicator of the probability of failure, or it is associated with structural reliability.

3. 6 Decision making

The reliability index gives the information about structure's condition and behavior. Once decision makers know the value of reliability index, they have more precise information about structure, and hence it is an important tool for the decision making for maintenance and rehabilitation schemes. This new information can be incorporated with the previously available information to have a better understanding and decisions. Figure 3- 3 shows the decision making flow chart.

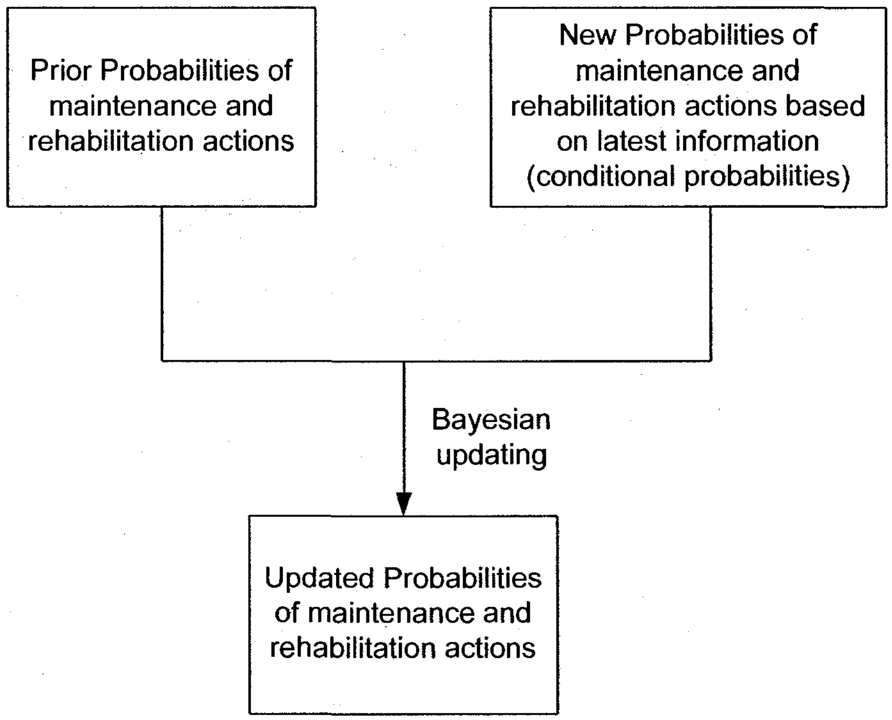


Figure 3- 3 Decision making approach

In general, the maintenance and rehabilitation actions have probabilities associated with them depending on the year of performance. These probabilities can always be updated based on new probabilities. For this Bayesian updating has been used in this study.

3. 7 Expected life cycle cost evaluation

The life cycle cost involves mainly four types of cost: initial cost, maintenance and rehabilitation cost, user cost, and failure cost. Other than initial cost, all costs are difficult to estimate precisely. The user cost and failure cost are very subjective, so calculations of these costs are very tedious. The calculation of maintenance and rehabilitation cost is also not an easy task as its keep changing during life of the structure. But, its expected value can be calculated over the time. There are two sorts of maintenance costs: preventive maintenance and essential maintenance costs. These costs are interdependent.

In this study, the life cycle cost and user cost have been calculated using expected monetary value criterion. It is assumed that both costs have triangle distribution.

Chapter 4

Structural Health Monitoring system for the Crowchild Bridge

4. 1 Crowchild Trail Bridge

The original Crowchild Trail Bridge in Calgary, Alberta, was a two-lane, three-span prestressed concrete box-girder bridge. The bridge was found to be under-strength as a result of deterioration over 20 years and increased traffic load on the bridge. Therefore, the bridge superstructure was replaced in June 1997 (Ventura et al, 2000). The new superstructure is the first continuous span steel free bridge deck in the world. The removal of internal steel reinforcement is made possible by providing lateral restraint to the supporting steel girders through evenly spaced transverse steel straps placed across the tops of the adjacent girders. Glass fiber reinforcements are used at the regions of interior supports and overhanging cantilevers. Prefabricated glass fiber reinforcing grid, NEFMAC, is used for the reinforcement of side barriers (Tennyson et al, 2000).

It is composed of five longitudinal steel girders (900 mm deep), a polypropylene fiber reinforced concrete slab deck and prefabricated glass fiber reinforced concrete barriers. The five longitudinal girders are spaced at 2 m. Four evenly spaced cross-frames in each span and steel girder diaphragms at the supports hold the main girders in place. The main girders are also connected by evenly spaced steel straps placed across the top of the girders to provide lateral restraint to them. The girders and straps are connected to the deck slab by stainless steel stubs. The deck is 9030 mm wide and does not contain any internal steel reinforcement. The slab thickness is 275 mm along the girders and 185 mm elsewhere (Ventura et al, 2000). Figure 4-1 shows overall view of Crowchild Bridge and Figure 4- 2 shows cross section area of it.

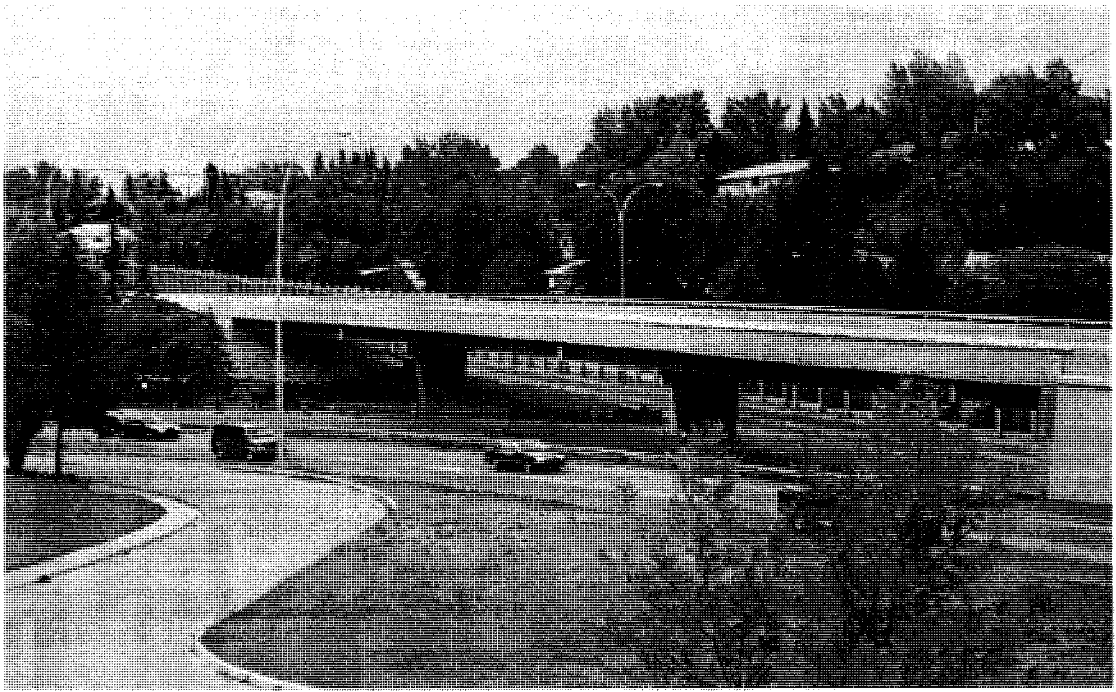


Figure 4- 1 Overall view of the University Drive/Crowchild Trail Bridge, Calgary, Alberta (Ventura et al, 2000).

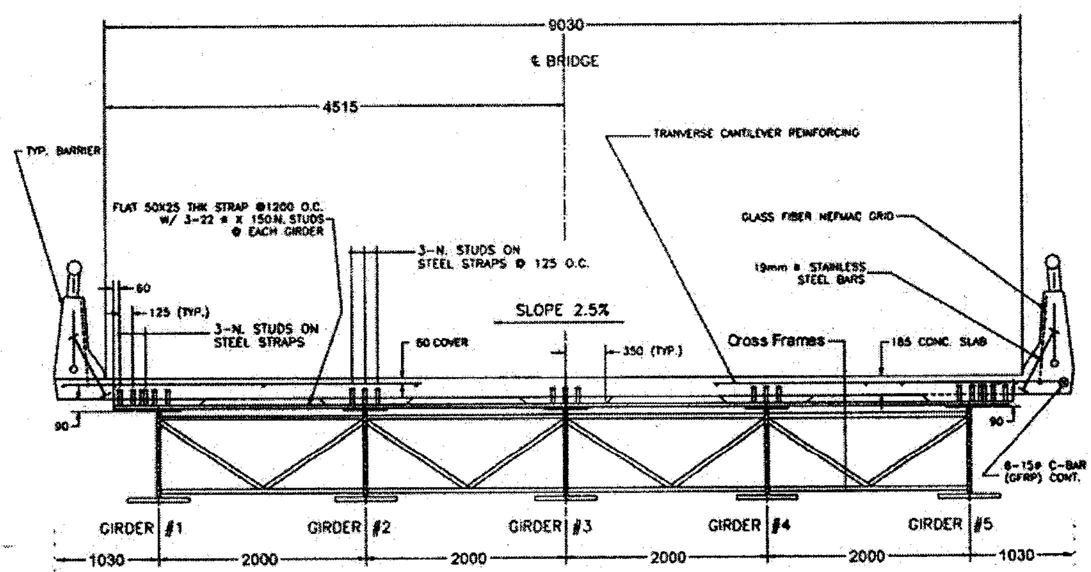


Figure 4- 2 Cross Section of Crowchild Bridge (Cheng and Afani,1999)

4. 2 Monitoring Setup

A total of 103 strain gages, two fiber optic strain sensors, and five thermisters were used in the monitoring program. The first tests (1997) consisted of a static truck load test, an ambient vibration test, an effect of temperature test, and dynamic measurements under passing trucks. The second tests (1998) consisted of static and dynamic truck load tests and ambient vibration test. To monitor strain distribution in the transverse direction of the bridge deck, 17 embedded strain gages were installed in a total of five precast blocks—three in the positive moment region and two in the negative moment region (Tennyson et al, 2000). Figure 4- 3 shows the location of embedded strain gauges.

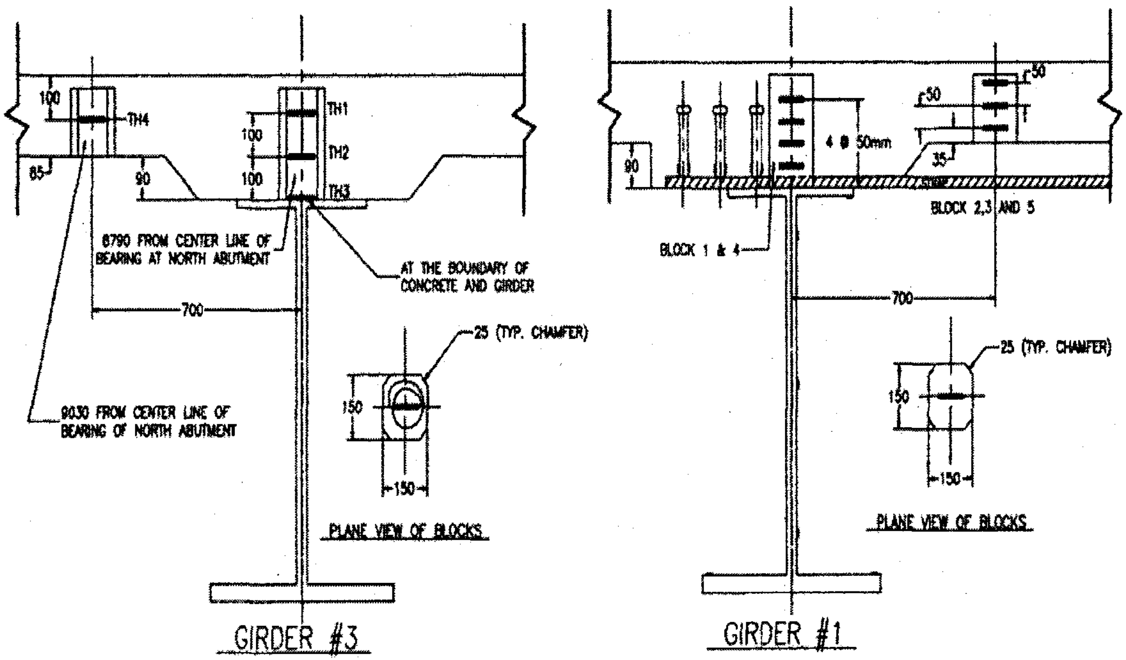


Figure 4- 3 Embedded Strain Gauges (ISIS Canada, 2004)

Eighteen gages monitored the performance of the steel straps. Six strain gages were used to monitor strains in end shear studs of the strap. Thirty-four strain gages were used to monitor steel girders. The webs of all five girders were instrumented with three gages at both positive and negative moment regions to monitor load sharing among the girders and moment distributions along the girders. Four gages were also installed on the flanges to measure any warping of the girders. The response of one cross frame was monitored by four strain gages. At the barriers, two strain gages were installed on a NEFMAC and two on a stainless steel stud. Six gages at the overhanging cantilevers and 14 gages at the pier monitored glass fiber reinforcement (Tennyson et al, 2000).

To evaluate the use of FOS technology, two commercially available sensors were installed on the glass fiber reinforcement at the same section as the electrical strain gages. The sensors were Fabry–Perot type and non-compensated for temperature. To measure deflections of the bridge under heavy traffic loads, a testing program was organized by the City of Calgary before the bridge was open to traffic. Two trucks, each loaded nominally to 355 kN, were used to produce nine different load cases (Ventura et al, 2000). Temperature profiles were recorded with the thermisters and strain measurements were taken using the strain indicator and the manual switching box. As the test took several hours, it was necessary to account for the thermal effects. The results provided preliminary information such as load sharing among the girders, location of the neutral axis, and moment distribution between mid-span and support. Similar information was later obtained from the results of the dynamic measurements. Measured strains were all less than 80 $\mu\varepsilon$ in the girders, and less than 40 $\mu\varepsilon$ in the steel straps. Concrete strains were insignificant (Ventura et al, 2000).

4. 3 Static Load Test

The first tests (1997) consisted of a static truck load test, an ambient vibration test, an effect of temperature test, and dynamic measurements under passing trucks. The second tests (1998) consisted of static and dynamic truck load tests and ambient vibration test.

The bridge consists of three continuous spans named as north span, interior span and south span which have length of 29.830m, 32.818m and 30.230m, respectively. During static load test in 1997, two 80,000 lbs trucks were placed at nine positions as shown in Figure 4- 4.

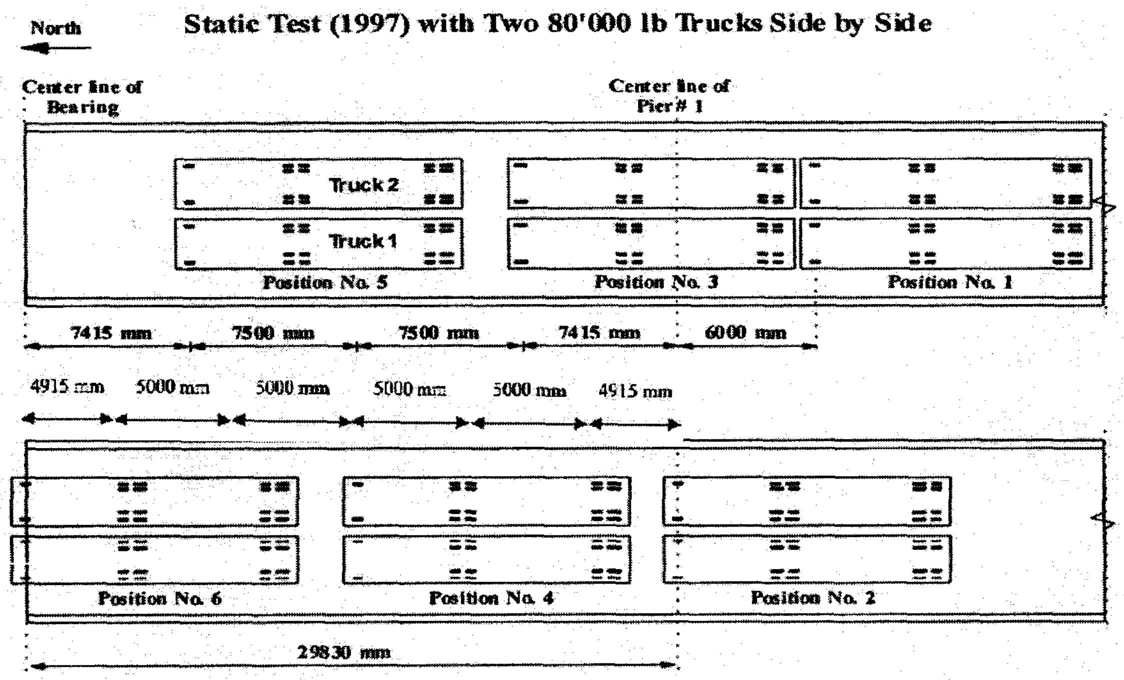


Figure 4- 4 Positions of trucks and equally spaced points (ISIS Canada, 2004)

North span was surveyed at five equally spaced points on each girder and deformation for each position was measured in mm. Distance between the points was 5000 mm. Points a and e were at 4915 mm from north abutment and pier no. 1 as shown in Figure 4- 5.

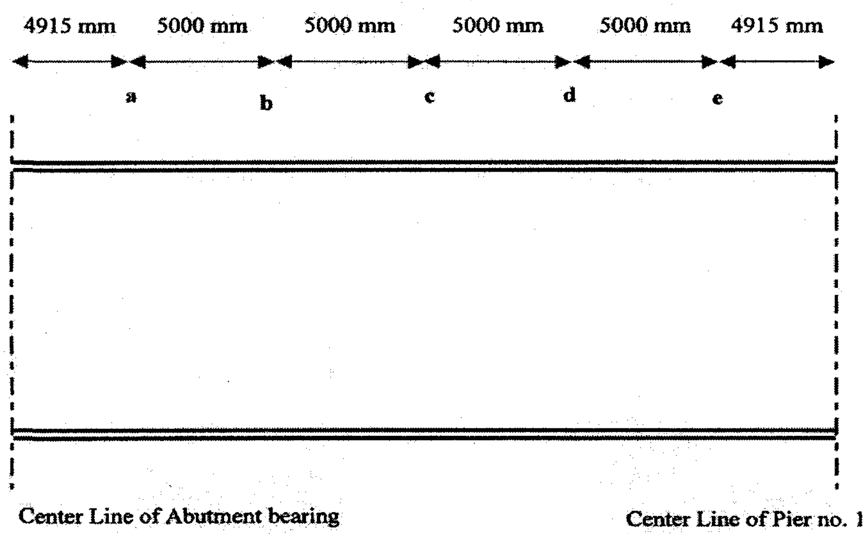


Figure 4- 5 Location of sections for Static load test, 1997 (ISIS Canada, 2004)

Figure 4- 6 illustrates the load sharing among girders based on the measured strain. As it's visible from the Figure 6 that exterior girders share major amount of load so these girders are considered critical in this study.

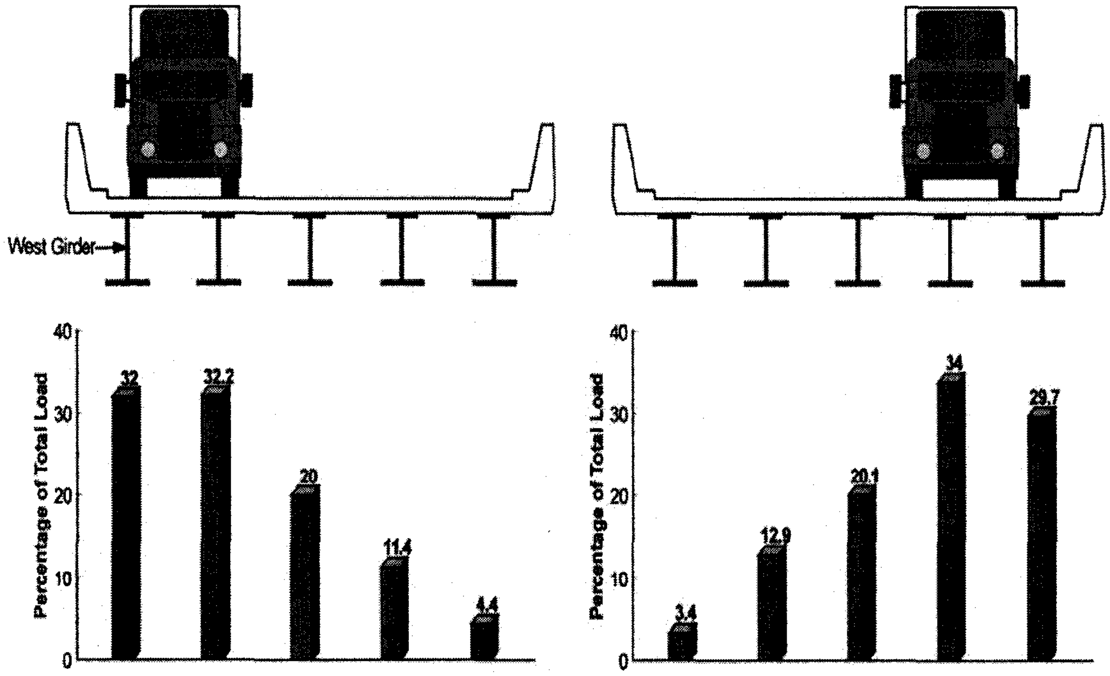


Figure 4- 6 Load sharing among girders based on dynamic strain measurements 1997 (Ventura et al, 2000).

4. 4 Simulation of the static load test on the FE Model

MFEM is Finite Element software which was developed at Carleton University (Bagchi et al.,). Using MFEM these conditions have been simulated and maximum deformation in each section has been measured for respective position. The Figure 4- 7 and Figure 4- 8 shows the finite element model of the Crowchild Bridge and simulated FE model for 6th position of static load test.

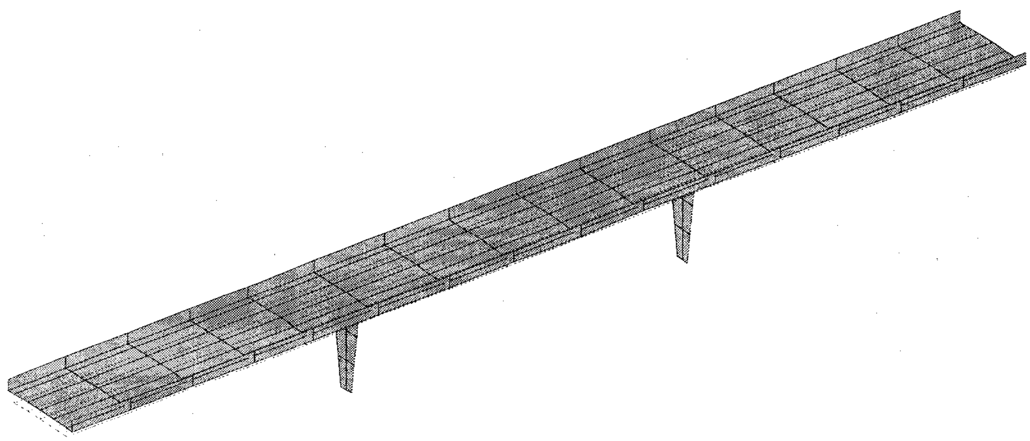


Figure 4- 7 Finite Element Model of Crowchild Bridge

Load of each truck was divided in 8 point loads according to axels positions. Table 4- 1 shows the amount of load and its coordinate for 6^{th} position of static load test. In this table fn and Fn, where n = 1, 2,……., 10, are same because both used trucks are of same type. North Span was surveyed at five equally spaced points on each girder as shown in Figure 4- 9.

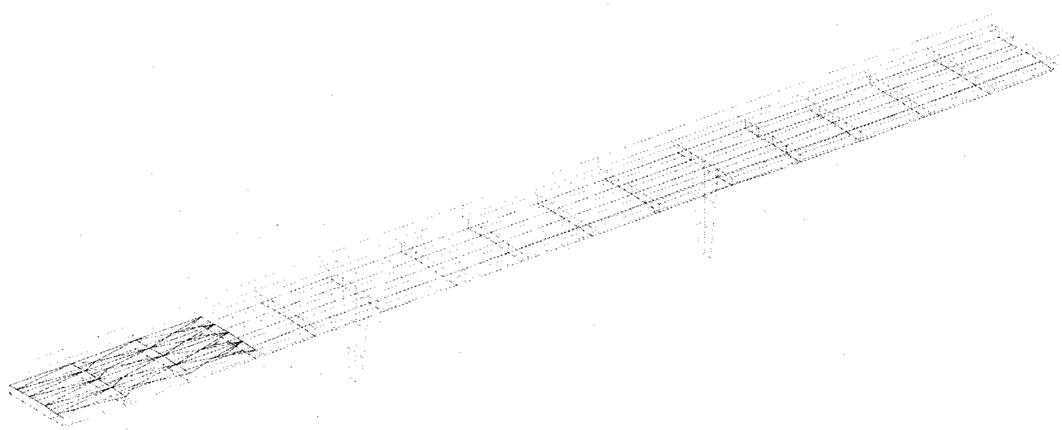


Figure 4- 8 FE model simulating 6^{th} position of static load test

Table 4- 1 Point loads and their coordinates for 6^{th} position

	Weight (in KN)	Cordinates (in mm)		
		x	y	z
f1	25	0	2.015	2.05
f2	25	0	3.845	2.096
f3	35.75	5.92	2.015	2.05
f4	35.75	5.92	3.845	2.096
f5	35.75	7.27	2.015	2.05
f6	35.75	7.27	3.845	2.096
f7	40.5	14.48	2.015	2.05
f8	40.5	14.48	3.845	2.096
f9	40.5	15.8	2.015	2.05
f10	40.5	15.8	3.845	2.096
F1	25	0	6.015	2.096
F2	25	0	7.845	2.05
F3	35.75	5.92	6.015	2.096
F4	35.75	5.92	7.845	2.05
F5	35.75	7.27	6.015	2.096
F6	35.75	7.27	7.845	2.05
F7	40.5	14.48	6.015	2.096
F8	40.5	14.48	7.845	2.05
F9	40.5	15.8	6.015	2.096
F10	40.5	15.8	7.845	2.05

Deformation results show that 6th position corresponds to maximum deformation in considered sections which is accepted because point loads are nearest to sections in the case of this position. Table 4- 2 shows a comparison between deformation values obtained from static load test and FEM, and it is found that both are in agreement.

Table 4- 2 Deflection in mm for static load test

Section	Girder	Deformation(in mm) from Load test	Deformation(in mm) from FEM
a	5	-5.2	-5.1
a	4	-5	-5.3
a	3	-5.5	-5.38
a	2	-5.7	-5.3
a	1	-5.5	-5.387
b	5	-10.7	-9.6
b	4	-11.5	-10.3
b	3	-11.7	-10.61
b	2	-12.7	-10.3
b	1	-9.5	-9.6
c	5	-11.5	-11.85
c	4	-12.7	-12.8
c	3	-11.7	-13.18
c	2	-10.7	-12.8
c	1	-11.7	-11.76
d	5	-10.5	-9.68
d	4	-9.5	-10.52
d	3	-10.5	-10.81
d	2	-9.5	-10.5
d	1	-7	-9.6
e	5	-6	-5.3
e	4	-5.5	-5.1
e	3	-6.2	-5.2
e	2	-5.5	-5.1
e	1	-3.7	-5.2

Figure 4- 9 shows the comparison among deformation in all sections for 1st position. The sections *a* and *e* have same deflection, and sections b and c have same deflections. It can be seen from Figure 4- 10 that the maximum deformation is in section c for all girders which is in agreement with the 1997 static load test results.

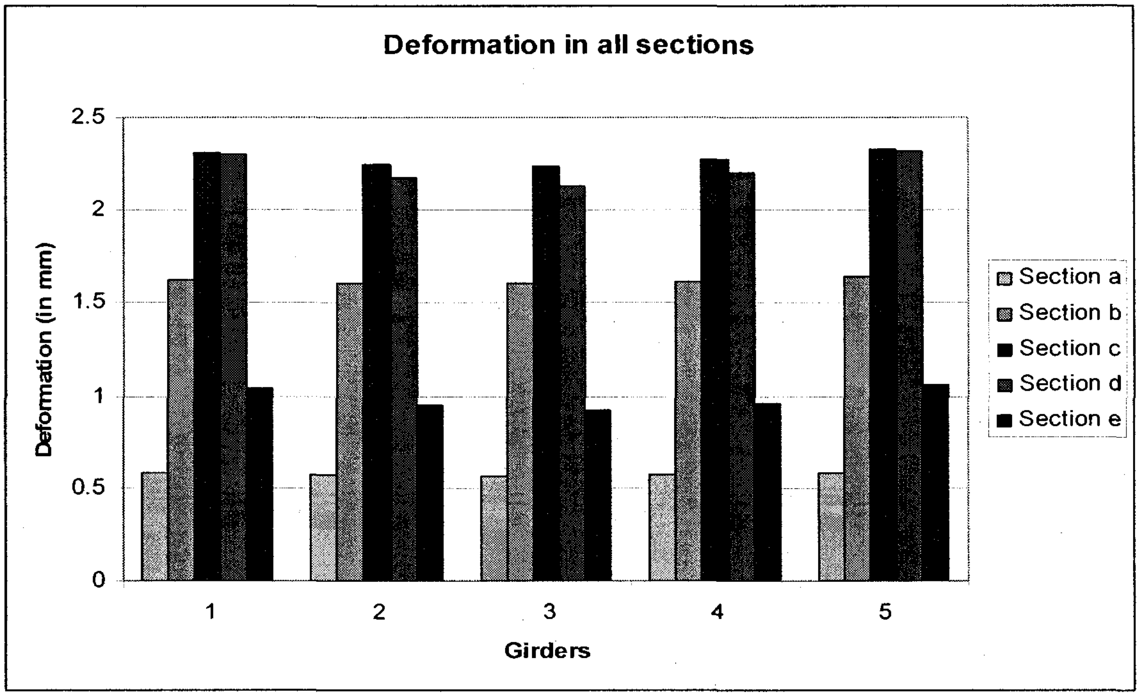


Figure 4- 9 Deformation for all sections for 1st position.

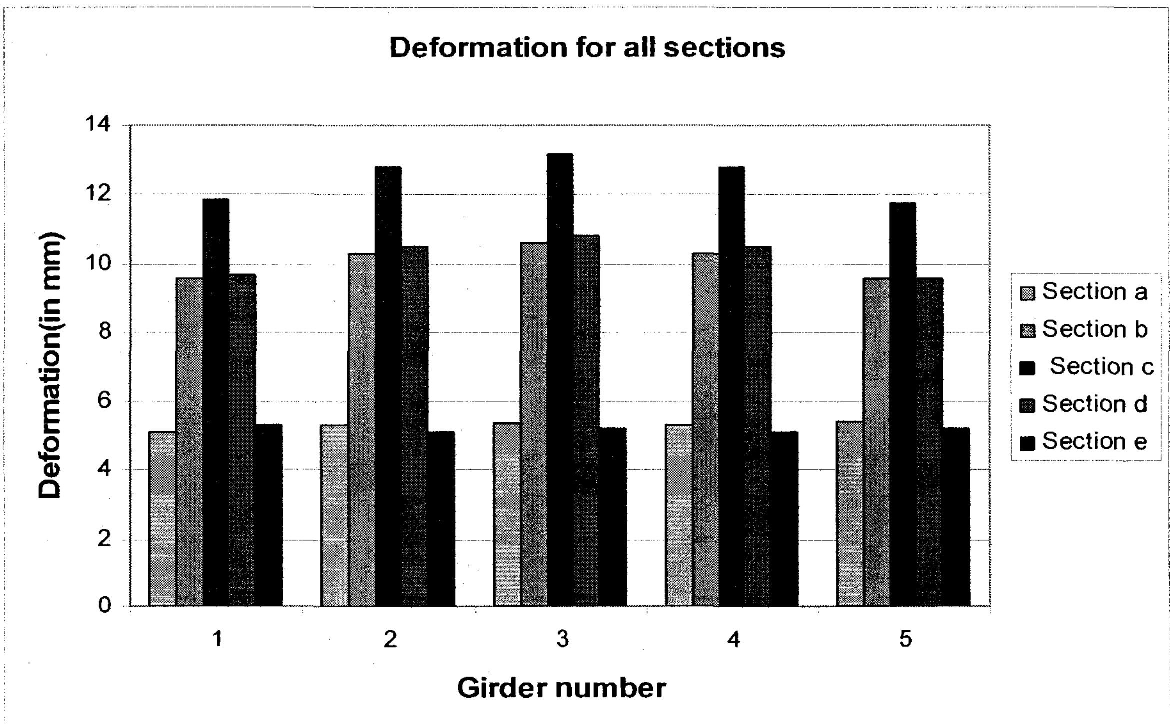


Figure 4- 10 Deformation for all section for 6th position.

The results show that maximum deformation in each section is for position 6 and the values are in agreement with static load test performed on this bridge in 1997. To develop degradation model this position has been selected as it corresponds to maximum deformation.

4. 5 Simulation of stiffness degradation

Degradation is a general function of stiffness reduction and in this study it is assumed that bridge degradation is proportional to reduction in stiffness.

Table 4- 3 Deformation for each reduced value of stiffness for 6th position

Section	Girder	Deformation (in mm)					
		$k=k_0$	$k=0.95k_0$	$k=0.90k_0$	$k=0.85k_0$	$k=0.8k_0$	$k=0.75k_0$
a	5	-5.1	-5.48	-5.60	-5.93	-6.46	-6.71
a	4	-5.3	-5.69	-5.82	-6.16	-6.71	-6.97
a	3	-5.38	-5.78	-5.91	-6.26	-6.81	-7.08
a	2	-5.3	-5.69	-5.82	-6.16	-6.71	-6.97
a	1	-5.387	-5.79	-5.92	-6.26	-6.82	-7.09
b	5	-9.6	-10.21	-10.55	-11.16	-12.15	-12.63
b	4	-10.3	-10.95	-11.51	-11.98	-13.04	-13.73
b	3	-10.61	-11.28	-11.85	-12.34	-13.43	-14.15
b	2	-10.3	-10.95	-11.51	-12.12	-13.04	-13.73
b	1	-9.6	-10.21	-10.73	-11.29	-12.15	-12.80
c	5	-11.85	-12.60	-13.24	-13.94	-15.00	-15.80
c	4	-12.8	-13.6	-14.30	-15.06	-16.20	-17.07
c	3	-13.18	-14.02	-14.73	-15.51	-16.68	-17.57
c	2	-12.8	-13.33	-14.30	-15.06	-16.00	-17.07
c	1	-11.76	-12.25	-13.14	-13.84	-14.70	-15.68
d	5	-9.68	-10.08	-10.82	-11.39	-12.10	-12.91
d	4	-10.52	-10.95	-11.69	-12.38	-13.15	-14.03
d	3	-10.81	-11.26	-12.01	-12.72	-13.51	-14.41
d	2	-10.5	-10.93	-11.67	-12.50	-13.13	-14.00
d	1	-9.6	-10	-10.67	-11.43	-12.00	-12.80
e	5	-5.3	-5.52	-5.89	-6.31	-6.63	-7.07
e	4	-5.1	-5.31	-5.67	-6.07	-6.38	-6.80
e	3	-5.2	-5.41	-5.78	-6.19	-6.50	-6.93
e	2	-5.1	-5.31	-5.67	-6.07	-6.38	-6.80
e	1	-5.2	-5.41	-5.78	-6.19	-6.50	-6.93

This concept has been used to develop the degradation model for Crowchild Bridge. To model the bridge deterioration in girders the stiffness value has been reduced by 1% up to 50%. The stiffness has been reduced by reduction in E values of used materials. Table 4- 3 shows the section deformation for each reduced value of stiffness.

In real situation, SHM provides the deformation in girders when certain amount of load passes trough the bridge but doesn't give any information, directly, about stiffness. To extract the information Artificial Neural Networks have been suggested in this study.

4. 6 Stiffness calculation using Artificial Neural Networks

In real situation, SHM provides data in terms of deformation, frequency, and vibration etc of the structure. This data certainly gives information about structure's condition, but extraction of that information is a challenging task. There is a need of a tool which can transfer this data in understandable information, and in this case ANN is found to be useful.

In this research, bridge load test has been simulated using finite element model of the bridge. This FEM provides deformation at certain nodes as discussed earlier. Table 4- 4 shows a data set of deformation values. The ANN has been trained and validated using this data set.

MATLAB is used in this study to compute NNs. The Multi Layer Perceptron (MLP) NN is used in this study. The MLP is supposed to have 2 layers feed forward networks. The weights and biases of the MLPs will be updates through error back-propagation algorithms. The NN has 25 input neurons in input layer and 1 output neurons in output layer as shown in Figure 4- 11. 50 data points have been created. The ANN will be trained with different sets of training data.

The type of NN used here is the so-called multi-layer perceptron (MLP). Previous researches have demonstrated that other NNs such as the radial basis function (RBF) fail on assessment of certain damage scenarios in this kind of structure (Bishop C M 1998). The MLP used are two-layer feed forward networks. The hidden units have the 'tanh' activation function, and the outputs units have the 'linear' one. The weights and biases of the MLPs were updated through the error back-propagation algorithm. The scaled conjugate gradient (SCG) method was used to minimize the error function during the MLPs training. The SCG is an efficient method of optimization that takes the minimum number of cycles to minimize the error function at the output of a MLP. The SCG can be regarded as a gradient descent method in which the learning rate and momentum are automatically optimized at each cycle of learning (Bishop C M 1998).

The ANN has 25 inputs neurons, and each input represents deformation of certain node on the bridge deck. The output layer only has one neuron which represents stiffness of the bridge deck. For training the neural network the randomly generated data, Table 4-3, set was divided in two subsets one training data set and other is validation data set.

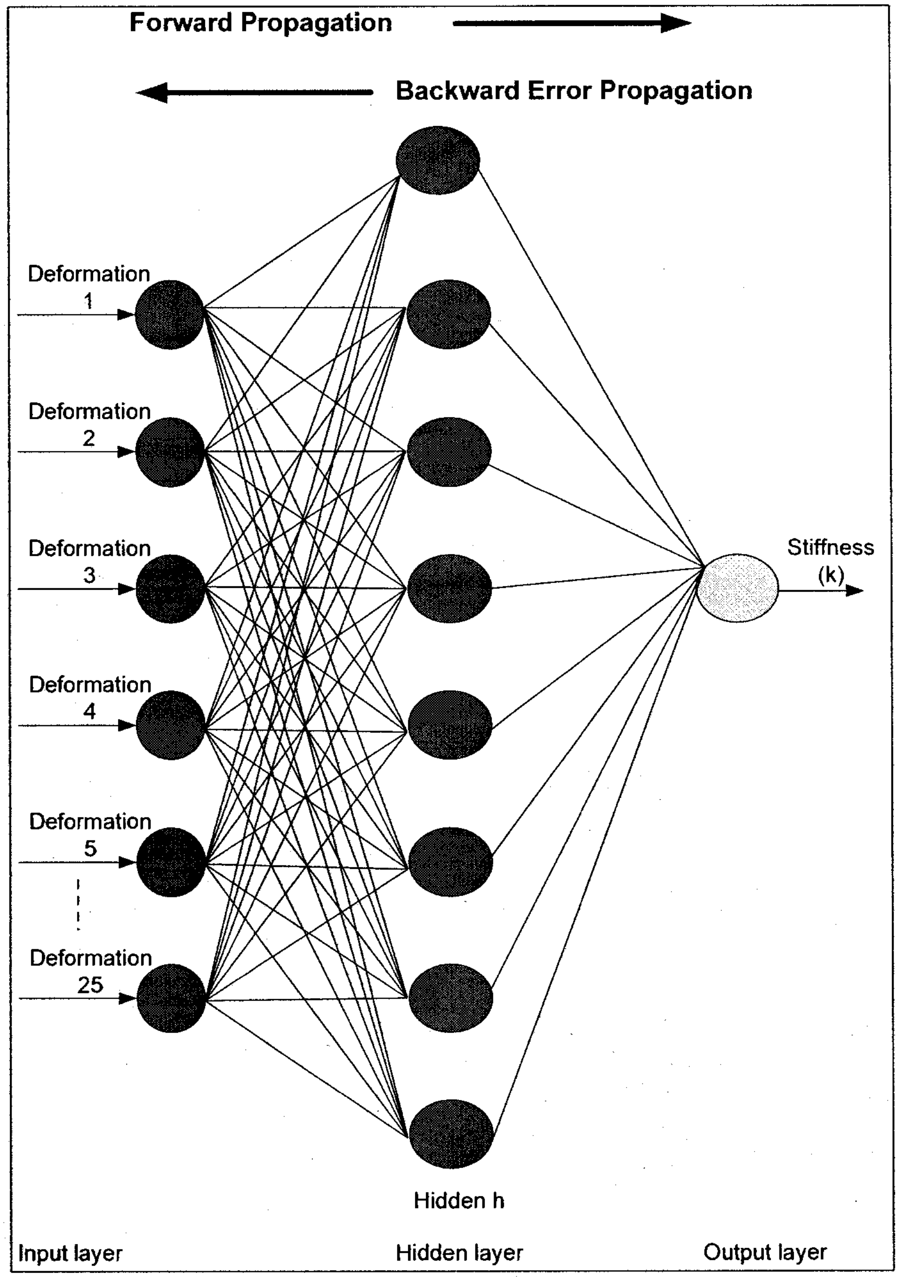


Figure 4- 11 Typical ANN architecture for stiffness calculation

The MATLAB is used to develop ANN. The advantage of using MATLAB is that it gives more control on the activation function, the rate of training and the training algorithm. The ANN learning rate is 0.05 and number of epochs has been chosen to 200. The root mean square error is 0.01%.

When stiffness of the bridge calculated using real test data it came out to be 95% of original stiffness. It shows that ANN results are in a good agreement with the real one.

Chapter 5

Reliability Analysis

5. 1 Introduction

In the chapter 4, the methodology to extract required information from SHM data has been discussed. The stiffness of the bridge deck has been computed using deformation values. This stiffness is an indication of structure's capacity against load moment. This chapter proposes the methodology to use this stiffness information to know the reliability of the structure (in this case bridge deck).

Estes (1997) developed limit state equation for slab, fails in moment, using random variables and moment equations. General form of a limit state equation for a slab would be as shown in equation 5-1

$$g(.) = M_{Capacity} - M_{Demand} = M_C - M_{dl} - M_U \qquad 5\text{- }1$$

where M_C is the ultimate moment capacity.

M_{dl} is the dead load moment capacity.

M_U is the live load moment

5. 2 Random variables

The first step in this process is to define the random variables and the nature of their distributions. In this study, dimensions that can be physically measured will be considered deterministic such as the spacing and length of girders and the dimensions of the steel girder cross sections. Dimensions which cannot be easily measured such as the spacing of reinforcement in concrete and dimensions which may vary throughout the structure such as concrete cover and asphalt thickness will be random.

Wherever possible, the random variables and their uncertainties will be taken from the literature. There have been an increasing number of reliability studies which quantify most of the random variables needed for these computations. While they may not apply perfectly to the Crowchild bridge, they are the most realistic values currently available without conducting a site specific investigation.

Table 5- 1 shows the random variables that will be used, their distribution, and the source from which they were taken. In many cases, these variables were described by a bias factor and coefficient of variation δ. The bias factor is a ratio between the mean value of the random distribution and the deterministic value of the variable. Table 5- 2 shows the terminology associated with each random variable.

5. 3 Ultimate Moment Capacity

The equation for ultimate moment capacity M_C is expressed as equation 5-2 (Estes, 1997)

$$M_C = \frac{A_t f_y d_{eff}}{12} - \frac{A_t^2 f_y^2}{244.8 f_c^{'}} \qquad 5\text{- }2$$

The random variables which account for the area of tension steel in a one-foot section of slab A_t and the effective depth of the slab d_{eff} are expressed as equation 5-3 and 5-4 respectively

$$A_t = (0.62in^2)\lambda_{rebar} \qquad 5\text{-}3$$

$$d_{eff} = (8.86)\lambda_{deff} \qquad 5\text{-}4$$

The effective depth for Crowchild Bridge deck (d_{eff}) is 8.86 inches. By substitution into equation 5.2, M_C can be expressed as shown in equation 5-5

$$M_C = \gamma_{mfc}[0.458\lambda_{rebar} f_y \lambda_{deff} - \frac{0.3844\lambda_{rebar}^2 f_y^2}{244.8 f_c^{'}}] \qquad 5\text{-}5$$

Table 5- 1 Random variables in reliability analysis of Crowchild bridge deck

Variable	Determinate	Random	Source	Bias	δ
λ_{rebar}	1.0	N[1.0, .015]	Nowak et. al. (1994)	1.0	0.015
f_y	50 ksi	N[56.0, 6.16]	Nowak (1995)	1.12	0.11
λ_{deff}	1.0	N[1.0, .02]	Lu et.al. (1994)	1.0	0.02
γ_{mfc}	1.0	N[1.02, .061]	Nowak-Yamani (1995)	1.02	0.06
λ^{*}_{trk}	1.0	N[2.37, .011]	Nowak (1993)	1.0	0.028
$f_c^{'}$	3 ksi	N[2.76, .497]	Nowak et. al. (1994)	0.92	0.92
λ_{asph}	1.0	N[1.0, 0.25]	Nowak (1993)	1.0	0.25
λ_{conc}	1.0	N[1.05, 0.105]	Nowak (1993)	1.05	0.10
* Variable is based on the 50 year load					

Table 5- 2 variables in reliability analysis of Crowchild bridge deck (Estes 1997)

Variable	Meaning
λ_{rebar}	Uncertainty factor: reinforcing steel area in concrete
f_y	Yield stress of steel reinforcing in concrete deck
λ_{deff}	Effective depth of reinforcing in concrete deck
γ_{mfc}	Model uncertainty factor: concrete flexure, deck
λ_{trk}	Uncertainty factor: HS-20 truck in analysis of deck
$f_c^{'}$	28 days yield strength of concrete
λ_{asph}	Uncertainty factor: weight of asphalt on deck
λ_{conc}	Uncertainty factor: weight of concrete on deck

5. 4 Dead Load Moment

The dead load moment on the slab includes the weight of the concrete w_{conc} and the weight of the asphalt w_{asph} which are normally distributed over the 6.56 feet (2 m) which separate any two interior girders. The unit weight of the concrete γ_{conc} and asphalt γ_{asph} are $150 lb/ft^3$ ($2403 kg/m^3$) and $144 lb/ft^3$ ($2307 kg/m^3$), respectively. The dead load moment M_{dl} is shown in equation 5-6

$$M_{dl} = \frac{ws^2C_f}{8} = \frac{w(6.656ft)^2(.8)}{8}\frac{kip}{1000} = 0.00404w \qquad 5\text{- }6$$
$$= 0.00404(w_{conc} + w_{asph}) = 0.00404(36\lambda_{asph} + 123.44\lambda_{conc})$$

$$= 0.145\lambda_{asph} + 0.4985\lambda_{conc}$$

5. 5 Live load

The live load moment M_U on the slab is based on a single wheel L_{trk} from the HS-20 truck placed in the center of the slab which produces a 16 kip point load between two girders. The live load moment (M_U) (AASHTO 92 (3.24.3.1), Estes 1997) includes both a continuity factor C_f and an impact factor I_f. Using these two factors, M_U can be further calculated as equation 5-7

$$M_U = \frac{L_{trk}(s+2)}{32} C_f I_f = \frac{16\lambda_{trk}(6.56+2)}{32}(0.8)(1.3) = 4.46\lambda_{trk} \qquad 5\text{- }7$$

5.5. 1 Nowak live load Model

A live load model which predicts the maximum truck moments and shears for different length spans was developed by Nowak (1993).The study covered 9,250 selected trucks from the Ontario Ministry of Transportation data base. The data base included number of axles, axle spacing, axle loads, and gross weight of the vehicles. The bending moments and shears were calculated for each truck in the survey for a wide range of spans. The cumulative distribution functions (CDF) of the span moments and shears were plotted on normal probability paper for spans ranging from 10 feet (3.05 m) to 200 feet (60.96 m). The maximum moments and shears for different time periods were extrapolated from these distributions. These CDFs were transformed to a standard normal distribution and the coefficients of variation for the maximum shears and moments were determined from the slope of the transformation. The end result was a series of graphs which provide a ratio of the mean shear and moment for the live load model to the shear and moment resulting from the standard HS-20 truck. This quantity is the bias factor needed for the random variable. The coefficients of variation for the maximum moment and shear are provided on other graphs. To read the graphs, one must know only the bridge span and the desired life of the bridge. The Nowak graphs were based on a measured two week traffic flow which equates to approximately 1,000 trucks per day. It is estimated that 1.5 million trucks will pass over the bridge in five years, 15 million trucks in 50 years, and 20 million trucks in 75 years (Estes, 1997). The Nowak graphs are based on the statistics of extreme values where the probability of encountering a large truck at the extreme tail of the distribution increases as the number of trucks passing over the bridge increases. As a result, the mean values of the maximum moment and shear increase over time and the coefficients of variation decrease. The Nowak graphs can be applied to a specific bridge where the daily traffic is known by reading the data for a single truck from the Nowak study and applying extreme value statistics to the actual traffic of the bridge under consideration.

For Crowchild Bridge which has a span of 30 m, the Nowak graphs (Nowak 1993) show that the ratio of the shear caused by one truck in the live load study to the shear caused by an HS-20 truck is 0.52 and the coefficient of variation is 0.29. Similarly, the ratio of the positive moment on a simple span for a single truck caused by the live load model to the moment caused by the HS-20 truck is 0.8 and the coefficient of variation is 0.42. As expected, the HS20 truck provides a conservative estimate of the single truck crossing the bridge. The AASHTO HS-20 truck does not account, however, for the increased probability that an extreme value truck will cross the bridge as the number of occurrences increases. Let the initial distribution of trucks crossing the bridge have a cumulative distribution function (CDF), $F_X(x)$, and probability density function (PDF), $f_X(x)$. The exact distribution of the maximum truck crossing the bridge CDF, F_{Mm}, and PDF, f_{Mn} is a function of the number of occurrences n (Ang & Tang 1984) as shown in equation 5-8 and 5-9

$$F_{M_n}(m) = [F_X(m)]^n \qquad 5\text{-}8$$

$$f_{M_n}(m) = n[F_X(m)]^{n-1} f_X(m) \qquad 5\text{-}9$$

Because the exact distribution is a function of another distribution and can contain many random variables, the computations can be very cumbersome. Fortunately, as the number of occurrences becomes larger, the extreme distribution approaches an asymtotic form which is not dependent on the original distribution. The normal and lognormal distributions approach a type I extreme value distribution with negligible differences as n is greater than 25. The type I extreme value distribution is only a function of the number of occurrences n, the mean value of the initial distribution μ, and the standard deviation of the original distribution σ (Ang & Tang 1984) as shown in equation 5-10 and 5-11

$$F_{M_n}(m) \approx e^{-e[\frac{\alpha_n}{\sigma}(m-\mu-\sigma\mu_n)]} \qquad 5\text{-}10$$

$$f_{M_n}(m) \approx (\frac{\sigma_n}{\alpha})e^{\left[\left(\frac{\alpha_n}{\sigma}\right)(m-\mu-\sigma\mu_n)\right]} e^{-e^{\left(\frac{-\alpha_n}{\sigma}(m-\mu-\sigma u_n)\right)}} \qquad 5\text{-}11$$

where

$$\alpha_n = \sqrt{2\ln(n)}$$

$$u_n = \alpha_n - \frac{\ln[\ln(n)] + \ln(4x)}{2\alpha_n}$$

To apply the live load model to the reliability analysis of the bridge, only the mean and standard deviation of the extreme distribution are needed. Using the central and dispersion characteristics of the type I extreme distribution, the mean μ and standard deviation σ can be computed as (Ayyub and White 1995)

$$\mu_{M_n} = \sigma\mu_n + \mu + (\gamma\sigma/\alpha_n) \qquad \text{5- 12}$$

$$\sigma_{M_n} = (\pi/\sqrt{6})(\sigma/\alpha_n) \qquad \text{5- 13}$$

where $\gamma = 0.577216$ (the Euler number).

The shear data from the Nowak graphs can be used to compute the equivalent truck 50 year live load to be used in the reliability analysis for the slab. The Nowak graphs show that ratio of the shear caused by one truck in the live load study to the shear caused by an HS-20 truck is 0.8 and the coefficient of variation is 0.365. The weight a wheel line on an HS-20 truck is 36kips (160 kN) which results in the mean μ and standard σ deviation for the single wheel line weight

$$\mu = 0.8(36kips) = 28.8kips$$

$$\sigma = 0.365(28.8) = 10.512kips$$

By substituting these values for n, μ, and σ into equations 5.12 and 5.13, the mean value of a wheel line for the 50 year truck is 85.35 kips (379.72 kN) with a standard deviation of 2.34 kips (10.53 kN). Since the weight of a wheel line on an HS-20 truck is 36 kips (160 kN), the uncertainty factor associated with the live load truck λ_{trk} used in equation 5.1 becomes

$$\lambda_{trk} = 85.35kips / 36.0kips = 2.37$$

$$\sigma_{\lambda_{trk}} = 2.34 kips / 45.65 kip(2.37) = 0.011$$

$$g(.) = \gamma_{mfc}[0.458\lambda_{rebar} f_y \lambda_{deff} - \frac{0.3844\lambda_{rebar}^2 f_y^2}{244.8 f_c^{'}}] - 0.145\lambda_{asph} - 0.4985\lambda_{conc} - 4.46\lambda_{trk} \quad 5\text{-}14$$

5. 6 Reliability index calculation

Using the limit state equation for the slab shown in the equation 5-14 and the values of random variables are shown in the Table 5- 1. A computer program in C++ has been developed to compute the probability of failure and reliability index using equation 5-15. This program uses Monte Carlo Simulation to calculate P_f for a given limit state equation, and then calculates the reliability index using equation 5-16. It is known that failure occurs when $g(.)<0$; therefore an estimate of the P_f can be found by

$$\overline{P_f} = N_f / N \quad 5\text{-}15$$

$$\beta = \Phi^{-1}(P_f) \text{ or } P_f = \Phi(-\beta) \quad 5\text{-}16$$

where N_f is the number of simulation cycles in which $g(.)<0$, and N is total number of simulation cycles. To check the accuracy of P_f the variance and covariance of estimated P_f have been calculated. The variance of the estimated P_f can be computed by assuming each simulation cycle to constitute a Bernoulli trial (Ayyub and McCuen, 1995).

The reliability index for bridge deck computed and found to be 5.1. The live load on the bridge is considered 50 years Nowak (1994) live load. In United States, the target reliability index for non-redundant bridges is 3.5. This target reliability index relates to the cumulative probability of failure of bridge remaining serviceable over 50-70 year lifetime without requiring any major rehabilitation (Stewart and Val, 1999). The accuracy of reliability index calculation depends on the accuracy of the input data and highly accurate input data can be obtained using SHM. In this study this value has been assumed sensitive enough to take rehabilitation decision.

Figure 5- 1 and Figure 5- 2 shows the probability of failure and reliability index over the period of time (age) respectively. It can be seen from Figure 5- 1 that the probability of failure drastically increases after that 45 years of age. Figure 5- 2 shows the effect of maintenance action over the reliability index. The maintenance actions improves the value of reliability index, hence it increases the life of bridges as shown in Figure 5-2.

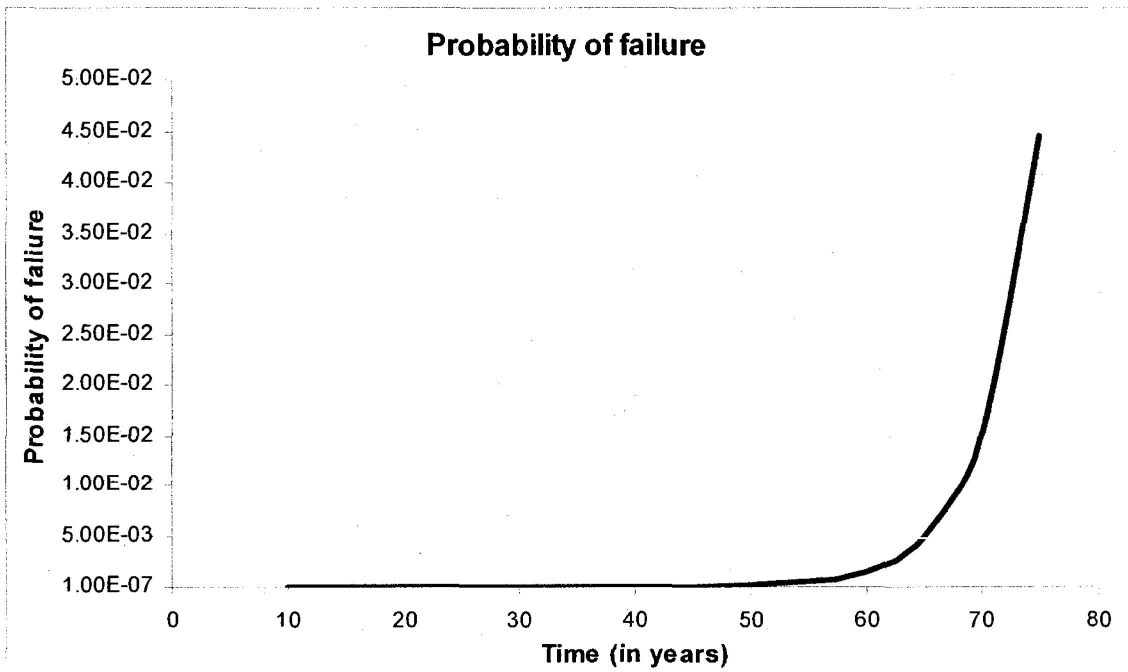


Figure 5- 1 Time Vs Probability of failure

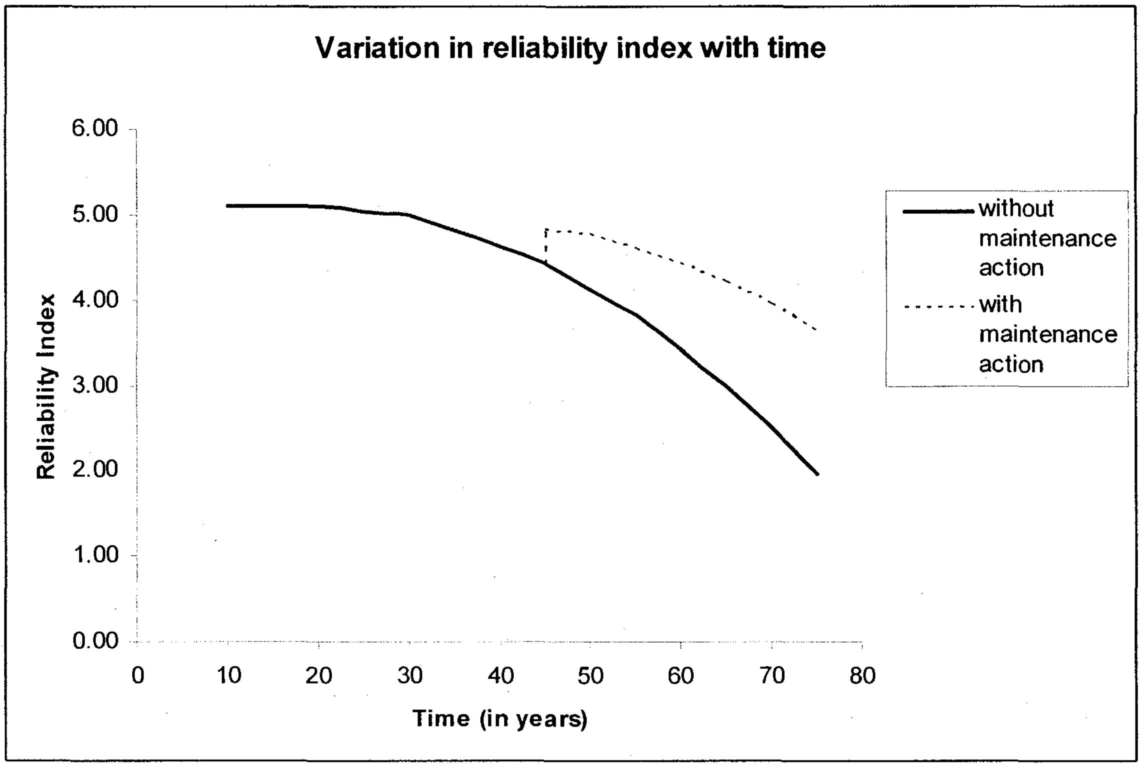


Figure 5- 2 Time Vs Reliability Index

Chapter 6

Decision Analysis

6. 1 Updating Probabilities

For all the sophisticated technology employed in bridge design and construction today, the maintenance and preservation of bridges still depends largely on regular visual inspection of the structures. So the best way to utilize SHM information would be incorporating this information with previously available information. This approach has the following advantages: (a) measurement errors are explicitly considered, (b) prior information based on engineering judgment or experience can be incorporated into the prediction of future deterioration, and (c) since inspection data merely alter (rather than replace) existing subjective data, the method provides a framework for incorporation of new inspection/monitoring data into the existing bridge management systems. Through the application of Bayesian methods, information from visual inspection data, SHM data and engineering judgment can be used to predict future behavior.

As mentioned in chapter 5, the reliability index for Crowchild bridge deck is 4.5. This value gives an idea to decision makers to predict the new rehabilitation scheme, and then the older scheme can be updated based on newer one.

6. 2 Bayesian Updating

Inspection results must often be supplemented with engineering or subjective judgment, particularly when the observed data are limited. Bayes theorem provides an error-free method for incorporating the prior information or judgment into prediction of future outcomes (Martz and Waller 1982). Bayesian methods are becoming increasingly popular for parameter updating (Miller and Freund 1977) and have also been applied to multiple events in the form of Bayesian networks (e-g., Normand and Trichler, 1992).

The uncertainty associated with some of the methods commonly used for acquiring bridge inspection data can be significant, particularly when the number of samples is relatively small. On the other hand, deterioration predictions based solely on data from historical records of similar bridges can be misleading, since the extent of damage to a bridge is often site-specific. One approach to the prediction of deterioration of RC bridges is to develop a baseline deterioration rate which can be updated as inspection data become available (Enright 1999). Suppose that, historically, the rate of strength deterioration of a particular class of bridges can be described by a random variable $\underline{\theta}$. If no inspection data are available, then bridge reliability estimates could be obtained at any time t, based on degradation rate $\underline{\theta}$. Suppose that an inspection is performed on the bridge, and the degradation rate from inspection measurements is described by a random variable *X*. A conditional probability density function for the new degradation rate can be identified based on the previously assumed degradation rate and on the inspection data. $g(\underline{\theta}/\underline{x})$. This pdf represents the predicted degradation rate based on one set of inspection evidence, and can be updated each time when new inspection data become available. An expression for the updated distribution, $g(\underline{\theta}/\underline{x})$, can be defined using Bayes Theorem as follows (Martz and Waller 1982) as equation 6.1

$$g(\underline{\theta}/\underline{x}) = \frac{f(\underline{x}/\underline{\theta}).g(\underline{\theta})}{\int f(\underline{x}/\underline{\theta})g(\underline{\theta})d\theta} \qquad 6.1$$

where $f(\underline{x}/\underline{\theta})$ = conditional pdf of $\underline{X}$ given $\underline{\theta}$ (sampling distribution), $g(\underline{\theta})$ = pdf of $\underline{\theta}$ (prior distribution), and $g(\underline{\theta}/\underline{x})$ = posterior pdf of $\underline{\theta}$ given $\underline{x}$ (posterior distribution), $\underline{\theta}$ = continuous parameter vector, and $\underline{x}$ = sample data.

Equation 6.1 can be applied to predict a posterior distribution of degradation rate based on previous data (prior distribution) and current data (sampling distribution). The procedure for computing the main descriptors (mean, coefficient of variation) and pdf of the posterior distribution is as follows (Enright 1999):

1. Evaluate the denominator of Eqn. 7.1, $\int f(\underline{x}/\underline{\theta})g(\underline{\theta})d\theta$, by numerical integration.
2. Compute the mean value of the posterior pdf, $E[g(\underline{\theta}/\underline{x})]$, by numerical integration.
3. Compute the coefficient of variation of the posterior pdf, $V[g(\underline{\theta}/\underline{x})]$ by numerical integration.
4. Plot $g(\underline{\theta}/\underline{x})$ versus Θ over the interval $E[g(\underline{\theta}/\underline{x})] \pm 5\sigma[g(\underline{\theta}/\underline{x})]$, where Θ means standard deviation.

To illustrate the mentioned approach an example is given. Let's assume that the probability of performing first rehabilitation at nth year is $P(\theta_n)$, where:

$$\sum_{i=0}^{n} P(\theta_i) = 1 \qquad 6.2$$

Now assuming that SHM test gives m output each for each true state of nature as shown in equation 6.3

$$\sum_{j=0}^{m} P(Z_j/\theta_i) = 1 \tag{6.3}$$

where $P(Z_j/\theta_i)$ is conditional probability of SHM test gives output of performing rehabilitation at j^{th} year when the rehabilitation is required at i^{th} year.

Using Bayesian Approach updated probability for first rehabilitation at ith year when test shows output Z_j is given by equation 6.4

$$P'(\theta_i) = P(\theta_i/Z_j) = \frac{P(Z_j/\theta_i)P(\theta_i)}{\sum_{i=0}^{n} P(Z_j/\theta_i)P(\theta_i)} \tag{6.4}$$

In general, the time of maintenance application and test output is random and their probability distribution can be described by a continuous random variable with a specified probability density functions (PDFs). Kong and Frangopol (2003) proposed an approach by replacing these PDFs by probability mass functions (PMFs) to calculate the superposed probability of rehabilitation (SPR) at a given time. SPR is defined as summation of all probabilities associated with a point in given time gives the superposed probability of (any) rehabilitation at that time. For instance, if $P_{R_2}(t_L^*) = 0.2$ and $P_{R_m}(t_L^*) = 0.1$, and all other probabilities are zero at t_L^*, then the SPR at t_L^* is 0.3. Considering all discrete intervention cycles, the SPR at a given point of time $t = t_L^*$ is as shown in equation 6.5

$$\sum_{all(i)} P_{T_1,T_2}[R_i(t = t_L^*)] = \sum_{all(i)} P[R_i(t_L^*)] \tag{6.5}$$

To evaluate how much the SPR change from time zero to t_L^*, the cumulative SPR can be evaluated as shown in equation 6.6

$$\sum_{all(i)} P_{T_1,T_2}[R_i(t \le t_L^*)] = \sum_{all\left(t \le t_L^*\right)} [\sum_{all} \lfloor P[R_i(t_L^*)] \rfloor \qquad 6.6$$

Kong and Frangopol (2003) recommend decision event tree approach to evaluate all possible rehabilitation scenarios as it reduces time significantly which is essential for solving practical problem associated with large stocks of deteriorating structures.

Figure1 (a) shows the probability distribution for first rehabilitation prior to SHM information.

Table 6-1 Reliability of SHM data

True State / Test outcome	Rehabilitation at 0^{th} year (from construction) θ_0 $(i=0)$	Rehabilitation at 2^{nd} year (from construction) θ_2 $(i=2)$	Rehabilitation at 3^{rd} year (from construction) θ_4 $(i=4)$
$P(Z_0/\theta_i)$	0.85	0.03	0
$P(Z_1/\theta_i)$	0.10	0.85	0.05
$P(Z_2/\theta_i)$	0.05	0.10	0.1
$P(Z_4/\theta_i)$	0	0.02	0.85

The reliability of the experimental results is as follows: if the bridge deck needs rehabilitation at 0^{th} year, the probability that the SHM data will indicate rehabilitation at 0^{th} year is 0.85, and corresponding probabilities for 2^{nd}, 3^{rd}, and 4^{th} year are 0.10, 0.05 and 0. On other hand, if the bridge deck needs rehabilitation at 2^{nd} year, the probabilities that the SHM data will show at 0^{th}, 1^{st}, $2n^{d}$ and 4^{th} year are 0.03, 0.85, 0.10 and 0.02. The probabilities are mentioned in the same manner for rehabilitation at 3rd year in the Table 6-1.

The distributions of updated probabilities for first rehabilitation is shown in Figure 6-1 (b), (c), (d) and (e) corresponding to test outcomes Z_0, Z_1, Z_2 and Z_4 respectively. It's evident from Figure 6-2 shows the probability distribution for 2nd and 3rd rehabilitations. Figure 6-1 that updating probabilities reduce the uncertainty associated with rehabilitation decision up to a great extent. This will, obviously, reflect in expected life cycle maintenance cost for bridge's deck.

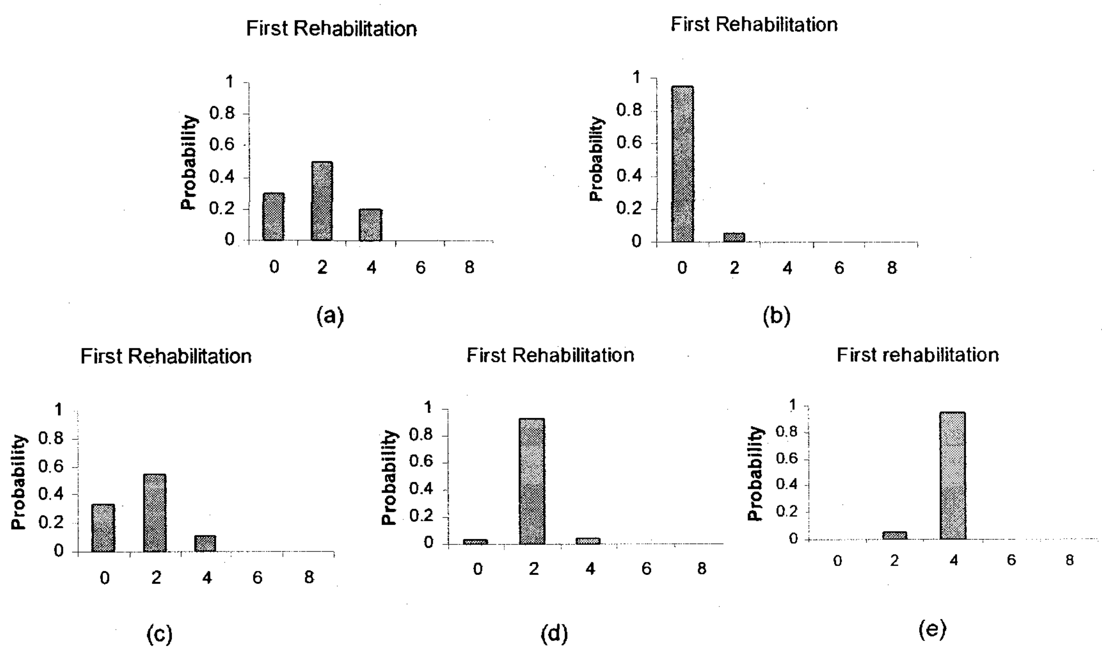
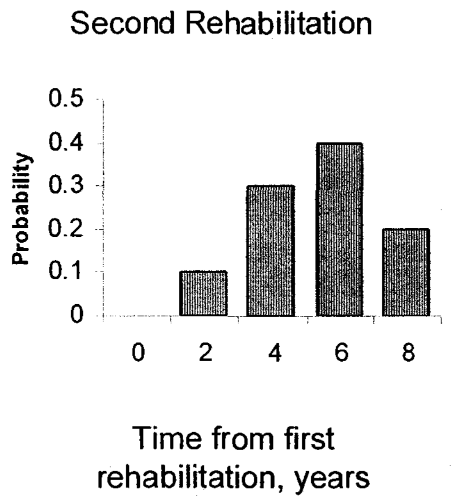


Figure 6-1 Distribution of first rehabilitation : (a) Prior to test; and Updated probabilities corresponding to (b) Test output Z_0; c) Test output Z_1; (d) Test output Z_2; (e) Test output Z_4.

The Table 6-2 shows the probability of joint rehabilitation prior to updating, while Table 6-3 , Table 6-5, Table 6- 6 and Table 6- 6 show the probability of joint rehabilitation after updating when test output is Z_0, Z_1, Z_2,and Z_4 respectively.



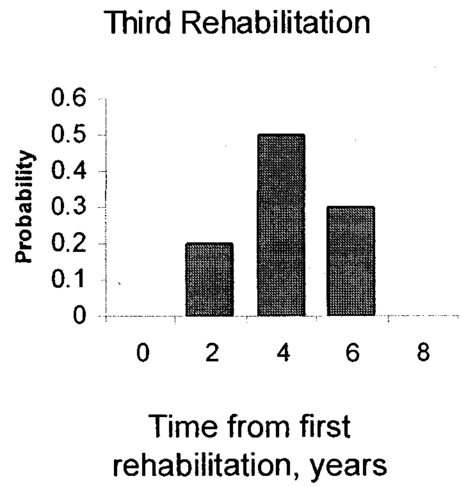


Figure 6-2 Distribution of first rehabilitation times for second and third subsequent cycles

It is evident from tables given below that updated probabilities give more precise information about the time of maintenance (or rehabilitation) actions. If test output is Z_0, the updated probabilities come out to be in favor of maintenance/rehabilitation actions at 0^{th} year.

This is the same case with all other test outputs.

Kong and Frangopol (2003) suggest a method of evaluation of annual rehabilitation cost with different maintenance cycles. Using that approach the expected cost of rehabilitation has been calculated for three different rehabilitation cycles. Figure 6- 3 shows the annual rehabilitation cost over time before updating the probabilities. Figure 6- 4, Figure 6- 5, Figure 6- 6, and Figure 6- 7 show the annual rehabilitation cost after updating the rehabilitation actions probabilities when SHM test outputs are Z_0, Z_1,

Z_2, and Z_4 respectively. It is evident from figures that probability updating has huge effect in evaluation of expected life cycle cost.

Table 6-2 Cumulative probabilities for rehabilitation scheme prior to test

First rehabilitation time		Second rehabilitation		Third rehabilitation		Second rehabilitation time (years, absolute time)	Third rehabilitation time (years, absolute time)	Probability of joint rehabilitation for two interventions ($P_1 x P_2$)	Probability of joint rehabilitation ($P_1 x P_2 x P_3$)
Time (years, absolute time)	Probability (P_1)	Time (years, relative time)	Probability (P_2)	Time (years, relative time)	Probability (P_3)				
0	0.3	2	0.1	2	0.2	2	4	0.03	0.006
0	0.3	2	0.1	4	0.5	2	6	0.03	0.015
0	0.3	2	0.1	6	0.3	2	8	0.03	0.009
0	0.3	4	0.3	2	0.2	4	6	0.09	0.018
0	0.3	4	0.3	4	0.5	4	8	0.09	0.045
0	0.3	4	0.3	6	0.3	4	10	0.09	0.027
0	0.3	6	0.4	2	0.2	6	8	0.12	0.024
0	0.3	6	0.4	4	0.5	6	10	0.12	0.06
0	0.3	6	0.4	6	0.3	6	12	0.12	0.036
0	0.3	8	0.2	2	0.2	8	10	0.06	0.012
0	0.3	8	0.2	4	0.5	8	12	0.06	0.03
0	0.3	8	0.2	6	0.3	8	14	0.06	0.018
2	0.5	2	0.1	2	0.2	4	6	0.05	0.01
2	0.5	2	0.1	4	0.5	4	8	0.05	0.025
2	0.5	2	0.1	6	0.3	4	10	0.05	0.015
2	0.5	4	0.3	2	0.2	6	8	0.15	0.03
2	0.5	4	0.3	4	0.5	6	10	0.15	0.075
2	0.5	4	0.3	6	0.3	6	12	0.15	0.045
2	0.5	6	0.4	2	0.2	8	10	0.2	0.04
2	0.5	6	0.4	4	0.5	8	12	0.2	0.1
2	0.5	6	0.4	6	0.3	8	14	0.2	0.06
2	0.5	8	0.2	2	0.2	10	12	0.1	0.02
2	0.5	8	0.2	4	0.5	10	14	0.1	0.05
2	0.5	8	0.2	6	0.3	10	16	0.1	0.03
4	0.2	2	0.1	2	0.2	6	8	0.02	0.004
4	0.2	2	0.1	4	0.5	6	10	0.02	0.01
4	0.2	2	0.1	6	0.3	6	12	0.02	0.006
4	0.2	4	0.3	2	0.2	8	10	0.06	0.012
4	0.2	4	0.3	4	0.5	8	12	0.06	0.03
4	0.2	4	0.3	6	0.3	8	14	0.06	0.018
4	0.2	6	0.4	2	0.2	10	12	0.08	0.016
4	0.2	6	0.4	4	0.5	10	14	0.08	0.04
4	0.2	6	0.4	6	0.3	10	16	0.08	0.024
4	0.2	8	0.2	2	0.2	12	14	0.04	0.008
4	0.2	8	0.2	4	0.5	12	16	0.04	0.02
4	0.2	8	0.2	6	0.3	12	18	0.04	0.012

Table 6-3 Cumulative probabilities for rehabilitation scheme when test output Z0

First rehabilitation		Second rehabilitation		Third rehabilitation		Second rehabilitation time (years, absolute time)	Third rehabilitation time (years, absolute time)	Probability of joint rehabilitation for two interventions (P1xP2)	Probability of joint rehabilitation (P1xP2xP3)
Time (years, absolute time)	Probability (P1)	Time (years, relative time)	Probability (P2)	Time (years, relative time)	Probability (P3)				
0	0.945	2	0.1	2	0.2	2	4	0.0945	0.0189
0	0.945	2	0.1	4	0.5	2	6	0.0945	0.04725
0	0.945	2	0.1	6	0.3	2	8	0.0945	0.02835
0	0.945	4	0.3	2	0.2	4	6	0.2835	0.0567
0	0.945	4	0.3	4	0.5	4	8	0.2835	0.14175
0	0.945	4	0.3	6	0.3	4	10	0.2835	0.08505
0	0.945	6	0.4	2	0.2	6	8	0.378	0.0756
0	0.945	6	0.4	4	0.5	6	10	0.378	0.189
0	0.945	6	0.4	6	0.3	6	12	0.378	0.1134
0	0.945	8	0.2	2	0.2	8	10	0.189	0.0378
0	0.945	8	0.2	4	0.5	8	12	0.189	0.0945
0	0.945	8	0.2	6	0.3	8	14	0.189	0.0567
2	0.055	2	0.1	2	0.2	4	6	0.0055	0.0011
2	0.055	2	0.1	4	0.5	4	8	0.0055	0.00275
2	0.055	2	0.1	6	0.3	4	10	0.0055	0.00165
2	0.055	4	0.3	2	0.2	6	8	0.0165	0.0033
2	0.055	4	0.3	4	0.5	6	10	0.0165	0.00825
2	0.055	4	0.3	6	0.3	6	12	0.0165	0.00495
2	0.055	6	0.4	2	0.2	8	10	0.022	0.0044
2	0.055	6	0.4	4	0.5	8	12	0.022	0.011
2	0.055	6	0.4	6	0.3	8	14	0.022	0.0066
2	0.055	8	0.2	2	0.2	10	12	0.011	0.0022
2	0.055	8	0.2	4	0.5	10	14	0.011	0.0055
2	0.055	8	0.2	6	0.3	10	16	0.011	0.0033
4	0	2	0.1	2	0.2	6	8	0	0
4	0	2	0.1	4	0.5	6	10	0	0
4	0	2	0.1	6	0.3	6	12	0	0
4	0	4	0.3	2	0.2	8	10	0	0
4	0	4	0.3	4	0.5	8	12	0	0
4	0	4	0.3	6	0.3	8	14	0	0
4	0	6	0.4	2	0.2	10	12	0	0
4	0	6	0.4	4	0.5	10	14	0	0
4	0	6	0.4	6	0.3	10	16	0	0
4	0	8	0.2	2	0.2	12	14	0	0
4	0	8	0.2	4	0.5	12	16	0	0
4	0	8	0.2	6	0.3	12	18	0	0

Table 6- 4 Cumulative probabilities for rehabilitation scheme when test output Z1

First rehabilitation		Second rehabilitation		Third rehabilitation		Second rehabilitation time (years, absolute time)	Third rehabilitation time (years, absolute time)	Probability of joint rehabilitation for two interventions (P1xP2)	Probability of joint rehabilitation (P1xP2xP3)
Time (years, absolute time)	Probability (P1)	Time (years, relative time)	Probability (P2)	Time (years, relative time)	Probability (P3)				
0	0.33	2	0.1	2	0.2	2	4	0.033	0.0066
0	0.33	2	0.1	4	0.5	2	6	0.033	0.0165
0	0.33	2	0.1	6	0.3	2	8	0.033	0.0099
0	0.33	4	0.3	2	0.2	4	6	0.099	0.0198
0	0.33	4	0.3	4	0.5	4	8	0.099	0.0495
0	0.33	4	0.3	6	0.3	4	10	0.099	0.0297
0	0.33	6	0.4	2	0.2	6	8	0.132	0.0264
0	0.33	6	0.4	4	0.5	6	10	0.132	0.066
0	0.33	6	0.4	6	0.3	6	12	0.132	0.0396
0	0.33	8	0.2	2	0.2	8	10	0.066	0.0132
0	0.33	8	0.2	4	0.5	8	12	0.066	0.033
0	0.33	8	0.2	6	0.3	8	14	0.066	0.0198
2	0.56	2	0.1	2	0.2	4	6	0.056	0.0112
2	0.56	2	0.1	4	0.5	4	8	0.056	0.028
2	0.56	2	0.1	6	0.3	4	10	0.056	0.0168
2	0.56	4	0.3	2	0.2	6	8	0.168	0.0336
2	0.56	4	0.3	4	0.5	6	10	0.168	0.084
2	0.56	4	0.3	6	0.3	6	12	0.168	0.0504
2	0.56	6	0.4	2	0.2	8	10	0.224	0.0448
2	0.56	6	0.4	4	0.5	8	12	0.224	0.112
2	0.56	6	0.4	6	0.3	8	14	0.224	0.0672
2	0.56	8	0.2	2	0.2	10	12	0.112	0.0224
2	0.56	8	0.2	4	0.5	10	14	0.112	0.056
2	0.56	8	0.2	6	0.3	10	16	0.112	0.0336
4	0.11	2	0.1	2	0.2	6	8	0.011	0.0022
4	0.11	2	0.1	4	0.5	6	10	0.011	0.0055
4	0.11	2	0.1	6	0.3	6	12	0.011	0.0033
4	0.11	4	0.3	2	0.2	8	10	0.033	0.0066
4	0.11	4	0.3	4	0.5	8	12	0.033	0.0165
4	0.11	4	0.3	6	0.3	8	14	0.033	0.0099
4	0.11	6	0.4	2	0.2	10	12	0.044	0.0088
4	0.11	6	0.4	4	0.5	10	14	0.044	0.022
4	0.11	6	0.4	6	0.3	10	16	0.044	0.0132
4	0.11	8	0.2	2	0.2	12	14	0.022	0.0044
4	0.11	8	0.2	4	0.5	12	16	0.022	0.011
4	0.11	8	0.2	6	0.3	12	18	0.022	0.0066

Table 6-5 Cumulative probabilities for rehabilitation scheme when test output Z1

First rehabilitation		Second rehabilitation		Third rehabilitation		Second rehabilitation time (years, absolute time)	Third rehabilitation time (years, absolute time)	Probability of joint rehabilitation for two interventions (P1xP2)	Probability of joint rehabilitation (P1xP2xP3)
Time (years, absolute time)	Probability (P1)	Time (years, relative time)	Probability (P2)	Time (years, relative time)	Probability (P3)				
0	0.0326	2	0.1	2	0.2	2	4	0.00326	0.000652
0	0.0326	2	0.1	4	0.5	2	6	0.00326	0.00163
0	0.0326	2	0.1	6	0.3	2	8	0.00326	0.000978
0	0.0326	4	0.3	2	0.2	4	6	0.00978	0.001956
0	0.0326	4	0.3	4	0.5	4	8	0.00978	0.00489
0	0.0326	4	0.3	6	0.3	4	10	0.00978	0.002934
0	0.0326	6	0.4	2	0.2	6	8	0.01304	0.002608
0	0.0326	6	0.4	4	0.5	6	10	0.01304	0.00652
0	0.0326	6	0.4	6	0.3	6	12	0.01304	0.003912
0	0.0326	8	0.2	2	0.2	8	10	0.00652	0.001304
0	0.0326	8	0.2	4	0.5	8	12	0.00652	0.00326
0	0.0326	8	0.2	6	0.3	8	14	0.00652	0.001956
2	0.924	2	0.1	2	0.2	4	6	0.0924	0.01848
2	0.924	2	0.1	4	0.5	4	8	0.0924	0.0462
2	0.924	2	0.1	6	0.3	4	10	0.0924	0.02772
2	0.924	4	0.3	2	0.2	6	8	0.2772	0.05544
2	0.924	4	0.3	4	0.5	6	10	0.2772	0.1386
2	0.924	4	0.3	6	0.3	6	12	0.2772	0.08316
2	0.924	6	0.4	2	0.2	8	10	0.3696	0.07392
2	0.924	6	0.4	4	0.5	8	12	0.3696	0.1848
2	0.924	6	0.4	6	0.3	8	14	0.3696	0.11088
2	0.924	8	0.2	2	0.2	10	12	0.1848	0.03696
2	0.924	8	0.2	4	0.5	10	14	0.1848	0.0924
2	0.924	8	0.2	6	0.3	10	16	0.1848	0.05544
4	0.0434	2	0.1	2	0.2	6	8	0.00434	0.000868
4	0.0434	2	0.1	4	0.5	6	10	0.00434	0.00217
4	0.0434	2	0.1	6	0.3	6	12	0.00434	0.001302
4	0.0434	4	0.3	2	0.2	8	10	0.01302	0.002604
4	0.0434	4	0.3	4	0.5	8	12	0.01302	0.00651
4	0.0434	4	0.3	6	0.3	8	14	0.01302	0.003906
4	0.0434	6	0.4	2	0.2	10	12	0.01736	0.003472
4	0.0434	6	0.4	4	0.5	10	14	0.01736	0.00868
4	0.0434	6	0.4	6	0.3	10	16	0.01736	0.005208
4	0.0434	8	0.2	2	0.2	12	14	0.00868	0.001736
4	0.0434	8	0.2	4	0.5	12	16	0.00868	0.00434
4	0.0434	8	0.2	6	0.3	12	18	0.00868	0.002604

Table 6- 6 Cumulative probabilities for rehabilitation scheme when test output Z2

First rehabilitation		Second rehabilitation		Third rehabilitation		Second rehabilitation time (years, absolute time)	Third rehabilitation time (years, absolute time)	Probability of joint rehabilitation for two interventions (P1xP2)	Probability of joint rehabilitation (P1xP2xP3)
Time (years, absolute time)	Probability (P1)	Time (years, relative time)	Probability (P2)	Time (years, relative time)	Probability (P3)				
0	0	2	0.1	2	0.2	2	4	0	0
0	0	2	0.1	4	0.5	2	6	0	0
0	0	2	0.1	6	0.3	2	8	0	0
0	0	4	0.3	2	0.2	4	6	0	0
0	0	4	0.3	4	0.5	4	8	0	0
0	0	4	0.3	6	0.3	4	10	0	0
0	0	6	0.4	2	0.2	6	8	0	0
0	0	6	0.4	4	0.5	6	10	0	0
0	0	6	0.4	6	0.3	6	12	0	0
0	0	8	0.2	2	0.2	8	10	0	0
0	0	8	0.2	4	0.5	8	12	0	0
0	0	8	0.2	6	0.3	8	14	0	0
2	0.055	2	0.1	2	0.2	4	6	0.0055	0.0011
2	0.055	2	0.1	4	0.5	4	8	0.0055	0.00275
2	0.055	2	0.1	6	0.3	4	10	0.0055	0.00165
2	0.055	4	0.3	2	0.2	6	8	0.0165	0.0033
2	0.055	4	0.3	4	0.5	6	10	0.0165	0.00825
2	0.055	4	0.3	6	0.3	6	12	0.0165	0.00495
2	0.055	6	0.4	2	0.2	8	10	0.022	0.0044
2	0.055	6	0.4	4	0.5	8	12	0.022	0.011
2	0.055	6	0.4	6	0.3	8	14	0.022	0.0066
2	0.055	8	0.2	2	0.2	10	12	0.011	0.0022
2	0.055	8	0.2	4	0.5	10	14	0.011	0.0055
2	0.055	8	0.2	6	0.3	10	16	0.011	0.0033
4	0.945	2	0.1	2	0.2	6	8	0.0945	0.0189
4	0.945	2	0.1	4	0.5	6	10	0.0945	0.04725
4	0.945	2	0.1	6	0.3	6	12	0.0945	0.02835
4	0.945	4	0.3	2	0.2	8	10	0.2835	0.0567
4	0.945	4	0.3	4	0.5	8	12	0.2835	0.14175
4	0.945	4	0.3	6	0.3	8	14	0.2835	0.08505
4	0.945	6	0.4	2	0.2	10	12	0.378	0.0756
4	0.945	6	0.4	4	0.5	10	14	0.378	0.189
4	0.945	6	0.4	6	0.3	10	16	0.378	0.1134
4	0.945	8	0.2	2	0.2	12	14	0.189	0.0378
4	0.945	8	0.2	4	0.5	12	16	0.189	0.0945
4	0.945	8	0.2	6	0.3	12	18	0.189	0.0567

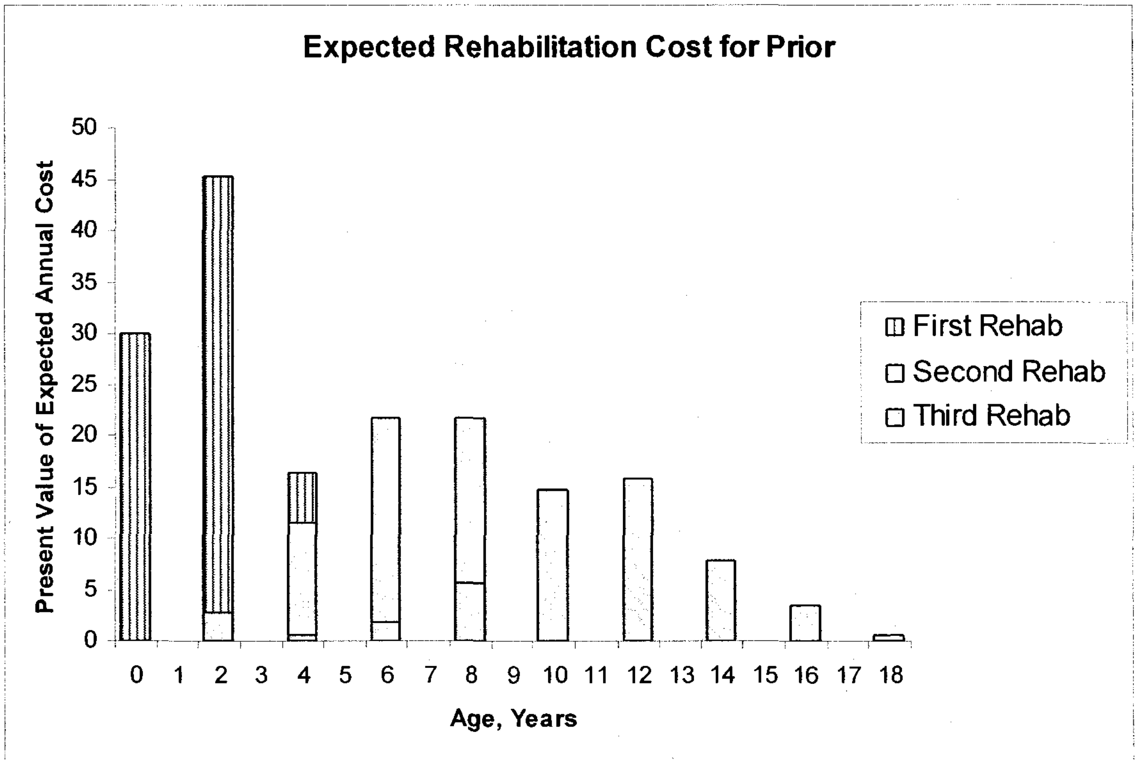


Figure 6- 3 Expected cost of rehabilitation prior to updating probability

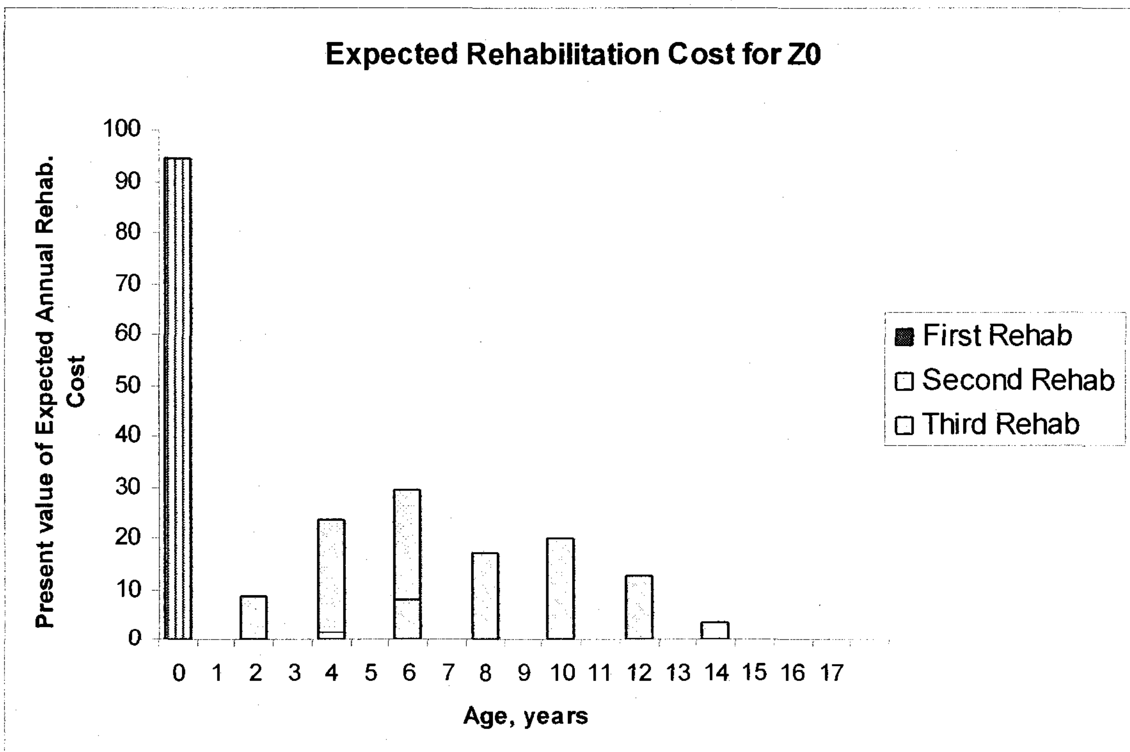


Figure 6- 4 Expected cost of rehabilitation corresponding to SHM test outcome Z_0

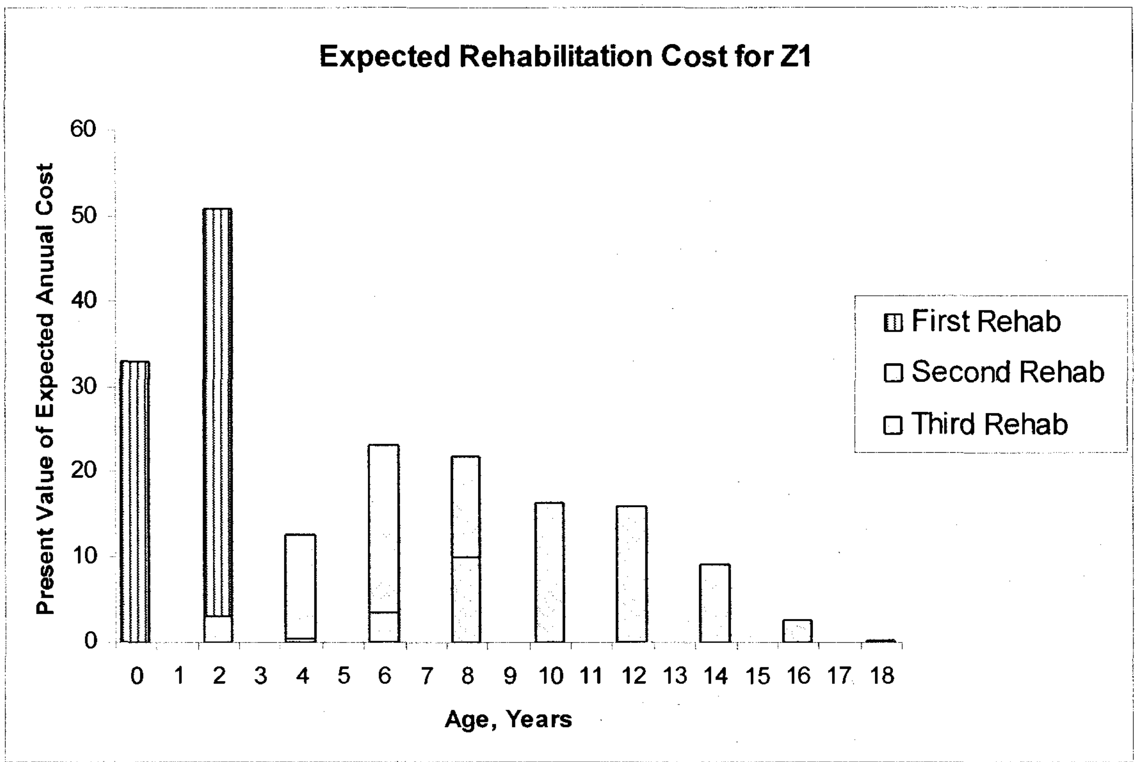


Figure 6- 5 Expected cost of rehabilitation corresponding to SHM test outcome Z_1

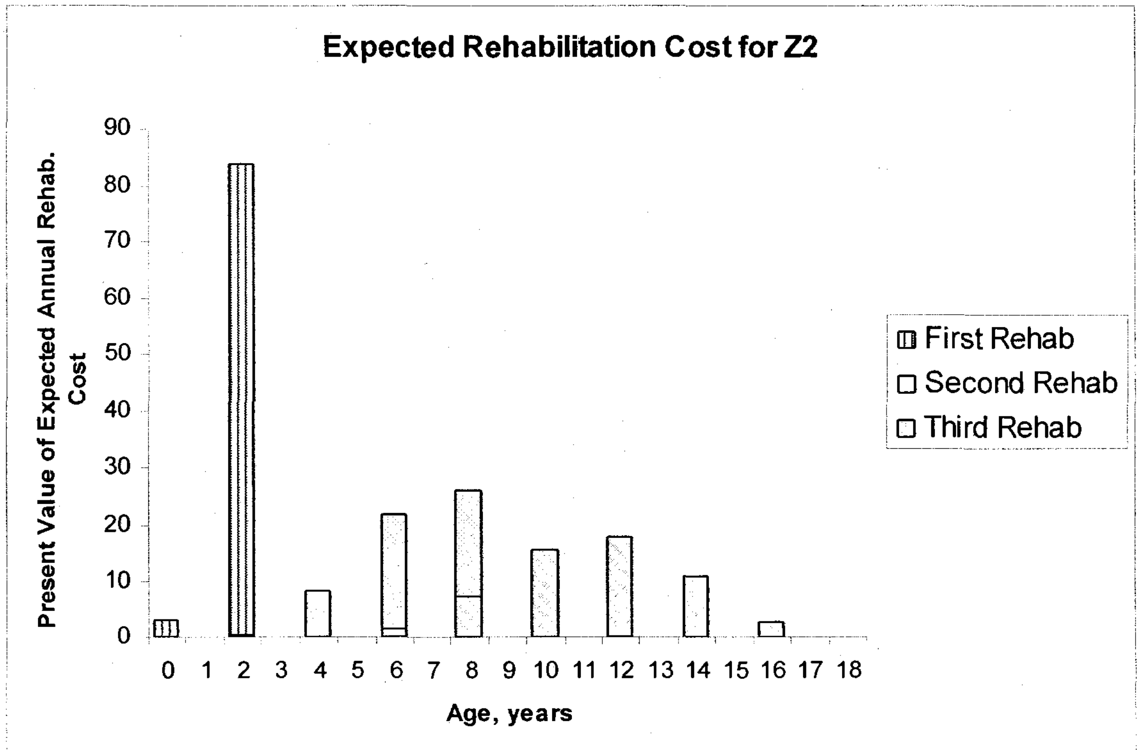


Figure 6- 6 Expected cost of rehabilitation corresponding to SHM test outcome Z_2

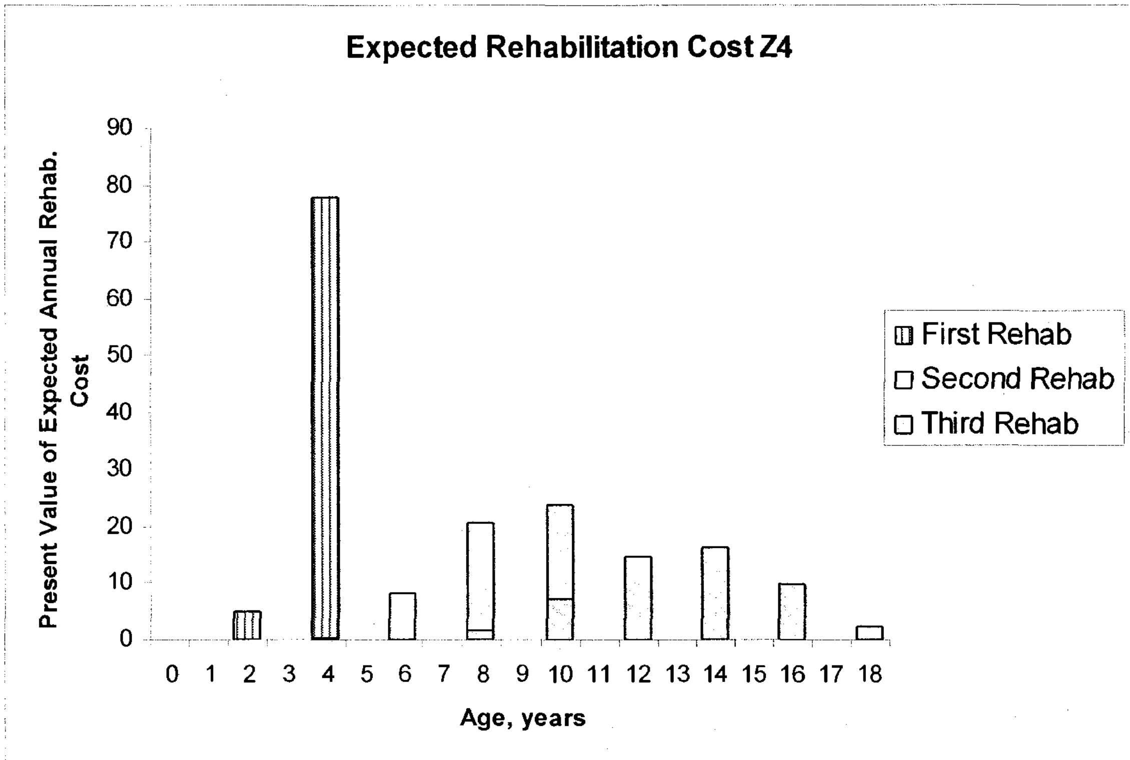


Figure 6- 7 Expected cost of rehabilitation corresponding to SHM test outcome Z_4

Prior to updating total expected cost for first rehabilitation is \$91.8 while after updating it is \$99.48, 92.84, 90.6, and 82.73 when SHM test outputs are Z_0, Z_1, Z_2, and Z_4 respectively. Table 6-7 cost relating to different rehabilitation actions.

Table 6- 7 Cost of rehabilitation actions over time

Corresponding to	Cost of first rehabilitation (EC_1 in \$)	Cost of second rehabilitation (EC_2 in \$)	Cost of third rehabilitation (EC_3 in \$)
Prior	91.8	70.81	50.07
Output Z_0	99.48	76.74	62.67
Output Z_1	92.84	71.61	58.48
Output Z_2	90.60	69.91	57.10
Output Z_4	82.73	63.81	52.11

6. 3 The Value of Information (VI)

To decide whether SHM information should be used or not, the value of SHM information needs to be calculated. VI is calculated as follows:

$$VI = E(T) - E(R) \tag{6. 7}$$

where *E(T)* is the expected cost of rehabilitation after updating the probabilities, excluding the cost of SHM information, and *E(R)* is the expected cost of rehabilitation calculated without considering the updated probabilities. *E(T)* is calculated using updating probabilities for the first rehabilitation. To calculate the expected rehabilitation cost and the cost associated with SHM a pre-posterior analysis needs to be done which involves a decision tree approach. This work is not in the scope of this present paper. If the value of information, *VI* exceeds the cost associated with the SHM system, C_{SHM} the SHM information will be regarded to be beneficial.

Let's assume that the SHM test gives for outputs, as mentioned earlier, having probability of each output as follows: $P(Z_0) = 0.2$; $P(Z_1) = 0.3$; $P(Z_2) = 0.3$; and $P(Z_4) = 0.2$. The *E(T)* can be calculated as

$$E(T) = \sum_{i=1}^{m} \left[P(Z_i) \times \sum_{j=1}^{n} EC_{j,i} \right] \tag{6. 8}$$

where, $P(Z_i)$ = probability of SHM test output will be Z_i

$EC_{j,i}$ = Expected cost of the jth rehabilitation actions when SHM test output is Z_i .

Using equation 6.8, $E(T)$ is evaluated \$ 219.67. The value of $E(R)$ is equal to \$ 212.68. The value of information is \$ 6.99. So if the cost of SHM system (C_{SHM}) is less than VI then SHM test should performed or SHM system should be implemented.

Chapter 7

Life Cycle Cost Analysis

7.1 Introduction

In previous chapter shows the updating of prior information on the basis of the new information. This chapter discusses how this update information has an effect on the life cycle cost of the structure.

In Canada more than 40% of the bridges currently in use were built over 50 years ago, and a significant number of these structures need strengthening, rehabilitation, or replacement (ISIS, Canada). Structural deterioration increases with the age of the bridge structure due to corrosion, fatigue, wear and rear and other methods of material deterioration. At the same time loads, vehicles and legal load limits for bridges have been increasing. When the aging bridge structures are subjected to these kinds of excessive loads, then the structural capability of it reduces. Therefore, a method to satisfy the ever increasing loads and traffic has to be found for a particular deteriorated bridge. This chapter aims to evaluate the expected life cost of the structure.

7.2 Life Cycle Cost Analysis

In a bridge maintenance and rehabilitation program, there are several costs and benefits involved during the service period. So calculation of Expected life Cycle Cost involves these all costs and then total cost needs to be minimized. Such a decision analysis is referred as a whole of life cycle costing, cost-benefit or cost-benefit-risk analysis (Setunge et. al., 2002). Life cycle costs will assess the cost effectiveness of design decisions, quality of construction or inspection, maintenance and repair strategies (Stewart 2001). The costs associated in a rehabilitation project may initially include:

- Initial cost
- Maintenance, monitoring and repair cost
- Costs associated with traffic delays or reduced travel time (Extra user cost)
- Failure cost

In order to be able to add and compare cash flows, these costs should be made time equivalent. It can e presented different ways, but the most commonly used indicator in road asset management is the Net Present Value (NPV) of the rehabilitation option. The Life Cycle Cost Analysis (LCCA) method converts all the costs to present values by discounting them to a common time, usually the base date. The present value analysis has to be considered together with Internal Rate of Return (IRR). There are several parameters to be considered in the present value analysis.

7.2.1 Study period

The study period begins with the base date, that is the date to which all cash flows are discounted. Because the cost of each alternative rehabilitation strategy can be compared reasonably, only if the benefits gained are the same, the alternatives should be compared over the same operational time period which is known as study period. As a rule of thumb, the analysis period should be long enough to incorporate all or significant component of each alternative's life cycle including one rehabilitation on each alternative (Setunge et. al., 2002). Generally, study period or the evaluation period is based on the economic life of major assets in the project. For bridges, the study period is normally longer than the pavements (more than 40 years). Assets with economic life longer than the evaluation period should be given a residual value (resale value).

7.2.2 Residual Value

This the net worth of a bridge structure at the end of the LCCA study period. Unlike other future costs, a particular alternative's residual value can be positive or negative, a cost or a value.

7.2.3 Discount rate and inflation

Discount rate is defined as "the rate of interest reflecting the investor's time value of money (Mearing et al. 1999). As the costs are incurred in a project in different times, the interest rate used to discount is a rate that reflects an investor's opportunity cost of money over time. It is the discount rate (interest rate) that would make an investor feel the same way if he receives a payment now or a large payment at sometime in the future. The LCCA can be performed in constant dollars or current dollars (Setunge et. al., 2002). Constant dollar analyses exclude the rate of general inflation. Current dollar analyses include the rate of genral inflation in all costs, discount rate and price escalation rates. Both methods give the identical present value.

It is obvious that the discount rates are normally influenced by the economic, social and political factors. Discount rates used by various countries are different. In Canada 3-4% discount rate is used. In this study 4% discount rate is considered.

7.2.4 Evaluation Factors

Table 7- 1 Evaluation factors for the analysis (Austroads, 1996)

Factor	Common value
Evaluation period	40 years
Price year	Current year
Discount rate	4%
Residual value	If the useful life of the asset exceeds the evaluation period an allowance should be made for the residual value. For projects with 30 year evaluation period this is taken as zero.

7.2.5 Formulation of whole life cycle cost

Objective function for the optimal bridge rehabilitation can be formulated as the maximization of W as shown in equation 7. 1,

$$W = B_{lifecycle} - C_{lifecycle} \qquad 7.\,1$$

where $B_{lifecycle}$ is the benefit which can be gained from the existence of the bridge after rehabilitation and $C_{lifecycle}$ is the cost associated with the bridge during its whole life. Since the benefit from the bridge will be the same irrespective of the method of rehabilitation, the objective function will be reduced to equation 7.2

$$W = C_{lifecycle} \qquad 7.\,2$$

As discussed above $C_{lifecycle}$ can be calculated using equation 7.3

$$C_{lifecycle} = C_{capital} + C_{repair} + C_{user} + C_{failure} + C_{SHM} \qquad 7.\,3$$

When all input costs are defined the NPV can be calculated easily. But inputs are associated with high degree of uncertainty. In order to deal with such uncertainties it is necessary to consider the probabilistic behaviour of the input costs. In the following part of this chapter all these components have been discussed in detail.

7.2.6 Initial Cost Calculation

Initial cost is considered as capital cost. For steel free bridge deck the capital cost is significantly different than conventional steel reinforced bridge deck. Table 7- 2 shows the difference in cost between steel free and conventional bridge deck for different bridges.

Table 7- 2 Cost comparison for both steel free and reinforced bridge decks (Mufti and Bakht, 2005)

Bridge Name	Difference in cost	Reason
Salmon River Bridge	The cost of steel free bridge deck is 6% higher than conventional steel reinforced.	Contractor had no experience in fibre concrete, and was apprehensive of the problems associated with this new concrete.
Chatham Bridge	Much higher than conventional one.	Use of expensive CFRP.
Crowchild Bridge	Lower than conventional one.	-
Waterloo Creek Bridge	-	-
Lindquist Creek Bridge	30% cheaper than the conventional one	-
US Highway 151	Material cost 60% more than conventional one, but saved 57% labour cost ($329/ m^2)	No experience with this technology.

Table 7- 3 shows initial cost of steel reinforced and GFRP bridge decks. Using Table 7- 3 data the initial cost for GFRP bridge deck comes out to be $443.16/m^2.

Table 7- 3 Initial and Maintenance cost for steel reinforced and GFRP Bridge decks (ISIS 2006)

	Variable	**Steel**	**GFRP**
Discount rate	i	6.0%	6.0%
Service life (years)	L_n	50	75
Initial costs			
-Design ($)	D_n	25,000	35,000
-Traffic control ($)	T	150,000	150,000
-Deck area (m^2)	A	6,000	6,000
-Unit rebar cost ($/$m^2$)	rc_n	25	94
-Unit concrete cost ($/$m^2$)	cc	300	300
Install rebar cost ($/$m^2$)	ic_n	25	20
Maintenance & Repair			
-M&R traffic control ($)	MT	75,000	75,000
-Concrete repair ($)	MC_n	5,000,000	2,500,000
-Concrete cycle (years)	SC_n	25	50
-Resurface ($)	MR	150,000	150,000
-Resurface cycle (years)	SR	25	25
Decommissioning Costs - Decommissioning ($)	DC	3,000,000	3,000,000

All these costs will incur in the base time of the project. Therefore the calculation of initial cost component is straight forward.

7.2.7 Maintenance Cost Calculation

Modeling of the future maintenance cost is complicated. Generally, future maintenance cost is calculated in probabilistic terms. There are two types of maintenance works in bridges: *preventive maintenance* if which is not done it will cost more at later stage to keep the structure in a safe condition, and *essential maintenance* which is required keep the structure safe (Noortwijk and Frangopol, 2004). Preventive maintenance are further divided in two types: *proactive* preventive maintenance (applied before any indication of deterioration is apparent) and *reactive* preventive maintenance (applied only after some deterioration is evidenced). The significance of preventive maintenance has always been questioned. Many engineers believed that these preventive measures are worthwhile in long term, but can not defend this point of view on reliability basis.

Table 7- 4 Estimated unit cost for superstructure of composite concrete bridges and reinforced concrete bridges (Maunsell Ltd. and Transport Research Laboratory 1998, 1999)

	With preventive maintenance ($/m^2)		Without preventive maintenance ($/m^2)	
Bridge Type	Reinforced bridge	Steel/Concrete Composite bridge	Reinforced bridge	Steel-Composite bridge
Cost Type				
Preventive maintenance cost	69	132	0	0
Essential maintenance cost	358	379	847	968
User cost for a preventive maintenance cost	157	177	0	0
User cost for essential maintenance	660	576	6408	3061

This is simply because basis does not exist. For this reason, a reliability based model has to be developed and used to identify optimal preventive strategies based on life time reliability and life cycle costs for different civil infrastructure systems. The literature shows that the maintenance costs get reduced by a significant amount if preventive maintenance work is performed, and it also increases the service life of the structure (Noortwijk and Frangopol, 2004). Figure 7- 1 shows the effect of preventive maintenance on the occurrence of the essential maintenance for the different bridge types. Table 7- 4 shows the comparison among various costs with preventive maintenance and without preventive maintenance.

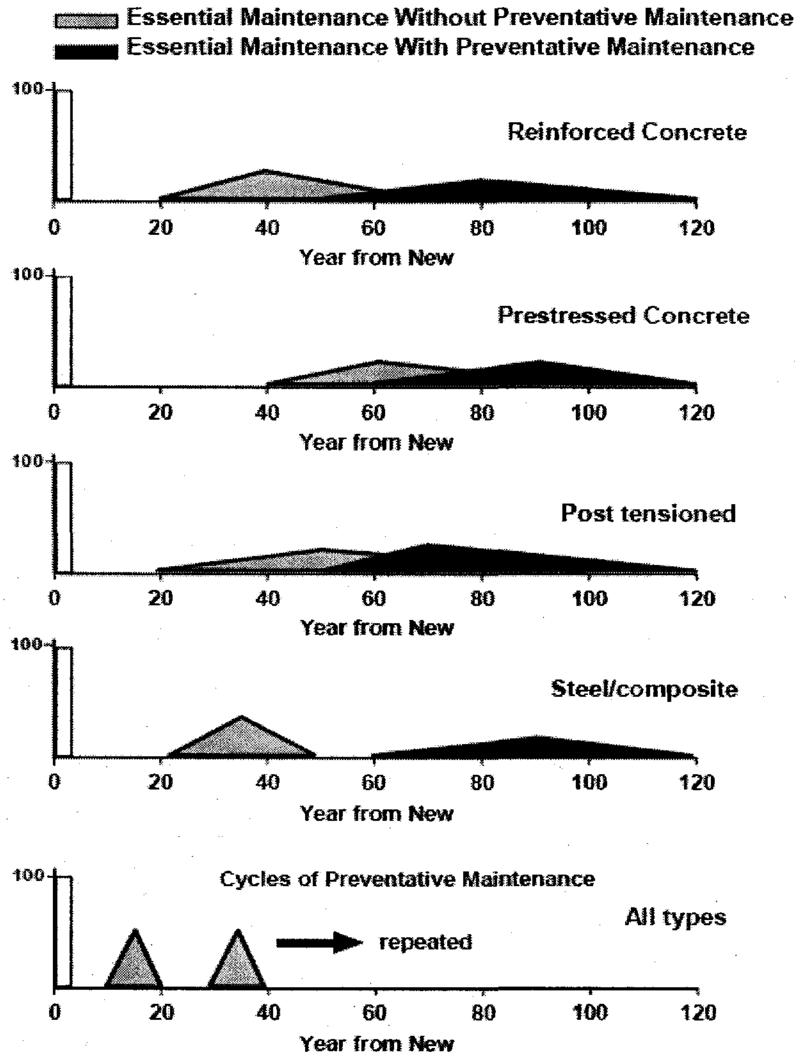


Figure 7- 1 Rehabilitation rate and maintenance cycles for different bridges (Das, 1999)

Preventive maintenances are cyclic maintenance and they are performed in intervals. But essential maintenances are generally performed once in lifetime. Table 7-5 shows the action types associated with maintenance work and their recurrence. It is obvious that preventive maintenance cycles are highly correlated. And it is clear from previous discussion that essential maintenance also depends on preventive maintenance.

So to calculate the expected life cycle maintenance cost these correlations has to be considered.

Table 7- 5 Action types (Kong and Frangopol, 2002)

Action types	Class types in life-cycle analysis of deteriorating structures	Example
Time controlled		
• Applied once	1	Essential maintenance based on a probability distribution of application time.
• Applied cyclically	2	Preventive maintenance every five years or painting steel components every 10 years.
Reliability controlled		
• Applied once	3	Member replacement required when the system reliability down crosses a given target level.
• Applied cyclically	4	Repair required when the system whenever the reliability of the system is in state 2.a

a Reliability states are defined in Frangopol et al. (2001)

These maintenance cycles' cash flows can be represented as several dependent projects' cash flows. Cassimates (1988) proposes an approach to calculate net present value (NPV) with interdependent cash flows (interdependent projects) using a series of conditional probability distributions. The solution is based on multistage decision tree analysis where separate probability distributions in year t follow each outcome in year t-1. For each series of probability is computed by multiplying the successive probabilities of all series are used to derive the project's expected net present value as in equation 7.4

$$\overline{NPV} = \sum_{x=1}^{s} JP_x \overline{DCF_x} \qquad 7.4$$

where JP_x is the joint probability of series x and $\overline{DCF}_x$ is the expected discounted cash flow of that series.

The cash flow's standard deviation is calculated by equation 7.5

$$\sigma = \sqrt{\sum_{x=1}^{s} (NPV_x - \overline{NPV})^2 P_x} \qquad 7.5$$

where NPV_x is the net present value for the series x. This method accounts for the correlation of crash flows from one year to the next, although the correlation is not perfect because a range of outcome is possible. A serious disadvantage of this approach is the amount of computation is necessary for multi year projects with many probability distributions. But due to advancement in computer technology this problem is no more a limitation.

Figure 7- 1 shows the preventive maintenance cycle, it follows a triangular distribution having min value at 10 years; max value at 20 years; and mode value at 15 years. The Table 7- 4 shows per unit cost for the preventive maintenance. According to (Setunge et. al., 2002) suggested minimum value for maintenance is -10% of estimate and suggested maximum value is +10%. Using Risk Analysis software, a histogram (Figure 7- 2) has been generated which represents the above mentioned probability distributions. Table 7- 6 can be obtained using Figure 7- 2 and Figure 7- 4.

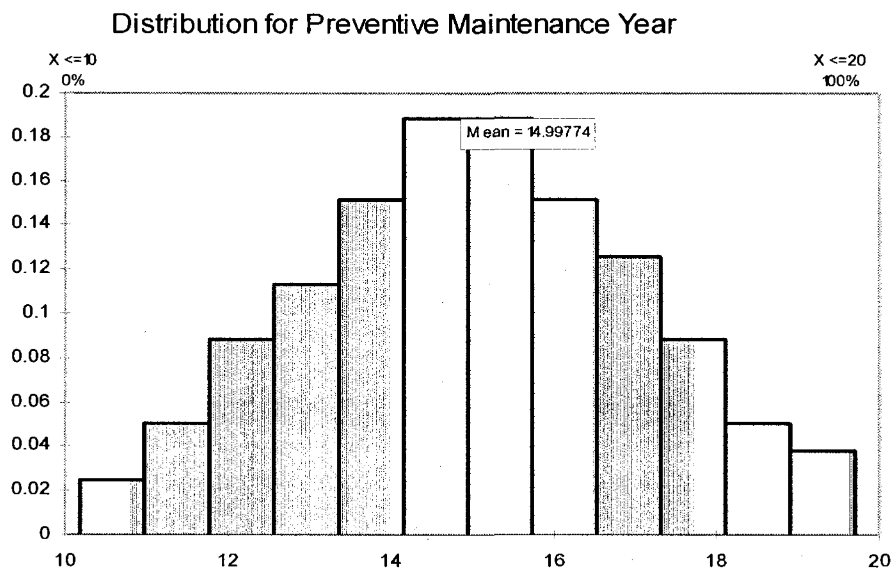


Figure 7- 2 Probability of preventive maintenance for respective years

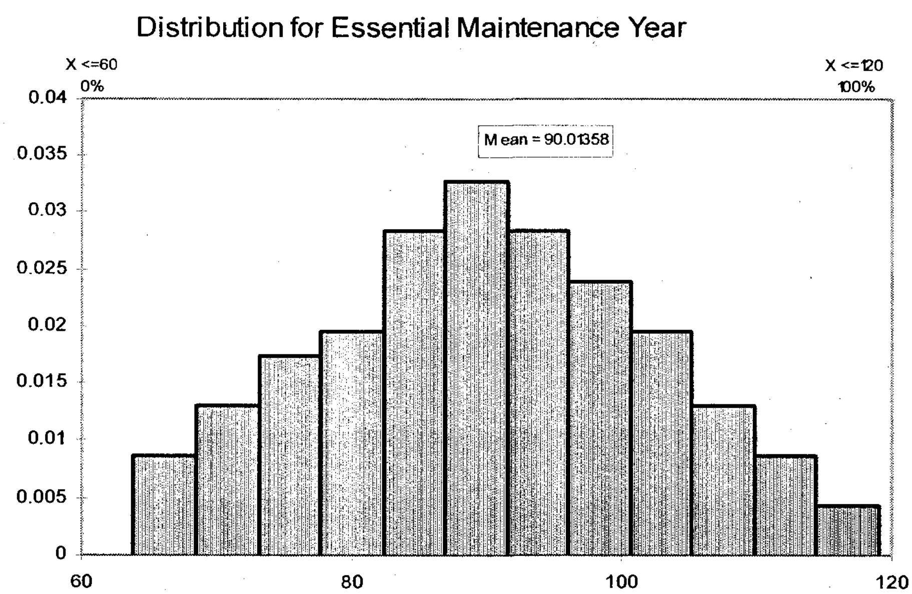


Figure 7- 3 Probability of essential maintenance for respective years

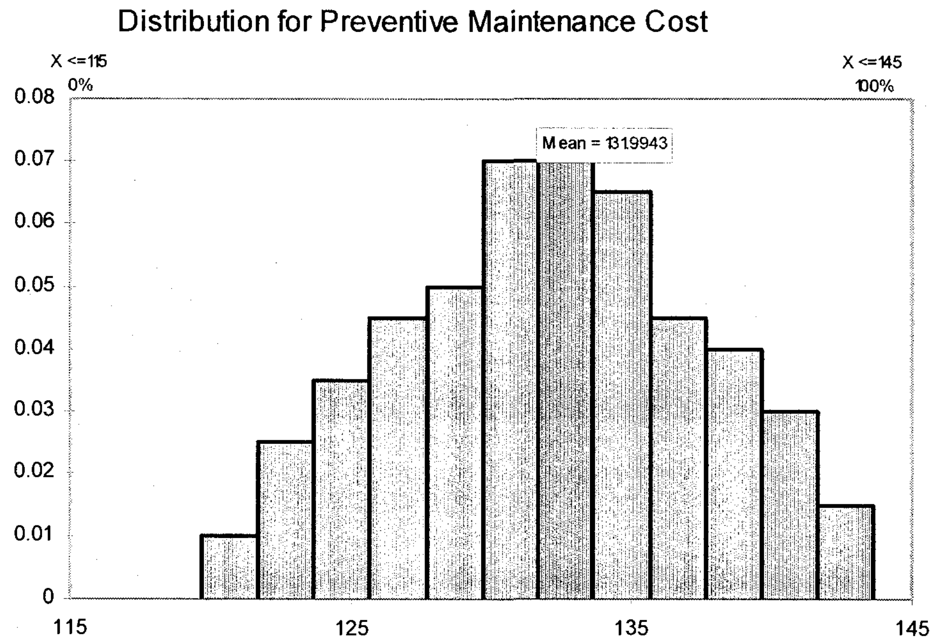


Figure 7- 4 Probabilities associated with preventive maintenance costs

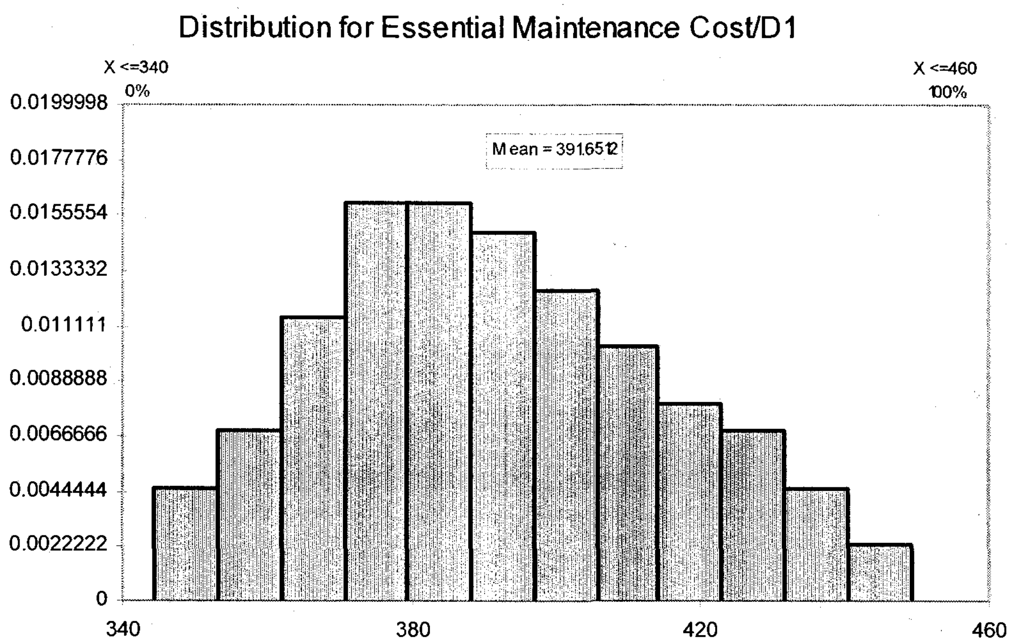


Figure 7- 5 Probabilities associated with essential maintenance costs

Table 7- 6 Preventive maintenance year and, corresponding cost and probability

Maintenance year from construction	Probability	Cost ($/m^2)
10	0.0273	121.5
12	0.0608	124.0
14	0.2333	129.0
16	0.3638	134.4
18	0.237	138.7
20	0.778	144.0

Table 7- 6 shows the year of preventive maintenance and, cost and probability associated with it. Here, it is assumed that most probable cost will correspond to most probable maintenance year or vice versa. The year of essential maintenance and cost are shown in Table 7- 7 . Using same method Table 7- 7 (from Figure 7- 3 and Figure 7- 5) has been created for essential maintenance. It should be noted that Table 7- 6 and Table 7- 7, both are for the steel/concrete composite bridges. For reinforced bridges same analysis is shown late in this chapter.

Table 7- 7 Essential maintenance year and, corresponding cost and probability

Maintenance Year from construction	Probability	Cost ($/m^2)
60	0.0381	352
70	0.1249	370
80	0.2196	386
90	0.2719	404
100	0.1882	422
110	0.1253	440
120	0.0320	448

Now using Equation (4) and (5), and decision tree approach the expected net present value ($\overline{NPV}$) has been calculated and lifetime (period of consideration) has been taken 120 years.

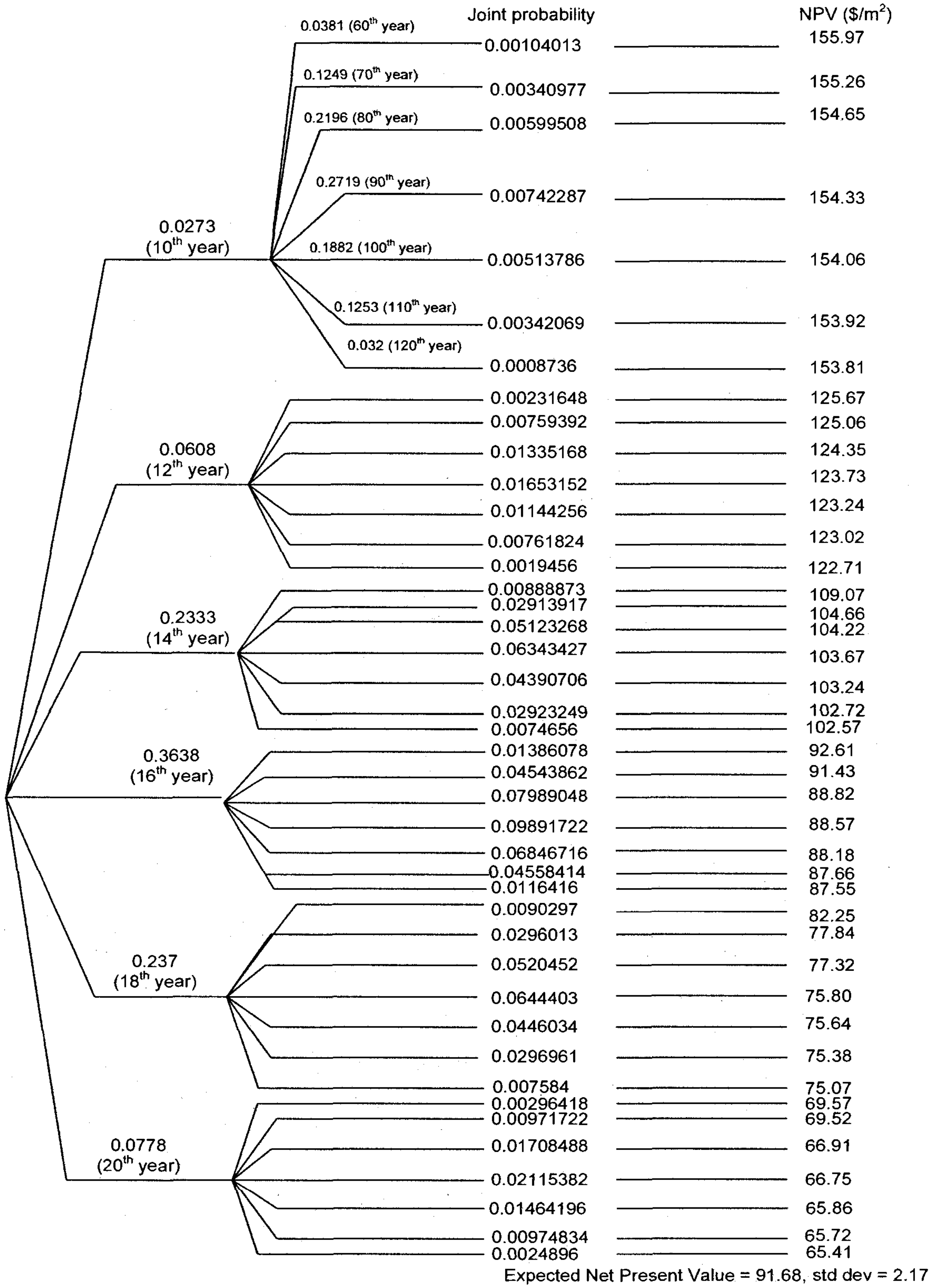


Figure 7- 6 Decision tree for preventive and essential maintenance cost

In order to calculate $\overline{NPV}$ of cash flow, first NPV and joint probability of each individual path in decision tree have to be evaluated. So total cost of preventive maintenance and essential maintenance need to be calculated. Now, the cost of preventive maintenance will depend on the number of maintenance cycles, which will further depend on the year of essential maintenance and the year of preventive maintenance. Hence, for each individual path in decision tree (Figure 7-6) the number of cycles will be different. This will, eventually, result in different preventive maintenance costs. The number of cycles can be calculated as equation 7.6

$$\left.\begin{array}{ll} N_c = \dfrac{t_e}{t_p} & \text{if } t_p \text{ is not divisor of } t_e \\ N_c = \dfrac{t_e}{t_p} - 1 & \text{if } t_p \text{ is divisor of } t_e \end{array}\right\} \qquad 7.6$$

where N_c is number of preventive maintenance cycles before essential maintenance is performed; t_p is the preventive maintenance cycle period; t_e is the year of essential maintenance is performed.

So the total preventive maintenance cost can be calculated maintenance cost can be given as equation 7.7

$$TPMC = \sum_{r=1}^{N_c} \frac{PMC_j}{(1+i)^{t_j * r}} \qquad 7.7$$

where $TPMC$ is the total preventive maintenance cost for each individual path (discounted) ; PMC_j is the preventive maintenance cost corresponding to cycle having time period of j year for each individual path ; i is the discount rate; and r is the cycle number.

If the time of consideration (lifetime period) is larger than the time of essential maintenance, the total preventive maintenance cost is given by equation 7.8

$$TPMC = \sum_{r=1}^{N_c} \frac{PMC_j}{(1+i)^{t_j * r}} + \sum_{r=N_c+1}^{N_{pt}} \frac{PMC_j}{(1+i)^{t_j * r}} \qquad 7.8$$

where N_{pt} is number of preventive maintenance cycle performed after the essential maintenance. It is calculated as equation 7.9

$$N_{pt} = \frac{t - t_e}{t_j} \qquad 7.9$$

where t is the life time period.

As mentioned in Table 7- 5, essential maintenance is performed once in lifetime. The total cost for essential is calculated as equation 7.10

$$EMC_d = \frac{EMC}{(1+i)^{t_e}} \qquad 7.10$$

where EMC_d is the discounted essential maintenance cost; EMC is the essential maintenance. Table 7- 8 shows the total discounted cost for preventive maintenance and essential maintenance for each path in decision tree. The number of preventive maintenance cycles has been calculated as discussed above. Table 7- 9 shows the joint probability and net present value (total discounted cost) for each path. And, in the end it calculates the expected net present value and standard deviation for cash flow.

Table 7- 8 Total discounted cost for preventive and essential maintenance

Case	Year of Preventive Maintenance	Dicount rate (in %)	Cost of preventive maintenance per cycle (in $/m^2$)	Year of Essential Maintenance	Cost of essential maintenance (in $/m^2$)	Number of preventive maintenance cycle before essential maintenance	Number of preventive maintenance cycleafter essential maintenance	Discounted cost for Preventive Maintenance before essential maintenance(in $/m^2$)	Total discounted cost for Preventive Maintenance after essential maintenance(in $/m^2$)	Discounted cost for Essential Maintenance (in $/m2)	Total discounted cost (in $/m2)
1	10	6	121.50	60	352.00	6	6	145.30	9.72	10.67	165.69
2	10	6	121.50	70	370.00	7	5	149.00	5.35	6.26	160.62
3	10	6	121.50	80	386.00	8	4	151.00	2.91	3.65	157.56
4	10	6	121.50	90	404.00	9	3	152.20	1.55	2.13	155.89
5	10	6	121.50	100	422.00	10	2	152.82	0.79	1.24	154.86
6	10	6	121.50	110	440.00	11	1	153.20	0.37	0.72	154.29
7	10	6	121.50	120	448.00	12	0	153.40	0.13	0.41	153.94
8	12	6	124.00	60	362.00	5	5	115.00	0.55	10.67	134.22
9	12	6	124.00	70	370.00	5	4	118.80	8.41	6.26	133.48
10	12	6	124.00	80	386.00	6	3	120.70	4.05	3.65	128.40
11	12	6	124.00	90	404.00	7	2	121.60	1.88	2.13	125.61
12	12	6	124.00	100	422.00	8	1	122.00	0.80	1.24	124.05
13	12	6	124.00	110	440.00	9	0	122.30	0.27	0.72	123.29
14	12	6	124.00	120	440.00	10	0	122.30	0.13	0.41	122.04
15	14	6	129.00	60	362.00	3	4	98.40	9.71	10.67	118.79
16	14	6	129.00	70	370.00	3	3	98.40	4.20	6.26	108.87
17	14	6	129.00	80	386.00	4	2	100.57	3.99	3.65	108.21
18	14	6	129.00	90	404.00	4	2	101.54	1.77	2.13	105.44
19	14	6	129.00	100	422.00	5	1	102.00	0.69	1.24	103.93
20	14	6	129.00	110	440.00	5	0	102.00	0.48	0.72	103.20
21	14	6	129.00	120	448.00	6	0	102.16	0.21	0.41	102.78
22	16	6	134.42	60	352.00	3	3	81.94	14.14	10.67	106.75
23	16	6	134.42	70	370.00	4	3	85.17	5.57	6.26	97.00
24	16	6	134.42	80	386.00	5	2	85.17	2.11	3.65	90.93
25	16	6	134.42	90	404.00	5	1	86.44	1.90	2.13	90.47
26	16	6	134.42	100	422.00	6	1	86.94	0.75	1.24	88.93
27	16	6	134.42	110	440.00	6	0	86.94	0.54	0.72	88.20
28	16	6	134.42	120	448.00	7	0	87.14	0.21	0.41	87.76
29	18	6	138.70	60	362.00	3	3	71.58	9.39	10.67	91.64
30	18	6	138.70	70	370.00	3	2	71.58	9.12	6.26	86.96
31	18	6	138.70	80	386.00	4	2	73.67	3.20	3.65	80.51
32	18	6	138.70	90	404.00	5	1	73.67	1.03	2.13	76.83
33	18	6	138.70	100	422.00	5	1	74.40	1.03	1.24	76.67
34	18	6	138.70	110	440.00	6	0	74.66	0.27	0.72	75.65
35	18	6	138.70	120	448.00	6	0	74.66	0.27	0.41	75.34
36	20	6	144.00	60	352.00	3	3	58.90	6.28	10.67	75.85
37	20	6	144.00	70	370.00	3	2	60.26	6.15	6.26	75.67
38	20	6	144.00	80	386.00	4	2	63.26	1.92	3.65	68.83
39	20	6	144.00	90	404.00	4	1	64.62	1.79	2.13	68.54
40	20	6	144.00	100	422.00	5	1	64.62	0.56	1.24	66.42
41	20	6	144.00	110	440.00	5	0	65.00	0.42	0.72	66.15
42	20	6	144.00	120	448.00	6	0	65.00	0.13	0.41	65.54

Table 7- 9 Total net present value for preventive and essential maintenance

Case	Time of first cycle of preventive maintenance	Probabillity of preventive maintenance	Time for essential maintenance	Probability of essential maintenance	Joint proabability (P)	NPV	P(NPV)
1	10	0.0273	60	0.0381	0.00104013	165.69	0.172337
2	10	0.0273	70	0.1249	0.00340977	160.62	0.547662
3	10	0.0273	80	0.2196	0.00599508	157.56	0.944605
4	10	0.0273	90	0.2719	0.00742287	155.89	1.157122
5	10	0.0273	100	0.1882	0.00513786	154.86	0.795636
6	10	0.0273	110	0.1253	0.00342069	154.29	0.52779
7	10	0.0273	120	0.0320	0.0008736	153.94	0.134485
8	12	0.0608	60	0.0381	0.00231648	134.22	0.310913
9	12	0.0608	70	0.1249	0.00759392	133.48	1.013621
10	12	0.0608	80	0.2196	0.01335168	128.40	1.714331
11	12	0.0608	90	0.2719	0.01653152	125.61	2.076566
12	12	0.0608	100	0.1882	0.01144256	124.05	1.419402
13	12	0.0608	110	0.1253	0.00761824	123.29	0.939256
14	12	0.0608	120	0.0320	0.0019456	122.84	0.239005
15	14	0.2333	60	0.0381	0.00888873	118.79	1.055849
16	14	0.2333	70	0.1249	0.02913917	108.87	3.172281
17	14	0.2333	80	0.2196	0.05123268	108.21	5.543942
18	14	0.2333	90	0.2719	0.06343427	105.44	6.688397
19	14	0.2333	100	0.1882	0.04390706	103.93	4.563327
20	14	0.2333	110	0.1253	0.02923249	103.20	3.016821
21	14	0.2333	120	0.0320	0.0074656	102.78	0.767334
22	16	0.3638	60	0.0381	0.01386078	106.75	1.479625
23	16	0.3638	70	0.1249	0.04543862	97.00	4.407485
24	16	0.3638	80	0.2196	0.07989048	90.93	7.26415
25	16	0.3638	90	0.2719	0.09891722	90.47	8.948965
26	16	0.3638	100	0.1882	0.06846716	88.93	6.088815
27	16	0.3638	110	0.1253	0.04558414	88.20	4.020517
28	16	0.3638	120	0.0320	0.0116416	87.76	1.021697
29	18	0.237	60	0.0381	0.0090297	91.64	0.827469
30	18	0.237	70	0.1249	0.0296013	86.96	2.57427
31	18	0.237	80	0.2196	0.0520452	80.51	4.19038
32	18	0.237	90	0.2719	0.0644403	76.83	4.95086
33	18	0.237	100	0.1882	0.0446034	76.67	3.419744
34	18	0.237	110	0.1253	0.0296961	75.65	2.246522
35	18	0.237	120	0.0320	0.007584	75.34	0.571363
36	20	0.0778	60	0.0381	0.00296418	75.85	0.224844
37	20	0.0778	70	0.1249	0.00971722	75.67	0.73534
38	20	0.0778	80	0.2196	0.01708488	68.83	1.17589
39	20	0.0778	90	0.2719	0.02115382	68.54	1.449837
40	20	0.0778	100	0.1882	0.01464196	66.42	0.972526
41	20	0.0778	110	0.1253	0.00974834	66.15	0.644838
42	20	0.0778	120	0.0320	0.0024896	65.54	0.163178
				Expected Net Present Value			**94.18**
				Standard Deviation			**2.22**

7.2.8 User cost

User cost may be calculated in terms of costs associated with traffic delay, and in case of using routes wear and tear of user vehicle. Most of the time, it is very hard to include all parameters. The calculation of user cost is similar to maintenance cost. Table 7- 4 shows the user cost for both preventive maintenance and essential maintenance for different bridge types. The cost can be calculated as equation 7.11

$$TUCPM = \sum_{r=1}^{N_c} \frac{UCPM_j}{(1+i)^{t_j * r}} \qquad 7.11$$

where $TUPMC$ is the total user cost for preventive maintenance (discounted); $UPMC_j$ is the user cost for preventive maintenance corresponding to cycle having time period of t_j year for each individual path. If the time of consideration (lifetime period) is larger than the time of essential maintenance, the total preventive maintenance cost is given by equation 7.12

$$TUCPM = \sum_{r=1}^{N_c} \frac{UCPM_j}{(1+i)^{t_j * r}} + \sum_{r=N_c+1}^{N_{pt}} \frac{UCPM_j}{(1+i)^{t_j * r}} \qquad 7.12$$

The total user cost for essential is calculated as equation 7.13

$$UCEM_d = \frac{UCEM}{(1+i)^{t_e}} \qquad 7.13$$

where $UCEM_d$ is the discounted essential maintenance user cost; $UCEM$ is the essential maintenance user cost. Figure 7- 7 shows the decision tree for user costs.

Table 7-10 shows the total discounted user cost for preventive maintenance and essential maintenance for each path in decision tree.

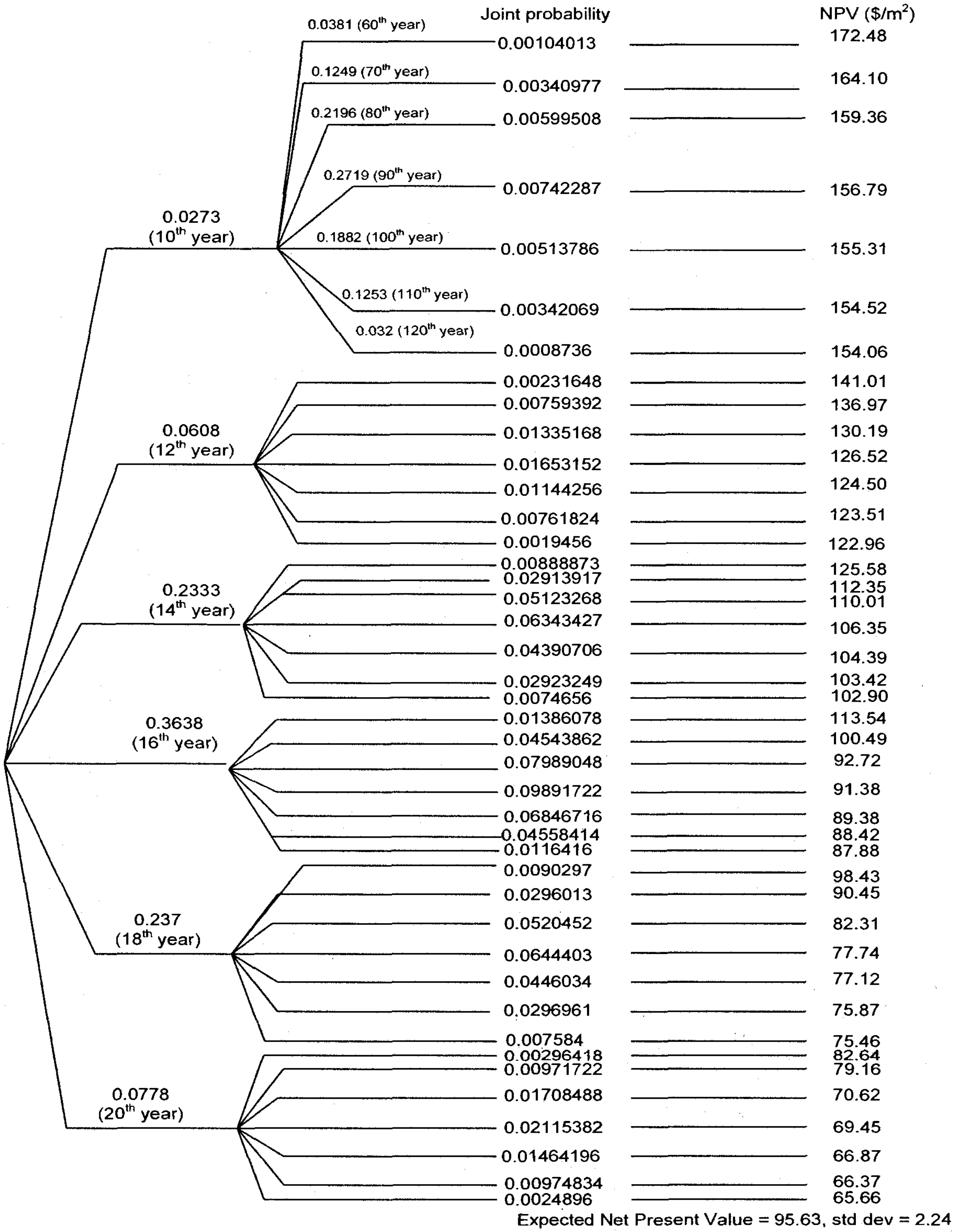


Figure 7- 7 Decision tree for user cost

Table 7-11 shows the joint probability and net present value (total discounted cost) for each path.

7.2.9 Failure Cost

Expected cost of failure needs to be considered in order to have more precise forecast of life cycle cost. Due to uncertainties associated with structural properties, loads and environmental conditions the cost failure is random variable (Setunge et al., 2002). This expected failure cost is included in the life cycle cost criterion based on Neumann-Morgenston (Von Neumann and Morgenston, 1944) decision theory under the assumption that utilities are express in monetary values. Failure of different alternatives may occur at different times so in order to obtain consistent results costs of failure are discounted to a present value (Val and Stewart 2004). The equation 7.14 shows the failure cost as

$$C_F(t) = \frac{c_F}{(1+i)^t} \tag{7. 14}$$

where c_F is the cost of failure set at the time of decision making, t, the time of failure and i the discount rate. The structural failure events are random events with time dependant probabilities of occurrence, due to uncertainties associated with the structural properties, the loads and the environmental conditions. It is common to consider failure at discrete points in time so that their probabilities are equal to the cumulative probability of failure over a corresponding time interval.

Table 7-10 Total discounted user cost

Case	Year of Preventive Maintenance	Dicount rate (in %)	User cost for preventive maintenance per cycle (in $/m^2$)	Year of Essential Maintenance	User cost for essential maintenance (in $/m^2$)	Number of preventive maintenance cycle before essential maintenance	Number of preventive maintenance cycleafter essential maintenance	Discounted cost for Preventive Maintenance before essential maintenance(in $/m^2$)	Total discounted cost for Preventive Maintenance after essential maintenance(in $/m^2$)	Discounted cost for Essential Maintenance (in $/m2)	Total discounted cost (in $/m2)
1	10	6	177.00	60	576.00	6	6	145.30	9.72	17.46	172.48
2	10	6	177.00	70	576.00	7	5	149.00	5.35	9.75	164.10
3	10	6	177.00	80	576.00	8	4	151.00	2.91	5.44	159.36
4	10	6	177.00	90	576.00	9	3	152.20	1.55	3.04	156.79
5	10	6	177.00	100	576.00	10	2	152.82	0.79	1.70	155.31
6	10	6	177.00	110	576.00	11	1	153.20	0.37	0.96	154.62
7	10	6	177.00	120	576.00	12	0	153.40	0.13	0.53	154.06
8	12	6	177.00	60	576.00	5	5	115.00	8.55	17.46	141.01
9	12	6	177.00	70	576.00	5	4	110.00	0.41	9.75	130.97
10	12	6	177.00	80	576.00	6	3	120.70	4.05	5.44	130.19
11	12	6	177.00	90	576.00	7	2	121.60	1.88	3.04	126.52
12	12	6	177.00	100	576.00	8	1	122.00	0.80	1.70	124.50
13	12	6	177.00	110	576.00	9	0	122.30	0.27	0.95	123.51
14	12	6	177.00	120	576.00	10	0	122.30	0.13	0.53	122.96
15	14	6	177.00	60	576.00	3	4	98.40	9.71	17.46	125.58
16	14	6	177.00	70	576.00	3	3	98.40	4.20	9.75	112.35
17	14	6	177.00	80	576.00	4	2	100.57	3.99	5.44	110.01
18	14	6	177.00	90	576.00	4	2	101.54	1.77	3.04	106.35
19	14	6	177.00	100	576.00	5	1	102.00	0.69	1.70	104.39
20	14	6	177.00	110	576.00	5	0	102.00	0.48	0.96	103.42
21	14	6	177.00	120	576.00	6	0	102.16	0.21	0.53	102.90
22	16	6	177.00	60	576.00	3	3	81.94	14.14	17.46	113.54
23	16	6	177.00	70	576.00	4	3	85.17	5.57	9.75	100.49
24	16	6	177.00	80	576.00	5	2	85.17	2.11	5.44	92.72
25	16	6	177.00	90	576.00	5	1	86.44	1.90	3.04	91.38
26	16	6	177.00	100	576.00	6	1	86.94	0.75	1.70	89.38
27	16	6	177.00	110	576.00	6	0	86.94	0.54	0.95	88.42
28	16	6	177.00	120	576.00	7	0	87.14	0.21	0.53	87.88
29	18	6	177.00	60	576.00	3	3	71.58	9.39	17.46	98.43
30	18	6	177.00	70	576.00	3	2	71.58	9.12	9.75	90.45
31	18	6	177.00	80	576.00	4	2	73.67	3.20	5.44	82.31
32	18	6	177.00	90	576.00	5	1	73.67	1.03	3.04	77.74
33	18	6	177.00	100	576.00	5	1	74.40	1.03	1.70	77.12
34	18	6	177.00	110	576.00	6	0	74.66	0.27	0.96	76.87
35	18	6	177.00	120	576.00	6	0	74.66	0.27	0.53	75.46
36	20	6	177.00	60	576.00	3	3	58.90	6.20	17.46	82.04
37	20	6	177.00	70	576.00	3	2	63.26	6.15	9.75	79.16
38	20	6	177.00	80	576.00	4	2	63.26	1.92	5.44	70.62
39	20	6	177.00	90	576.00	4	1	64.62	1.79	3.04	69.45
40	20	6	177.00	100	576.00	5	1	64.62	0.55	1.70	66.87
41	20	6	177.00	110	576.00	5	0	65.00	0.42	0.95	66.37
42	20	6	177.00	120	576.00	6	0	65.00	0.13	0.53	65.66

Table 7-11 Net present value for user cost

Case	Time of first cycle of preventive maintenance	Probabillity of preventive maintenance	Time for essential maintenance	Probability of essential maintenance	Joint proabability (P)	NPV	P(NPV)
1	10	0.0273	60	0.0381	0.00104013	172.48	0.1794
2	10	0.0273	70	0.1249	0.00340977	164.10	0.559552
3	10	0.0273	80	0.2196	0.00599508	159.36	0.955372
4	10	0.0273	90	0.2719	0.00742287	156.79	1.163861
5	10	0.0273	100	0.1882	0.00513786	155.31	0.797968
6	10	0.0273	110	0.1253	0.00342069	154.52	0.528556
7	10	0.0273	120	0.0320	0.0008736	154.06	0.134588
8	12	0.0608	60	0.0381	0.00231648	141.01	0.326643
9	12	0.0608	70	0.1249	0.00759392	136.97	1.040101
10	12	0.0608	80	0.2196	0.01335168	130.19	1.738309
11	12	0.0608	90	0.2719	0.01653152	126.52	2.091574
12	12	0.0608	100	0.1882	0.01144256	124.50	1.424595
13	12	0.0608	110	0.1253	0.00761824	123.51	0.940961
14	12	0.0608	120	0.0320	0.0019456	122.96	0.239234
15	14	0.2333	60	0.0381	0.00888873	125.58	1.116207
16	14	0.2333	70	0.1249	0.02913917	112.35	3.273891
17	14	0.2333	80	0.2196	0.05123268	110.01	5.635951
18	14	0.2333	90	0.2719	0.06343427	106.35	6.745984
19	14	0.2333	100	0.1882	0.04390706	104.39	4.583255
20	14	0.2333	110	0.1253	0.02923249	103.42	3.023364
21	14	0.2333	120	0.0320	0.0074656	102.90	0.768212
22	16	0.3638	60	0.0381	0.01386078	113.54	1.573745
23	16	0.3638	70	0.1249	0.04543862	100.49	4.565931
24	16	0.3638	80	0.2196	0.07989048	92.72	7.407626
25	16	0.3638	90	0.2719	0.09891722	91.38	9.038764
26	16	0.3638	100	0.1882	0.06846716	89.38	6.11989
27	16	0.3638	110	0.1253	0.04558414	88.42	4.03072
28	16	0.3638	120	0.0320	0.0116416	87.88	1.023067
29	18	0.237	60	0.0381	0.0090297	98.43	0.888784
30	18	0.237	70	0.1249	0.0296013	90.45	2.677491
31	18	0.237	80	0.2196	0.0520452	82.31	4.283848
32	18	0.237	90	0.2719	0.0644403	77.74	5.009361
33	18	0.237	100	0.1882	0.0446034	77.12	3.439989
34	18	0.237	110	0.1253	0.0296961	75.87	2.253168
35	18	0.237	120	0.0320	0.007584	75.46	0.572255
36	20	0.0778	60	0.0381	0.00296418	82.64	0.244972
37	20	0.0778	70	0.1249	0.00971722	79.16	0.769224
38	20	0.0778	80	0.2196	0.01708488	70.62	1.206573
39	20	0.0778	90	0.2719	0.02115382	69.45	1.469041
40	20	0.0778	100	0.1882	0.01464196	66.87	0.979171
41	20	0.0778	110	0.1253	0.00974834	66.37	0.64702
42	20	0.0778	120	0.0320	0.0024896	65.66	0.163471
				Expected Net Present Value			**95.63**
				Standard Deviation			**2.24**

Thus, $C_F(t)$ is a discrete random variable which at failure time t_i assumes different values, c_i, as equation 7.15

$$C_F(t) = \frac{c_F}{(1+i)^t} \qquad 7.15$$

With probabilities of occurrence p_i, for a single structure, which can fail only once during T years of service, and when c_F is assumed the same for all possible failure modes, expected cost of failure is defined by the Stewart et al. (2004) as, equation 7.16

$$E[C_F(T) = \sum_{i=1}^{M} p_i c_i \qquad 7.16$$

where M is number of points in time at which the possibility of failure occurrence is considered. An alternative with the minimum expected life cycle cost may then be selected as the optimal alternative, which is included the risk of each alternative in monetary value.

The first step of including failure cost to the decision analysis based on probabilistic life cycle cost is to evaluate failure probabilities of a structure over its service life, which is obtained by a probabilistic time-dependent analysis of the structure taking to into account uncertainties associated with the structural properties and the environmental conditions. The probability distribution of the cost of failure is then necessary to combine with the probability distribution of other variables.

According to (Setunge et. al., 2002) for a single structure with only one possible failure during its service life the probability distribution of the cost of failure with taking into account the discount rate is as shown in equation 7.17

$$f(C_F\begin{cases} P_f(t_i)-P_f(t_{i-1}) & C_F=\dfrac{c_F}{(1+r)^{t_i}} \\ 1-P_f(t_M) & C_F=0 \end{cases} \qquad 7.17$$

where $P_f(t_i)$ is cumulative probability of failure at time t_i (i = 1,2,3,….,M), M the number of point in time at which may occur, t_0 = 0 and t_M denotes the latest possible time of failure. It is assumed that repair/replacement of a failed structure will occur immediately after the structure is inspected. The time between inspections, Δt , is define as $\Delta t = t_i - t_{i-1}$.

The failure cost is very subjective, so it's difficult to calculate. In this study failure cost has not been taken into consideration.

7.2.10 Salvage cost

The salvage cost of the structure often comes equal to the decommissioning (dismantle) cost. So generally salvage cost is not considered in the calculations. In this study it is assumed to be zero.

7.2. 11 Cost of SHM system

Due to unavailability of the SHM system cost data, this cost couldn't be included in this study. But maximum value C_{SHM} can be assumed equal to value of information (VI). As theoretically it should be greater than this.

7.2.12 Total cost calculation

Using Equations 7.3, 7.8, 7.10, 7.12 and 7.13 the total cost can be given as shown in equation 7.18

$$C_{lifecycle} = C_{capital} + \sum_{i=1}^{N_c} \frac{PMC_j}{(1+r)^{j*i}} + \sum_{i=N_c+1}^{N_{pt}} \frac{PMC_j}{(1+r)^{t_j*i}} + \frac{EMC}{(1+r)^{t_e}} + \sum_{i=1}^{N_{pt}} \frac{UCPM_j}{(1+r)^{j*i}}$$

$$+ \sum_{i=1}^{N_{pt}} \frac{UCPM_j}{(1+r)^{t_j*i}} + \frac{UCEM}{(1+r)^{t_e}} \qquad 7.18$$

$$C_{lifecycle} = 443.16 + 94.18 + 95.63 = \$632.97 / m^2$$

The sensitivity analysis is not necessary in this case as all the probabilities associated with the maintenance and user costs, and their application time have been considered. The $C_{lifecycle}$ is a linear function of $C_{capital}$, so it increases with $C_{lifecycle}$.

Chapter 8

Summary, Conclusions and Recommendations

8. 1 Summary and results

The present research work leads to proposing a methodology, first, to use SHM information to assess structure's condition, and then evaluation of expected life cycle cost based on this assessment.

SHM is emerging as a promising technique to assess the structure (in this case bridge) behaviour more precisely. It's a powerful tool for better understanding of bridge condition during its service life.

As discussed that present bridge management systems are not adequate as they are mainly based on visual inspection and, they generally don't consider the history of bridge maintenance actions. Many researchers have suggested that these problems can be overcome using reliability based maintenance approach. The reliability of structure is indication of probability of failure. This probability of failure can be calculated by accounting structure's resistance and load moment applied during its service life. As structure's resistance is property of strength of materials used it degrades as time passes. This degradation of resistance can be modeled using SHM information.

In past, researchers have proposed many deterministic and probabilistic approaches to evaluate life cycle maintenance cost of a structure. Using reliability based maintenance strategies these costs can be calculated more accurately. This study proposes a structure's resistance degradation model based on available SHM information for a bridge type, and later develop an approach to evaluate life cycle maintenance cost for the bridge.

The Finite Element model of the Crowchild Bridge has been used to generate the information. This model contains 351 elements, 247 nodes and 1399 active degrees of freedom. The density of steel and concrete is assumed to be 76 and 24 kN/m3, respectively. The concrete compressive strength is taken as 35 MPa. The modulus of elasticity for concrete is assumed to be 30 GPa for the deck and 27 GPa for the barrier and pier; for steel it is assumed to be 200 GPa. In this study FEM has been validated against the field test data. The accuracy of the method, certainly, depends on how accurately finite element model simulate the real bridge conditions. In 1997 static load test were performed on the bridge by ISIS Canada. In this test the deformation values were recorded on particular points under 9 different load conditions. During the test two trucks were used for the first six load conditions and later 3 conditions had one truck load. One truck was represented by 10 point loads. The same load conditions have been simulated using FE model. The results show that maximum deformation values correspond to the 6th position. The Table 4-2 shows that the field test deformation values and FEM deformation values are in the agreement.

Using the FE model the stiffness of the bridge deck is reduced and the deformation values on certain nodes have been noted down. This is done to simulate the real behaviour of the bridge when it is undergoing to deterioration as in this study it is assumed that stiffness is general function of bridge deterioration. The stiffness value for bridge deck has been reduced up to 25% on interval of 5%. This process provides a set of data which consist of deformation values on certain nodes for a particular value of stiffness. In a real situation SHM system provides the deformation on certain points (where the sensors have been installed) but doesn't give any information explicitly about stiffness or degradation. But this deformation values can be used to estimate the degradation of the structure. In this study ANN has been used for estimating the stiffness degradation using the measured values of displacement at the censor locations. It has 25 neurons in input layer, which correspond to 25 nodes where deformation values were taken, and one neuron in output layer for stiffness. The ANN has been trained using the data set generated from FEM and also validated against field data. The difference between actual stiffness value and value calculated using ANN is 5%.

The trained ANN can be used to calculate the stiffness of bridge deck using the deformation values at any given point of time. In this study, the stiffness is calculated using field test deformation values and it comes out to be 0.95K0 , where K0 is the original stiffness (without any degradation). As it is assumed that degradation in the bridge deck is directly proportional to the stiffness degradation, the ultimate moment capacity can be calculated by multiplying it with K_t/K_0, where K_t is the stiffness at any given point of time. In this case ultimate moment capacity is 95% of the designed capacity. A limit state equation has been developed for the Crowchild bridge deck using ultimate moment capacity, dead load moment, and live load moment. Live load moment calculation has been done using Nowak live load model. This limit state equation is used to calculate the probability of failure of the deck at any given point of time. In order to do that Monte Carlo Simulation has been used in this research. A program in C++ has been developed to perform the simulation. The live load on the bridge has been considered for 50 years. The reliability index β, an indicator of probability of failure, for the Crowchild bridge deck has been computed to be 5.1.

The ultimate purpose of this research lies in the incorporation of this new available information (SHM data) with the previously available information. To make the best use of SHM based information, the prior information based on the statistical data of the bridge inspection should be updated. In order to achieve it the Bayesian approach has been used. An illustration has been taken to quantify the effect of Bayesian updating. Updated information, in this study updated probabilities of expected maintenance and rehabilitation actions, provides better understanding of structure's behaviour and condition. The reliability of updated information, eventually, depends on the accuracy of the source (SHM system) of the information. Higher degree of sophistication SHM system has, more accurate information is obtained. But more sophisticates system cost more. It leads to the life cycle cost calculation in order to show the significance and benefits of SHM system.

To quantify the effect of updated information on the whole life cycle cost of structure a comparison between expected costs evaluated based on un-updated and updated information has been done. The life cycle cost analysis has been performed considering Initial cost, Maintenance, monitoring and repair cost, and user cost. The failure cost hasn't been included in this research because of the highly subjective nature of it. None of these costs is deterministic except initial cost. It is found in the literature that steel free bridge deck is not always costlier than the reinforced one. But the difference varies between -30% to + 60% of the cost the reinforced bridge deck. In this study the initial cost for the steel free bridge deck is found around $443.16/m^2$.

The steel free bridge deck may be most of the time costly in terms of initial cost than the reinforced bridge deck. But the maintenance and rehabilitation schemes for this type of decks are inexpensive than the traditional one. One of the main reasons for this is absence of steel. The main reason of degradation in reinforced bridge deck is corrosion, and as steel free bridge deck doesn't have steel so the chances of corrosion are very less. Other factor in maintenance and rehabilitation cost is type of maintenance work. Literature shows that if the preventive maintenance is performed the overall cost of preventive and rehabilitation maintenance reduces significantly. The maintenance and rehabilitation cost (if preventive maintenance performed) is calculated $ 94.18/m2 with standard deviation of 2.22.

The calculation of the user cost is complex as it is very subjective in nature. In this study user cost is calculated $95.63/m2 with standard deviation of 2.24.

8. 2 Conclusions

The main objective of this research was to develop a methodology to use SHM information, and on the basis of this information to plan the maintenance and rehabilitation strategies. The commission which was setup to investigate causes of Laval overpass collapse, Montreal has also recommended to make inspection and monitoring methods more reliable using the emerging technologies such as SHM system.

The Finite Element Model of the Crowchild Bridge is found to be very useful to simulate the real bridge conditions. Once the model is validated against the field data, it provides flexibility to analyze the effect of degradation under the different load conditions. One can try different load conditions with various extent of the degradation in order to have an idea about bridge behaviour under different scenarios. In this study the deformation values provided by SHM system have been used to study the effects of the loads. The maximum deformations in the studied section of the bridge deck have been noted for the 6th position. Further, it has been validated against field static load deformation and as shown in Table 4-2 they are found to be in agreement.

The degradation is simulated by reducing the stiffness of the bridge deck as it is assumed to be a general function of stiffness reduction. To interpret the SHM data it is imperative to remove redundant and useless data. One of the suggested methods in literature advocates the data should be recorded when it crosses a particular limit. For example in this study the deformation values are recorded for the 6th position as maximum deformation corresponds to this position. So critical live load moment will be created by this load position and which will further contribute in bridge (bridge deck) deterioration. After intelligent data (deformation values in this case) collection the extraction of the information about bridge condition from it, is essential. The ANN is found to be useful to serve this purpose. The bridge deck stiffness output from the ANN model has only 5% difference with original bridge deck stiffness. So once again it proves the validity of Finite Element Model of the Crowchild Bridge as well as accuracy of ANN model.

Once the stiffness of bridge deck is known it can be used to perform reliability analysis on the deck. The ultimate moment capacity of bridge deck is assumed to be proportional to the stiffness, so the reduction in bridge stiffness represents reduction in the ultimate moment capacity. The probability of failure for the Crowchild bridge deck has been calculated to 1.7 x 10-7 which corresponds to reliability index 5.1. In literature it's stated that the target reliability for non redundant bridges is 3.5. Other important thing to be noted is the bridge deck is constructed using FRPs so the ultimate moment capacity is much higher compare to the traditional steel reinforced. However, because of the lake of the information about moment capacity of FRP materials the ultimate moment capacity of steel reinforced is considered in this research. This makes it a bit conservative estimate of probability of failure. Other than that the dead load of FRP bridge deck is considerably lower than the steel reinforced one. Hence, dead load moment of the former is lesser than the later which results in further decrease in probability of failure.

This information is beneficial for decision makers to plan maintenance and rehabilitation actions. But this information serves its purpose at the best when incorporated with the previous available information. The visual inspection is widely used method of monitoring in the bridge community as it is easy and fast to perform. So a lot of knowledge about bridges' conditions is contained in the form of visual inspection data. Using Bayesian approach this previous information is updated in this study. The example taken this study shows that the probability of first maintenance at 0^{th} year increases from 0.3 to 0.945 when updated based on SHM test output of rehabilitation should perform at 0^{th} year. The accuracy of SHM test is considered 85%. The more accuracy SHM test has, the more precise information about maintenance and rehabilitation actions is obtained. The joint probabilities of different rehabilitation and maintenance actions also get changed. The decision regarding whether to implement SHM system or not can be made using the value of information concept. Though practically it's recommended irrespective of the cost as it is concerned with users' safety and that is utmost important.

The life cycle cost analysis includes mainly four costs: capital cost, rehabilitation and maintenance cost, user cost, and failure cost. Mostly the capital cost is easy to calculate as it occurs at the base time of project. Modeling of future maintenance and rehabilitation cost, and user cost is complicated as so many subjective factors are involved in calculation. The user cost also depends on the maintenance and rehabilitation strategy, if preventive maintenance are preformed on the structure the over all user cost is less compare to when preventive maintenance are not performed. In this study user cost is found to be more than the maintenance and rehabilitation cost. The maintenance and rehabilitation strategy chosen in this research is with preventive maintenance. In case of FRP bridge deck it's important to note that though capital cost may be higher than the conventional steel reinforced but over the life maintenance and rehabilitation cost are much lesser. It is due to the fact that main reason of deterioration in bridge deck (or in bridge) is corrosion and FRP bridge deck is steel free. According to this study the maintenance and rehabilitation cost contributes 14.88% of total life cycle cost while the user cost shares 15.12% and rest of it is the capital cost.

8. 3 Recommendations and future work

Recommended future of this research can be described as follows:

Current study enhancement area:

- Incorporate more SHM data like frequency, load, strain, temperature, acceleration etc, to calculate stiffness of structure.
- Cost of SHM system should also be considered.
- Bayesian updating should be done using Numerical Integration method.
- Evaluation of failure cost.

Current study extension area:

- Development of degradation model.
- Incorporate the time variant reliability analysis.

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