PROBABILISTIC LIFE CYCLE COST OPTIMIZATION OF BRIDGES

by

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A dissertation submitted to the

Graduate School-New Brunswick

Rutgers, The State University of New Jersey

In partial fulfillment of the requirements

For the degree of

Doctor of Philosophy

Graduate Program in Civil and Environmental Engineering

Written under the direction of

Professor Hani H. Nassif

And approved by

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New Brunswick, New Jersey

OCTOBER 2013

ABSTRACT OF THE DISSERTATION

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The main goal is to develop time-dependent bridge deterioration curves and to perform a Probabilistic-Bridge-Life-Cycle-Cost-Optimization (BLCCO-p) using them. Currently, a commonly accepted methodology considering the bridge as a system with all of its components doesn't exist according to a report by the National Cooperative Highway Research Program published in 2003. Therefore, research is needed to develop a methodology for the BLCCO taking a systems approach and ensure that the most appropriate course of action is taken for bridge improvements.

Bridge deterioration curves are developed here based on nonlinear optimization analysis of the **N**ational **B**ridge **I**nventory (NBI) database using Markov-Chain process. Most of the existing models consider only the age of the bridge and the **A**verage **D**aily **T**raffic (ADT) and disregard other important factors such as climatic regions, bridge length and material type in formulating the objective functions. The deterioration curves developed in this dissertation categorize each bridge according to their climatic region, length, ADT and material type. Bridge deterioration curves are formed for superstructure, deck and substructure of the bridges. This deterioration is simulated by a Markov-Chain process. Unexpected event (seismic, scour, etc.) occurrences are also considered.

In the BLCCO model, which is essentially a set of economic principles and computational procedures to obtain the most economical strategy for ensuring that a bridge will provide the services for which it was intended, bridge deterioration curves are used as the decision tool for the repair or the replacement of the bridge. In addition, detour traffic analysis, cost and effectiveness of rehabilitation or replacement activities, user, accident, agency costs and discounting models are included. The BLCCO is formulated as a mixed-integer nonlinear optimization model.

The BLCCO model uses genetic algorithm to reach the optimal total cost and Monte-Carlo simulation as a risk analysis technique to do the optimization probabilistically (BLCCO-p).

Additional major outcomes of this dissertation are (1) updating of deterioration curves using Bayesian methodology, (2) a combined methodology for reliability index and bridge condition index to be used in deterioration models, (3) performing a parametric study for BLCCO using traffic volume growth, discount rate, user cost weight, probability of unexpected events.

Keywords: Bridge, Deterioration, Nonlinear Optimization, Markov Chain, Monte Carlo Simulation, Bridge Condition Rating, Probabilistic Life Cycle Cost Analysis, Risk Optimization, Genetic Algorithm.

Acknowledgements

I wish to express my sincere gratitude to my advisor Professor Hani H. Nassif and Professor Kaan Ozbay for their guidance and continued support during my studies. I would also like to thank the other members of my Ph.D. committee who took effort in reading the thesis and provided me with valuable comments: Professor Husam Najm and Professor David Coit.

I would like to thank my wonderful daughters and my wife, who are my precious pearl drops, for their support. I also would like to thank my mother and father for their continuous encouragement.

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Chapter One

INTRODUCTION

1.1. Problem Statement

Current regulatory requirements recognize the benefits of Bridge Life Cycle Cost Analysis (BLCCA) and point its importance for the infrastructure investments, including the highway bridge program (FHWA (2003)). The Transportation Equity Act for the 21st Century (TEA-21) defines Life Cycle Cost Analysis (LCCA) as *"a process for evaluating the total economic worth of a usable project segment by analyzing initial costs and discounted future costs, such as maintenance, user, reconstruction, rehabilitation, restoring, and resurfacing costs, over the life of the project segment."* (FHWA (2011)).

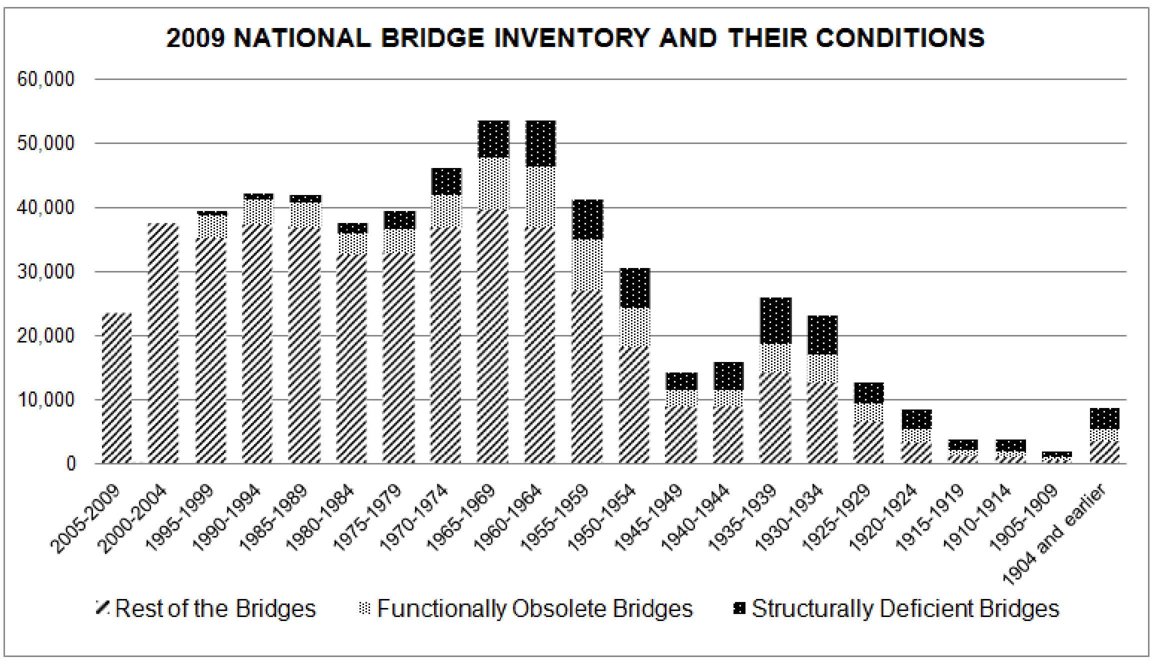

Figure 1.1a: Nationwide Number of Structurally Deficient and Functionally Obsolete Bridges

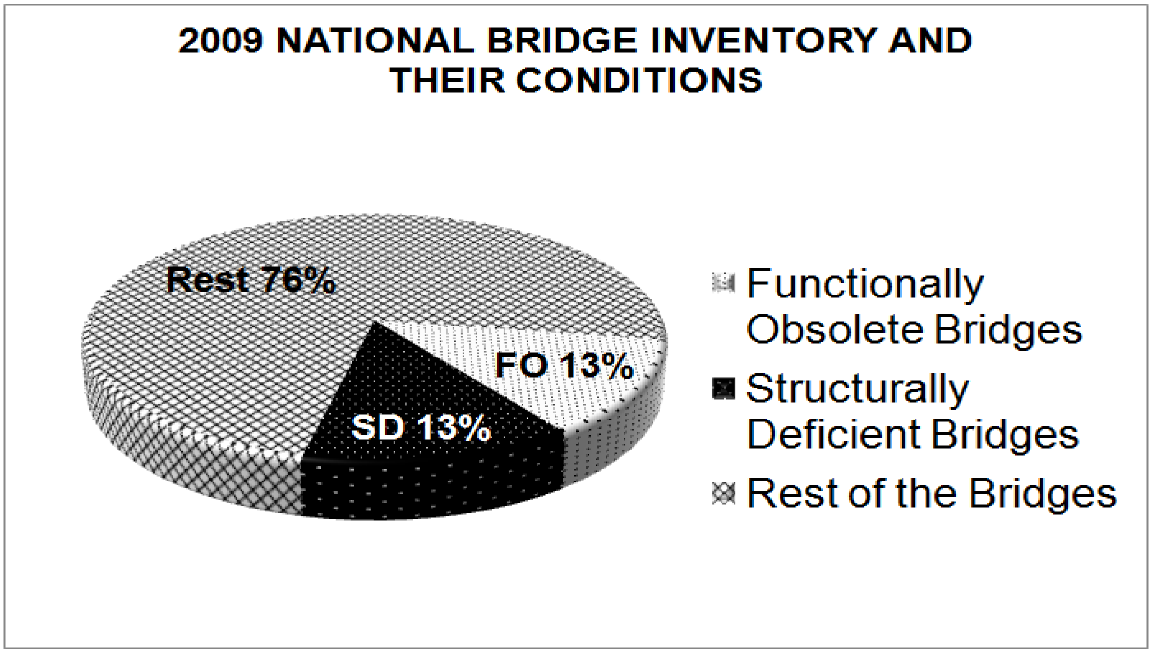

Figure 1.1b: Nationwide Percentage of Structurally Deficient and Functionally Obsolete Bridges

U.S. National Bridge Inventory's (NBI) 2009 data indicates that, of the 603,309 bridges nationwide, 13% are structurally deficient and 13% are functionally obsolete, which means that 26% of the nation's bridges need replacement or major repairs (Figures 1.1a & 1.1b). It, of course, takes millions of dollars to replace and maintain these older bridges, and bridge life cycle cost optimization (BLCCO) methodologies are considered to be invaluable tools for allocating relatively scarce public resources efficiently. Therefore reliable optimization techniques and algorithms are needed for BLCCO. Bridges are the single most expensive elements within our transportation system. Therefore, it is imperative to develop an efficient BLCCO methodology for these highway assets.

1.2. Research Objectives and Approach

Bridges constitute a special class of structures that are influenced by a continuously changing load and environmental conditions. Due to these conditions bridges are subjected to deterioration more than the other structure types. Considering the increasing rate of traffic and deterioration rates, every bridge will need rehabilitation repair or reconstruction in its life time. Therefore, a BLCCO algorithm needs to be developed to utilize all the resources in a cost effective manner for the future actions.

Currently, there is no consensus on the required level of detail in performing a BLCCA concerning the number of elements that should be studied to attain a certain level of accuracy. However, some analysts, as mentioned in FHWA (2003), suggest that considering three elements (substructure, deck, and superstructure) yields an adequately detailed description of most highway bridges. The majority of the previous studies however, focused more on developing algorithms for one individual element of the bridge such as deck, superstructure, substructure or pavement, or the bridge itself as one element, rather than all the components as a system.

The work presented in this research expands the current state-of-the art in several important ways, by:

- Utilizing 18 years of NBI data from 1992 to 2009 for all the bridges in the Northeast Region of US to validate the used deterioration model and utilizing an extensive data set; 603,309 bridges obtained from the year 2009 NBI Database for developing the deterioration curves
- Taking into consideration a variety of parameters including the superstructure material, bridge length, ADT, climatic conditions, for the development of deterioration curves
- Having flexible deterioration curves, which can be updated with the availability of reliable bridge deterioration data

- Incorporating unexpected events (seismic, scour, terrorist attacks) into the bridge deterioration model
- Showing the two bridge deterioration models (bridge condition rating based and bridge reliability based) are not different than each other
- Combining the deterioration, cost, and optimization models (superstructure, substructure, deck, and pavement) and performing BLCCO
- Performing probabilistic BLCCO (BLCCO-p) by introducing Monte Carlo simulation into the optimization to take into account the uncertainties
- Performing detailed sensitivity analyses for BLCCO-p and determining the most sensitive parameters
- Performing parametric study for BLCCO using the sensitive parameters

The deterioration model used is based on nonlinear optimization using Markov Chain analysis of the Structure Inventory and Appraisal data (SI&A). A total of 216 different deterioration curves which are represented by Markov vectors are prepared considering the bridge members, superstructure material, bridge length, ADT and climatic conditions.

In order to develop the BLCCO-p, a probabilistic mixed-integer nonlinear optimization model, based on economic theory of life cycle cost analysis, which combines the dynamic engineering and economic models over the lifetime of the bridge, is used. The BLCCO-p uses genetic algorithm to reach the optima and Monte Carlo simulation as a risk analysis technique. Adding Monte Carlo simulation to the optimization model enhances the power of BLCCOM-p by considering the uncertainties that are available in the model and carry out the optimization probabilistically.

1.3. Dissertation Outline

Chapter 2 reviews the previous studies in the literature that address bridge deterioration modeling and BLCCA studies. The techniques for dealing with these important issues have long been studied in the literature and there is abundant number of studies that deal with the subject. The literature search yielded many documents related to the evaluation and rehabilitation of the bridge structures nationwide. The most related studies are reviewed in Chapter 2 in detail. A table representing the history of BLCCO through the literature review is also presented in this chapter.

Chapter 3 deals with choosing and improving of a bridge deterioration modeling system using Markov Chain process. It shows the comparisons of the simulated deterioration curves with the real life observed deterioration of bridges through their condition ratings and it presents the deterioration curves (total of 216) in Markov vector format.

This chapter further shows different implementations of the deterioration models.

- Bayesian updating of the bridge deterioration models whenever there is reliable bridge data.
- Application of unexpected events (seismic, scour, terrorist attacks) into the deterioration model.
- A study to relate bridge condition rating with the bridge reliability index.

Chapter 4 presents BLCCOM. The flowcharts, methodologies and techniques used are mentioned in details. BLCCO is applied on a bridge in NJ to demonstrate how the optimization process works.

Chapter 5 shows BLCCO-p application. Again the flowcharts, methodologies and techniques used are mentioned in detail. BLCCO-p is applied on a bridge in NJ to show how the optimization process works. Sensitivity analysis and a parametric study are performed on BLCCO-p.

Findings, a summary of the proposed research and possible future research directions are presented in Chapter 6.

Chapter Two

LITERATURE REVIEW

In this chapter, a review of the existing studies that deal with the bridge deterioration problem and the lifetime performance analysis is presented. The literature search yielded many documents related to the evaluation and rehabilitation of the bridge structures nationwide.

2.1 The Literature on LCCA

Hassanain et.al (2003) covered the literature review for the last 10 years for bridge LCCA for concrete bridges. The authors concluded that there needed for more research in this area.

Singh et.al (2005) presented a detailed procedure for developing a framework for life cycle cost analysis (LCCA) of highway bridges in Myanmar. The paper discussed various cost components and other statistical factors that need to be taken into consideration while assessing the life cycle cost (LCC) of a highway structure. A stepwise procedure to determine various cost components that come into LCC calculation was also illustrated. No such models capable of predicting the deterioration frequency had been developed for the bridge stock in Myanmar. So, the authors assumed that due to high durability of Grade 45 PCC used for bridge decks, the initial defect free period of the bridge would be extended from 20 to 25 years and after that time they calculated the user costs according to major maintenance season average daily traffic data (MSADT). The study has made a call for the development of comprehensive life cycle costing framework for transportation-related projects in Myanmar in order to be able to strike a balance between the need for maintenance and replacement of highway structures and limited funds available for their upkeep. The approach presented in this paper was somewhat crude and imprecise. If viewed from proper perceptive, i.e., a decision support tool to be considered with engineering judgments and other factors, even a crude LCCA based on educated professional guesses usually leads to better decisions than no LCC considerations at all.

2.2 The Literature on Deterioration Model

Roelfstra et.al (1999) proposed improving deterioration predictions of bridges by using a segmental approach, accurate deterioration models and non-destructive testing for Swiss Federal Institute of Technology. According to the author, the main deterioration mechanism was chloride-induced corrosion. Fick’s second law (law of mass conservation) was used to simulate diffusion of chloride through the concrete to the steel rebars. After that, 3 different corrosion speeds of concrete were used due to observed field conditions (good, average, bad) as a deterioration model. According to the author three results were concluded from this study :

- Non-destructive tests give quantitative values from which to predict the condition evaluation of the bridge.
- Chloride corrosion initiation time has a larger influence on the service life of the structure than the corrosion speed of the steel.
- Deterioration curves are discontinuous. The safety coefficients have several plateaus due to multiple load paths.

Chase et.al (1999) proposed to development of bridge deterioration models using nonlinear optimization analysis. Using the expanded data sets available from the combined NBI and GIS databases, three different methods were applied to model the relationship between condition state and plausible factors causing deterioration. The variables included in the study were age, ADT, precipitation, frequency of deicing, temperature range, freeze-thaw cycles and type of bridge construction. Different models were developed for deck, superstructure and substructure deteriorations. Generalized linear models, generalized additive models and a combination of the two were applied. According to the author the generalized linear model gave the best prediction.

Chase et.al (2000) proposed a system to improve the software Pontis, which was the predominant bridge management system at that time, implemented in the US, considered load-carrying capacity as static during an incremental benefit-cost approach, by modeling the interrelationship between the load rating and physical bridge deterioration. The approach was based on a combination of regression analysis and deterioration modeling using Markov chains. The method was applied to historic bridge data from the US (409,741 bridges from NBI database were included in the analysis) and Hungary.

Madanat and Mayet (2002) worked on incorporating seismic hazard and risk analysis considerations, which are concerned with the occurrence of earthquakes and the vulnerability of structures, into bridge management systems. They developed a decision model for optimizing bridge management policies that takes into account the occurrence of earthquakes. The model that they presented was not meant to be very detailed or comprehensive, but rather to allow others to obtain qualitative implications of seismic considerations in bridge management systems.

Akgul and Frangopol (2004) came up with a deterioration model for steel members of steel girder bridges. They analyzed the life-time performance of the bridges with the deterioration model. For time-variant performance analysis, special emphasis was placed on the corrosion penetration modeling in the girders. They attempted to adopt such models to the atmospheric and environmental conditions of Colorado. For their steel deterioration model, the authors used a nonlinear equation which they obtained from Townsend and Zoccola (1982). They also mentioned that the deterioration equation might not give accurate results at the end of the service life of girders due to under estimation of the corrosion rate. The focus of the paper was on the formulation and the overall methodology rather than the analysis of the results.

Another study by Akgul and Frangopol (2004) investigated the time-dependent relationship between the reliability-based analysis results, representing the future trends in bridge evaluation, and the load ratings for different types of bridges located in an existing bridge network. The comparisons between live load rating factors and reliability indices were made over the lifetime of each bridge in the network. The rating–reliability profile and rating–reliability interaction envelope concepts were introduced. Furthermore, the rating–reliability profiles were collectively examined in order to evaluate the time-dependent performance of the overall bridge network. The study demonstrated that it was possible to predict the load rating and reliability index of a bridge using live load and resistance deterioration models integrated into a single computational platform.

Roelfstra1 et.al (2004) proposed a mathematical chloride induced corrosion deterioration model for the bridges in Switzerland. The authors compared the deterioration model with Markov chain model which forecasted the condition states of any given element at any given time. Markov chains were used in KUBA-MS to represent condition evaluation and the transition probabilities were determined using regression analysis of pairs of inspections. Five different condition states were used which were defined in Swiss bridge management system (Good, Acceptable, Damaged, Bad Condition, Alarming). For the chloride diffusion to concrete, Fick's first law was used and mass of the corroded reinforcement was calculated by the Faraday Equation. Numerical simulations of the condition evaluation for different values of model parameters were performed. The simulation results have been mapped to condition states as defined in KUBA-MS and Markov transition matrices have been calibrated to fit simulation results. According to the authors, it was concluded that it seemed feasible at least in the foreseeable future to continue to use Markov chain models in BMSs due to the complexity of mathematical models that were developed.

Akgul and Frangopol (2004), presented a computational platform for predicting the life time system reliability profile for different structure types located in an existing network. The computational platform had the capability to incorporate time-variant live load and resistance models. Following a review of the theoretical basis, the overall architecture of the computational platform was described. Finally, numerical examples of three existing bridges, a steel, a prestressed concrete, and a hybrid steel-concrete bridge located in a network, were briefly presented to demonstrate the capabilities of the proposed computational platform.

Neves et.al (2004), used the reliability index as a measure of structural performance. The time dependent reliability index and the effect of maintenance actions were described using a model. Inspite of the importance of the cost of a maintenance action and of its effects on the reliability index, there was very limited information on the relation between the cost and the effect of maintenance actions. A model considering the interaction between maintenance cost and its effect on the reliability index was proposed. This model was used to compare the cost effectiveness of several maintenance strategies for a deteriorating structure. The effect of the parameters associated with the cost model on the optimal maintenance scenario was also analyzed.

Thompson et.al (2005) performed a preliminary analysis on the California bridge data set, to quantify the deterioration transition probabilities actually observed, and to determine whether it is yet possible to validate the key assumptions of Markovian bridge deterioration models. The authors obtained inconclusive results from their study and concluded that in the short term, the results indicate the necessity of continuing to use deterioration models developed from expert elicitation, not switching to historical-based models unless maintenance activity is reliably included in the database.

Mei et.al (2005) investigated the utility of the 2001National Household Travel Survey Kentucky standard and add-on samples for statewide, rural county and small urban area travel demand modeling. The weaknesses of the Kentucky standard sample for deriving trip rates and average trip lengths were identified, which included greater uncertainty caused by a small sample size and suspiciously low trip rates for urban clusters (urban areas with less than 50,000 population). It was shown that the Kentucky add-on sample could be used to enhance the Kentucky standard sample for developing trip rates and average trip lengths. Combining the two samples using Bayesian updating resulted in improved trip rates and average trip lengths.

Furuta et.al (2005) evaluated LCC for road networks by observing the seismic risk. RC bridge piers were considered as analysis models. Earthquake occurrence probabilities were calculated using the hazard curve, and the damage probability was calculated using the damage curve. The seismic risk was then defined in relation to the expected loss due to the earthquake. Through numerical examinations, the following conclusions were derived:

- The damage degree was defined by using the maximum response displacement obtained by the dynamic analysis and the horizontal force and displacement of RC bridge piers.
- Through the LCC calculation of several representative road net-works, it was found that differences in road networks greatly influence the seismic risk.
- By comparing the cases with and without user costs, it was clear that the effect of seismic risk was small if user cost was not considered; however, it became quite large if user cost was considered.
- Examining the change of LCC with respect to the change of the maximum acceleration of the earthquake showed that the seismic risk decreased as the maximum acceleration increased.
- Effect of the discount rate was examined by varying its value. It was found that the discount rate has a large influence on the estimation of LCC, implying that it was very important to determine an appropriate discount rate in the calculation of LCC.
- Examining the effect of damage degree showed that the medium damage level had a large ratio, 45%, whereas the severe and rather severe damage levels were at 28% and 27%, respectively. In this study, only bridge piers were considered in evaluating LCC, since they sustain the most damage when earthquakes occur. Results obtained in the study could be extended to the entire bridge model; it was

not difficult to consider the initial construction cost and damage cost of components other than bridge piers, because they were only slightly damaged. It was necessary to examine more road networks with different road characteristics to investigate the effects of detour, road network, and damage degree.

Agrawal et.al (2009) worked on a bridge deterioration model using bridge inspection ratings to calculate deterioration rates of bridge elements by considering effects of environmental (climate, ice, salt), geographical (highways below or above bridges), ownership, material types, design types, and other factors. They used Markov Chain and Weibull Distribution based approaches. The authors mentioned that they would perform further investigations to obtain deterioration rates of different bridge elements affected by factors such as climate, bridge element materials, average daily truck traffic, ice, salts, etc. This project was done for New York State Department of Transportation. They concluded that, since the Weibull-based method utilizes actual scatter in duration data for a particular rating and considers this duration as a random variable, it has been found to be more reliable for calculating deterioration rates for bridge elements. Hence, deterioration curves and equations using the Weibull-based method have been generated and were presented for use.

2.3 The Literature on both LCCA and Deterioration Model

Mohammadi et.al (1995) proposed a new method called the VI model, considering bridge age, condition rating and cost which were mentioned in this study as the most important factors in decision making process. A parameter referred to as the value index (VI), was introduced to incorporate these variables for the optimization strategy. This enabled rational decisions to be made regarding the type of work to be performed that best suited a bridge’s needs within the appropriate constraints. The “objective function”, the function to be optimized, was written in terms of the key factors that control the decision making process. This method was applied to several case studies for highway bridges in Illinois. As a deterioration model for deck, superstructure and substructure, the models that were recommended from Transportation System Center (TCS) of the U.S. Department of Transportation were used. These deterioration curves were linear and all depended on age and ADT of the bridges. The solution of the Life Cycle Cost optimization was carried out by numerical means. According to the author, the model developed in the study could be used to make decisions on the time of the bridge works within a designated life cycle.

Liu et.al (1997a & 1997b) proposed an optimization method for the bridge deck rehabilitation. As a deterioration model, the authors used a nonlinear equation representing the deterioration at bridge deck. The deterioration level of bridge deck was assumed to vary between 0 and 1, where 0 represents like new condition and 1 represents potentially hazardous condition. As an optimization method, Genetic Algorithm (GA) was used. According to the authors, it was found that this method can find a satisfactory optimal set in a short calculation time.

Enright et.al (1999) considered using Bayesian techniques in a way that the information from both inspection data and engineering judgment can be combined and used in a rational manner to better predict future bridge conditions. In their study, the influence of inspection updating on time-variant bridge reliability was illustrated for an existing reinforced concrete bridge. Inspection results were combined with prior information in a Bayesian light. The approach was illustrated for a reinforced concrete bridge located near Pueblo, Colorado. For this bridge, the effects of corrosion initiation time and rate on time-variant strength were illustrated using simulation. Inspection results were combined with prior information using Bayesian updating. Time-variant bridge reliability computations were performed using a combined technique of adaptive importance sampling and numerical integration. The approach allowed accounting for inspection results in the quantitative assessment of condition of bridges and demonstrated how to incorporate quantitative information into bridge system and component condition prediction.

Frangopol et.al (1999) proposed a completely new bridge management decision system. According to the authors, most of the existing bridge management systems were based on visual or subjective condition assessment, and did not predict optimum maintenance requirements based on balancing life-cycle cost and bridge system reliability requirements. The authors proposed a methodology by integrating maintenance, repair and replacement decisions in bridge management based on reliability, optimization, and life-cycle cost. In the paper, the framework of the methodology was provided and the approach was illustrated for both new and existing highway bridges. For deterioration of steel and concrete, simply deterioration rates per year were assumed. For the optimization, several life cycle cost scenarios were tested and the optimum was chosen. The benefits of the new proposed system were summarized as given below:

- Identification of total life-cycle cost associated with maintaining a bridge at or above a target reliability level.
- Identification of maintenance strategies, which minimize total life-cycle cost and satisfy reliability constraints.

- Establishing future bridge maintenance needs based on safety and serviceability rather than on the visible condition state of the structure.
- Providing a rational basis for prioritization of bridge maintenance fund allocations.

Neff (1999) presented a methodology for incorporating statistical reliability considerations into corrosion service life prediction and life cycle cost analysis that was developed as part of a FHWA study of corrosion resistant reinforcement. According to the author this approach gave the engineer the ability to statistically consider different material, environmental, structural, and corrosion protection factors in computing the life cycle costs, and was applicable to any corrosion protection system. According to the author, if viewed in the proper perspective (i.e., as a decision support tool to be considered with engineering judgment and other factors), even a crude life cycle cost analysis based only on educated professional guesses will usually lead to better decisions than no consideration of future costs at all.

In his Master Thesis (2001), Pratik Roychoudhury developed a prototype life cycle cost-estimating model for FRP bridge decks called “bridgeMATE”. The model considered the agency costs, user costs and the third party costs to establish the life cycle cost of an FRP bridge deck. The model featured two condition indices for maintenance and for repair that could estimate the condition of the bridge deck in any given year. Based on the limiting values set on the indices, a decision for maintenance and repair was made when the index value for a year exceeds the limiting value. The model theoretically estimated the deterioration rates of FRP bridge decks based on the physical and chemical properties of fiber reinforced polymers. Two kinds of repair strategies were considered, one putting an overlay on the bridge deck and the other was replacing the deck. Based on the repair strategy selected, both of the condition indices were reduced to represent the improvement in the quality of the bridge deck. Using this deterioration model, LCCA was performed considering User and Third party costs besides the Agency costs. The sensitivity analysis revealed that the life cycle cost of the FRP bridge deck is greatly affected by the Initial Fabrication cost and the Initial Construction cost. These constitute about 75% of the life cycle cost. In order for FRP bridge decks to substitute concrete bridge decks, the most important factors to be minimized were the fabrication cost and the initial construction cost, since they were the main cost drivers of the life cycle cost.

Estes and Frangopol (2001) proposed a probabilistic framework for optimizing the timing and the type of maintenance over the expected useful life of a deteriorating concrete deck of a bridge with a 45-year service life. The approach focused on the likelihood of events occurring, defined the decision points and used the best information available at the time. As a deterioration model, the authors used nonlinear equations dependents on chloride induced corrosion of the steel. Four different cases of probability of replacing deck was formed and previous inspection results were used for the decision process. For the optimization model, all possible alternatives were formed using the deterioration model and the probability of deck replacement for the deck replacements at years 10,19 and 35 and the optimum result was chosen. According to the authors, additional research was needed in the areas of quantifying the probabilistic capability of inspection techniques, probability of making repairs and modeling of deterioration.

Zayed et.al (2002) made a life cycle cost analysis comparison of steel bridge paint systems between deterministic and stochastic methods. By using paint deterioration systems, the authors applied both deterministic (Economic Analysis “EA”) and stochastic (Markov Decision Process “MDP”) methods on INDOT and MDOT bridge data. For Economic Analysis they used a paint deterioration system developed using regression and for Markov Decision Process they used a paint deterioration system developed using Markov chains. According to the authors, EA was superior to MDP for INDOT data and they gave same performance for MDOT data. Also the EA method’s advantages (simple to use and understand, applied widely) were offset by the MDP’s ability to incorporate the inherent stochastic nature of the phenomenon being modeled.

Ozbay and Javad (2003, 2004, 2005, 2006, 2007) and a dissertation study carried out by Jawad (2003) focused on a topic very similar to the one studied in this dissertation, with application on highway pavement structures. These studies involved the development of a hybrid model for optimizing life cycle cost in transportation infrastructures, particularly for pavement structures at the project level using genetic algorithms as the search tool for arriving at the optima. Major aspects of the research presented in those studies included the investigation of methodologies of applying LCCA to project decision-making process and utilization of LCCA as an economic evaluation technique to achieve life cycle cost optimization. The authors conducted a multi-stage direct survey of the LCCA practices in all State departments of transportation (DOTs). A literature review of technical reports and engineering journals was also carried out. Those studies also provided an insight into the discount rate to be used in the LCCA of traditional (i.e., highway and bridge) transportation projects and in the cost-benefit analysis of Intelligent Transportation Systems (ITS). The authors formulated a Monte Carlo optimization routine and its submodels using the Riskoptimizer software upon construction of the LCCA genetic algorithm. Reportedly, the model was then implemented in a real-life case study using a road network data for NJ obtained from the New Jersey Pavement management System database. One of the major conclusions of the study was that, it is best to use a probability distribution constructed by best-fitting the real treasury discount rates with special consideration to the distribution bounds. The study also suggested that ITS projects should be evaluated at higher discount rates than other traditional transportation projects due to their shorter life-times and higher risk factors involved in such projects. Finally, according to the authors, the study demonstrated the feasibility of pairing genetic algorithm as an optimization tool with Monte Carlo simulation as a risk analysis tool into a single tool capable of performing probabilistic optimization in infrastructure management.

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. Authors stated that 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 was suitable for application to both new and existing civil infrastructures under various maintenance strategies. The ultimate objective of the paper was to evaluate the costs of alternative maintenance strategies and determine the optimum maintenance regime over a specified time horizon. In its format at the time, the first line of application of the method was for highway bridges. According to the paper, however, the method could be used for any structure, or group of structures, requiring maintenance in the foreseeable future. The proposed method could be programmed and incorporated into an existing software package for life-cycle costing of civil infrastructures. In the paper, an existing reinforced concrete bridge stock was analyzed to illustrate the proposed methodology and to reveal the cost-effectiveness of preventive maintenance interventions. The overall maintenance costs of different maintenance scenarios were compared. As a result, it was illustrated that the scenario associated with preventive maintenance is more economical than the one associated with essential maintenance beyond a certain time horizon. Preventive maintenance interventions were shown to reduce significantly the expected total cost.

In another paper, Kong and Frangopol (2003) considered the uncertainties involved in life-cycle analysis of deteriorating structures by providing a reliability-based framework and showed that the identification of the optimum maintenance scenario was a straightforward process. The authors used a computer program for Life-Cycle Analysis of Deteriorating Structures which could consider the effects of various types of actions on the reliability index profile of a group of deteriorating structures. In their paper, the authors considered only the effect of maintenance interventions. The paper presented numerical examples of deteriorating bridges to illustrate the capability of the proposed approach.

Ayyub and Popescu (2003) proposed methods for performing reliability computations and managing information that are suitable for risk-informed expenditure allocation in lifecycle management. The methods included structural reliability assessment methods and the analytic hierarchy method for multi criteria ranking. The paper also presented the advantages of using web-based computing through an example of system reliability assessment software that can be used in an interactive web environment.

Stewart et.al (2004) investigated the effect of limit state selection, strength versus serviceability, on bridge deck life-cycle costs and thus on optimal repair strategies in order to determine whether safety or functionality (or both) are important criteria when optimizing bridge life-cycle performance and costs. The structural element under consideration was a reinforced concrete bridge deck; namely, a State Highway Bridge in Colorado. Two limit states were considered: ultimate strength and serviceability. The life-cycle cost analysis presented in the paper included expected replacement costs as well as the random variability of material properties, loads, section dimensions, model errors, chloride penetration, and corrosion rates. Authors of the paper concluded that life-cycle costs for deck replacement based on a serviceability limit state were generally larger than those obtained for the strength limit states. Hence, according to the paper, an unrealistically optimistic life-cycle cost would result when serviceability was not included in the analysis.

Liu and Frangopol (2004) presented a maintenance planning procedure for deteriorating bridges, which considered the uncertainties evaluated by means of a Monte Carlo Simulation. A multi-linear model was used as the deterioration model for the reinforced concrete. Two performance indices were used in the study: the condition index and the safety index. Condition index was obtained from discrete values of 0,1,2 and 3 representing the visual inspection. The safety index was defined as the ratio of available to required live load capacity, describing approximately the reliability level of a deteriorating bridge component. A larger safety index value indicated a more reliable level accordingly. As a maintenance strategy, two options were considered: silane (reducing the chloride penetration in reinforced concrete structures) and do nothing which the characteristic parameter data of these strategies were collected in the United Kingdom. In order to optimize the maintenance problem Genetic Algorithm (GA) was used. According to the authors, the significance of uncertainties in deterioration process was shown by using Monte Carlo Simulation.

Noortwijk and Frangopol (2004) described and compared two maintenance models for deteriorating civil infrastructures that can be used to insure an adequate level of reliability at minimal life-cycle cost. These models, referred to as Rijkswaterstaat's model and Frangopol's model have been independently developed by the authors. Noortwijk and Frangopol mentioned that the former model had been applied by the Netherlands Ministry of Transport, Public Works and Water Management and that it can be used for justification and optimization of maintenance measures. The latter model, according to the authors, contributed to the further development of the bridge management methodology that had been set up by the UK Highways Agency. The paper stated that, although the two maintenance models were quite similar, several differences could be identified. The former model was reliability-based and treated the multicomponent, multi-failure mode and multi-uncertainty case. The latter model was condition-based and treated only one component, one failure mode and one uncertainty. Another difference was that Frangopol's model used Monte Carlo simulation, whereas Rijkswaterstaat's model was analytical.

Yang et.al (2004) proposed a model using lifetime functions to evaluate the overall system probability of survival of existing bridges, under maintenance or no maintenance conditions. In the model, bridges were modeled as systems of independent and/or correlated components. Deterioration of the bridges was simulated by probability of survival of deteriorating component which decreased with time. Probability of survival was approximated by lifetime distribution functions (LDFs). In general, exponential and Weibull survivor functions were used as LDFs. In the study, the maintenance actions considered were defined by the replacement of one, several, or all components of a system. The proposed model was applied to an existing bridge located in Denver, Colorado, and the optimal maintenance strategy of this bridge was obtained in terms of service life extension and cumulative maintenance cost. The results showed that the optimum cost was strongly dependent on the system model.

Huang et.al (2004) presented the development of a project-level decision support tool for ranking maintenance scenarios for concrete bridge decks deteriorated as a result of chloride-induced corrosion. The approach was based on a mechanistic deterioration model and a probabilistic life-cycle cost analysis. The analysis included agency and user costs of alternative maintenance scenarios and considered uncertainties in the agency cost and the corrosion rate in the deterioration model. Based on the results obtained using three existing bridge decks, it was shown that the total life-cycle cost (user cost plus agency cost) was a nonlinear function of the maximum tolerable condition of the deck and that the relationship between total life-cycle cost and the maximum tolerable condition was convex.

Morcous et.al (2005) presented an approach to determining the optimal set of maintenance alternatives for a network of bridge decks using genetic algorithms. A Markov-chain model was used for predicting the deterioration model of the bridge deck because of its ability to capture the time-dependence and uncertainty of the deterioration process, maintenance operations, and initial condition, as well as its practicality for network level analysis. Four different deteriorating groups were formed according to the environmental factors (Benign, Low, Moderate, Severe Environments). Transition probability matrices of concrete bridge decks with asphalt concrete (AC) overlay were formed after analyzing data for 9181 concrete bridge decks obtained from Ministe´re des Transports du Que´bec (MTQ) database. Genetic algorithms were applied to maintenance optimization because of their robust search capabilities that resolve the computational complexity of large-size optimization problems. According to the authors, the output of this approach comprised the percentages of the bridge deck areas in each group that requires a specific maintenance action in every year of the planning horizon. These percentages minimized the total maintenance costs and ensured that the overall average condition of each group was within acceptable limits.

Bolukbasi et.al (2005) investigated the cost-effectiveness of the timings of bridge component rehabilitation both for individual components and for combinations of components, with the objective of finding optimum bridge life and the most cost-effective rehabilitation schedule. The optimization model made use of: (1) deterioration curves to predict the condition of bridge components with respect to bridge age; (2) benefit/cost analysis to identify the most cost-effective rehabilitation schedule. The study was conducted using Illinois data for steel bridges covering the period 1976–1998. The results showed that the most cost-effective timings of rehabilitating individual bridge components were significantly different if the rehabilitations of different components were combined and treated as parts of a system. The results of the analysis indicated that optimum bridge life was 74 years with a best time of combined rehabilitation of both the deck and superstructure in year 44. The study was an attempt to improve the existing methods of scheduling bridge maintenance, repair and rehabilitation.

Zonta et.al (2007) described the bridge management methodology applied since 2004, at the Autonomous Province of Trento, Italy. The Bridge Management System entirely based on reliability concepts. The system operates on the web, and includes sections for (1) condition state evaluation, (2) safety assessment, and (3) prioritization. Condition appraisal is based on visual inspections, and acknowledges the general rules of the AASHTO Commonly Recognized Standard Element system. Normally, the system conservatively estimates the prior reliability of each bridge, based on the sole inspection data. Where the condition of the bridge gives cause for concern, its reliability was evaluated in a more formal manner using multi-step procedures of increasing refinement. Decision-making was driven by a principle where by priority is given to those actions that, within a certain budget, minimizes the risk of occurrence of an unacceptable event in the whole network.

Robelin and Madanat (2007) developed a framework for bridge maintenance optimization using a deterioration model that took into account aspects of the history of the bridge condition and maintenance, while allowing the use of efficient optimization techniques. Markovian models are widely used to represent bridge component deterioration. In existing Markovian models, the state is the bridge component condition, and the history of the condition is not taken into account, which is a limitation. The method was described to formulate a realistic history-dependent model of bridge deck deterioration as a Markov chain, while retaining aspects of the history of deterioration and maintenance as part of the model. The model was then used to formulate and solve a reliability-based bridge maintenance optimization problem as a Markov decision process.

The authors presented an approach to formulate a complex history-dependent deterioration model as a Markovian model with augmented state, as well as its use in a Markov decision process to determine optimal maintenance and replacement policies for one facility. Additional research was needed to address the problem of determining optimal maintenance and replacement policies for a system of facilities.

Ertekin et.al (2008), the authors of this dissertation and coauthors, presented a BLCCO framework that considered all the components of the bridge as a system rather than an individual element. Bridge data from NBI and Markov-Chain approach was used for the deterioration models. Agency, user and third party costs were considered and Genetic Algorithm was used for the optimization process. The effectiveness of the suggested algorithm was evaluated by using a case study with hypothetical bridge data. The results of the case study were presented. Considering the bridge elements as a system ensured making more precise calculations. (i.e. only considering deck results in user cost for speed restriction due to deck condition, however including the superstructure in the model as well ensures checking the user cost for speed reduction due to operating rating of the bridge, plus user cost for bridge weight limit due to operating rating of the bridge). Moreover, the probabilistic optimization resulted in a better understanding of the bridge life cycle cost, showing final cost as a range rather than a single value. This probabilistic approach was superior to the deterministic methods as it reflects the real-world cases to a greater extent and gives the agencies a better view for how to manage their resources. Main results showed that the recommended approach can be successfully applied to concrete or steel superstructure bridges that constitute the majority of bridges.

Okasha et.al (2011), attempted to present an integrated framework for the life-cycle management of highway bridges in the form of a detailed computational platform. The elements integrated into the framework included the advanced assessment of life-cycle performance, analysis of system and component performance interaction, advanced maintenance optimization, and updating the life-cycle performance by information obtained from structural health monitoring and controlled testing.

A framework for the life-cycle management of highway bridges was presented in the form of a detailed computational platform. The elements integrated included the assessment of life-cycle performance, analysis of system and component performance interaction, advanced maintenance optimization, updating the life-cycle performance by information obtained from SHM and controlled testing. These developments and establishing the means for their integration into a detailed life-cycle management process were among the main findings of the study. Given all the uncertainties inherent in the material properties, prediction of applied loads, and degradation of the resistance, the framework was formulated probabilistically. The performance prediction was one that takes into account the interaction among the structural components and evaluates the overall system performance. For enhancement of the efficiency of this framework and reduction in the uncertainty of its outcomes, data from SHM and controlled testing was integrated. The planning of maintenance interventions was executed based on a state-of-the-art optimization formulation and solved using advanced tools. The target of this framework was highway bridges. However, this framework, according to the authors, could be modified to become applicable to other structures such as buildings, off shore platforms, nuclear power plants and naval ships.

A summary of the previous studies and their contributions to the research subject is shown in Tables 2.1 and 2.2. As seen from the tables, the research performed in this dissertation is the most comprehensive study performed for BLCCO in the literature.

Author	**Year**	**Life Cycle Cost**
Mohammadi et.al	1995	OPTIMIZATION, NUMERICAL MEANS, AGENCY COST,
Liu et.al	1997	OPTIMIZATION, GENETIC ALGORITHM, AGENCY COST, USER COST,
Frangopol et.al	1999	OPTIMIZATION, NUMERICAL MEANS, AGENCY COST, USER COST,
Chase et.al	2000	---
Pratik, Roychoudhury	2001	OPTIMIZATION, NUMERICAL MEANS, AGENCY COST, USER COST,
Estes & Frangopol	2001	OPTIMIZATION, NUMERICAL MEANS, AGENCY COST, USER COST, PROBABILISTIC
Madanat and Mayet	2002	OPTIMIZATION, NUMERICAL MEANS, AGENCY COST, USER COST,
Kong & Frangopol	2003	OPTIMIZATION, NUMERICAL MEANS, AGENCY COST, USER COST,
Ozbay & Jawad	2003	OPTIMIZATION, GENETIC ALGORITHM, AGENCY COST, USER COST, PROBABILISTIC, MONTE CARLO SIMULATION
Liu & Frangopol	2004	OPTIMIZATION, GENETIC ALGORITHM, AGENCY COST, PROBABILISTIC, MONTE CARLO SIMULATION
Neves et.al	2004	---
Morcous et.al	2005	OPTIMIZATION, GENETIC ALGORITHM, AGENCY COST, USER COST,
Bolukbasi et. al	2005	OPTIMIZATION, NUMERICAL MEANS, AGENCY COST,
Furuta et.al	2005	OPTIMIZATION, NUMERICAL MEANS, AGENCY COST, USER COST,
Robelin & Madanat	2007	OPTIMIZATION, NUMERICAL MEANS, AGENCY COST, PROBABILISTIC, MONTE CARLO SIMULATION
Zonta et.al	2007	OPTIMIZATION, NUMERICAL MEANS, AGENCY COST,
Ertekin et. al	2008	OPTIMIZATION, GENETIC ALGORITHM, AGENCY COST, USER COST, PROBABILISTIC, MONTE CARLO SIMULATION
Agrawal et.al	2009	---
Okasha & Frangopol	2011	OPTIMIZATION, GENETIC ALGORITHM, AGENCY COST, USER COST, PROBABILISTIC, MONTE CARLO SIMULATION

Table 2.1: Summary of Bridge Life Cycle Cost Optimization Studies

Author	Year	Deterioration Model	
		Deck	**Superstructure**
Mohammadi et.al	1995	LINEAR EQUATION	LINEAR EQUATION
Liu et.al	1997	NON LINEAR EQUATION,	
Frangopol et.al	1999	LINEAR EQUATION, RELIABILITY BASED	LINEAR EQUATION, RELIABILITY BASED
Chase et.al	2000	NON LINEAR OPTIMIZATION MARKOV CHAIN, CONDITION RATING	NON LINEAR OPTIMIZATION MARKOV CHAIN, CONDITION RATING
Pratik, Roychoudhury	2001	NON LINEAR EQUATION	---
Estes & Frangopol	2001	NON LINEAR EQUATION, CHLORINE INDUCED DETERIORATION	---
Madanat and Mayet	2002	---	---
Kong & Frangopol	2003	---	---
Ozbay & Jawad	2003	---	---
Liu & Frangopol	2004	---	---
Neves et.al	2004	---	---
Morcous et.al	2005	EJ MARKOV CHAIN	---
Bolukbasi et. al	2005	NON LINEAR EQUATION	NON LINEAR EQUATION
Furuta et.al	2005	---	---
Robelin & Madanat	2007	NON LINEAR REGRESSION MARKOV CHAIN, CONDITION RATING	---
Zonta et.al	2007	---	---
Ertekin et. al	2008	NON LINEAR OPTIMIZATION MARKOV CHAIN, CONDITION RATING, UNEXPECTED EVENT	NON LINEAR OPTIMIZATION MARKOV CHAIN, CONDITION RATING, UNEXPECTED EVENT
Agrawal et.al	2009	---	---
Okasha & Frangopol	2011	NON LINEAR REGRESSION, STRUCTURAL HEALTH MONITORING	NON LINEAR REGRESSION, STRUCTURAL HEALTH MONITORING

Table 2.2: Summary of Bridge Deterioration Studies

Author	Year	Deterioration Model		
		Substructure	**Pavement**	**General**
Mohammadi et.al	1995	LINEAR EQUATION	---	---
Liu et.al	1997	---	---	---
Frangopol et.al	1999	---	---	---
Chase et.al	2000	NON LINEAR OPTIMIZATION MARKOV CHAIN, CONDITION RATING	---	---
Pratik, Roychoudhury	2001	---	---	---
Estes & Frangopol	2001	---	---	---
Madanat and Mayet	2002	---	---	NON LINEAR REGRESSION MARKOV CHAIN, CONDITION RATING, UNEXPECTED EVENT
Kong & Frangopol	2003	---	---	RELIABILITY BASED, MONTE CARLO SIMULATION
Ozbay & Jawad	2003	---	NON LINEAR EQUATION	---
Liu & Frangopol	2004	---	---	LINEAR EQUATION
Neves et.al	2004	---	---	LINEAR EQUATION, RELIABILITY BASED
Morcous et.al	2005	---	---	---
Bolukbasi et. al	2005	NON LINEAR EQUATION	---	---
Furuta et.al	2005	NON LINEAR EQUATION, UNEXPECTED EVENT	---	---
Robelin & Madanat	2007	---	---	---
Zonta et.al	2007	---	---	NON LINEAR EQUATION, CONDITION RATING
Ertekin et. al	2008	NON LINEAR OPTIMIZATION MARKOV CHAIN,CONDITION RATING, UNEXPECTED EVENT	NON LINEAR EQUATION	---
Agrawal et.al	2009	---	---	NON LINEAR OPTIMIZATION MARKOV CHAIN, CONDITION RATING
Okasha & Frangopol	2011	NON LINEAR REGRESSION, STRUCTURAL HEALTH MONITORING	---	---

Table 2.2 (continued): Summary of Bridge Deterioration Studies

Chapter Three

BRIDGE DETERIORATION MODEL

3.1. Introduction and Problem Definition

In order to prepare an efficient and powerful Bridge Life Cycle Cost Analysis (BLCCA) model, it is important to understand the major components of it (Figure 3.1a). These are; a deterioration model of the bridge, rehabilitation, repairs or reconstruction of the bridge and the agency, and user costs and the budget. In this cha`pter, the first two of these components will be covered and the third one will be covered in Chapter 4 of this dissertation.

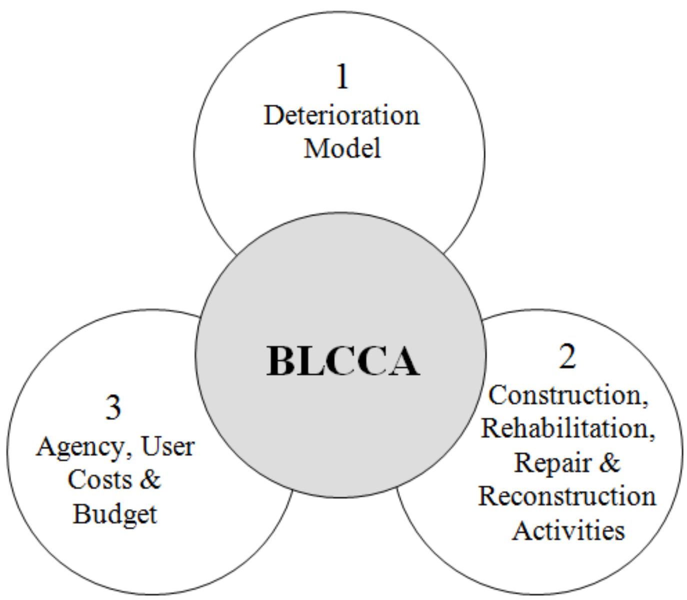

Figure 3.1a: BLCCA Components

It is essential to use a deterioration model, which can accurately simulate the service life of the bridge and facilitate the proper selection of rehabilitation, repair or reconstruction activities to optimize the costs and efforts (Figure 3.1b). The fact that 26% of the bridges need repair or replacement nationwide (2009 NBI Data), also justifies the importance to use a detailed and accurate bridge deterioration model.

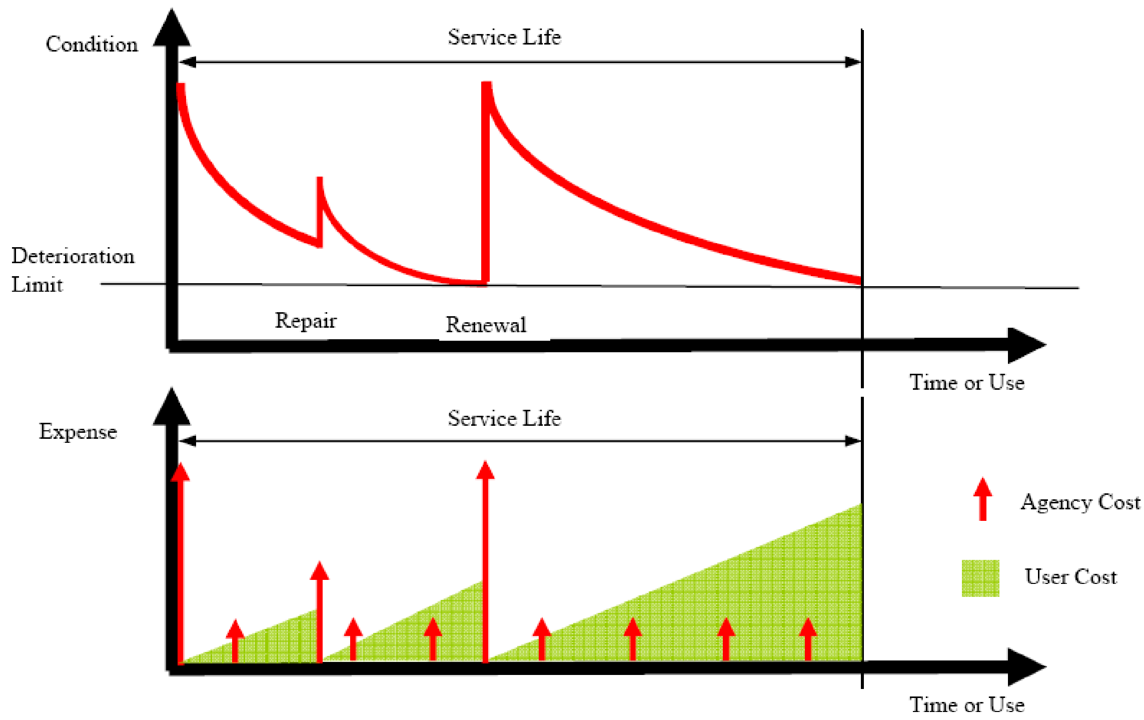

Figure 3.1b: Possible Activities during the Service Life of a Bridge.

It is recommended in the report NCHRP 12-43 that the deterioration model can be formed through, statistical regression analysis where relationships between condition measures and parameters presumed to have a causal influence on condition are built. These relationships maximize the likelihood that the output parameter (i.e., condition) will be in the particular range calculated if the causal parameters are in their particular assumed range (NCHRP 12-43, 2003).

There have been some efforts towards using regression or nonlinear optimization analysis for bridge deterioration modeling in the previous studies. . In many of them, only the age of the bridge and ADT were used as the variables describing the bridge and only a few of the previous studies considered the environmental factors on bridge deterioration (Chase, 1999).

In this dissertation, for the bridge deterioration modeling, a constrained non-linear optimization analysis method utilized through Markov chain methodology is used. In the used model, the map of the U.S. is divided into nine different regions as defined by the National Climatic Data Center (NCDC) (Figure 3.2) with different environmental conditions. For each of these regions, bridges are grouped according to the material of the superstructure, the length of the bridges and the ADT on the bridges. Structure Inventory and Appraisal (SI&A) data for deterioration analysis was obtained from National Bridge Inventory (NBI) Database. All 603,309 bridges available in the database for the year 2009 are considered in this study.

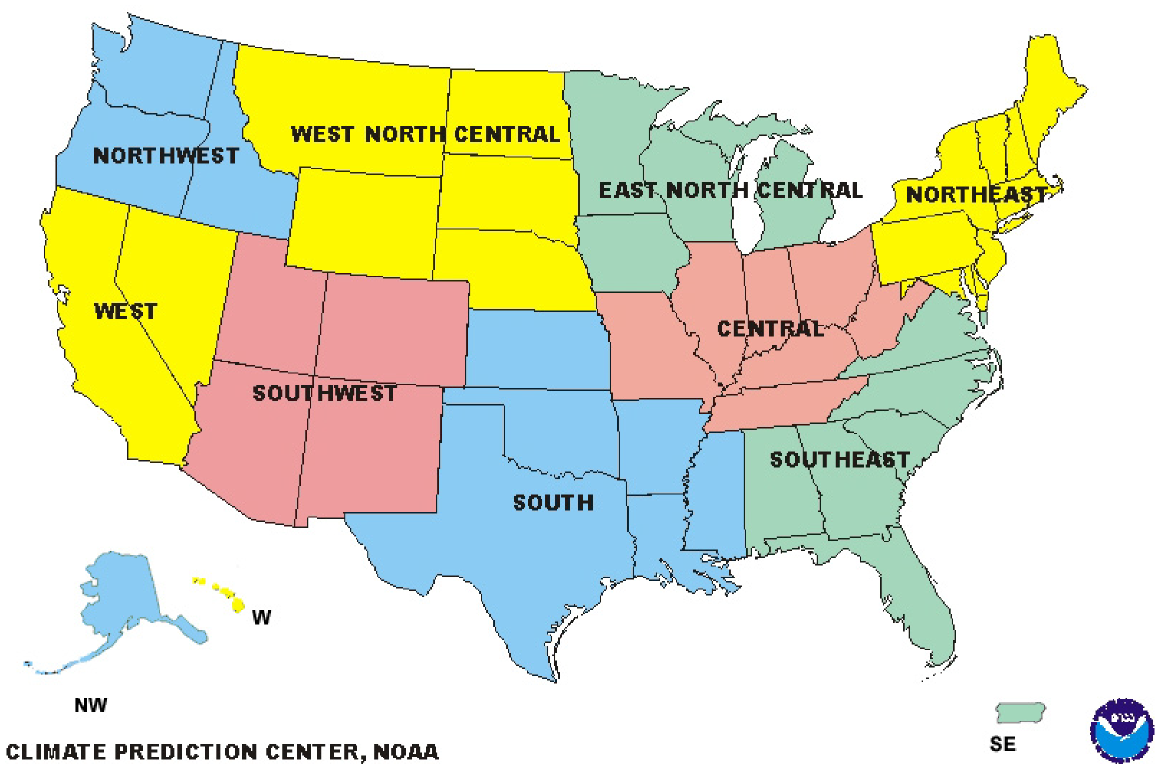

Figure 3.2: The nine regions as defined by NCDC.

3.2. NBI Database Description

The NBI Database has the most extensive and detailed data on highway bridges in the US. The NBI is a collection of information (database) covering all of the Nation's bridges located on public roads, including Interstate Highways, US highways, State and county roads, as well as publicly-accessible bridges on Federal lands. It presents a State by State summary analysis of the number, location, and general condition of highway bridges within each State.

Collection of NBI data is authorized by statute, 23 U.S.C. 151 (National Bridge Inspection Program), and implemented by regulation, 23 CFR 650.301 *et seq*. In accord with these authorities, the FHWA established National Bridge Inspection Standards (NBIS) for the safety, inspection and evaluation of highway bridges; and each State is required to conduct periodic inspections of all bridges subject to the NBIS, prepare and maintain a current inventory of these structures, and report the data to the FHWA using the procedures and format outlined in the Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges.

After evaluation of the inspection data, the FHWA provides States with a list of bridges that are eligible for replacement or rehabilitation. The FHWA uses the data to submit a required biannual report to Congress on the status of the Nation's bridges, to publish an Annual Materials Report on New Bridge Construction and Bridge Rehabilitation in the Federal Register, and to apportion funds for the Highway Bridge Program.

NBI Database is a very important resource for examination and evaluation of the US highway bridge population.

3.3. Analysis Used to Obtain Deterioration Curves

Deterioration curves are prepared for superstructure, substructure, deck and pavement. For the first three of them, deterioration level is quantified using condition rating indices, which were also used by NBI. This is a numeric ranking system from “0” to “9”, where “0” represents “Failed Condition” and “9” represents “Excellent Condition” (Table 3.1). Such condition rating data is available for deck, superstructure and substructure in the NBI database. NJDOT uses the same condition rating system as well. A condition rate of “6” (Satisfactory Condition) or above indicates that there is no need to do any repair on the bridge. “5” (Fair Condition) means; minor repair that can be done by maintenance units of the DOT. “4” (Poor Condition) means; major repair that requires a contractor. A condition rate below “3” (Serious Condition) is the case when it might be considered to replace the bridge. In this study, this rating system will be used for the decision making process for repair type through the service life of the bridge. This topic will be covered in more detail later in Chapter 4 of this dissertation.

Condition Rating	**Interpretations**
9	Excellent Condition
8	Very Good Condition – no problems noted.
7	Good Condition – some minor problems.
6	Satisfactory Condition – some minor deterioration of structural elements.
5	Fair Condition – minor section loss of primary structural elements.
4	Poor Condition – advanced section loss of primary structural elements.
3	Serious Condition – seriously deteriorated primary structural elements.
2	Critical Condition – facility should be closed until repairs are made.
1	Imminent Failure Condition – facility is closed. Study if repairs feasible.
0	Failed Condition – facility is closed and beyond repair.

Table 3.1: NBI Bridge Condition Ratings

Markov chain methodology is used to simulate bridge deterioration. The Equation 3.1. below shows condition state transition probability matrix T, which basically defines the probability of a bridge transitioning from condition state CS_i (Initial condition state of the bridge) to CS_{i+1} (final state of the bridge) over a given time interval**.** As the bridge is inspected or tested periodically, a different condition state number is assigned based upon the results of the inspection or testing and criteria assigned to each of the 10 unique condition states given in Table 3.1. As a bridge deteriorates, the condition state for the bridge will change, where the probability that the condition state of a bridge will change from CS_i to CS_j for a given time interval is given by $\mathbf{T}_{ij}$. Assuming that bridges are inspected every year or every other year, the time between observations is short enough, the observed condition state transitions should be limited to transitions between two adjacent condition states. Considering there are 10 unique condition states, T will be a 10 x 10 matrix. The diagonals (P_{99}, P_{88}, P_{77},… P_{00}) represent the probability of the condition to remain the same for the next year. The cells next to the diagonal (P_{98}, P_{87}, P_{76},…P_{10})

$$T = \begin{bmatrix} p_{99} & 1-p_{99} & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & p_{88} & 1-p_{88} & 0 & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & p_{77} & 1-p_{77} & 0 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & p_{66} & 1-p_{66} & 0 & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & p_{55} & 1-p_{55} & 0 & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & p_{44} & 1-p_{44} & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & p_{33} & 1-p_{33} & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & p_{22} & 1-p_{22} & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & p_{11} & 1-p_{11} \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & p_{00} \end{bmatrix} \quad \text{(Eq. 3.1)}$$

represent the probability of the condition to change in the next year. The rest of the cells will be all “0”. It is assumed that at every transition period the bridge can only deteriorate to the next lower condition state. P_{00} is always 1, because this is the worst condition state and the bridge cannot continue deteriorating after this condition.

The whole transition matrix can also be represented by a vector P which contains the diagonal of the matrix.

$$P = [p_{99} \quad p_{88} \quad p_{77} \quad p_{66} \quad p_{55} \quad p_{44} \quad p_{33} \quad p_{22} \quad p_{11} \quad p_{00}] \qquad \text{(Eq. 3.2)}$$

Final condition state probability distribution CS_f for a bridge part with an initial condition state probability distribution CS_i after N number of transitions can be shown as:

$$CS_f = CS_i T^N \qquad \text{(Eq. 3.3)}$$

In order to better light on the preceding discussion, consider the following illustrative (hypothetical) example. A new bridge has a condition state probability distribution of {1,0,0,…0}, then the condition state probability distribution of bridge after N transition is: $\{1,0,0,\ldots 0\}T^N$.

The constrained nonlinear optimization to calculate the transition probabilities from one condition state to another is summarized as follows: Considering a population of bridges to be studied (e.g. bridges with steel superstructure in the Northeast region) with differing ages and observed condition states for every bridges, a matrix is formed where the rows correspond to the number of transitions, N and the columns correspond to bridge condition states in different formulations, M. Four different formulations are considered for M.

1- Number of bridges in each condition state, which the weighted condition ratings for every row of N are obtained for further analysis. Note that the obtained condition ratings are discrete and ordered values.

2- Probability of bridges being in each condition rating for every row of N. Note that at each raw, the numbers of bridges are divided to the sum of the bridges in that raw (normalized). The values are not discrete.

3- Odds of bridges being in each condition rating for every row of N. Note that all the numbers in the matrix are:

   Probability of bridges being at that condition rating (x) divided by (1-x). The values are not discrete.

4- Logit methodology: Logarithm of odds of bridges being in each condition rating for every row of N. The values are not discrete.

If the transition interval is selected as 1 year, then the number of transitions will be equal to the age of the bridge, which is considered to be 75 years (or transitions). These bridge matrixes are called R, which is an M x N matrix. For the Markov chain simulation, a Markov matrix is prepared from the vector P= [p_{99}(<1), p_{88}(<1), p_{77} (<1), p_{66}(<1) p_{55}(<1), p_{44}(<1), p_{33} (<1), p_{22}(<1), p_{11}(<1),1] and N=75 transitions are calculated (T^{75}). Using N transitions of the simulated condition state probability distribution CS, which is a size M vector, a simulated bridge condition matrix called S can be obtained which is also an M x N matrix. Getting the P vector which will give the best possible simulation of the R matrix is the essence of the whole process. An optimization subroutine is prepared which changes the P vector values between “0” to “1” and yields the best possible simulation S matrix. The aim here is depending on the formulations used for M in R matrix to determine the P vector values that minimize the Equation 3.4; where S_{ij} is the simulated matrix, R_{ij} is the observed matrix and CR is the condition rating.

Optimization for formulation 1 for M

$$\text{Min}\ [\Sigma_i^N \{\Sigma_j^M (S_{ij} * CR_j) / \Sigma_j^M (S_{ij}) - \Sigma_j^M (R_{ij} * CR_j) / \Sigma_j^M (R_{ij})\}^2] \qquad \text{(Eq. 3.4a)}$$

Optimization for formulation 2, 3 and 4 for M

$$\text{Min}\ [\Sigma_i^N \Sigma_j^M (S_{ij} - R_{ij})^2] \qquad \text{(Eq. 3.4b)}$$

To analyze which formulations work best, the year 2009 NBI database for the Northeast Region steel bridges are used in a case study. Minimization of the Equation 3.4 is performed for the superstructure of the steel bridges condition ratings for the first 30 years (transitions) of the bridges. The reason for this time constraint is to minimize the number of bridges with repairs or reconstruction in the analysis. This topic will be discussed in detail in the following pages.

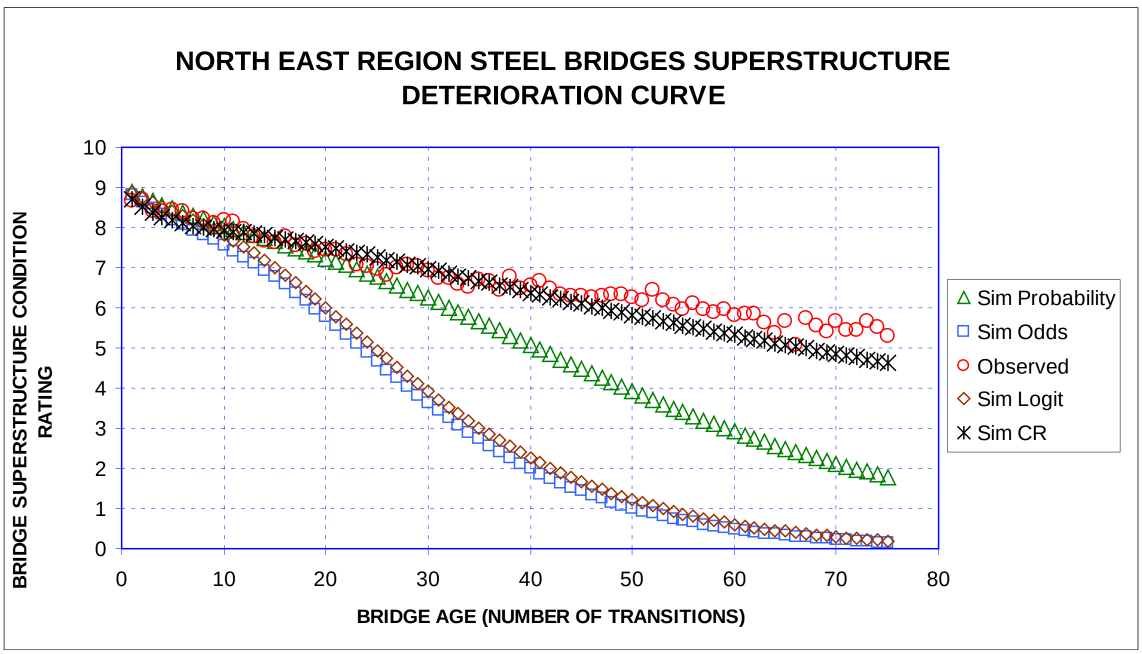

Figure 3.3: Comparisons of Different Optimization Formulations for Markov Chain Deterioration Model

Observed deterioration curve (ODC) is the weighted sum of the observed normalized condition ratings which is shown in the Equation 3.5.

$$ODC_N = \Sigma_i^M (R_{iN} \times CR_i) \qquad \text{(Eq. 3.5)}$$

Simulated deterioration curve (SDC) is the weighted sum of the simulated normalized condition ratings which is shown in the Equation 3.6. These normalized condition ratings are simulated according to the 4 formulations.

$$SDC_N = \Sigma_i^M (S_{iN} \times CR_i) \qquad \text{(Eq. 3.6)}$$

Among these formulations 1st one (number of bridges, CR) performed the simulation with an absolute mean error of 1.71%, 2nd one (probabilities of bridges) with an absolute mean error of 1.59%, 3rd one (odds of bridges) with an absolute mean error of 10% and 4th one (Logit) with an absolute mean error of 7.5%. Therefore it is decided to proceed with 2nd formulation (probability of bridges being in each condition rating) and all the deterioration curves in this dissertation are formed by using it.

Since the formulation method for the simulation is determined, the minimization process is illustrated with a case study. The year 2009 NBI database for the Northeast Region steel and prestressed bridges are chosen to show the result of the process and moreover to explain how the process is modified to get more accurate deterioration models. Minimization of the Equation 3.4b is performed for the superstructure of the steel and prestressed bridges condition ratings.

For the steel bridges a 10x10 Markovian transition probability matrix is determined through a constrained non linear optimization procedure that minimizes the Equation 3.4b above. The constraints are that the lower triangular elements of the transition probability matrix are zero (no condition improvements are allowed), all the transition probabilities must be greater than or equal to zero and that the probabilities for each condition state must add up to “1”.

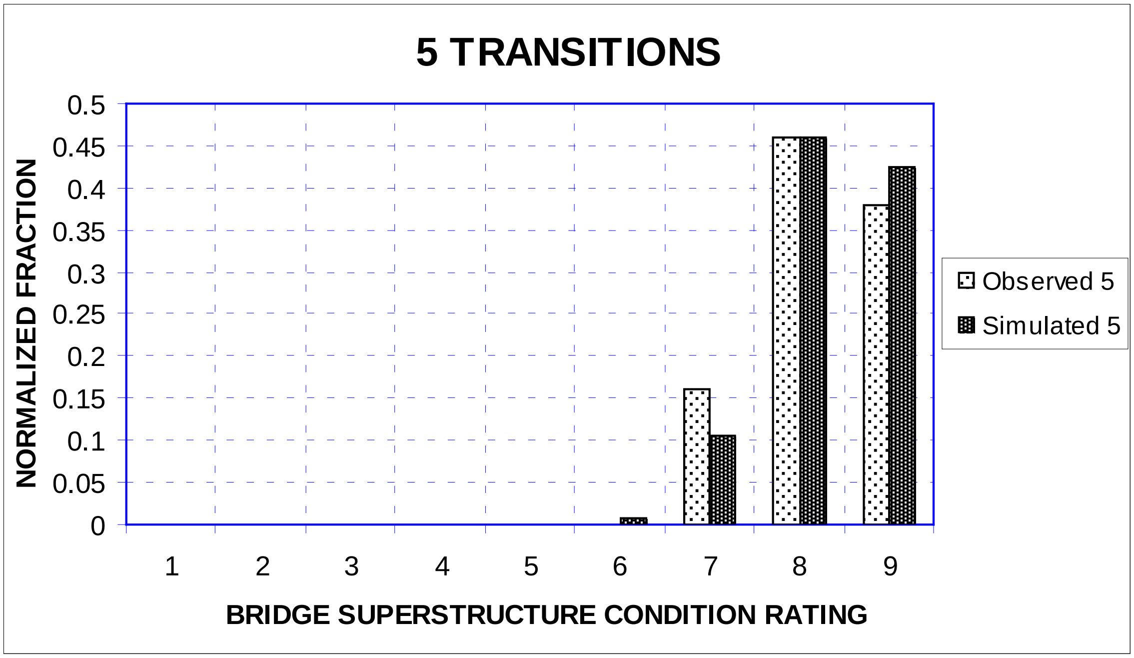

Figure 3.4a: Comparison of Observed and Simulated Normalized Superstructure Condition State Distributions for Bridges with Steel Superstructure.

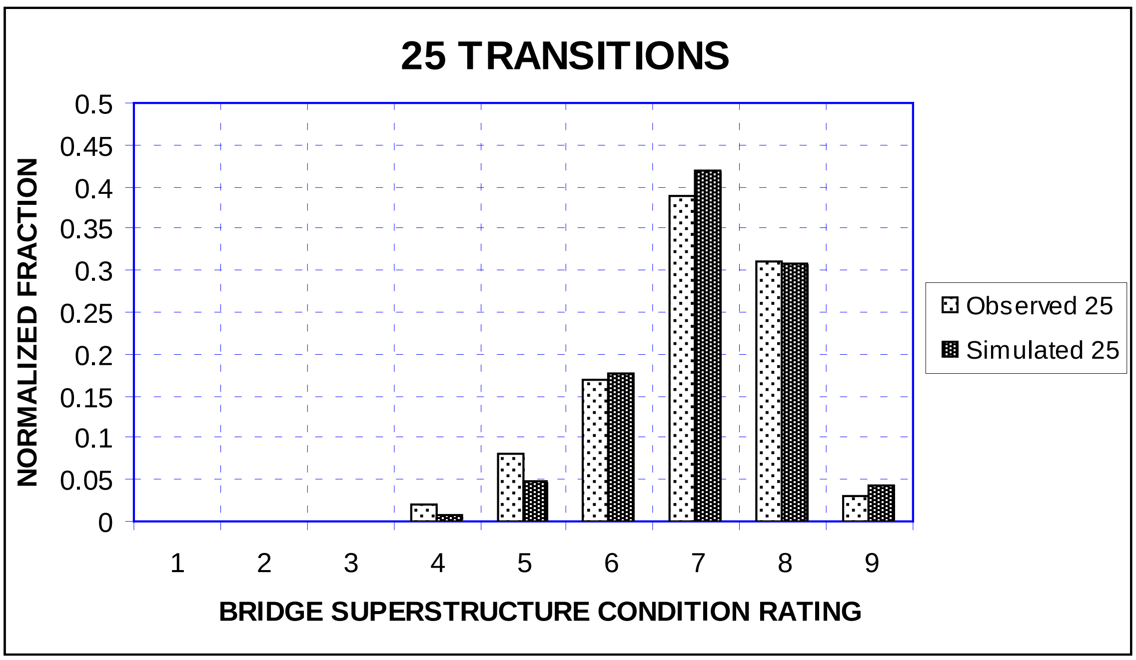

Figure 3.4b: Comparison of Observed and Simulated Normalized Superstructure Condition State Distributions for Bridges with Steel Superstructure.

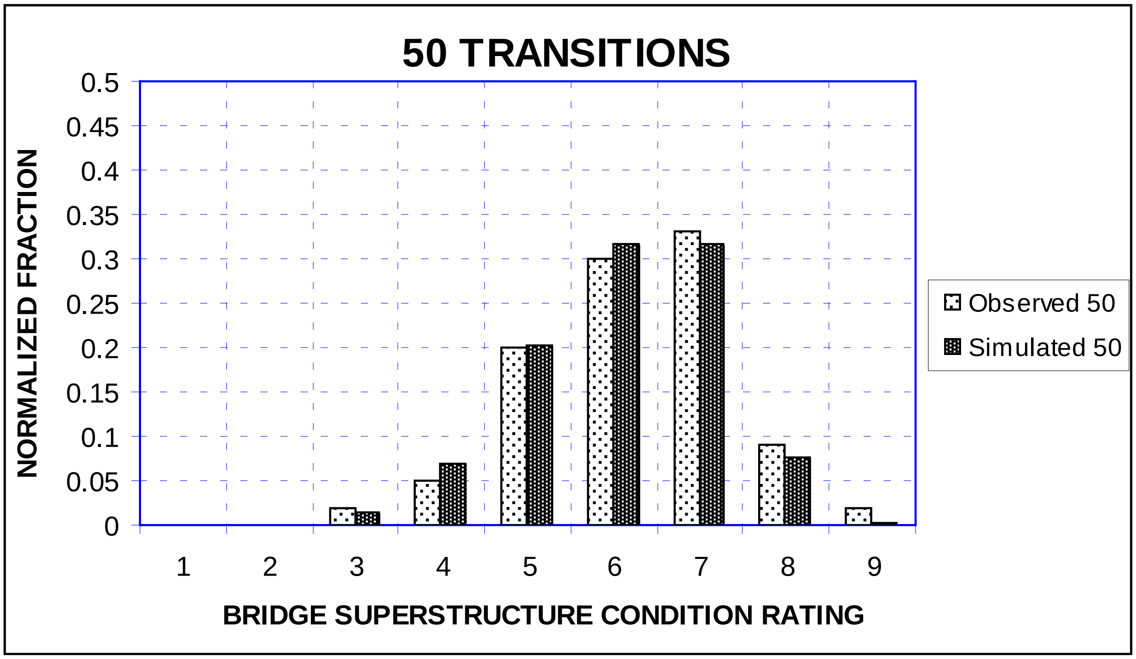

Figure 3.4c: Comparison of Observed and Simulated Normalized Superstructure Condition State Distributions for Bridges with Steel Superstructure.

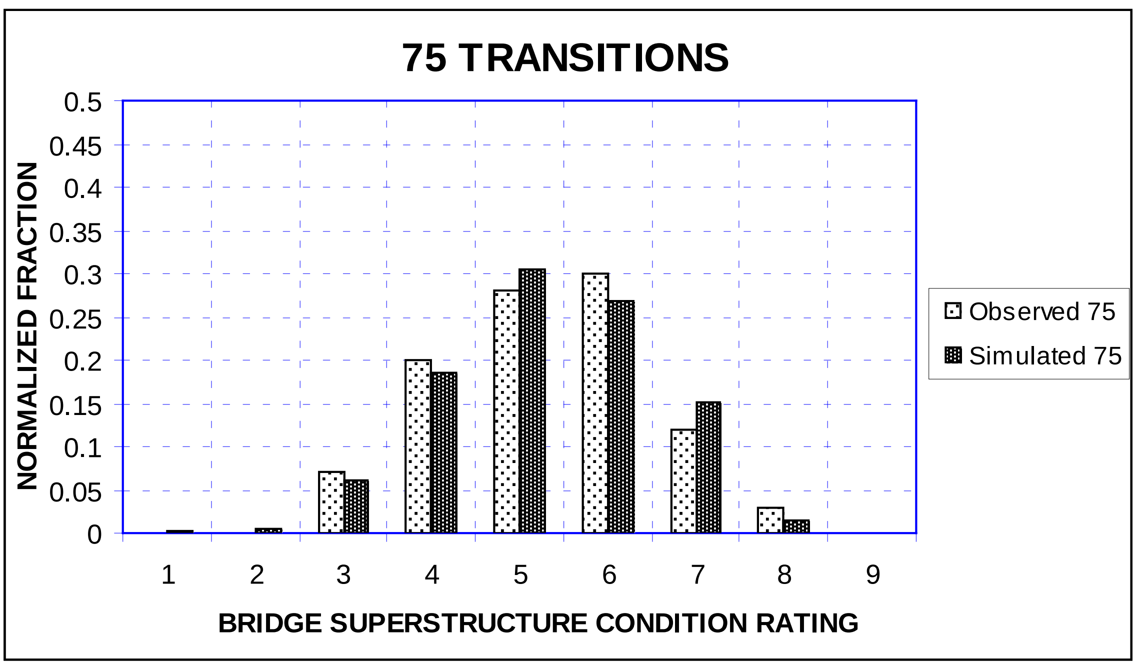

Figure 3.4d: Comparison of Observed and Simulated Normalized Superstructure Condition State Distributions for Bridges with Steel Superstructure.

The resulting transition probability vector is:

P= [0.885035,0.935756,0.960147,0.963163,0.972871,0.979787,0.980736,0.793055,0.754435, 1]

The simulated and the observed values are close, with an absolute mean error of 2.00%, when applied to the available 75-year observed (hypothetical) data.

Bridges with steel superstructure are categorized according to the observed and simulated superstructure condition data and their age forming 10x75 matrixes. All rows (transition) in these matrixes are normalized. The observed and the simulated normalized data for transitions 5, 25, 50 and 75 are shown in Figures 3.4a-b-c & d. Comparison of the Markov Simulation and Observed Superstructure Condition is shown in Figure 3.5. In the same figure, observed ± Standard Deviation graphs are also shown to give a better feeling of the minimum and maximum deterioration ranges.

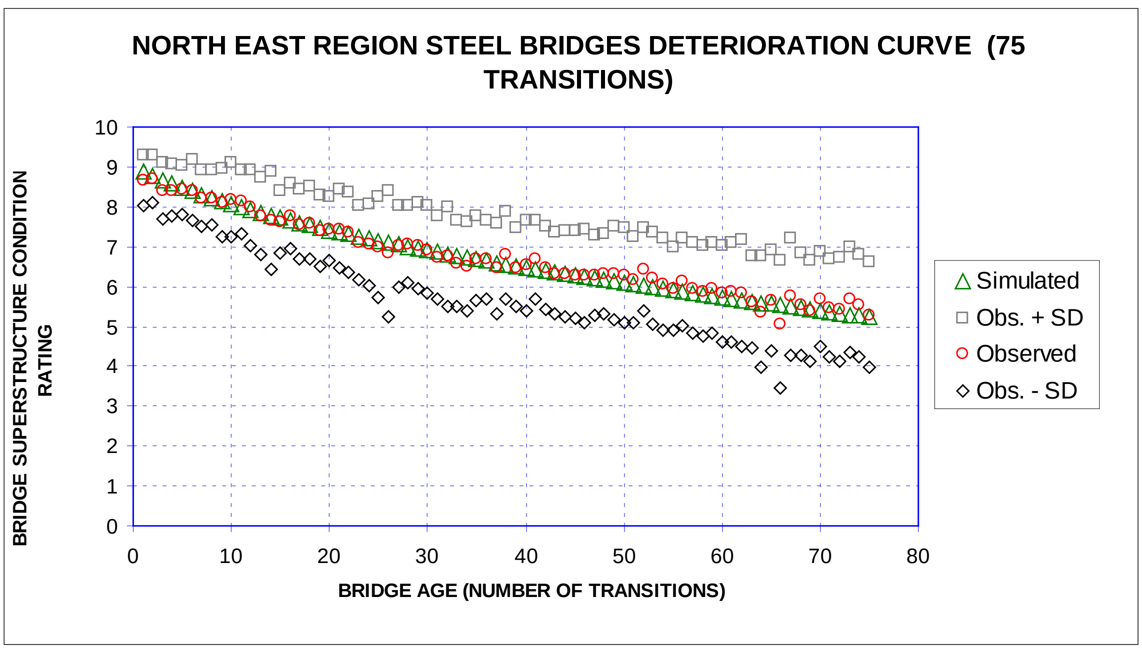

Figure 3.5: Comparison of Markov Simulation and Observed, Observed + Standard Deviation and Observed – Standard Deviation Superstructure Condition for Bridges with Steel Superstructure.

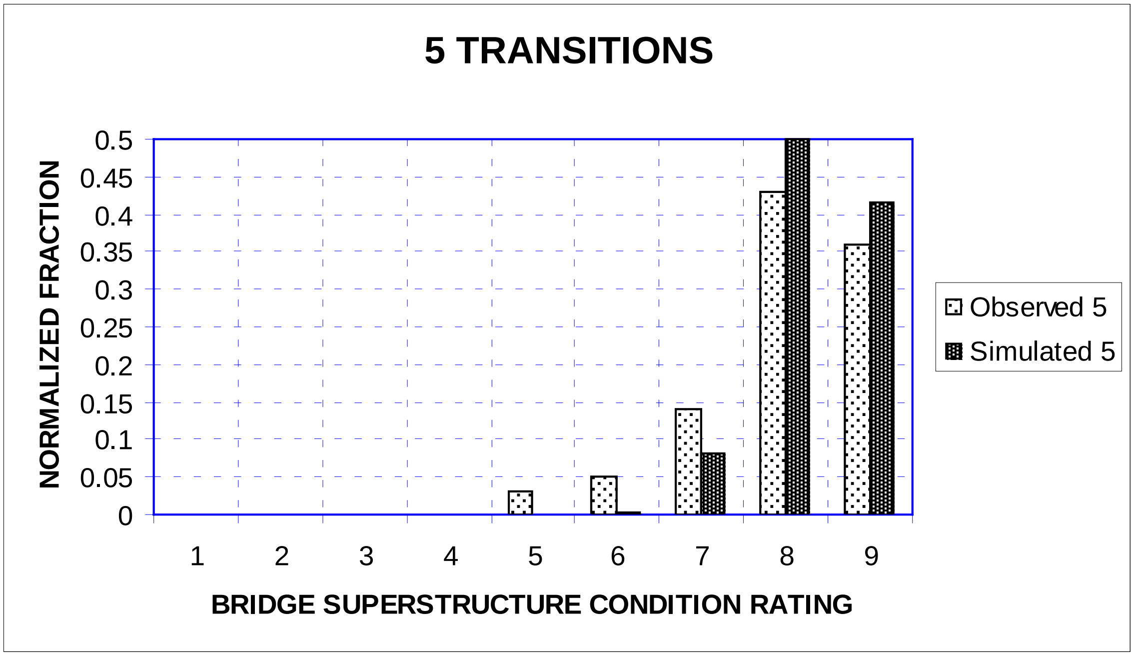

Figure 3.6a: Comparison of Observed and Simulated Normalized Superstructure Condition State Distributions for Bridges with Prestressed Superstructure.

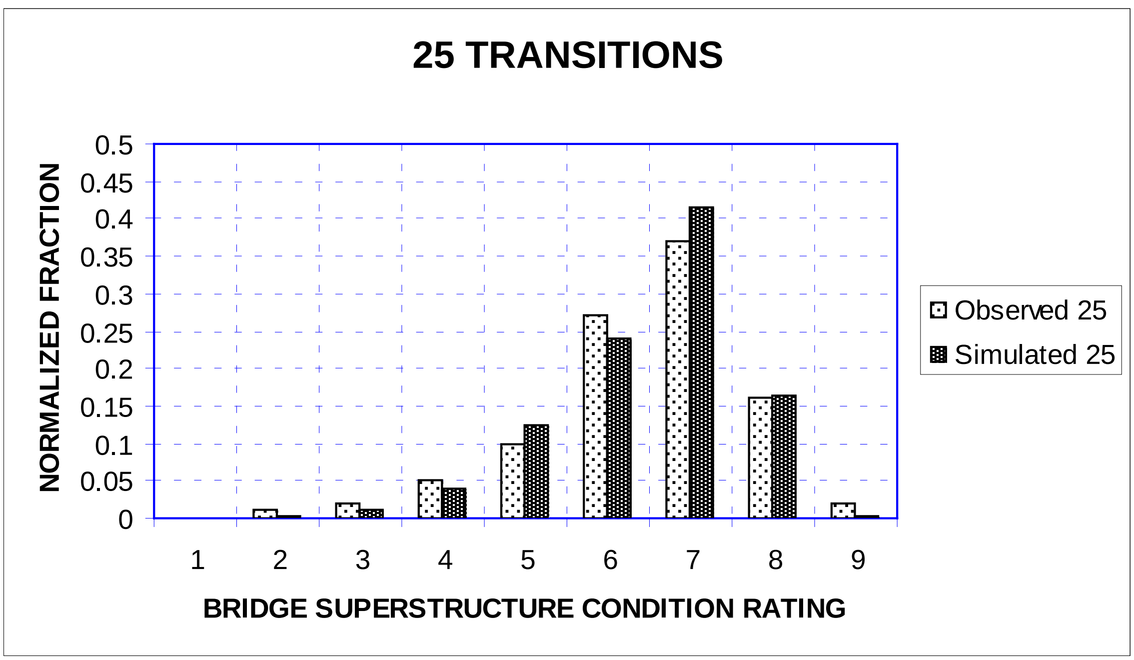

Figure 3.6b: Comparison of Observed and Simulated Normalized Superstructure Condition State Distributions for Bridges with Prestressed Superstructure.

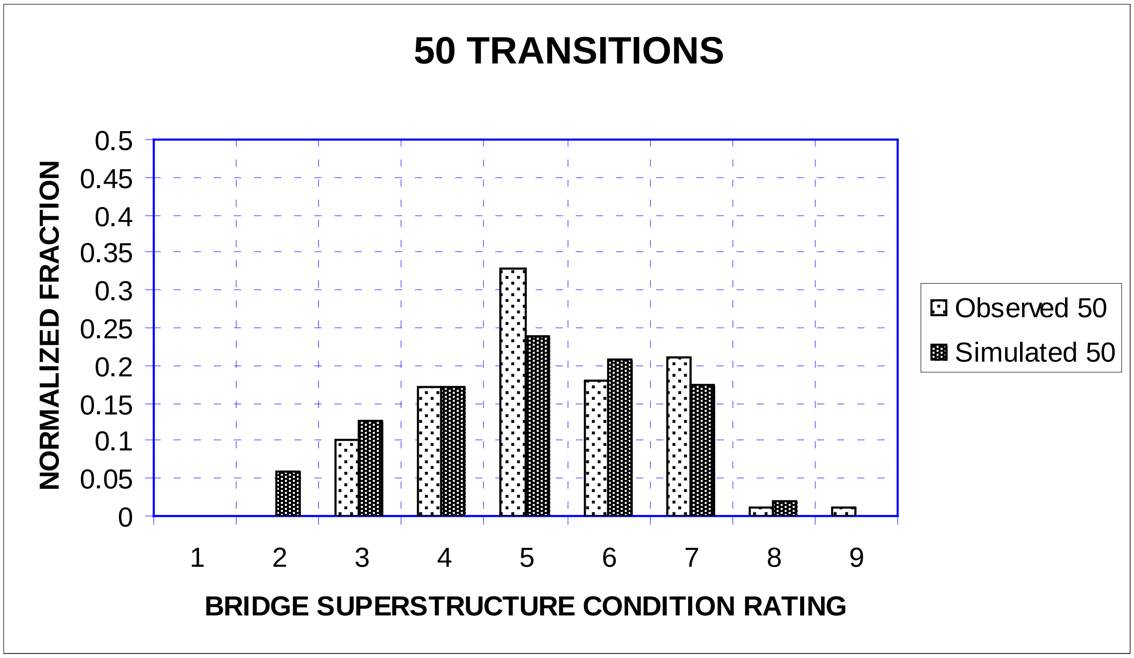

Figure 3.6c: Comparison of Observed and Simulated Normalized Superstructure Condition State Distributions for Bridges with Prestressed Superstructure.

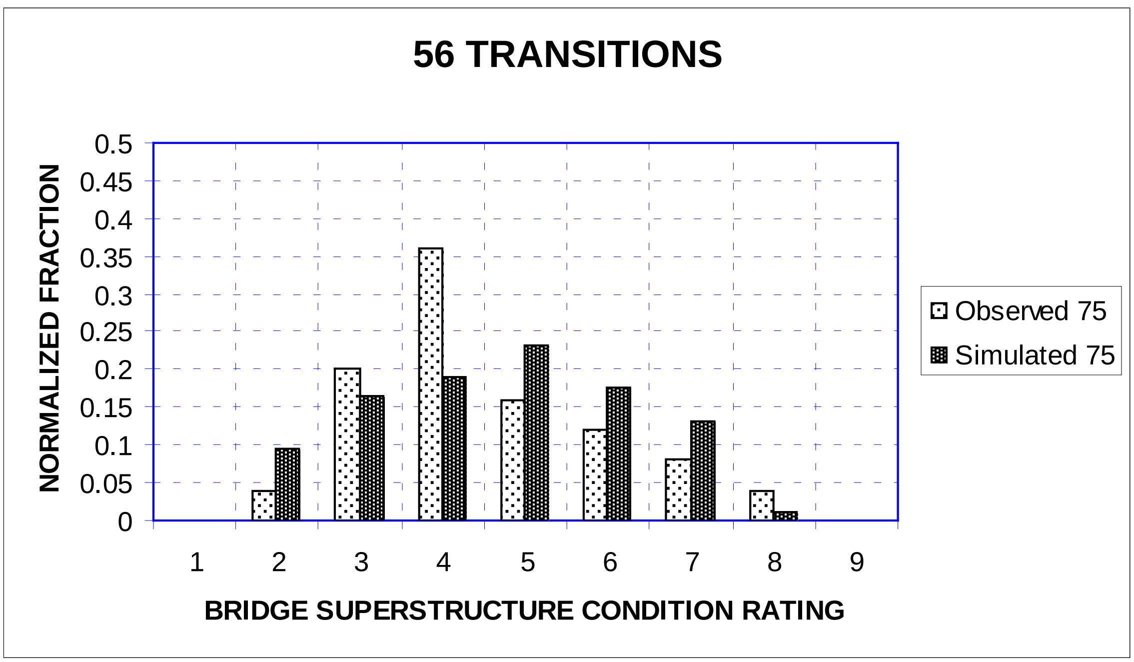

Figure 3.6d: Comparison of Observed and Simulated Normalized Superstructure Condition State Distributions for Bridges with Prestressed Superstructure.

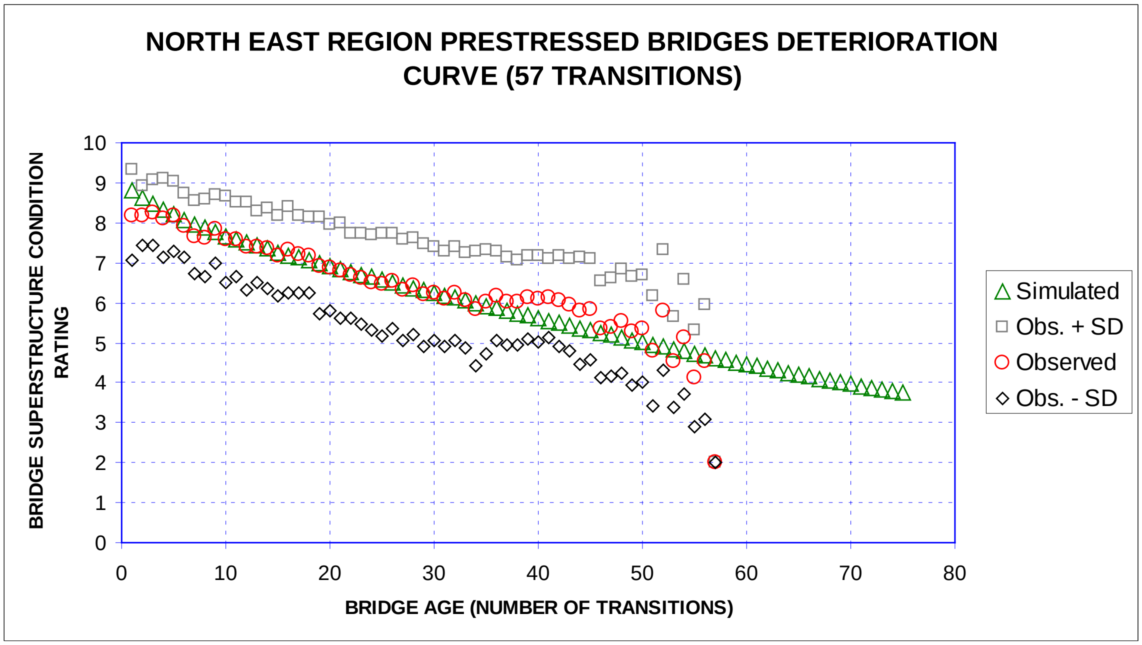

Figure 3.7: Comparison of Markov Simulation and Observed, Observed + Standard Deviation and Observed – Standard Deviation Superstructure Condition for Bridges with Prestressed Superstructure.

A 10x10 Markovian transition probability matrix for the prestressed bridges is determined through a constrained non linear optimization procedure that minimizes the square of the deviations between the simulated and observed condition states' probability distributions for each transition period. The constraints are that the lower triangular elements of the transition probability matrix are zero (no condition improvements are allowed), all the transition probabilities must be greater than or equal to zero and that the probabilities for each condition state must add up to "1". The resulting transition probability vector is:

P= [0.802714, 0.913835, 0.945408, 0.929317, 0.936746, 0.932781, 0.959219,1, 0.810226, 1]

The simulated and the observed values are close, with an absolute mean error of 5.91%, when applied to the available 57-year observed data. Only 57-year data is used since very few prestressed bridges were built in the Northeast region before 1952 (57 years ago) and that the limited data available is not enough to be used in the simulation and get a reliable model.

Bridges with prestressed concrete superstructure are categorized according to the observed and simulated superstructure condition data and their age, forming 10x75 matrixes. All rows (transition) in these matrixes are normalized. The observed and the simulated normalized data for transitions 5, 25, 50 and 56 are shown in Figures 3.6a-b-c & d. Comparison of Markov Simulation and Observed Superstructure Condition is shown in Figure 3.7. In the same figure, observed ± Standard Deviation graphs are also shown to give a better feeling of minimum and maximum deterioration range. S, R and Markov matrixes for steel superstructure bridges are shown in Appendix A1 to illustrate how the process works.

This approach has a small problem. The deterioration progresses steeper in the first 20 to 40 year (average 30 years) of the bridge life time, as it can be seen from Figures 3.5 and 3.7. After 30 years the deterioration curve becomes more inward curve which indicates that the deterioration is slowing down as the years pass by. The reason for this is an anomaly in the NBI database. The database does not reflect the effect of the bridge repair activities and their dates. Due to this, the condition rating data of the repaired and non-repaired bridges are mixed. The repaired bridges show a higher condition rating compared to bridges that are continuously deteriorated and this situation artificially increases the condition ratings in the deterioration curve for those years. As the bridges get older, more of them get repaired and that is why the bridge deterioration curves become more inward as the number of the transitions increase. The formation of this situation is observed after 20 to 40 transitions in the deterioration curves and it is

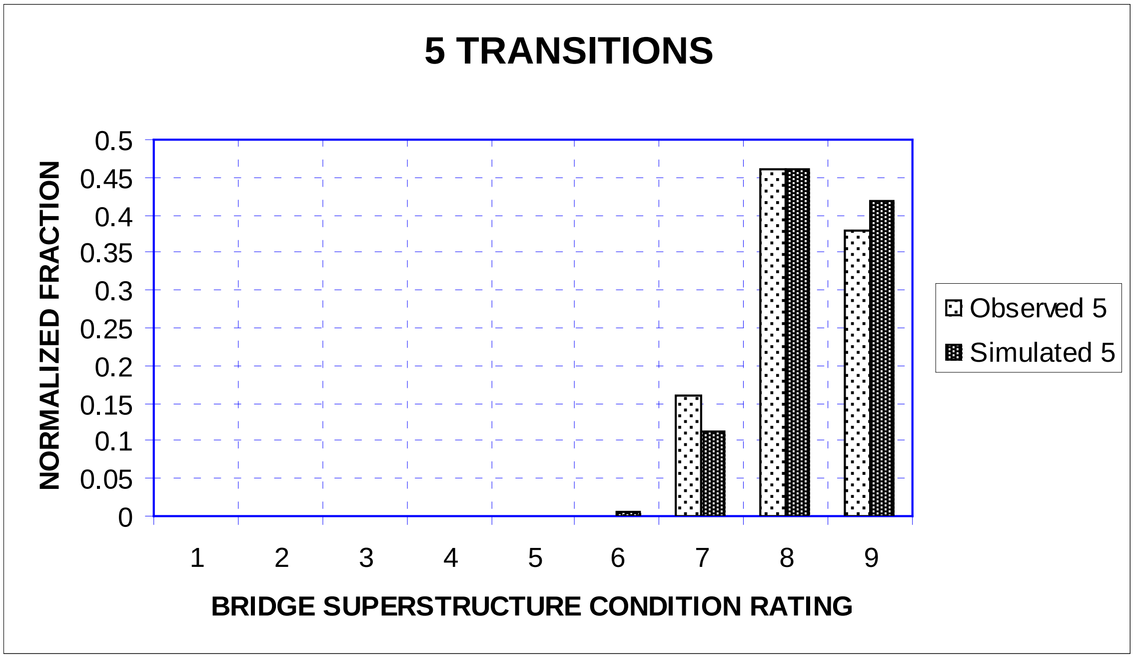

Figure 3.8a: Comparison of Observed and Simulated Normalized Superstructure Condition State Distributions for Bridges with Steel Superstructure for the Simulation of 30 Transitions.

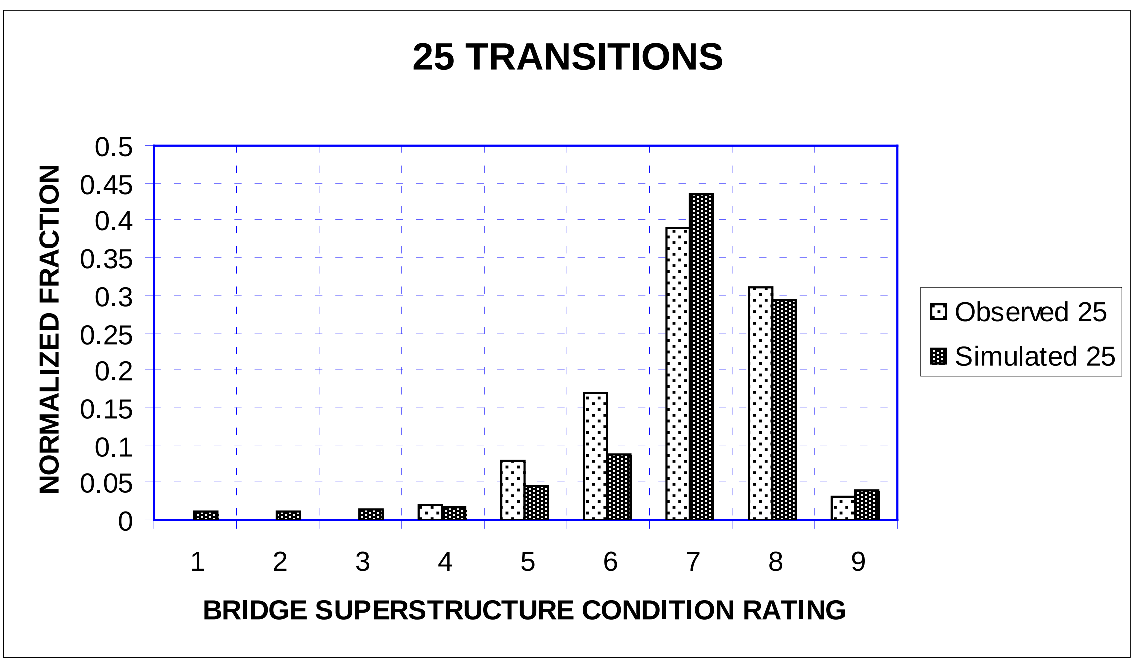

Figure 3.8b: Comparison of Observed and Simulated Normalized Superstructure Condition State Distributions for Bridges with Steel Superstructure for the Simulation of 30 Transitions.

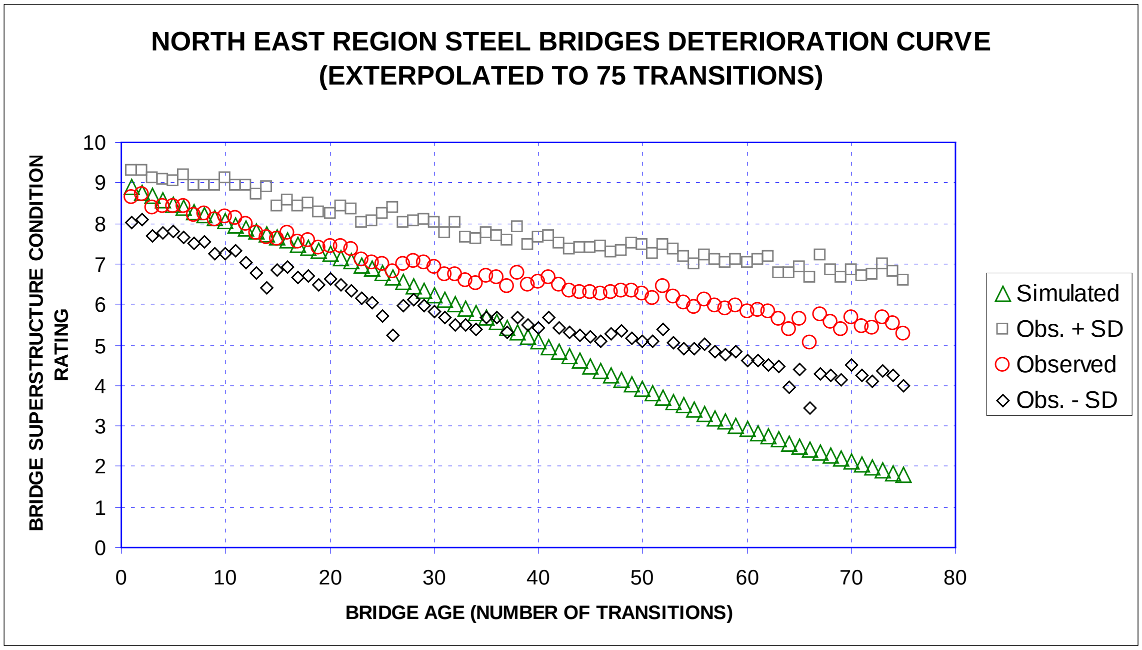

Figure 3.9: Comparison of Markov Simulation and Observed, Observed + Standard Deviation and Observed – Standard Deviation Superstructure Condition for Bridges with Steel Superstructure for the Simulation of 30 Transitions.

decided to prepare a Markov deterioration simulation matrix for the first 30 years of the bridge life span and extrapolate it to 75 years which is the life time of the bridges. The observed and the simulated normalized data at transitions 5 and 25 for steel and prestressed concrete bridges are compared in Figures 3.8a & b and 3.10a & b respectively. Comparison of Markov Simulation and Observed Superstructure Condition are shown in Figures 3.9 and 3.11. The graphs show that, after 30 transitions, the deterioration curves are not inward anymore.

The resulting transition probability vector for the bridges with steel superstructure is:

P= [0.883425,0.933111,0.961949,0.839587,0.744845,0.398882,0.340316,0.278984,0.216664,1]

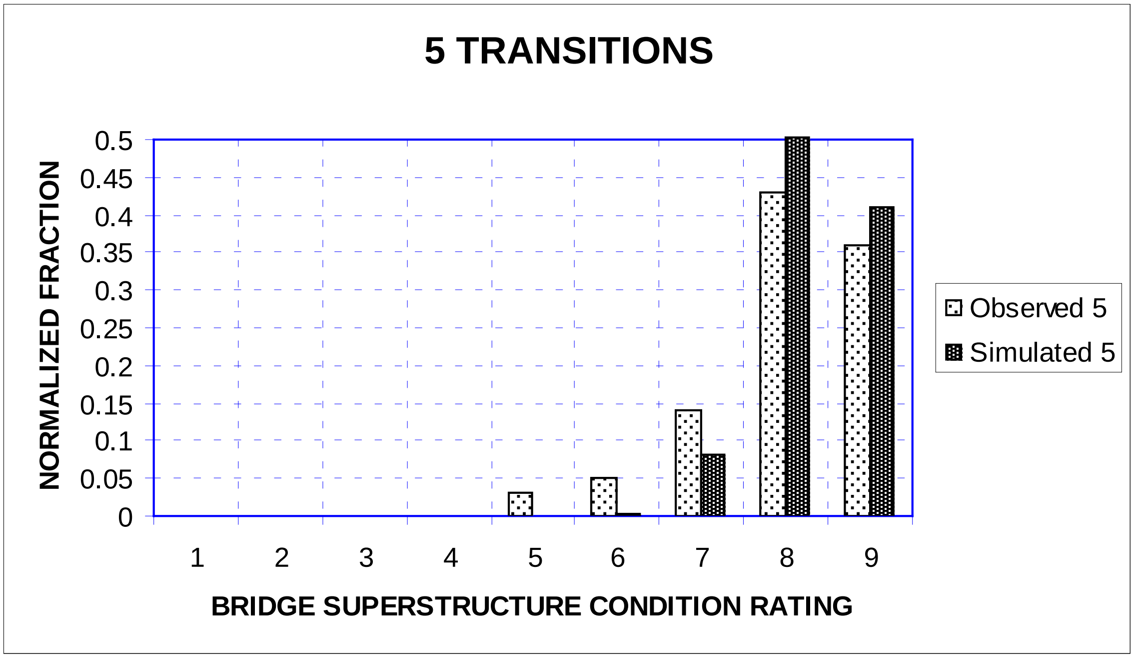

Figure 3.10a: Comparison of Observed and Simulated Normalized Superstructure Condition State Distributions for Bridges with Prestressed Superstructure for the Simulation of 30 Transitions.

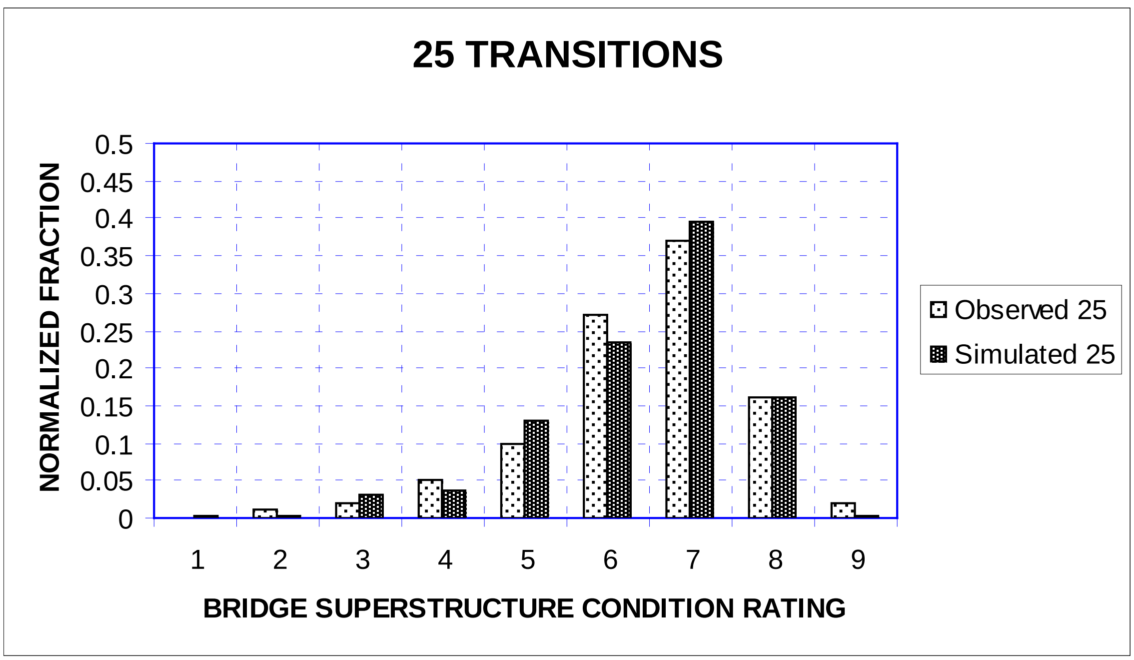

Figure 3.10b: Comparison of Observed and Simulated Normalized Superstructure Condition State Distributions for Bridges with Prestressed Superstructure for the Simulation of 30 Transitions.

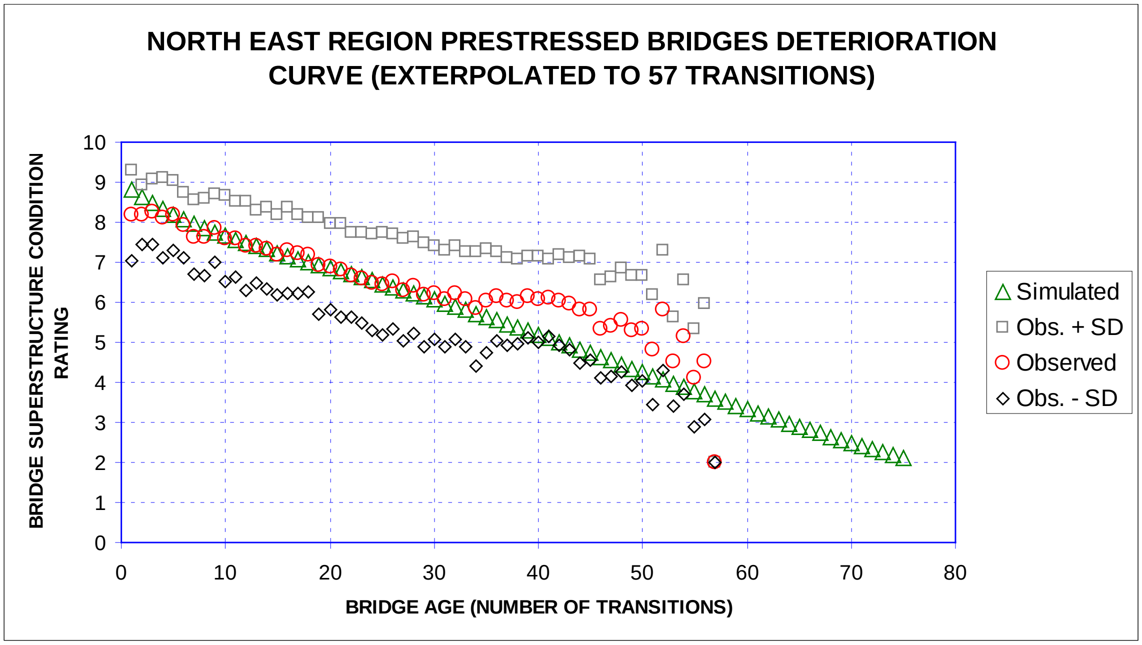

Figure 3.11: Comparison of Markov Simulation and Observed, Observed + Standard Deviation and Observed – Standard Deviation Superstructure Condition for Bridges with Prestressed Superstructure for the Simulation of 30 Transitions.

The simulated and the observed values are close, with an absolute mean error of 1.59%, when applied to the 30-year observed data.

The resulting transition probability vector for the bridges with prestressed superstructure is:

P= [0.800516,0.913123,0.941149,0.918976,0.919164,0.812091,0.923709,0.532096,0.470407, 1]

The simulated and the observed values are close, with an absolute mean error of 1.65%, when applied to the 30-year observed data.

3.4. Validation of Proposed Analysis

The validity of this assumption (performing the Markov deterioration simulation matrix for the first 20-40 years of the bridge life span and extrapolate it to 75 years which is the life time of the bridges) is investigated. Two different methods are applied.

First method considers the bridge database. The logic is simple; bridges are being closed to traffic when their condition rating decreases down to “3”. From condition rating “9” to “3” there are seven condition rating transitions and bridges are designed for 75 years. Therefore reducing condition rating by a minimum of “1” condition rating every 10 years will add up to a minimum of 70 years which is almost the design life span of the bridge. Therefore, it is assumed that the steel bridges condition rating will go down by a minimum of “1” condition rating every 10 years. Any bridge which has a condition rating more than the assumed value has a potential of repair activity and they are not included in the deterioration curve. Consequently, the bridge population is reduced in a way that, 1 to 10 years old bridges can have a condition rating of 9 to 1, 11 to 20 years old bridges can have a condition rating of 8 to 1, 21 to 30 years old bridges can have a condition rating of 7 to 1, 31 to 40 years old bridges can have a condition rating of 6 to 1, 41 to 50 years old bridges can have a condition rating of 5 to 1, 51 to 60 years old bridges can have a condition rating of 4 to 1 and 61 to 70 years old bridges can have a condition rating of 3 to 1. In order to illustrate, consider the following: When the bridges between the ages 41 to 50 years old are considered, the ones which have a condition rating 6 and above are not included and the ones with the condition rating 5 and below are included in the observed database. Modified observed data as described above and its Markov chain simulation is compared and presented in Figure 3.12.

The resulting transition probability vector for the bridges with steel superstructure is:

P= [0.851233,0.906870,0.964328,0.909108,0.808351,0.697264,0.570778,0.428015,0.184481,1]

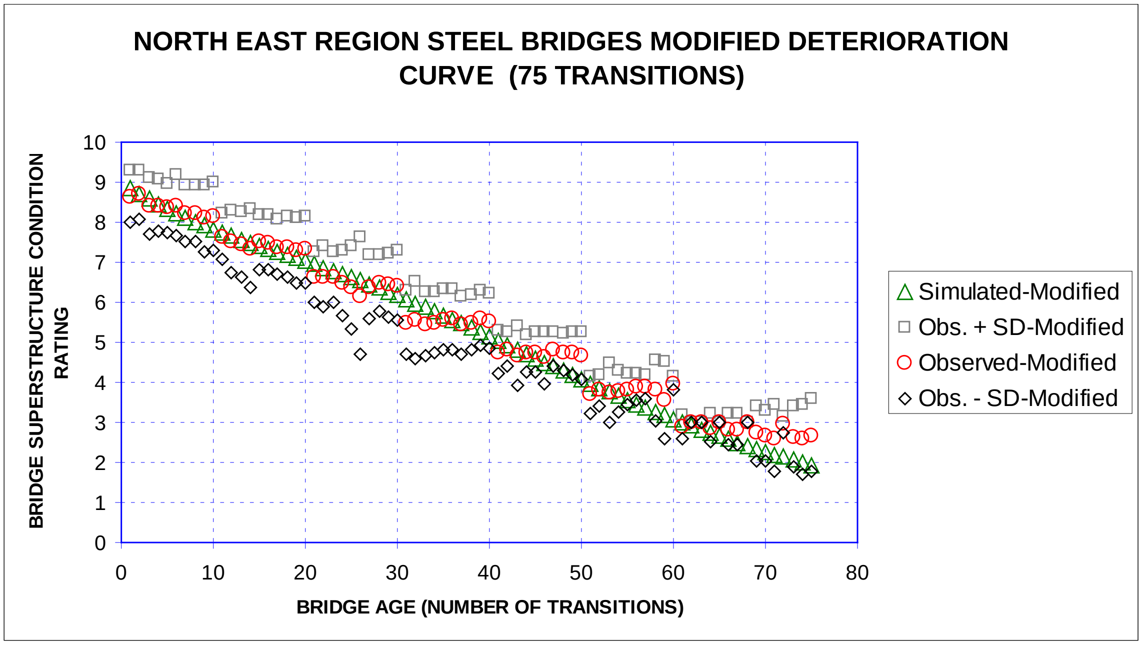

Figure 3.12: Comparison of Modified Markov Simulation and Modified Observed, Modified Observed + Standard Deviation and Modified Observed – Standard Deviation Superstructure Condition for Bridges with Steel Superstructure for the Simulation of 75 Transitions.

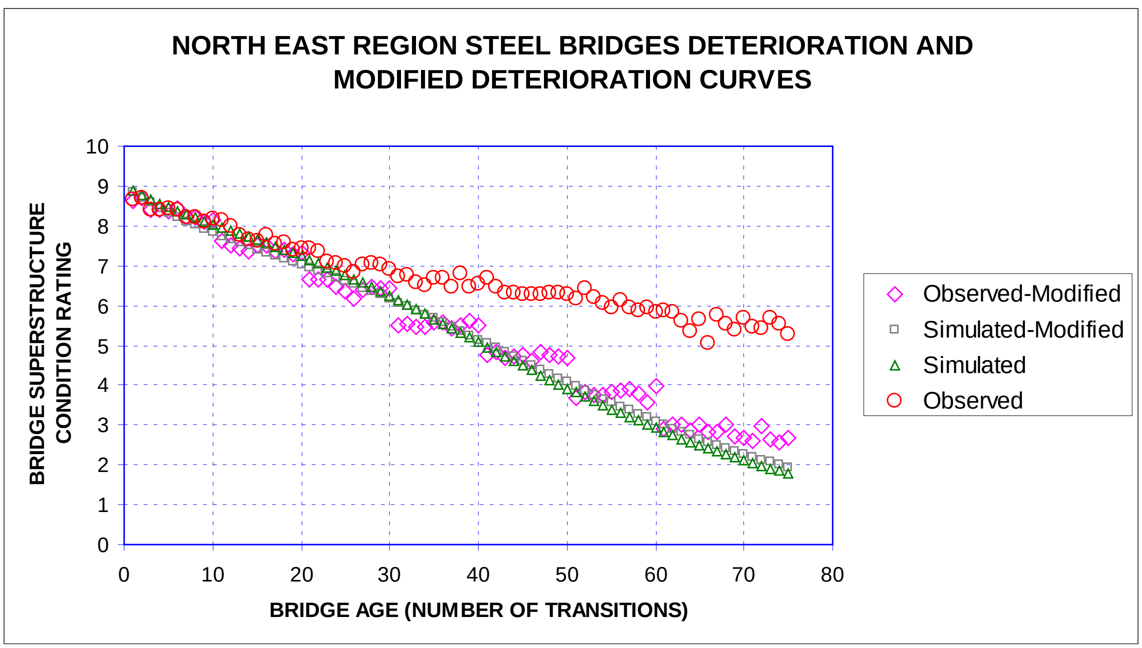

Figure 3.13: Comparison of Modified Markov Simulation (75 Transitions) and Modified Observed, Markov Simulation (30 Transitions) and Observed Superstructure Condition for Bridges with Steel Superstructure.

The simulated and the observed values are close, with an absolute mean error of 6.48%, when applied to the 75-year observed data.

Modified Markov Simulation (75 Transitions) and Markov Simulation (30 Transitions) curves for Bridges are compared and shown in Figure 3.13. Modified Observed deterioration and Observed deterioration curves are also shown in the same figure. As mentioned before the observed deterioration curve is becoming a more convex curve around after 30 transitions. The modified observed deterioration curve shows a steeper decline. Simulated deterioration curve with 30 transitions and the modified simulated deterioration curve with 75 transitions are very similar.

Second method considers the bridge deteriorations through an 18-year period and compares these deterioration data with the simulated deterioration. For this method, 18 years of NBI data from 1992 to 2009 for all the bridges in the Northeast Region of the US is considered. Data set for a total of 1,615,322 bridges is used. This makes an average of 89740 bridges for every year. Using Fortran subroutines, the raw data is compiled into 8 different groups. Each of these groups represents the bridge condition rating at ages 0, 1, 5, 10, 15, 20, 25 and 30. The bridges chosen for this analysis are the ones which never went through any repair or reconstruction activities. These bridges continuously deteriorated since their original construction date. The mean and the standard deviation of each group is calculated. Since most of the condition ratings in each group are from the same bridges at their different ages, combining all these data and plotting them gives us the continuous average deterioration of bridges in the Northeast Region for a period of 30 years.

In order to illustrate, consider a bridge which was built in 1962 and since then it did not go through any repair or was not reconstructed. In 1992, this bridge was 30 years old and was also inspected and its information was recorded. Considering that the NBI data from 1992 to 2009 is available, the condition rating information of this bridge is also readily accessible and it is included in the bridge condition rating data group of “30 years old”. Similarly a bridge which was constructed in 1979 and had no repair or reconstruction history can be placed in the groups “15 years old” (inspection data from 1994 NBI), “20 years old” (inspection data from 1999 NBI), “25 years old” (inspection data from 2004 NBI) and “30 years old” (inspection data from 2009 NBI) as long as its condition rating data is available during those years. As a final example, consider a bridge which was constructed in 1999 and had no repair or reconstruction history can be placed in the groups “0 year old” (inspection data from 1999 NBI), “1 year old” (inspection data from 2000 NBI), “5 years old” (inspection data from 2004 NBI) and “10 years old” (inspection data from 2009 NBI) as long as its condition rating data is available during those years.

This methodology facilitates inclusion of all the bridges in several age groups, which lets the groups to be related with each other and a continuous deterioration curve can be plotted. The age limit for the bridges is chosen as 30 years since the bridges are getting repaired more often as they get older and this changes the true characteristics of the deterioration curve. The bridges which were reconstructed are also excluded from the analysis group because it is observed that many of the reconstructed bridge data does not reflect new bridge condition information. This may be due to not replacing the existing bridge data with the new one in the database or it may be due to partial reconstruction which does not change the bridge condition.

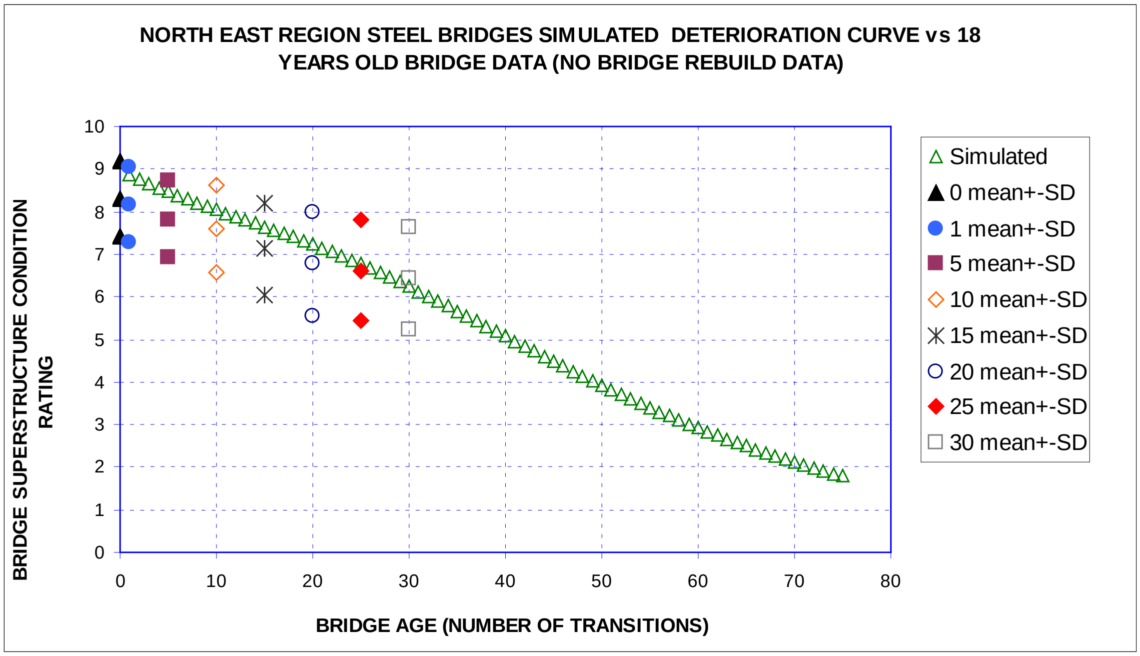

Figure 3.14: Comparison of Simulated Superstructure Deterioration Curve with Mean & Mean ± SD of Superstructure Condition Rating Obtained from 18 years of NBI Data from 1992 to 2009 Calculated for the Steel Bridges with no Repair or Reconstruction History when They were 0, 1, 5, 10, 15, 20, 25 & 30 Years Old in Northeast US.

	BRIDGE AGE DURING THE INSPECTION							
	0	**1**	**5**	**10**	**15**	**20**	**25**	**30**
Bridges with No Repair or Replacement History	2466	1547	2078	2712	3001	3056	4711	5521
All the Bridges	4799	3802	4988	6259	5806	5658	5821	6830

Table 3.2: Number of Bridges Used in the Age Groups in Figure 3.14, Northeast Region Steel Bridges

As explained above, 18 years of NBI data from 1992 to 2009 is available. Bridges with any repair or reconstruction history are eliminated and the data group is analyzed for the Northeastern US bridges. Mean and mean ± SD of superstructure condition ratings when the bridges were 0, 1, 5, 10, 15, 20, 25 & 30 years old are compared with the simulated superstructure deterioration curves (see Figure 3.14). As seen in the graph, the mean of the 30 years of continuous bridge data compares well to the simulated data. In Table 3.2, the number of bridges which were inspected during the 18-year period at each age group (all the bridges) and the ones which are used for the evaluation (bridges with no repair or replacement history) are shown. The same graph and table is shown for prestressed concrete bridges in Figure 3.15 and Table 3.3. Again, the results show a great agreement between the simulated data and the mean of the 30 years of continuous bridge data.

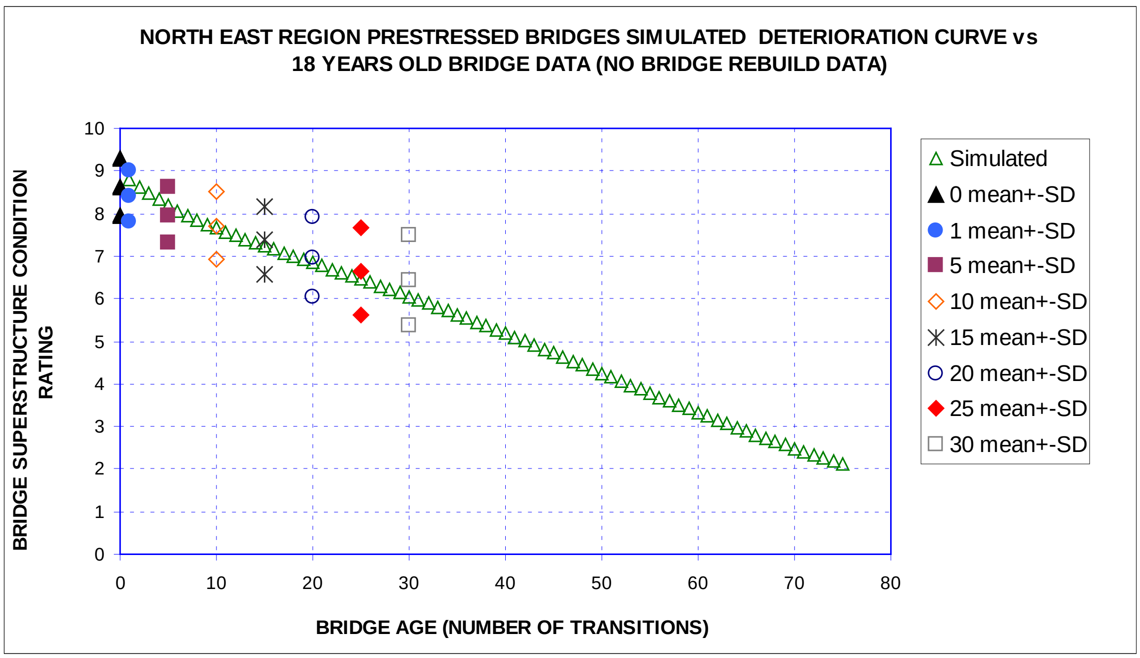

Figure 3.15: Comparison of Simulated Superstructure Deterioration Curve with Mean & Mean ± SD of Superstructure Condition Rating Obtained from 18 Years of NBI Data from 1992 to 2009 Calculated for the Prestressed Concrete Bridges with no Repair or Reconstruction History When They were 0, 1, 5, 10, 15, 20, 25 & 30 Years Old in Northeast US.

	BRIDGE AGE DURING THE INSPECTION							
	0	**1**	**5**	**10**	**15**	**20**	**25**	**30**
Bridges with No Repair or Replacement History	2475	1868	1919	2366	1780	1853	1957	2259
All the Bridges	3302	2505	2695	3455	2534	2359	2177	2415

Table 3.3: Number of Bridges Used in the Age Groups in Figure 3.15, Northeast Region Prestressed Concrete Bridges

These two methods clearly show that the methodology “Applying the Markov chain simulation for the first 30 transitions of the bridge and extrapolating it to 75 transitions” captures the true behavior of bridge deterioration.

3.5. Deterioration Curves

Deterioration curves for the deck, substructure and superstructure for the nationwide bridges are prepared. Several different parameters are considered These are;

- Material type. Steel (S) or Prestressed (PS) Superstructure).
- Length (L). Bridges less than 20 meters and bridges longer than 20 meters.
- Average Daily Traffic (ADT). ADT<10000 and ADT>10000.
- Climatic Regions in the US (Central, East North Central, Northeast, Northwest, South, Southeast, Southwest, West, West North Central).

In view of these parameters, a total of 72 deterioration curves are formed for bridge deck, substructure and superstructure.

In this dissertation the Northeast region bridges are used for the analysis. The bridge deck, substructure and superstructure, another 72 deterioration curves, which simulate the mean and the standard deviation of the deterioration values are developed. All these Markov Chain deterioration curves are shown in the Markov vector (P) format in Tables A2.1 through A2.4 in Appendix A2.

3.6. Effect of the Parameters on Deterioration Curves

The effects of the parameters, which are presented in the previous section, on the deterioration curves are investigated and shown in Figures 3.16 through 3.19. It is observed that the material type does not have a significant effect on the deterioration of bridge deck, substructure and superstructure, however long bridges have a steeper

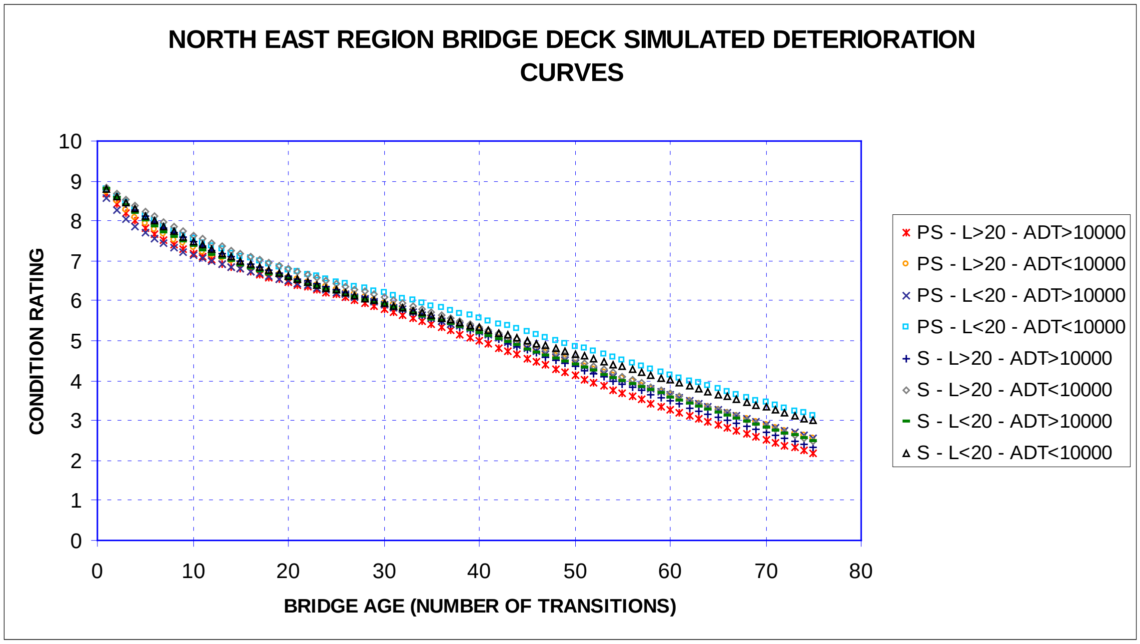

Figure 3.16: Comparison of Northeast Region Bridge Deck Simulated Deterioration Curves.

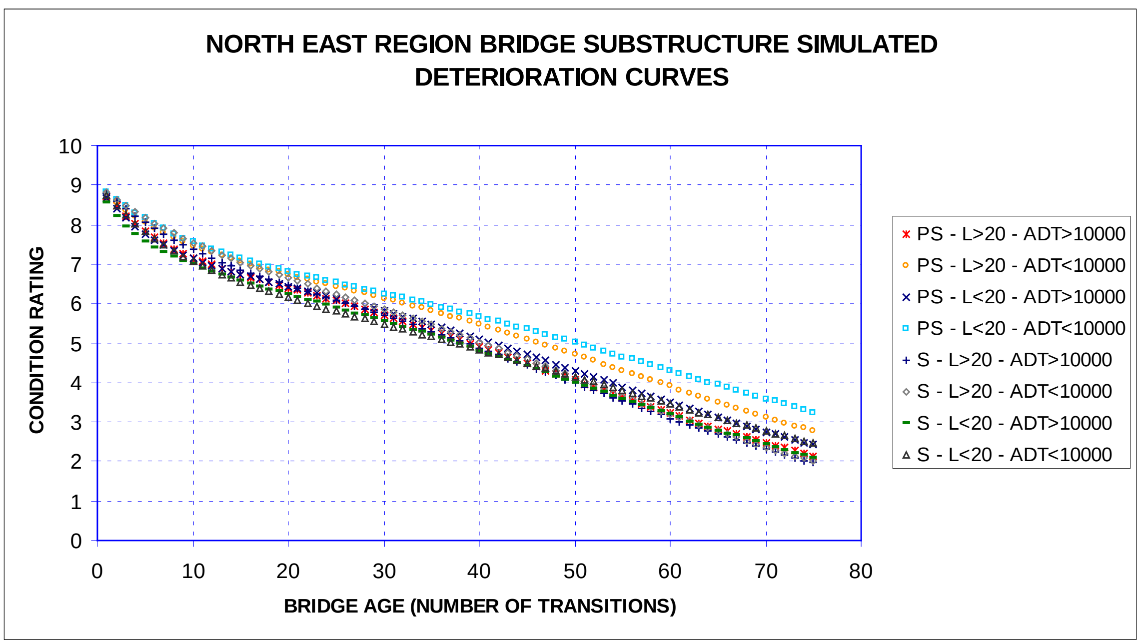

Figure 3.17: Comparison of Northeast Region Bridge Substructure Simulated Deterioration Curves.

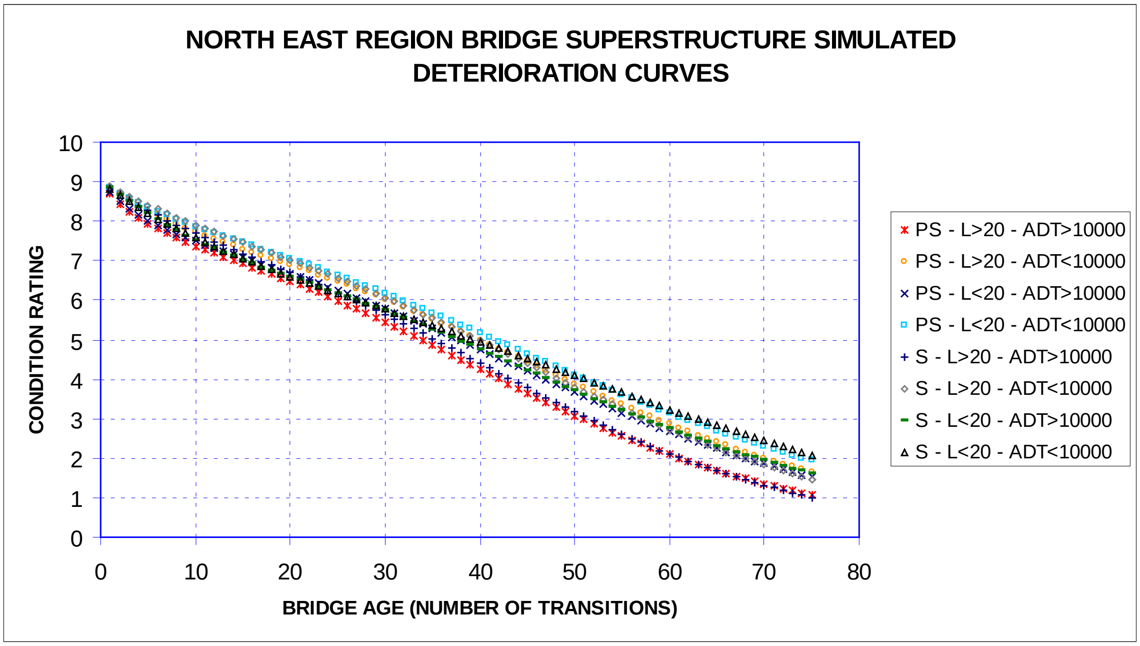

Figure 3.18: Comparison of Northeast Region Bridge Superstructure Simulated Deterioration Curves.

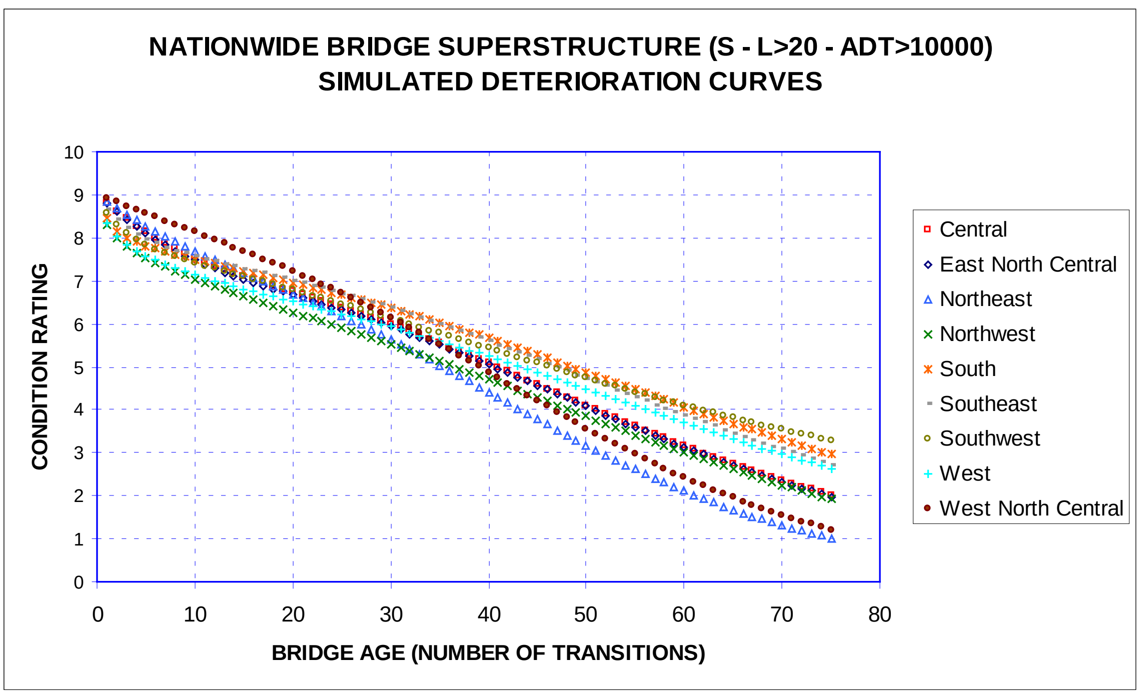

Figure 3.19: Comparison of Nationwide Bridge Superstructure (S – L>20 – ADT>10000) Simulated Deterioration Curves for Different Climatic Regions.

deterioration rate than the short ones. The bridges with more ADT deteriorates at a higher rate than the ones with less ADT. Finally, comparison of the bridge deterioration curves in different climatic regions shows that the bridges in the Northern side of the US has steeper deterioration rates than the ones in the Southern regions of the US.

3.7. Rehabilitation, Repair or Replacement Markov Models

In order to simulate the rehabilitation, repair or reconstruction activities for bridge deck, substructure and superstructure, 10 X 10 Markovian transition probability matrixes are formed. These matrixes can be prepared for every state in the bridge management system. In this research it is assumed that a bridge part that went through a rehabilitation activity shall have minimum condition of 5 (Fair), a bridge part that had a repair shall have minimum condition of 7 (Good), and a bridge part that had a replacement shall have condition of 9 (Excellent). The proposed transition probability vectors are in the format of:

$$P_{rehabilittion} = [p_{99}(<1), p_{88}(<1), p_{77}(<1), p_{66}(<1), 1, 1, 1, 1, 1, 1]$$

$$P_{repair} = [p_{99}(<1), p_{88}(<1), 1\ 1, 1, 1, 1, 1, 1, 1]$$

$$P_{replacement} = [1, 1, 1, 1, 1, 1, 1, 1, 1, 1] \quad \text{(Eq. 3.7)}$$

3.8. Bridge Deterioration – Cost Relationship

The relationship between the bridge deterioration and the agency cost is very obvious. As the bridge deteriorates, the repair activities or the bridge replacement will cost the agency, material and man-work hours. The relationship between the bridge deterioration and the user cost is a little more complicated. Basically, the whole idea is, when the bridge deteriorates, its condition rating decreases so is the operating rating of the bridge. This causes the heavy trucks to detour their routes. This will result in more time and gas consumption and it will be the direct user cost for the deteriorating bridges.

Linear regression equations for the different regions of the US representing the operating rating of the bridge due to the Bridge Condition Rating is developed (Equation 3.8). According to this rating, the allowable truck weight on the bridge is calculated and this process was included in the BLCCA for rehabilitation decision and the user cost purposes.

$$\psi = \theta * V + C \qquad \text{(Eq. 3.8)}$$

where

ψ: Operating rating

θ: Regression coefficient

C: Constant

V: Bridge condition rating

Table 3.4 shows the values of the regression coefficients for different regions in the US. The relationship between the condition rating and the operating rating of the bridges is further investigated and explained in the following pages.

Regions	**α**	**C**	**R^2**	**t-stat**
Central	5.714	3.731	0.94	10.50
East North Central	7.043	1.903	0.96	13.86
Northeast	8.103	9.830	0.92	8.74
Northwest	6.096	7.408	0.97	13.15
South	6.668	-6.747	0.98	16.75
Southeast	8.955	-9.858	0.99	24.34
Southwest	6.835	2.335	0.89	6.97
West	5.205	22.866	0.90	6.71
West North Central	7.205	-6.186	0.96	13.61

Table 3.4: Linear Regression Coefficients for Bridge Condition Rating and Operating Rating

3.9. Bridge Pavement Deterioration

Bridge and approach pavement condition rating data is not available in the NBI, therefore a different approach than the Markov chain simulation is considered for the bridge and approach pavement deterioration model. The International Roughness Index (IRI) is used. Pavement roughness, which is generally defined as an expression of the irregularities in the pavement surface, is used as a measure of pavement condition. Roughness is typically quantified using some form of either present serviceability rating (PSR), ride quality index (RQI), IRI, with IRI being the most prevalent. According to the FHWA, IRI is an objective measure of pavement roughness and is accepted as a standard in the pavement evaluation community. IRI is based on the accumulated suspension of a vehicle (inches or mm) divided by the distance traveled by the vehicle during the measurement (miles or kilometers). The lower values of IRI correspond to higher quality pavements. IRI value typically increases as the pavement ages. The roughness prediction model is used in BLCCA to estimate the bridge and approach deterioration rates.

IRI depends on traffic volume. The consideration of traffic should be consisting of loading magnitude and number of load repetitions. The annual average daily traffic data (AADT) collected by highway agencies is converted into the equivalent single axle load (ESAL), which is the number of repetitions of a standard 18-kip axle load. In addition to the AADT, the conversion accounts for the heavy vehicle proportions, the lane distributional factor and the directional distributional factor.

$$\mathrm{ESAL} = \mathrm{ADT} * \mathrm{HV} * \mathrm{LEF} * \mathrm{D} * \mathrm{L} * 365 \qquad \text{(Eq. 3.9)}$$

$$\mathrm{CESAL} = \mathrm{ESAL} * \frac{(1+g_c)^n - 1}{g_c} \qquad \text{(Eq. 3.10)}$$

where

ADT : Average Daily Traffic

g_c : Combined axle weight and traffic volume growth rate

HV : Heavy vehicle proportions

LEF : Load equivalency factor

D : Directional distribution proportion

L : Lane distribution proportion

ESAL : 18-kip equivalent single axle load

CESAL : Cumulative 18-kip equivalent single axle load

The equation of IRI is:

$$IRI = [\frac{CESAL}{1000}]^{0.25} * 10^{Z} \qquad \text{(Eq. 3.11)}$$

where

$$\begin{aligned} Z &= 0.0403 + 0.00014 * AC_{VISC} + 0.0704 * AC_{void} + 0.314 * \log(AC_{thick}) - 0.00162 * B_{thick} \\ &- 0.00165 * DGT + 0.00001628 * FI * AC_{void} \end{aligned} \qquad \text{(Eq. 3.12)}$$

where

CESAL : Cumulative 18-kip equivalent single axle load

IRI : International Roughness Index

AC_{visc} : Asphalt viscosity

AC_{void} : Asphalt air voids

B_{thick} : Base layer thickness

DGT : Annual days of temperature above 90 F°

FI : Freeze index

3.10. Deterioration Model Supplementary Improvements

The process of designing a detailed and accurate deterioration model was described in previous pages. In this section several supplementary improvements for the deterioration model are described. These are:

- Bayesian updating of the model
- Including unexpected events in the model (seismic, scour, terrorism)
- Relating bridge condition rating with bridge reliability index.

Since they are not necessarily required to be applied in the life cycle cost calculations, they are not included in the optimization process. They can be used in future research and can even be further investigated as individual research topics.

3.10.1. Bayesian Updating of the Model

The method of Bayesian updating is based on Bayes Theorem, which has been widely used for statistical inference. It starts with prior information and a measure of certainty regarding the prior information. When new sample data are available they are incorporated with the prior into a new answer, which is also called the posterior. With more sample data, the uncertainty regarding the new answer diminishes and the following answers improve.

Bridge condition rating transitions probability distribution functions are assumed to be normally distributed (Figure 3.20) consequently the variance is known. The mean and variance of the posterior can be expressed as a function of the mean and variance of the prior and the updating data in the functional form as shown in the equations 3.13 and 3.14 (Atherton and Ben- Akiva 1976).This functional form is known as a normal-normal conjugate prior. The posterior produced with this function is also normally distributed.

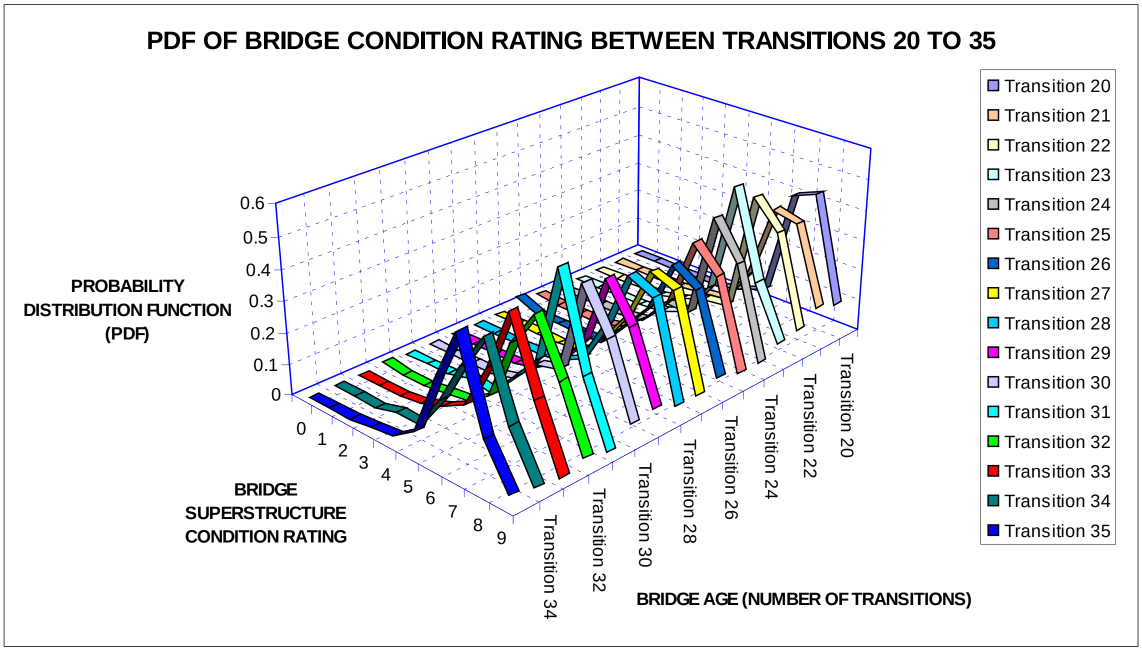

Figure 3.20: Probability Values of Bridge Condition Rating between Transitions 20 to 35 for Northeast Region Steel Bridges (2009)

$$\theta_{CS_{updated}} = \frac{\dfrac{\theta_{CS_{prior}}}{\sigma^2_{CS_{prior}}} + \dfrac{\theta_{CS_{updating}}}{\sigma^2_{CS_{updating}}}}{\dfrac{1}{\sigma^2_{CS_{prior}}} + \dfrac{1}{\sigma^2_{CS_{updating}}}} \qquad \text{(Eq. 3.13)}$$

and

$$\sigma^2_{CS_{updated}} = \frac{1}{\dfrac{1}{\sigma^2_{CS_{prior}}} + \dfrac{1}{\sigma^2_{CS_{updating}}}} \qquad \text{(Eq. 3.14)}$$

where

θ_i : Mean of the Condition State for Transition i

σ_i^2 : Variance of the Mean of the Condition State for Transition i

$CS_{updated}$: Bayesian updated Bridge Condition State

CS_{prior} : Prior Bridge Condition State

$CS_{updating}$: Updating Bridge Condition State

In the equations 3.13 and 3.14, data values from the data sources are weighted by the inverse of their variance to achieve a value for the updated data item. This is a very good feature, because data values with greater certainty contribute more to the estimate of the updated data item than those with less certainty. When the prior data are reliable, a relatively small sample can be used for updating. However, in the cases where the prior data are not very reliable, a relatively large updating sample is more likely to be needed. In both cases, the variance of the posterior data will always be less than that of both the prior and the updating sample.

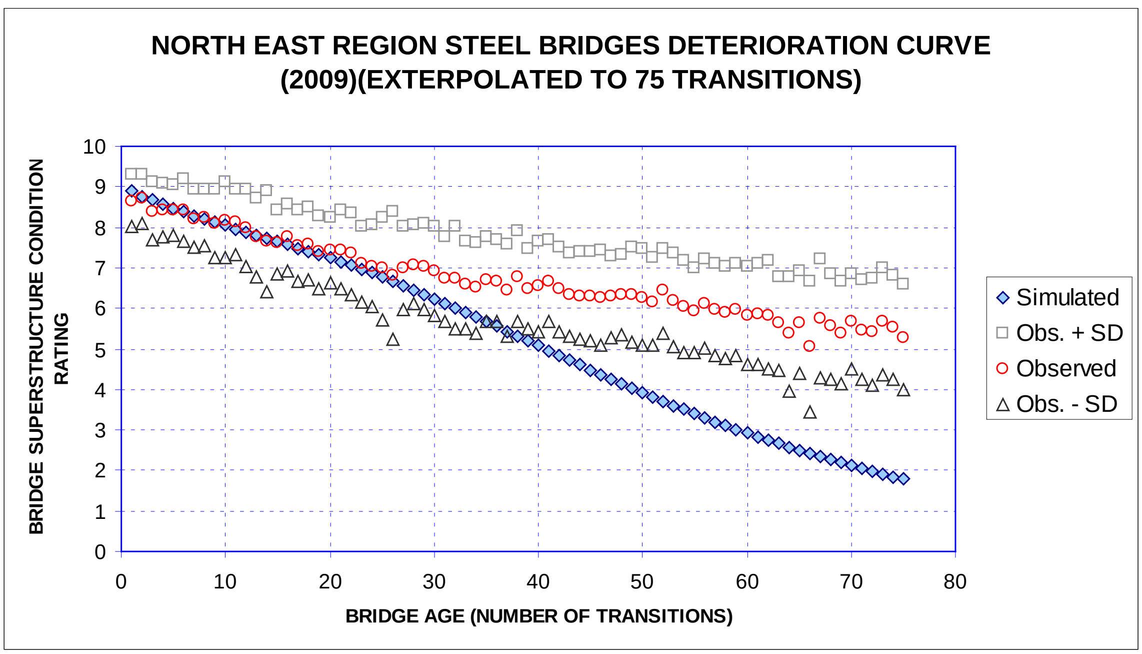

Figure 3.21: Comparison of 2009 Markov Simulation and Observed, Observed + Standard Deviation and Observed – Standard Deviation Superstructure Condition for Bridges with Steel Superstructure for the Simulation of 30 Transitions.

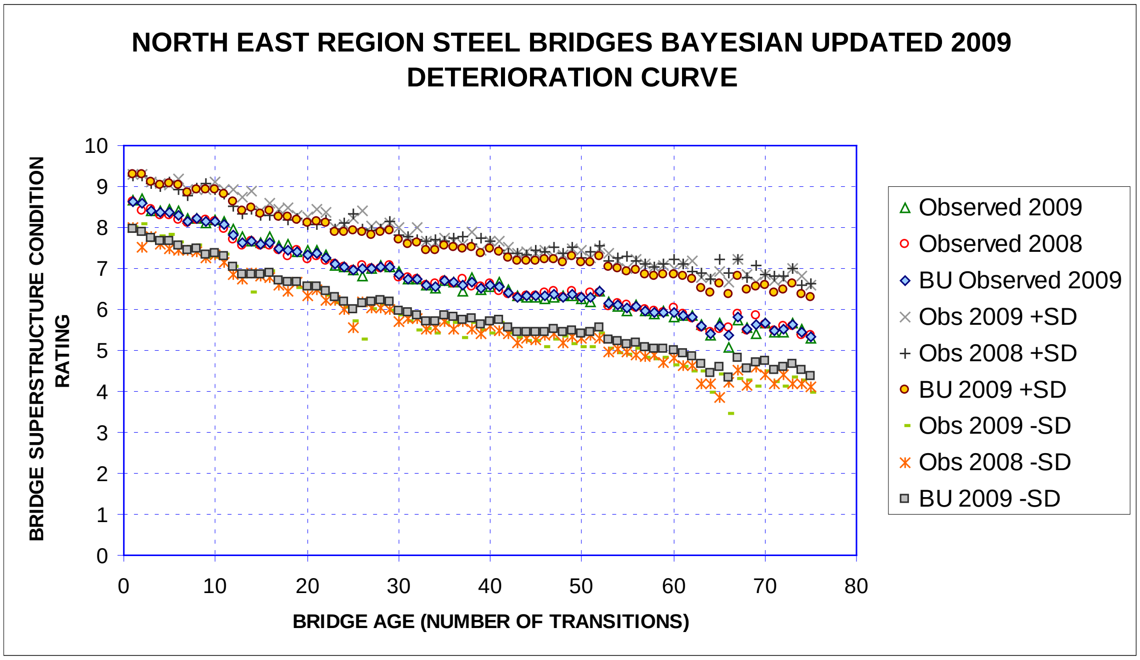

Figure 3.22: Comparison of 2008-2009 Observed and Bayesian Updated, 2008-2009 Observed and Bayesian Updated + Standard Deviation, 2008-2009 Observed and Bayesian Updated – Standard Deviation Superstructure Condition for Bridges with Steel Superstructure.

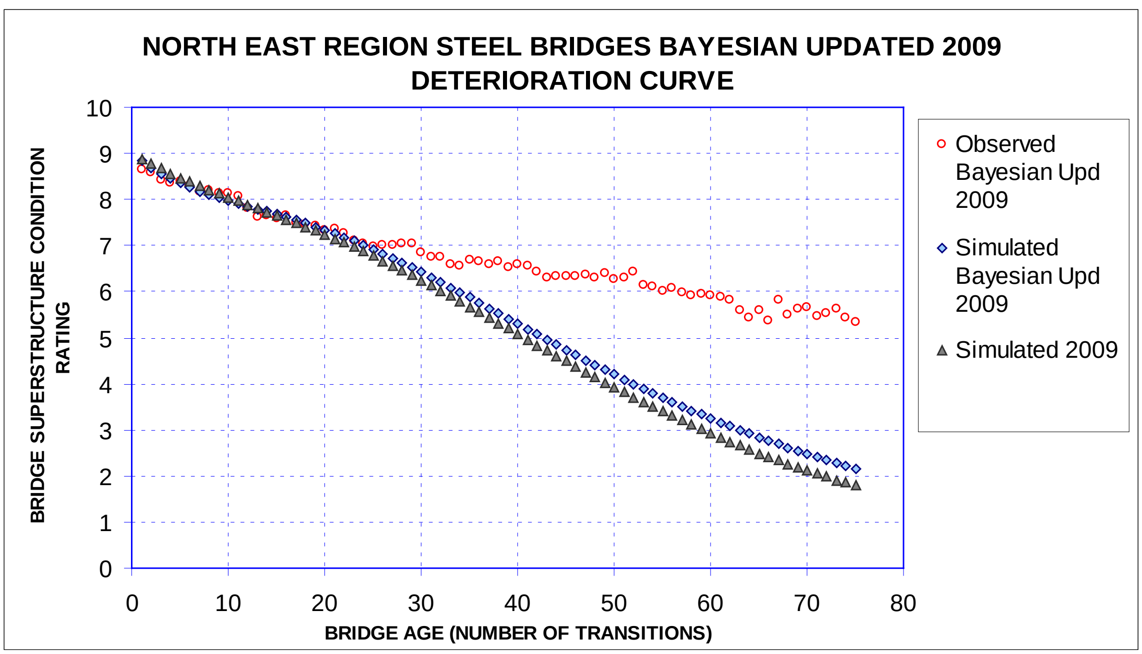

Figure 3.23: Comparison of 2009 Simulated, 2009 Bayesian Updated Simulated and Observed Superstructure Condition for Bridges with Steel Superstructure.

The Bayesian Updating of the model can be explained best with an illustrative case study. In this case study, CS_{prior} is the year 2009 Northeast region steel bridges deterioration model (Figure 3.21). Assuming the year 2008 Northeast region steel bridges deterioration model was very reliable, it is used as $CS_{updating}$. The observed deterioration curve (ODC) and ±SD for 2008 and 2009 data is calculated. Using equations 3.13 and 3.14, $CS_{updated}$ is obtained (see Figure 3.22). In the figure (BU) represents Bayesian Updated, (Obs) represents Observed and (SD) represents Standard Deviation. At this stage, Markov Simulation Matrix is applied for the first 30 years of the bridge life span on Bayesian Updated Observed 2009 deterioration curve and extrapolated to 75 years. Finally, 2009 simulated condition rating data is compared with 2009 Bayesian updated simulated condition rating data for bridges with steel superstructure (see Figure 3.23). As can be seen, when the reliable bridge deterioration data is available, it is possible to improve the model and make it more accurate.

3.10.2. Including Unexpected Events in the Model

In the previous sections, a deterioration system was described where the component deteriorations are represented by Markov transition probabilities. The component deteriorations are gradual processes which are caused by the environmental factors, bridge physical or geometrical properties or by the effect of the traffic. Therefore, these effects do not include the unexpected events such as earthquake, scour or terrorism. In this section, these unexpected events are incorporated into the deterioration system.

The probability of an unexpected event occurring and causing the bridge condition to drop to a certain condition state or making the bridge non-functional is incorporated into the Markov matrix. Since the DOTs start replacing and rebuilding the bridges damaged or deteriorated to “Serious Condition State”, as the worst case scenario the bridge condition state “3” was considered. It is assumed that an unexpected event damages the bridge primary structural elements seriously. It is closed to traffic and becomes nonfunctional. The new Markov matrix is formulated in Eq 3.15. In this matrix, the sum of each row is equal to 1.

$$T = \begin{bmatrix} p_{99} & 1-p_{99}-\Sigma p_{ue} & p_{ue79} & p_{ue69} & p_{ue59} & p_{ue49} & p_{ue39} & 0 & 0 & 0 \\ 0 & p_{88} & 1-p_{88}-\Sigma p_{ue} & p_{ue68} & p_{ue58} & p_{ue48} & p_{ue38} & 0 & 0 & 0 \\ 0 & 0 & p_{77} & 1-p_{77}-\Sigma p_{ue} & p_{ue57} & p_{ue47} & p_{ue37} & 0 & 0 & 0 \\ 0 & 0 & 0 & p_{66} & 1-p_{66}-\Sigma p_{ue} & p_{ue46} & p_{ue36} & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & p_{55} & 1-p_{55}-\Sigma p_{ue} & p_{ue35} & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & p_{44} & 1-p_{44}-\Sigma p_{ue} & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & p_{33} & 1-p_{33} & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & p_{22} & 1-p_{22} & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & p_{11} & 1-p_{11} \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & p_{00} \end{bmatrix}$$

(Eq. 3.15)

A case study is now presented to show the effect of unexpected events incorporated into the deterioration model. Northeast Region Steel Bridges are used in the study. Initially, it is assumed that the unexpected event makes the bridge non-functional and the condition state drops to “Serious, 3”. The variable is the probability of the unexpected event which drops the bridge condition state to “3”. The probability of occurrence of this event is assumed to be changing from 0% to 5%. The details are shown in Eq. 3.16 where:

$0\% < p_{ue3i} < 5\% \quad (4 \leq i \leq 9)$

$\sum T[1,j] = \sum T[2,j] = \sum T[3,j] = \sum T[4,j] = \sum T[5,j] = \sum T[6,j] = \sum T[7,j] = \sum T[8,j]$

$= \sum T[9,j] = \sum T[10,j] = 1 \qquad (1 \leq j \leq 10)$

$$T = \begin{bmatrix} p_{99} & 1-p_{99}-p_{ue39} & 0 & 0 & 0 & 0 & p_{ue39} & 0 & 0 & 0 \\ 0 & p_{88} & 1-p_{88}-p_{ue38} & 0 & 0 & 0 & p_{ue38} & 0 & 0 & 0 \\ 0 & 0 & p_{77} & 1-p_{77}-p_{ue37} & 0 & 0 & p_{ue37} & 0 & 0 & 0 \\ 0 & 0 & 0 & p_{66} & 1-p_{66}-p_{ue36} & 0 & p_{ue36} & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & p_{55} & 1-p_{55}-p_{ue35} & p_{ue35} & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & p_{44} & 1-p_{44}-p_{ue34} & 0 & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & p_{33} & 1-p_{33} & 0 & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & p_{22} & 1-p_{22} & 0 \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & p_{11} & 1-p_{11} \\ 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & p_{00} \end{bmatrix}$$

(Eq. 3.16)

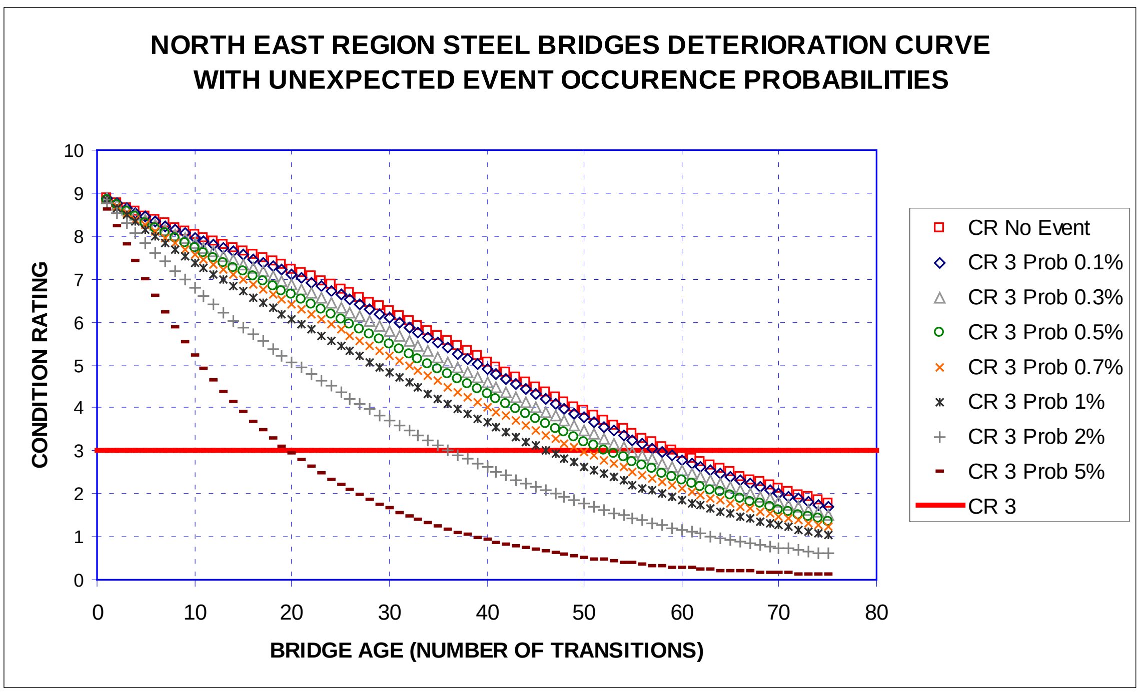

Figure 3.24: Comparison of 2009 Simulated Superstructure Condition for Different Unexpected Event Occurrence Probabilities for Northeast Region Bridges with Steel Superstructure.

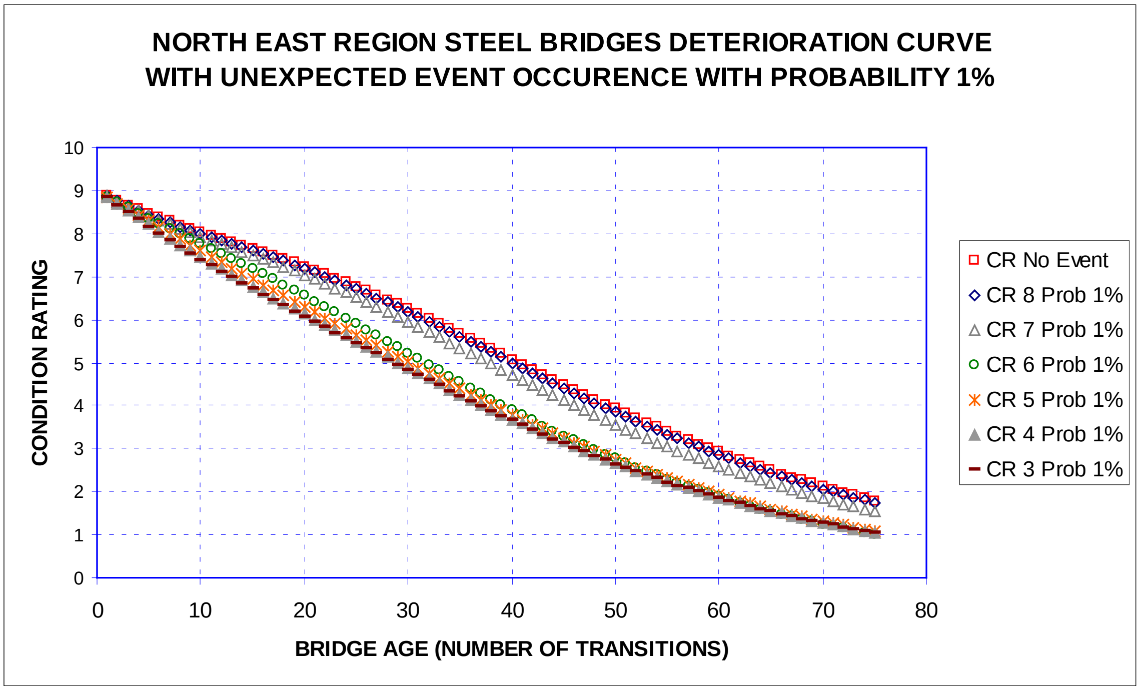

Figure 3.25: Comparison of 2009 Simulated Superstructure Condition by 1% Unexpected Event Occurrence Probability for Different Condition Ratings for Northeast Region Bridges with Steel Superstructure.

The results of the case study are shown in Figures 3.24 and 3.25. In Figure 3.24, as the probability of an unexpected event that drops the structures condition rating to “3” increases, the deterioration curve decreases more rapidly. In Figure 3.25, the concept is investigated by assuming a constant probability of 1% for the unexpected event occurrence at and applying this to condition ratings from “8” to “3”. As expected, the deterioration curve of smaller condition ratings decreases more rapidly than the higher condition ratings considering the same unexpected event occurrence probability.

“Bayesian Updating” and “Unexpected Events” are two deterioration model improvements which enables the deterioration model to adapt to different situations in real life. Therefore they can be incorporated into the proposed Life Cycle Cost

Optimization Model whenever necessary. These topics can be further investigated in future research endeavors.

3.10.3. Bridge Condition Rating and Bridge Reliability Indices (β)

3.10.3.1. Description of the New Approach

In this dissertation, so far the bridge deterioration is simulated by the bridge condition rating evaluation. The main reasons for this are:

- DOTs use bridge condition rating as the main criteria when deciding which bridges need to be repaired or replaced.
- The last 18 years of condition rating data of the bridges is available in the NBI database. Instead of trying to create bridge deterioration data (instrumentation or detailed modeling) which will be bridge-specific and limited to the certain number of bridges for the research, it is decided to use the readily available bridge data.

Many researchers attempted to use different methodologies for their bridge deterioration models which are the basis of the decision making for the repair activities. Using a bridge reliability index is the most well known method. . In many of the previous studies, the deterioration models prepared from bridge condition ratings and reliability indices were compared and their differences were shown. However, in reality, they are not much different than each other. Bridge condition ratings in the US are recorded every two years and this information is placed in the Structure Inventory & Appraisal (SI&A) sheets. In the meantime, if a section loss progresses further or a previous problem is repaired on the bridge, then the engineers responsible for the inspection are also responsible to prepare an inventory and operating rating analysis according to the new section properties using the design truck for that bridge. The resulting new analysis data is also recorded in the SI&A sheets. If a bridge reliability index can be set from inventory

Span (ft)	average	1 day	2 weeks	1 month	2 months	6 months	1 year	5 years	50 years	75 years
10	0.62	0.97	1.12	1.18	1.23	1.30	1.37	1.46	1.63	1.65
20	0.71	1.15	1.25	1.31	1.36	1.41	1.47	1.56	1.66	1.68
30	0.74	1.20	1.32	1.37	1.42	1.47	1.52	1.61	1.70	1.72
40	0.75	1.31	1.42	1.46	1.50	1.55	1.58	1.64	1.72	1.74
50	0.72	1.32	1.43	1.47	1.52	1.56	1.60	1.65	1.73	1.75
60	0.72	1.37	1.47	1.52	1.56	1.60	1.64	1.69	1.77	1.79
70	0.74	1.42	1.51	1.56	1.60	1.64	1.68	1.74	1.81	1.83
80	0.77	1.47	1.55	1.60	1.64	1.68	1.73	1.79	1.86	1.89
90	0.79	1.51	1.60	1.64	1.68	1.72	1.78	1.84	1.92	1.94
100	0.82	1.55	1.64	1.68	1.72	1.76	1.82	1.89	1.98	2.00
110	0.84	1.60	1.68	1.72	1.76	1.81	1.86	1.94	2.03	2.05
120	0.85	1.63	1.72	1.76	1.80	1.85	1.90	1.97	2.06	2.08
130	0.86	1.66	1.75	1.80	1.83	1.87	1.92	1.99	2.08	2.10
140	0.86	1.67	1.76	1.80	1.83	1.87	1.92	1.99	2.08	2.10
150	0.85	1.64	1.73	1.78	1.81	1.84	1.88	1.96	2.05	2.07
160	0.84	1.60	1.68	1.73	1.76	1.80	1.84	1.91	2.01	2.03
170	0.81	1.56	1.63	1.69	1.72	1.76	1.80	1.87	1.96	1.98
180	0.78	1.50	1.58	1.64	1.67	1.71	1.75	1.82	1.91	1.94
190	0.75	1.45	1.53	1.58	1.62	1.66	1.70	1.77	1.86	1.88
200	0.70	1.38	1.48	1.54	1.57	1.60	1.64	1.71	1.80	1.82

Table 3.5: “Table B-2” of NCHRP – Report 368. Mean Maximum Moments for Simple Spans Due to a Single Truck (Divided by Corresponding HS20 Moment)

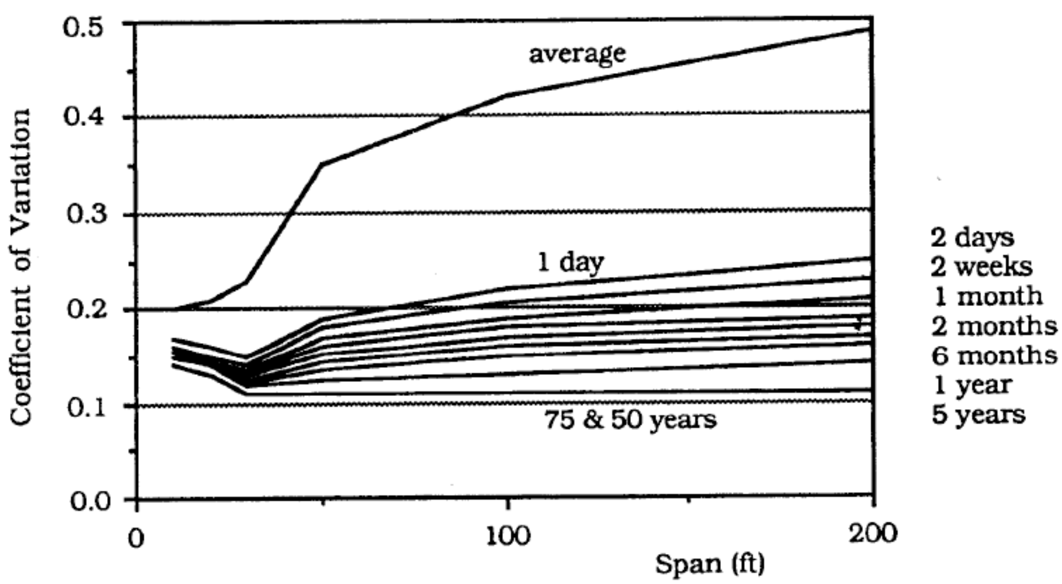

Figure 3.26: “Fig. B-11” of NCHRP – Report 368. Coefficient of Variation of the Maximum Moment Due to a Single Truck

or operating rating then any change on the condition rating of a structural load carrying member will have direct effect on the inventory or operating rating and so is on the reliability index.

3.10.3.2. Case Study

In this research, a methodology for obtaining a reliability index from the SI&A data in NBI database is developed. Using the definition of reliability index given by Equation (eq. 3.17), the parameters needed are the mean resistance, standard deviation of resistance, mean load and standard deviation of load. The operating rating of the bridges can easily be used as resistance data which gives the current live load capacity of the bridges and the load data can easily be obtained from NCHRP – Report 368. And the resistance and load values for bridge dead load can be obtained from Nowak et.al, 1979 and from "Development of a Probability Based Load Criterion for American National Standard A58, 1980".

A case study is presented here to show how the system works for the prestressed bridges from the Northeast region of the US:

For the live load values, the Table B-2 of NCHRP – Report 368, “Mean Maximum Moments for Simple Spans Due to a Single Truck (Divided by Corresponding HS20 Moment)” is used (Table 3.5). For the standard deviation of the load values Fig. B-11 of NCHRP – Report 368, “Coefficient of Variation of the Maximum Moment Due to a Single Truck” is used (Figure 3.26). For the live load resistance part, prestressed bridges from the Northeast region of the US for the year 2009 NBI data are selected. Among these bridges, the ones whose operating rating was calculated according to HS20 truck are used for the study. For the dead load and resistance, standard prestressed sections

(AASHTO I to VI and B I to VI) with known dead loads are used and mean to nominal ratio of "1.03" and the coefficient of variation value of "0.04" are used from the tables in Nowak et.al, 1979.

$$\beta = \frac{(\mu_R - \mu_Q)}{\sqrt{\sigma_R^2 + \sigma_Q^2}} \qquad \text{(Eq. 3.17)}$$

where

β : Reliability index for the bridge

μ_R : Mean resistance ($\mu_{\text{live load resistant}} + \mu_{\text{dead load resistance}}$, Sum of the Operating Rating of the year 2009 NBI data for the Northeastern region prestressed bridges for HS20 truck and dead load of the prestressed girder with its effective deck width divided by mean to nominal ratio, Nowak et.al, 1979. These are calculated for different bridge lengths and different bridge condition ratings.

μ_Q : Mean load ($\mu_{\text{live load}} + \mu_{\text{dead load}}$,. Sum of mean maximum moments for simple spans due to a truck (normalized with corresponding HS20 moment) from NCHRP – Report 368 and dead load of the prestressed girder with its effective deck width

σ_R : Standard deviation of resistance ($\sqrt{\sigma^2_{\text{live load resistant}} + \sigma^2_{\text{dead load resistance}}}$, Combination of standard deviation of Operating Rating of year 2009 NBI data for the Northeastern region prestressed bridges for HS20 truck. Calculated for different bridge lengths and different bridge condition ratings and standard deviation of dead load of the prestressed girder with its effective deck width divided by mean to nominal ratio)

σ_Q : Standard deviation of load ($\sqrt{\sigma^2_{\text{live load}} + \sigma^2_{\text{dead load}}}$, Standard deviation of mean maximum moments for simple spans due to a truck (normalized with corresponding

HS20 moment) from NCHRP – Report 368 and standard deviation of dead load of the prestressed girder with its effective deck width.

In the NBI database, the operating rating is shown in metric tons. Simply, the capacity-demand ratio is multiplied with the design truck (HS20 in this dissertation). It is not possible to know which part of the bridge the operating rating is for if the bridge reports are not individually investigated, however in the majority of the bridges, they are critical for their maximum moment carrying capacities. Therefore, it is assumed that all the data from the NBI reflects the live load moment capacities of the bridges. In NCHRP – Report 368, the loading is shown as the moment capacity of the bridges in HS20 units. Simply multiplying them with 32.4 (HS20 metric ton weight) will give the load values in metric tons. Dead loads and resistances are also calculated in metric tons. Now that both the resistance and the load values are in metric tons, it is possible to proceed and calculate β values.

In Appendix A3, the calculations of β values for bridges with different lengths and different condition ratings are presented in Tables A3.1 through A3.7. The mean age which is also calculated using the NBI database for each case, is also shown in the same tables. Using these tables, the graph for β vs. Span Length for different bridge condition ratings is plotted (Figure 3.27). From this graph it is clear that the β values decrease as the bridge condition rating decreases. In Figure 3.28, β vs. Bridge Age graph is represented. As expected, as the bridges get older their β values decrease.

The β vs. Bridge Condition Rating graph is shown in Figure 3.29, which clearly shows the same result as Figure 3.27. As bridge condition rating decreases, β value also decreases, which agrees with the inspection procedures; "any deterioration which will

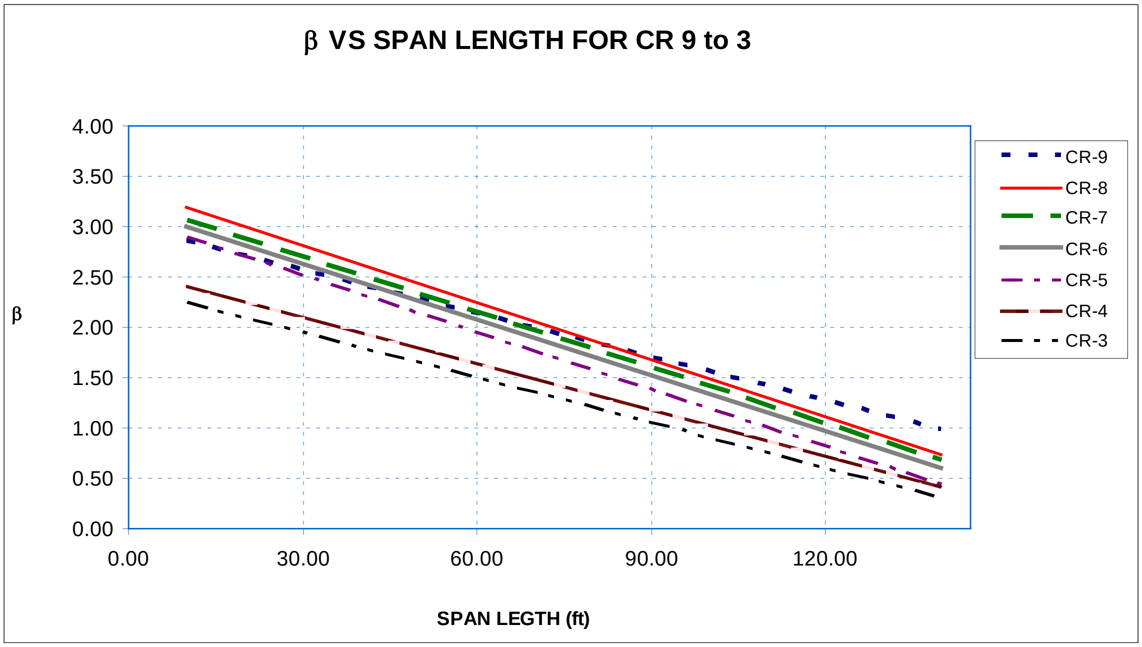

Figure 3.27: Comparison of β Curves for Different Bridge Condition Ratings for Different Bridge Length Using 2009, HS20 Rated, Prestressed, Northeast Region Bridges.

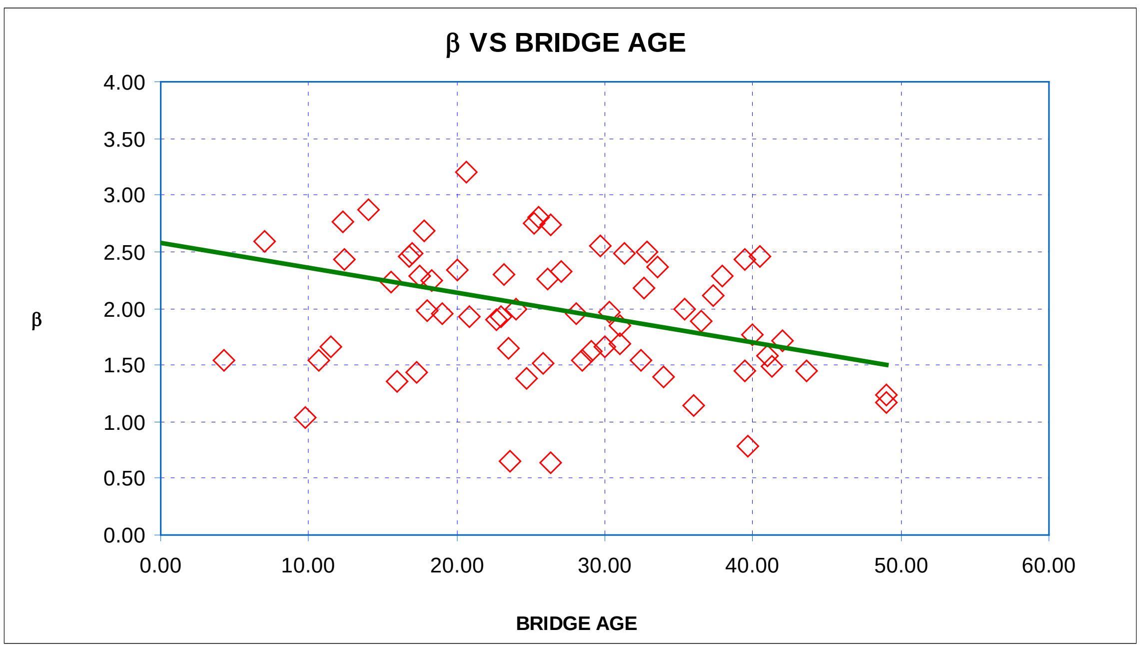

Figure 3.28: β vs Bridge Age Graph Using 2009, HS20 Rated, Prestressed, Northeast Region Bridges.

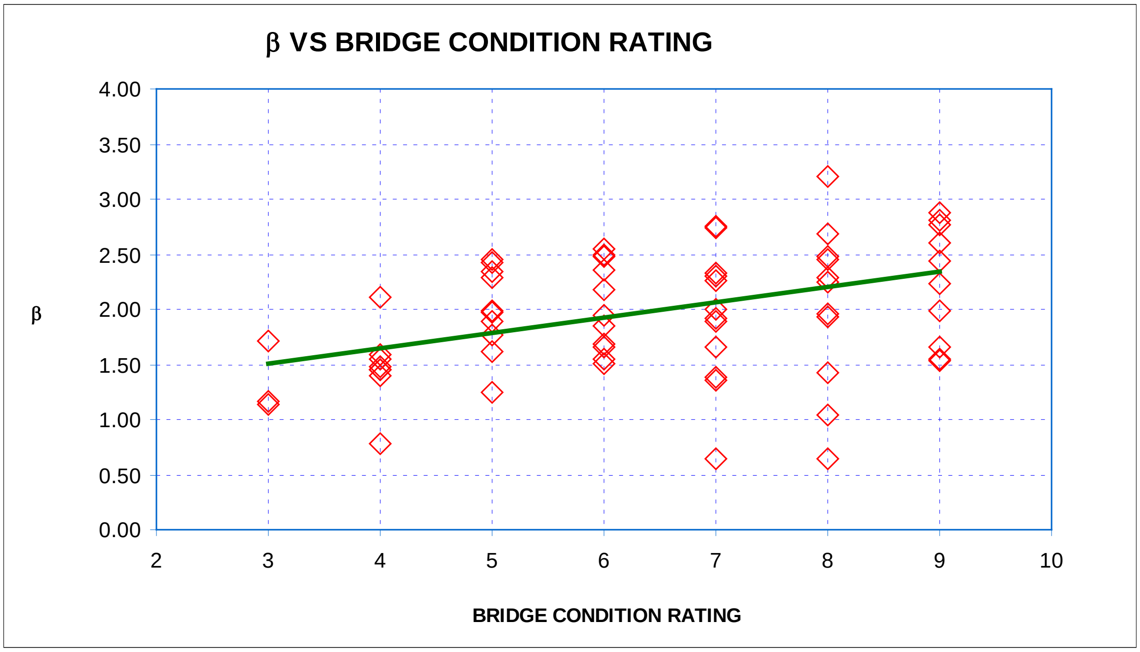

Figure 3.29: β vs Bridge Condition Rating Graph Using 2009, HS20 Rated, Prestressed, Northeast Region Bridges.

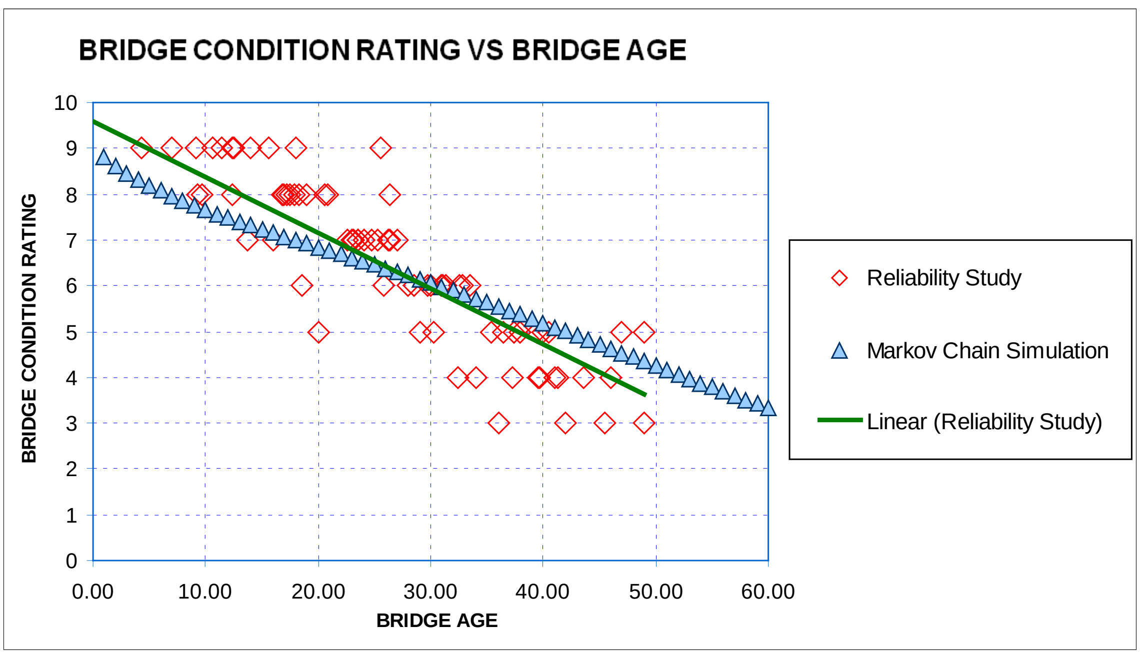

Figure 3.30: Bridge Condition Rating vs Bridge Age Graph Using 2009, HS20 Rated, Prestressed, Northeast Region Bridges for both Reliability Study and Markov Chain Simulation.

change the bridge condition rating, gets evaluated for the bridge operating rating calculation as well".

Finally, bridge condition rating vs. bridge age graph is shown in Figure 3.30. Deterioration curve obtained from bridge condition ratings using Markov chain simulation is also shown in the same graph.

In sum, a reliability methodology that can be used for the deterioration model in the BLCCA is presented in this study. Bridge reliability indices are calculated which can also be used to obtain bridge deterioration curves. This study can be further improved by developing the reliability indices for steel superstructure bridges and formulating the reliability based deterioration curves and integrating them into the BLCCO. Developing the reliability indices for all the bridges can be done by preparing charts for the average weight and the capacity of bridges according to their lengths or it can be really easy if this information being included in the NBI database starting with the next cycle of inspections of the bridges. Since a bridge database is readily available, it would be better to use it or improve it rather than trying to focus on individual bridges. This can be a future research topic.

Chapter Four

BRIDGE LIFE CYCLE COST OPTIMIZATION MODEL (BLCCOM)

4.1. Introduction and Problem Definition

Life-cycle cost analysis (LCCA) has received increasing attention as a tool to assist transportation agencies in making investment decisions as well as in managing assets (PIARC 1991, FHWA 1994).

Transportation agencies using federal funds must often conduct LCCA to justify their planning and design decisions, because the federal agencies providing funds require that. Sections 1024 and 1025 of the Intermodal Surface Transportation Efficiency Act of 1991 (ISTEA) specified that consideration should be given to life cycle costs in the design and engineering of bridges, tunnels, and pavements. The National Highway System Designation Act of 1995 requires that states conduct an LCCA for each proposed National Highway System (NHS) project segment costing $25 million or more. Federal Executive Order 12893, signed by President Clinton in January 1994, requires that all federal agencies use “systematic analysis of expected benefits and costs... appropriately discounted over the full life cycle of each project” in making major infrastructure investment decisions; the Federal Highway Administration(FHWA) and other executive-branch agencies have issued more detailed guidance for implementing this Executive Order and been more explicit in adopting the terms of LCCA in describing their requirements, (Part II of NCHRP Report 483, 2003). Despite such requirements, LCCA is not universally used in US transportation agencies. There is currently no commonly accepted methodology for LCCA, particularly as it might be applied to bridge management.

That is why in this research, the main emphasis was on developing a BLCCOM for general use nationwide, using genetic algorithm to reach the optima.

Ozbay and Javad (2003, 2004, 2005, 2006, and 2007) focused on an LCCA methodology with application on highway pavement structures. These academic studies formed the foundation of the BLCCOM developed in this dissertation.

4.2. BLCCA Model Components and Formulations

Model components for BLCCA are shown below and illustrated in a flow chart (Figure 4.1):

- Characterize bridge and its elements
- Define planning horizon, analysis scenario(s) and base case
- Select appropriate deterioration models and parameters
- Define alternative bridge management strategies
- Estimate cost (Agency and user costs)
- Calculate net present values
- Review results
- Modify management strategies
- Select preferred strategy

Bridge deterioration equations were calculated in chapter 3, where the user and agency cost equations due to the rehabilitation activities or condition of the bridge were calculated.

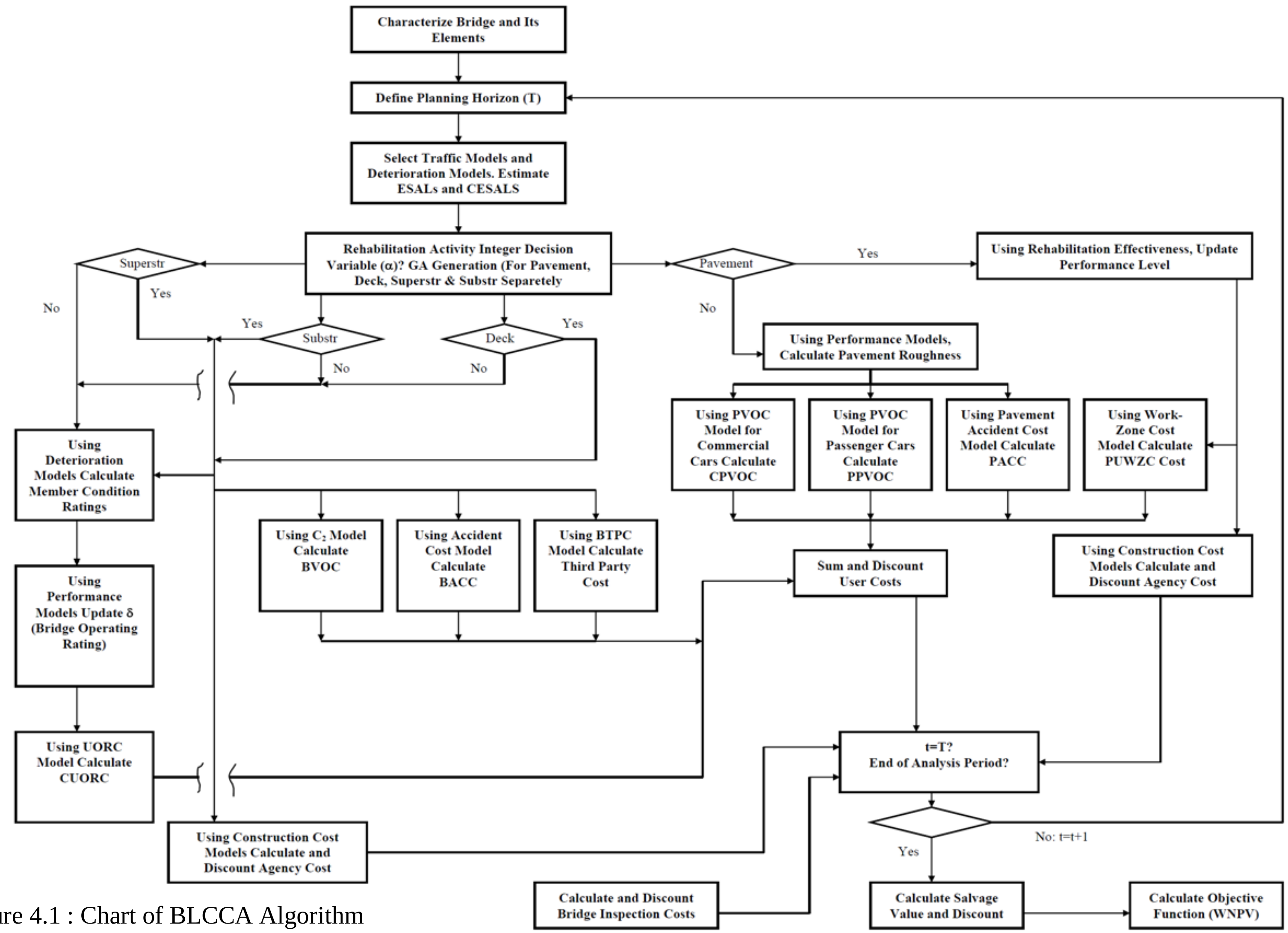

Figure 4.1 : Chart of BLCCA Algorithm
(Modified from Ozbay and Javad (2003) LCCA Algorithm)

Vehicle Operation Cost (PVOC) Model due to Pavement Condition:

The PVOC model equations are adapted from Prancl (1999).

$$PVOC = C1 * IRI + C2 \quad \text{(Eq. 4.1)}$$

$$TAPVOC_t = PVOC_p * ADT_p * 365 * (1+g)^t + PVOC_c * ADT_c * (1+g)^t \quad \text{(Eq. 4.2)}$$

Vehicle	C1	C2
Passenger Car	0.00021	0.576
Commercial Car	0.00088	0.9962

where

TAPVOC : Total Annual Pavement Vehicle Operation Cost ($)

PVOC : Pavement Vehicle Operating Cost ($), p: passenger car, c: commercial vehicle

g : Traffic Volume Annual Growth Rate (decimal)

ADT : Average Daily Traffic (vehicles), p: passenger car, c: commercial vehicle

t : Year of Analysis

Accident Cost Model (PACC) due to Pavement Condition:

The PACC model equations are also adapted from Prancl (1999).

$$PACC = e^{a(IRI+b)} \quad \text{(Eq. 4.3)}$$

Facility Type	a	b
Two Lane	0.0014	-4.222
Undivided Multi-Lane	0.0041	-3.848
Divided Multi Lane	0.0015	-4.916
Average Cost	0.0030	-4.254

where

PACC: Accident costs per mile ($)

Delay Time Cost During Pavement Work Zone Operations (PUWZC):

The cost of delay time during pavement work zone operations used in BLCCOM is based on the approximating it by taking it as some percentage of the rehabilitation agency cost.

The PUWZC is adapted from Jawad (2003).

$$PUWZC = \beta_a AC_a \quad \text{(Eq. 4.4)}$$

where

PUWZC : User cost resulting from delays during pavement work zone activities ($/mile)

β_a : Parameter representing the percentage of user costs during pavement work zone to the agency cost of rehabilitation activity α

AC_a : Agency costs of rehabilitation activity α ($)

User Costs Due To Operating Rating of the Bridge (UORC):

$$UORC = [(L_d/S_d)-(L_n/S_n)]*ADT_T*R*365 \quad \text{(Eq. 4.5)}$$

where

UORC : User Cost due to Operating Rating of the Bridge

L_d : Length of detour

S_d : Traffic speed for the detour

L_n : Length of the bridge roadway between detour locations

S_n : Normal traffic speed on the bridge

ADT_T : Number of the trucks (daily) affected by operating rating restrictions

R :Vehicle operating cost ($/hour)

Vehicle Operation Cost (BVOC) Model due to Bridge Rehabilitation Activity:

The BVOC is adapted from Roychoudhury (2001).

$$BVOC = \{(L/S_a) - (L/S_n)\} * \{(ADT_c * C) + (ADT_p * P)\} * N \qquad \text{(Eq. 4.6)}$$

where

BVOC : Total vehicle operating cost for the entire period of the roadwork ($)

L : Length of affected roadway on which the vehicles travel

S_a : Traffic speed during bridge work activity

S_n : Normal traffic speed.

ADT_c : ADT of commercial vehicle

C : Commercial vehicle operating cost and driver travel cost ($/hour)

ADT_p : ADT of passenger vehicles

P : Passenger vehicle operating cost and driver travel cost ($/hour)

N : Number of days of work.

Accident Cost (BACC) Model due to Bridge Rehabilitation Activity:

The BACC is adapted from Roychoudhury (2001).

$$BACC = L * ADT * N * (A_c - A_n) * C_a \qquad \text{(Eq. 4.7)}$$

where

BACC : Average cost of accident ($)

A_c : Accident rate during construction

A_n : Normal accident rate

C_a : Average cost per accident ($/accident)

L : Length of the affected roadway on which vehicles travel

ADT : Average daily traffic

N : Number of days of roadwork

Third Party Cost (BTPC) Model due to Bridge Rehabilitation Activity:

The BTPC is adapted from Roychoudhury (2001).

Third party cost is basically taken as 40% of their regular business profit.

$$BTPC = 0.4 * (N_G * G + N_F * F) * N \quad \text{(Eq. 4.8)}$$

where

BTPC : Cost due to the Lost Business ($)

N_G : Number of gas stations

N_F : Number of food or beverage stores

G : Average daily sales for gas stations

F : Average daily sales for food or beverage stores

N : Number of days of roadwork

4.3. Genetic Algorithm

Genetic algorithms are parts of evolutionary computing, which is a rapidly growing area of artificial intelligence. Genetic algorithms are inspired by Darwin's theory of evolution. Simply put problems are solved by an evolutionary process resulting in a best (fittest) solution (survivor) - in other words, the solution is evolved.

Evolutionary computing was introduced in the 1960s by I. Rechenberg in his work "*Evolution strategies*" (*Evolutionsstrategie* in original). His idea was then further developed by other researchers. Genetic Algorithms (GAs) were invented by John Holland and then further developed by him and his students and colleagues. This led to Holland's book "*Adaption in Natural and Artificial Systems*" published in 1975. In 1992, John Koza has used genetic algorithms to evolve programs to perform certain tasks. He called his method "genetic programming" (GP). LISP programs were used, because programs in this language could beexpressed in the form of a "parse tree", which is the object the GA works on.

Algorithm begins with a set of solutions (represented by chromosomes) called population. Solutions from one population are taken and used to form a new population. This is motivated by a hope, that the new population will be better than the old one. Solutions which are then selected to form new solutions (offspring) are selected according to their fitness - the more suitable they are the more chances they have to reproduce. This is repeated until some condition (for example number of populations or improvement of the best solution) is satisfied.

A chromosome should in some way contain information about solution that it represents (Figure 4.2). The most used way of encoding is a binary string. A chromosome then could look like this:

Chromosome 1	0101100100110110
Chromosome 2	1101111000011110

Figure 4.2: GA Coding

Each chromosome is represented by a binary string. Each bit in the string can represent some characteristics of the solution. Another possibility is that the whole string can represent a number - this has been used in the basic GA applet.

Of course, there are many other ways of encoding. The encoding depends mainly on the problem to be solved. For example, one can encode directly integer or real numbers; sometimes it is useful to encode some permutations and so on.

Once an encoding method is selected, , the crossover operation can be done. The crossover operates on selected genes from parent chromosomes and creates new offspring (Figure 4.3). The simplest way to do that is to randomly select a crossover point and copy everything before that point from the first parent and then copy everything after the crossover point from the other parent.

The crossover operation can be illustrated as follows: (| represents the crossover point):

Chromosome 1	01011 | 00100110110
Chromosome 2	11011 | 11000011110
Offspring 1	01011 | 11000011110
Offspring 2	11011 | 00100110110

Figure 4.3 : Crossover Operation in GA

There are other ways to make crossovers. For instance, more crossover points can be selected. The crossover operation can be quite complicated and depends mainly on the encoding of the chromosomes. A specific crossover made for a specific problem can improve the performance of the genetic algorithm.

After a crossover operation is performed, mutation takes place (Figure 4.4). Mutation is intended to prevent falling of all solutions in the population into a local optimum of the solved problem. Mutation operation randomly changes the offspring that resulted from crossover. In case of binary encoding, it can be switched a few randomly chosen bits from 1 to 0 or from 0 to 1. Mutation can then be illustrated as follows:

Original offspring 1	0101111000011110
Original offspring 2	1101100100110110
Mutated offspring 1	0100111000011110
Mutated offspring 2	1101101100110110

Figure 4.4 : Mutation Operation in GA

Genetic Algorithms are different from other traditional optimization techniques in three aspects:

1. The selection of the fittest solution(s) at every iteration in GA is not gradient based and does not require information about differentiability, convexity, or other auxiliary properties. The fitness of the string is measured directly by the objective function.

2. GA operates by manipulating a pool of solutions instead of one each time. This enables the search process to explore properties simultaneously in different directions.

3. Genetic algorithms improve the search process in an adaptive manner using probabilistic transition rules to generate new solutions from the existing solutions. This introduces perturbations to move out of the local optima (Fwa et al, 2000; Liu et al, 1997)

To summarize, GA is selected as an optimization technique in this research for the following reasons:

1. There are no mathematical limitations for the decision variables, objective functions and formulations of the constraints.

2. GA is already proven to be a robust and effective optimization tool, which can reach the optima in a relatively short time compared to other search algorithms.

3. GA is simple to use. It does not show “Black Box Syndrome”, which means a very complicated methodology that analysts cannot understand clearly the solution.

The application of GA can overcome this syndrome, because it can be performed using available user friendly add-ins (ie, Evolver, Riskoptimizer and Solver) in spreadsheet programs such as Microsoft Excel.

Figure 4.5 demonstrates GA as an optimization technique in BLCCOM.

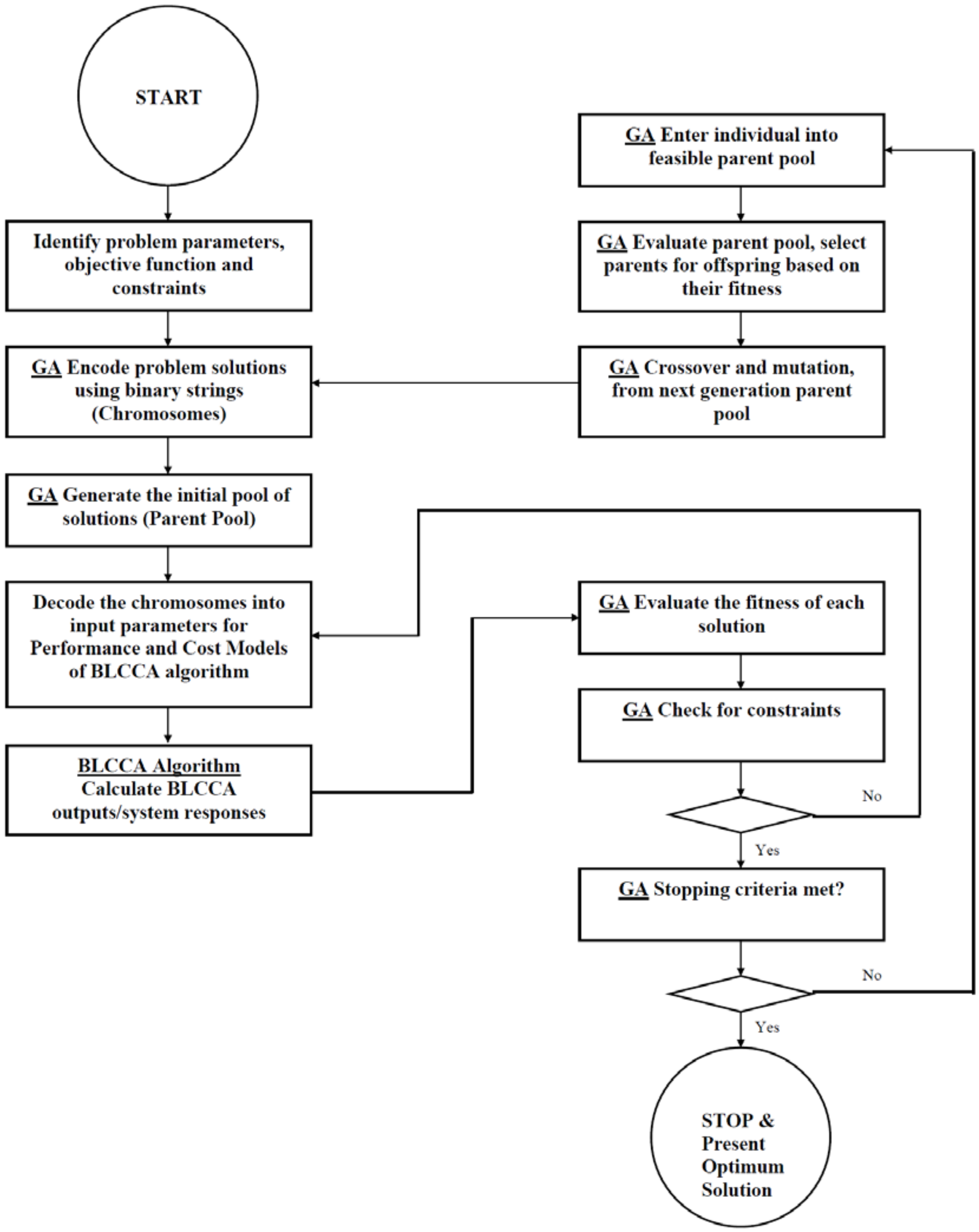

Figure 4.5 : Chart of BLCCOM
(Modified from Ozbay and Javad (2003) LCCO-p Chart)

4.4. Optimization Formulations

4.4.1. Decision Variables

The decision variables (string structure) are:

α_{at} = an integer $\in$ {0,1,2,3,..m} where m represents the type of activity

t = the time periods (year) in the analysis period

a = member type (overlay, deck, superstructure, substructure) that decision variable belongs to.

4.4.2. Constraints

$$\sum_{t} Count(\alpha_{at}) \leq n \qquad \text{Eq. 4.9}$$

where

$$\alpha_{at} \geq 1 \qquad \text{Eq. 4.10}$$

n is the maximum possible number of rehabilitation activities, and Count is a function that counts the number of α_t that is larger than or equal to zero.

$$WNPV = \sum_{t=0}^{T} \frac{[\sum_{k \in K} \sum_{j \in J} \omega_k * Cost(k, j, t)]}{(1+r)^t} \qquad \text{Eq. 4.11}$$

where

WNPV : Weighted Net Present Value

ω_k : weight of cost k mentioned above in the decision making process

K : Set of costs classified on the basis of the bearing entity ($) (ie, Agency Cost, User Costs, Social Costs)

J : Set of costs incurred by each entity (element) in K and classified by their nature ($) (ie. Agency costs can consist of costs of material, labor, overhead, engineering, salvage)

t : year at which the cost is incurred

T : Analysis period

r : Discount rate

This objective assigns different weights for different types of costs. By this way, it prevents the user costs from overwhelming the analysis. The aim here is to minimize the net present economic worth of the project over its lifetime.

$$OR \geq n \qquad \text{Eq. 4.12}$$

Where: OR is the operating rating of the bridge and n is a number less than one. The lowest value of the OR for the bridges needs to be determined based on traffic volumes and the location of the bridge. If that rating value is exceeded, the bridge needs a major repair or reconstruction.

$$CR \geq m \qquad \text{Eq. 4.13}$$

Where: CR is the condition rating of the bridge member (deck, superstructure or substructure) and m is a real number less than 4. If that condition rating value is exceeded, then the bridge member needs a major repair. Also, any repair activity on deck, superstructure or substructure is carried out such that it raises the condition state of the bridge member to desired levels, as the effectiveness of the repair work is obviously very important.

4.4.3. Objective Function

$$WNPV_I(T) = \sum_{t}^{T} \frac{\omega_1\{AC_t(\alpha_t) - SV_t\} + \omega_2\{TAVOC_t(IRI_t) + ACC_t(IRI_t) + UORC(\delta_t) + UC_t(\alpha_t)\}}{(1+r)^t} \qquad \text{Eq. 4.14}$$

where:

α : an integer representing the type of rehabilitation activity to be scheduled in year t and equal to zero if no rehabilitation activity is planned for pavement, deck, superstructure and substructure.

ω : the weight of the cost in decision making process.

δ : operating rating factor of the bridge.

r : discount rate (decimals)

$WNPV_i(T)$: the weighted net present economic worth of the life cycle strategy in project, due to the agency and user costs over a planning period of T years.

IRI_t : a function modeling the deterioration of pavement roughness

AC_t : the agency cost in year t depending on construction, rehabilitation or reconstruction activity.

SV_t : the salvage value at the end of the service life (T).

$TAVOC_t$ & ACC_t : the total annual vehicle operating costs and the total annual accident costs in year t, respectively which are functions of the pavement roughness in that year.

$UORC_t$: user cost (detour) due to operating rating of the bridge in that year.

UC_t : the user work zone costs during the rehabilitation activities for pavement, deck, superstructure and substructure.

4.4.4. Steps in Implementing BLCCOM

Step A: Initial Input Entry – Determine input parameters such as discount rate, traffic characteristic and deterioration characteristic. Prepare their probability distributions. Choose genetic algorithm operators.

Step B: GA Initialization- generate a pool of initial solutions (parent pool): for each solution assign a value for 4 different "α" values (pavement, deck, superstructure, substructure) for all t values randomly (each solution represents a life cycle strategy and it is always better to prepare a logical parent pool for a quick and accurate optimization process).

Step C: BLCCA Algorithm

Step C-1-2: For each solution in the parent pool

Step C-1-1: Bridge Condition – For all t, using traffic equations and nonlinear optimization with Markov Chain, calculate the condition of the bridge members. (Eqs 3.1 to 3.12).

Step C-1-1-1: Updating facility condition – If α >0 (for pavement) then, update IRI_t otherwise calculate roughness level using performance models (Eqs. 3.9 to 3.12).

Step C-1-1-2: Updating facility condition – If α >0 (for deck) then, update condition rating using proper repair vector (Eq. 3.7); otherwise calculate condition rating by using transition matrix (Eq. 3.1).

Step C-1-1-3: Updating facility condition – If α >0 (for substructure) then, update condition rating using a proper repair vector (Eq. 3.7); otherwise calculate condition rating by using transition matrix (Eq. 3.1).

Step C-1-1-4: Updating facility condition – If α >0 (for superstructure) then, update condition rating using a proper repair vector (Eq. 3.7); otherwise calculate the condition rating by using transition matrix (Eq. 3.1).

Also no matter what α value is, update the operating rating of the bridge (Eq. 3.8).

Step C-1-1-5: Cost Estimation – If $\alpha>0$ for pavement, deck, superstructure or substructure, calculate the agency costs for the repair or replacement activity and user work zone costs (Eqs. 4.1, 4.2, 4.4, 4.5 & 4.6).

Step C-1-1-6: Cost estimation – Calculate total annual accident cost and also calculate the annual inspection and third party costs for the bridge (Eqs. 4.3, 4.7 & 4.8).

Step C-1-1-7: Discounting – Calculate the present value of the total cost (user costs and agency costs) (Eq. 4.11).

Step C-1-2: Objective function calculation – Record system response for the iteration and calculate the objective function (Eq. 4.14).

Step C-2: Legitimacy of Solution – Check for constraints (Eqs. 4.9 to 4.13). If constraints are violated, disregard the solution. Otherwise, keep the solution in the parent pool.

Step D: Stopping Rule – Repeat steps F, G, H then C until the preset stopping criteria is met.

Step E: Optimum Solution – Stop and present the optimum life cycle strategy that yields the minimum value for the objective function.

Step F: GA Selection of parent offspring – Evaluate the valid solutions based on their fitness, and select the best two solutions as the parents for the offspring.

Step G: GA Next pool generation – Generate the next parent pool of solutions.

Step H: GA Operations Performance – GA operations of cross over and mutation on the parent pool of the next generation and go back to Step C.

4.5. Implementation of Genetic Algorithm in BLCCOM

To validate the effectiveness of the suggested algorithm, results from a case study are presented. The bridge No. 18G0702 from NJ over Raritan River is used for this study. Below are the map and information about the bridge No. 18G0702.

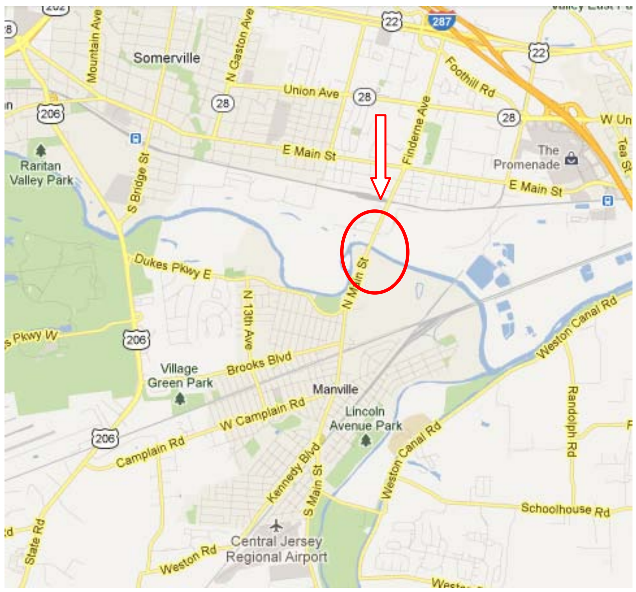

Figure 4.6 : Location of Bridge No. 18G0702 in NJ

4.5.1. Bridge No. 18G0702 Properties from the NBI Database

Name: S MAIN ST (CR533) over RARITAN RIVER

Structure number: 18G0702

Location: .3 MI NORTH OF DUKES PKWY

Purpose: Carries 4-lane highway and pedestrian walkway over waterway

Route classification: Minor Arterial (Urban) [16]

Length of largest span: 81.0 ft. [24.7 m]

Total length: 559.4 ft. [170.5 m]

Roadway width between curbs: 56.1 ft. [17.1 m]

Deck width edge-to-edge: 64.6 ft. [19.7 m]

Owner: County Highway Agency [02]

Year built: 2007

Historic significance: Bridge is not eligible for the National Register of Historic Places

Design load: MS 22.5 / HS 25 [9]

Main span material: Steel continuous [4]

Main span design: Stringer/Multi-beam or girder [02]

Deck type: Concrete Cast-in-Place [1]

Number of main spans: 9

Wearing surface: Monolithic Concrete (concurrently placed with structural deck) [1]

December 2009 Inspection

Status: Open, no restriction [A]

Average daily traffic: 36,518 [as of 2009]

Truck traffic: 4% of total traffic

Deck condition: Very Good [8 out of 9]

Superstructure condition: Very Good [8 out of 9]

Substructure condition: Very Good [8 out of 9]

Structural appraisal: Equal to present desirable criteria [8]

Deck geometry appraisal: Somewhat better than minimum adequacy to tolerate being left in place as is [5]

Water adequacy appraisal: Equal to present minimum criteria [6]

Roadway alignment appraisal: Equal to present desirable criteria [8]

Channel protection: Banks are protected or well vegetated. River control devices such as spur dikes and embankment protection are not required or are in a stable condition. [8]

Scour condition: Bridge foundations determined to be stable for the assessed or calculated scour condition. [8]

Inventory rating: 73.8 tons [67.1 metric tons]

Sufficiency rating: 84.1

4.5.2. Optimization Model Input Parameters

The optimization input parameters are the bridge, pavement, vehicle, deterioration, genetic algorithm, agency and user cost parameters. Most of the bridge input parameters are obtained from the NBI database, pavement parameters are from the previous studies, agency and user cost parameters are from the NJDOT bridge repair and construction practice and previous studies. Genetic algorithm parameters are taken as the default values. Vehicle and deterioration parameters were determined in Chapter 3 according to the location and properties of the bridge, namely the Northeast Climatic Region, steel superstructure, longest span longer than 65.6 feet (20m) and ADT more than 10000 vehicles/day. All these parameters are shown in Table 4.1.

BRIDGE #18G0702 CHARACTERISTICS		
Bridge Length =	559	feet
Bridge Width =	65	feet
ADT (total) =	35802	veh/day
% ADT (Commercial) =	4	
%Traffic Growth Rate =	2.5	
Length Betw Det Loc=	0.5	mile
Speed =	55	mile / hour
Work Zone Speed =	25	mile / hour
Detour Length =	8	mile
Detour Speed =	35	mile / hour

VEHICLE SPEED RESTRICTIONS		
Condition Rating 5 =	50	mile / hour
Condition Rating 4 =	45	mile / hour
Condition Rating 3 =	35	mile / hour

OPERATING RATING PARAMETERS	
Intercept "C" =	1.903
X Variable "β" =	7.043

PAVEMENT CHARACTERISTICS		
$IRI_{t=0}$ =	45	in/mile
AC_{visc} =	1600	Poise
AC_{void} =	6	% volume
AC_{thick} =	6	in
B_{thick} =	14	in
DGT =	9	Days
FI =	925	F^o/day
D =	0.5	
L =	0.9	

GENETIC ALGORITHM PARAMETERS	
Population Size =	50
Cross Over Rate =	50%
Mutation Rate =	5%

ECONOMIC PARAMETERS		
Period =	75	years
r (discount rate) =	0.04	

DECK COST INPUT		
Minor Repair Cost =	200	$\$/ft^2$
Major Repair Cost =	600	$\$/ft^2$
Replacement Cost =	1000	$\$/ft^2$

Minor Repair Time =	40	ft^2/hour
Major Repair Time =	10	ft^2/hour
Replacement Time =	5	ft^2/hour

SUPERSTRUCTURE COST INPUT		
Minor Repair Cost =	1000	$/ft
Major Repair Cost =	2000	$/ft
Replacement Cost =	5000	$/ft
# of Girders =	12	

Minor Repair Time =	16	ft/hour
Major Repair Time =	4	ft/hour
Replacement Time =	1	ft/hour

SUBSTRUCTURE COST INPUT		
Minor Repair Cost =	20	$\$/ft^3$
Major Repair Cost =	150	$\$/ft^3$
Volume of Piers + Abutments=	8000	ft^3

PAVEMENT COST INPUT		
Replacement Cost =	65.71	$\$/yd^2$

USER COST PARAMETERS		
% User Cost Weight =	15	
Passenger V. O. Cost =	5	$/hour
Commercial V. O. Cost =	30	$/hour

BRIDGE MC DETERIORATION VECTORS										
P_{deck} =	0.803	0.850	0.946	0.944	0.948	0.831	0.737	0.638	0.561	1.000
$P_{superstr}$ =	0.839	0.892	0.927	0.905	0.901	0.794	0.682	0.555	0.488	1.000
P_{substr} =	0.793	0.829	0.934	0.937	0.945	0.884	0.746	0.625	0.328	1.000

Table 4.1 : Input Parameters for the BLCCO Model

4.5.3. Developing the Life Cycle Cost Management Plan for the Model

An initial life time management plan is prepared for the Bridge No. 18G0702 to show how the BLCCO model works. Logical individual maintenance plans are prepared for deck, superstructure, substructure and pavement using several assumptions. The evaluation is performed every other year.

For the deck:

- It is assumed that the deck condition will be maintained at a minimum condition rating of 6 and up as a serviceability constraint
- It is assumed that there can be up to 6 repairs throughout the life span of the bridge.
- The activities are assigned the following codes:

  1 : Do nothing, 2 : Minor Repair, 3 : Major Repair, 4 : Replace

For the superstructure:

- It is assumed that the superstructure condition will be maintained at a minimum condition rating of 4 and up as a serviceability constraint
- It is assumed that there can be up to 4 repairs throughout the life span of the bridge.
- The activities are assigned the following codes:

  1 : Do nothing, 2 : Minor Repair, 3 : Major Repair, 4 : Replace

For the substructure:

- It is assumed that the substructure condition will be maintained at a minimum condition rating of 5 and up as a serviceability constraint

Year	Deck Condition	Deck	Super Condition	Super	Sub Condition	Sub	Pave. Cond.	Pavement
1	9	1	9	1	9	1	45	1
3	8	1	9	1	8	1	60	1
5	8	1	8	1	8	1	69	1
7	8	1	8	1	8	1	77	1
9	8	1	8	1	7	1	83	1
11	7	1	8	1	7	1	89	1
13	7	1	7	1	7	1	95	1
15	7	1	7	1	7	1	100	1
17	7	1	7	1	7	1	105	1
19	7	1	7	1	7	1	110	1
21	7	1	7	1	6	1	115	1
23	6	1	6	1	6	1	120	1
25	6	3	6	1	6	1	125	1
27	7	1	6	1	6	1	130	2
29	7	1	6	1	6	1	63	1
31	7	1	6	1	6	1	77	1
33	7	1	5	1	5	1	86	1
35	6	1	5	1	5	1	95	1
37	6	1	5	1	5	1	102	1
39	6	4	5	4	5	2	109	1
41	9	1	9	1	6	1	115	1
43	8	1	8	1	5	1	121	1
45	8	1	8	1	5	1	127	1
47	8	1	8	1	5	1	133	2
49	7	1	8	1	5	1	72	1
51	7	1	7	1	5	1	87	1
53	7	1	7	1	5	2	98	1
55	7	1	7	1	5	1	107	1
57	7	1	7	1	5	1	115	1
59	7	1	7	1	5	1	123	1
61	6	1	6	1	5	1	130	2
63	6	1	6	1	5	2	78	1
65	6	1	6	1	5	1	94	1
67	6	1	6	1	5	1	107	1
69	6	1	6	1	5	1	117	1
71	6	2	5	1	5	2	126	2
73	6	1	5	1	5	1	83	1
75	6	1	5	1	5	1	100	1

Table 4.2: Initial Lifetime Repair and Reconstruction Plan for Bridge No. 18G0702

Year	Deck Condition	Deck	Super Condition	Super	Sub Condition	Sub	Pave. Cond.	Pavement
1	9	1	9	1	9	1	45	1
3	8	1	9	1	8	1	60	1
5	8	1	8	1	8	1	69	1
7	8	1	8	1	8	1	77	1
9	8	1	8	1	7	1	83	1
11	7	1	8	1	7	1	89	1
13	7	1	7	1	7	1	95	1
15	7	1	7	1	7	1	100	1
17	7	1	7	1	7	1	105	1
19	7	1	7	1	7	1	110	2
21	7	1	7	1	6	1	60	1
23	6	1	6	1	6	1	73	1
25	6	1	6	1	6	1	82	1
27	6	3	6	1	6	1	90	1
29	7	1	6	1	6	1	97	1
31	7	1	6	1	6	1	103	1
33	7	1	5	1	5	1	109	2
35	7	1	5	1	5	1	66	1
37	6	1	5	1	5	1	79	1
39	6	1	5	1	5	1	90	1
41	6	1	4	1	5	1	98	2
43	6	4	4	3	5	3	69	1
45	9	1	7	1	7	1	83	1
47	8	1	7	1	7	1	94	1
49	8	1	7	1	7	1	103	2
51	8	1	6	1	6	1	72	1
53	7	1	6	1	6	1	88	1
55	7	1	6	1	6	1	99	1
57	7	1	6	1	6	1	108	2
59	7	1	6	1	6	1	76	1
61	7	1	5	1	6	1	92	1
63	7	1	5	1	6	1	104	2
65	6	1	5	2	6	1	79	1
67	6	1	5	1	5	1	96	2
69	6	1	5	1	5	1	81	1
71	6	1	5	1	5	1	98	2
73	6	1	5	1	5	1	83	1
75	6	1	4	1	5	1	100	1

Table 4.3: Optimized Lifetime Repair and Reconstruction Plan for Bridge No. 18G0702 (BLCCO)

- It is assumed that there can be up to 6 repairs throughout the life span of the bridge.
- The activities are assigned the following codes:

  1 : Do nothing, 2 : Minor Repair, 3 : Major Repair

For the pavement:

- It is assumed that the road condition index will be maintained at a minimum value of 135 and down as a serviceability constraint
- It is assumed that there can be up to 8 repairs throughout the life span of the bridge.
- The activities are assigned the following codes:

  1 : Do nothing, 2 : Replace

4.5.4. Comparison and Results

In Table 4.2, the manually prepared initial management plan for the bridge is shown. BLCCO is performed using the Risk Optimizer software, which is used as add-in software in Microsoft Excel, and can perform genetic algorithm optimization both deterministically and stochastically using Monte Carlo simulation. The resulting optimized bridge life span management plan is given in Table 4.3. As can be seen, the optimization changed the type and time of the scheduled activities wherever necessary according to equation 4.14. The effect of the repair-replacement activities on the bridge components deterioration curves are shown in Figure 4.7 through 4.14 for both the initial and optimized bridge management strategies.

As explained in section 4.4.3, the objective function is minimized by the optimization process. The objective function is “the weighted net present economic worth of the life cycle strategy in the project due to the agency and the user costs over a planning period of 75 years". The graphs comparing the cumulative user and agency costs of the initial and the optimized scenarios are shown in Figures 4.15 through 4.20. The summary of the percentage of cost change between the initial and the optimized scenario is shown in Table 4.4. In order to give a better idea of the change in the user and agency costs, they are also shown individually as well as the total cost. The values at the

	AGENCY COST		
	Cost	Net Present Cost	Net Present, Weighted Cost
Initial Agency Cost	94,191,748	22,635,595	22,635,595
Optimized Agency Cost	76,834,603	17,127,650	17,127,650
% change	18%	24%	24%

	USER COST		
	Cost	Net Present Cost	Net Present, Weighted Cost
Initial User Cost	675,090,695	117,170,277	17,575,542
Optimized User Cost	640,744,988	124,110,226	18,616,534
% change	5%	-6%	-6%

	TOTAL COST		
	Cost	Net Present Cost	Net Present, Weighted Cost
Initial Total Cost	769,282,443	139,805,872	40,211,136
Optimized Total Cost	717,579,591	141,237,876	35,744,184
% change	7%	-1%	11%

Table 4.4: Percentage of Cost Change Summary of Initial and Optimized Bridge Management Scenarios for Bridge No. 18G0702 (BLCCO)

end of 75 years for cost, the net present cost (discounted cost) and the net present weighted cost which is the objective function are presented. For the agency cost, according to the initial solution, the net present, weighted cost for the 75-year life span of the bridge is $22,635,595 and the optimized solution (optimization is done for the total cost, not only for the agency cost) is $17,127,650, which is %24 lower than the initial value. For the user cost, according to the initial solution, the net present, weighted cost for the 75-year life span of the bridge is $17,575,542 and the optimized solution (optimization is done for the total cost too, not only for the user cost) is $18,616,534, which is %6 higher than the initial value. Finally, for the total cost, according to the initial solution, the net present, weighted cost for the 75-year life span of the bridge is $40,211,136 and the optimized solution is $35,744,184, which is %11 lower than the initial value. The optimization changed the planning in a way that while the agency net present, weighted cost decreased, the user net present, weighted cost increased. However, the total net present, weighted cost decreased and the BLCCO process seemed to work well.

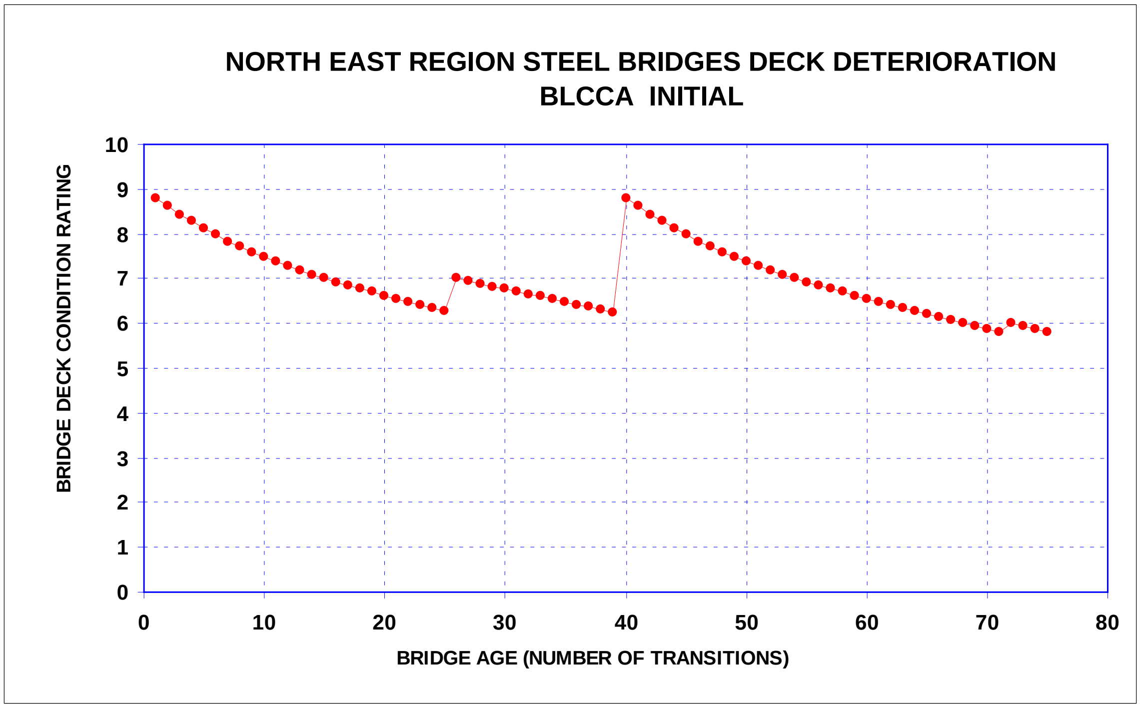

Figure 4.7: Deterioration Curve of Initial Management Strategy for Bridge Deck

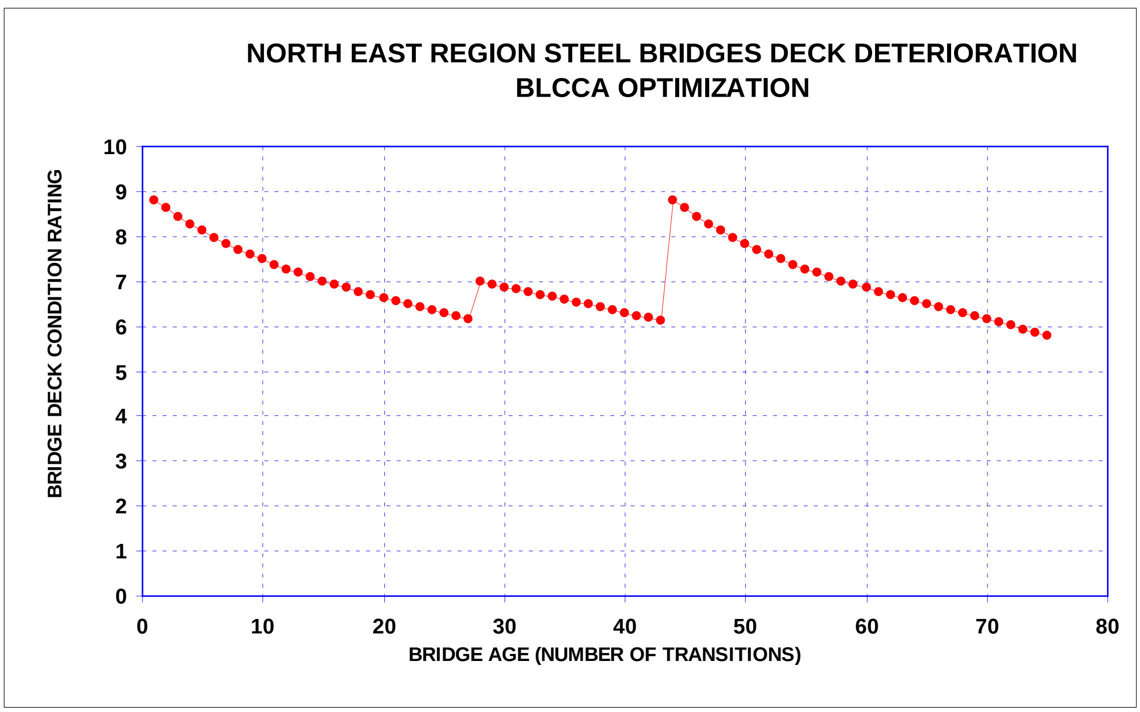

Figure 4.8: Deterioration Curve of Optimized Management Strategy for Bridge Deck

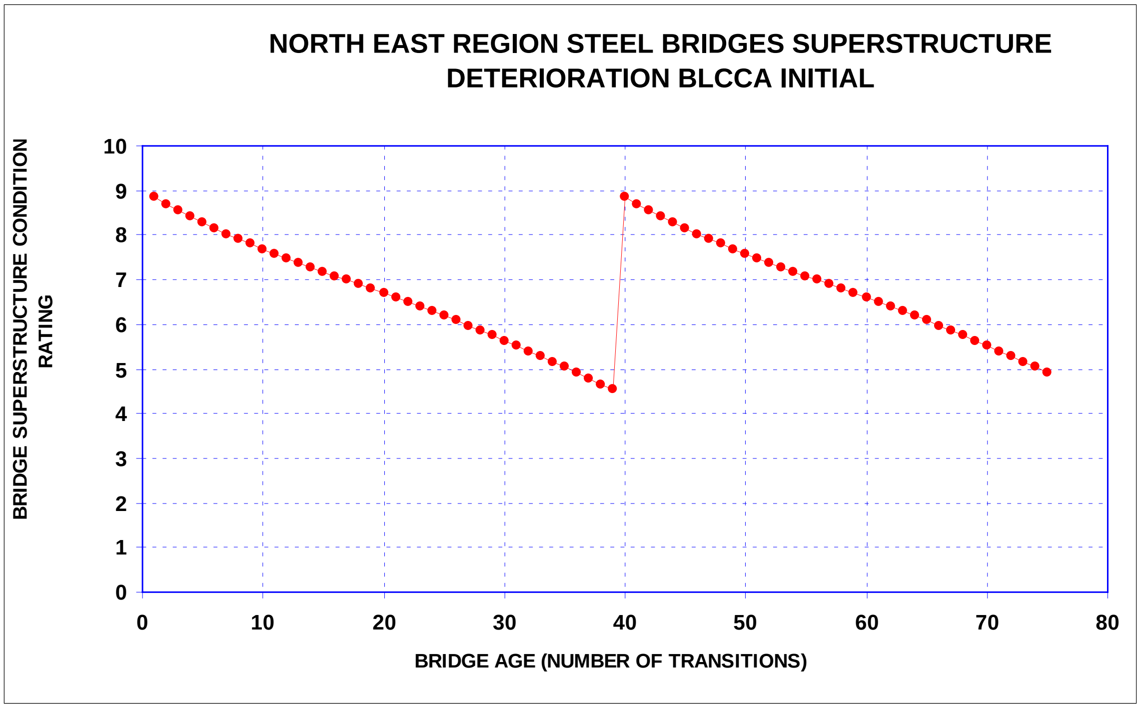

Figure 4.9: Deterioration Curve of Initial Management Strategy for Bridge Superstructure

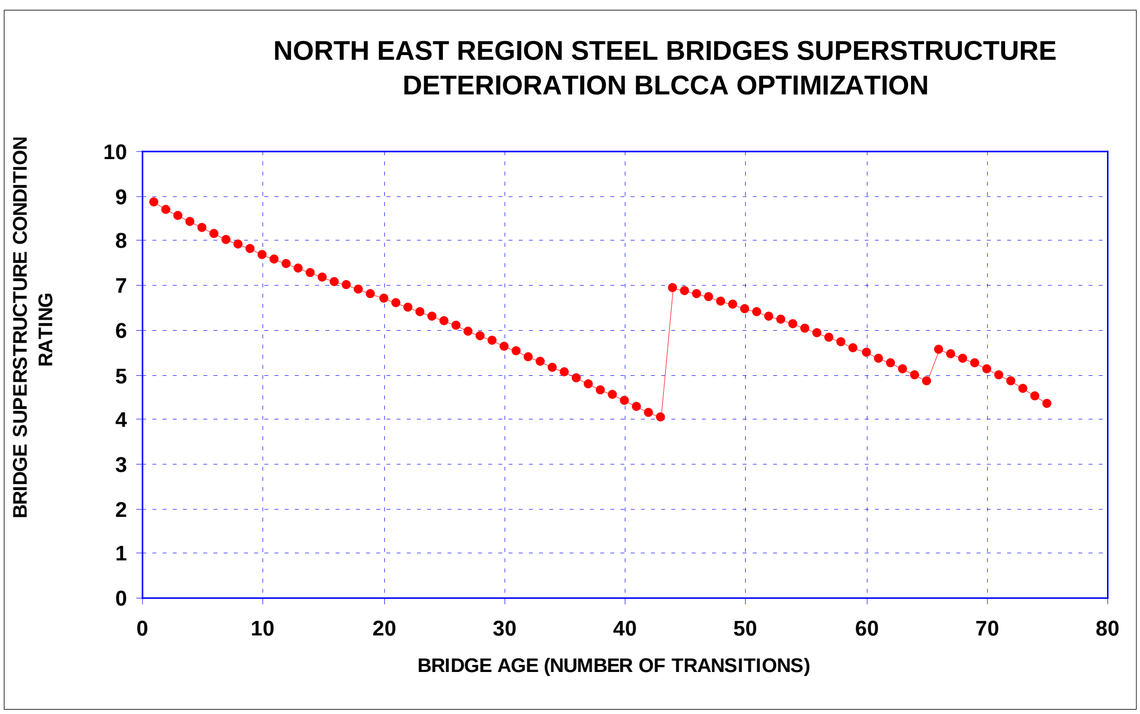

Figure 4.10: Deterioration Curve of Optimized Management Strategy for Bridge Superstructure

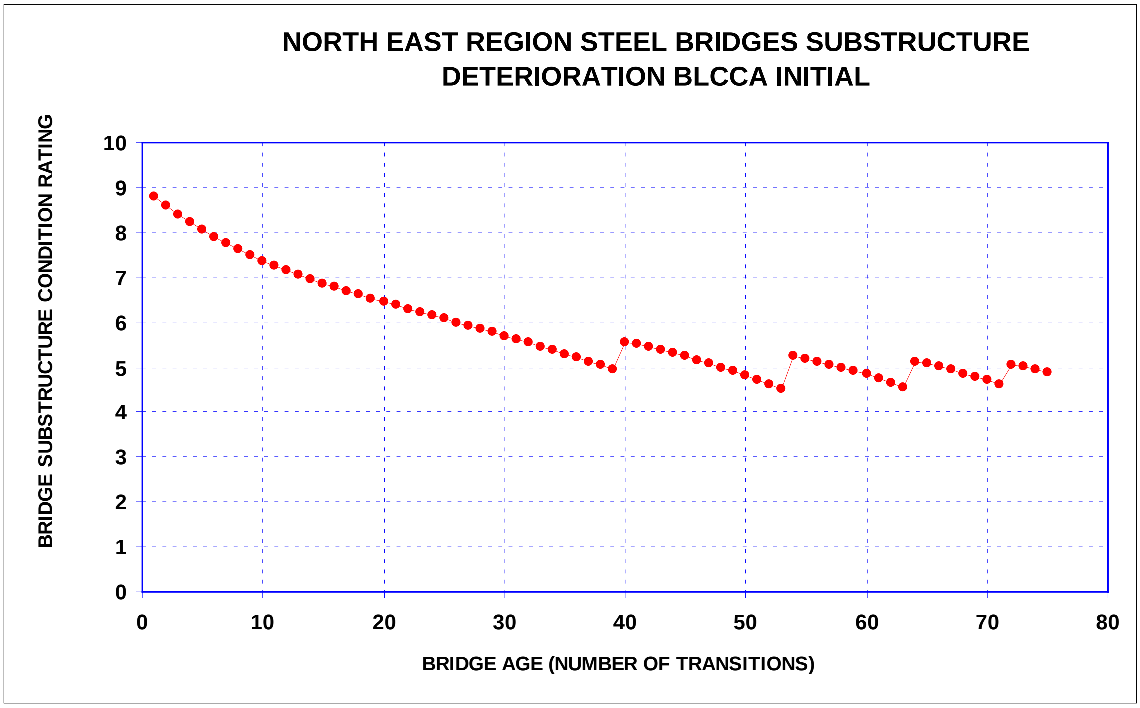

Figure 4.11: Deterioration Curve of Initial Management Strategy for Bridge Substructure

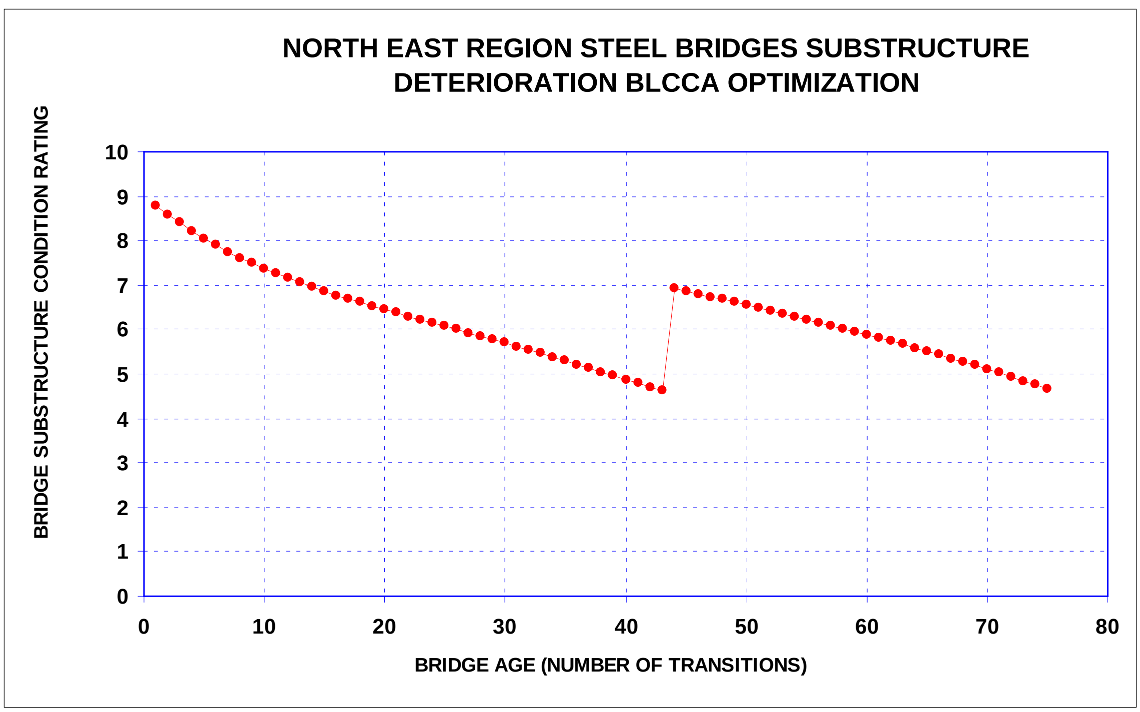

Figure 4.12: Deterioration Curve of Optimized Management Strategy for Bridge Substructure

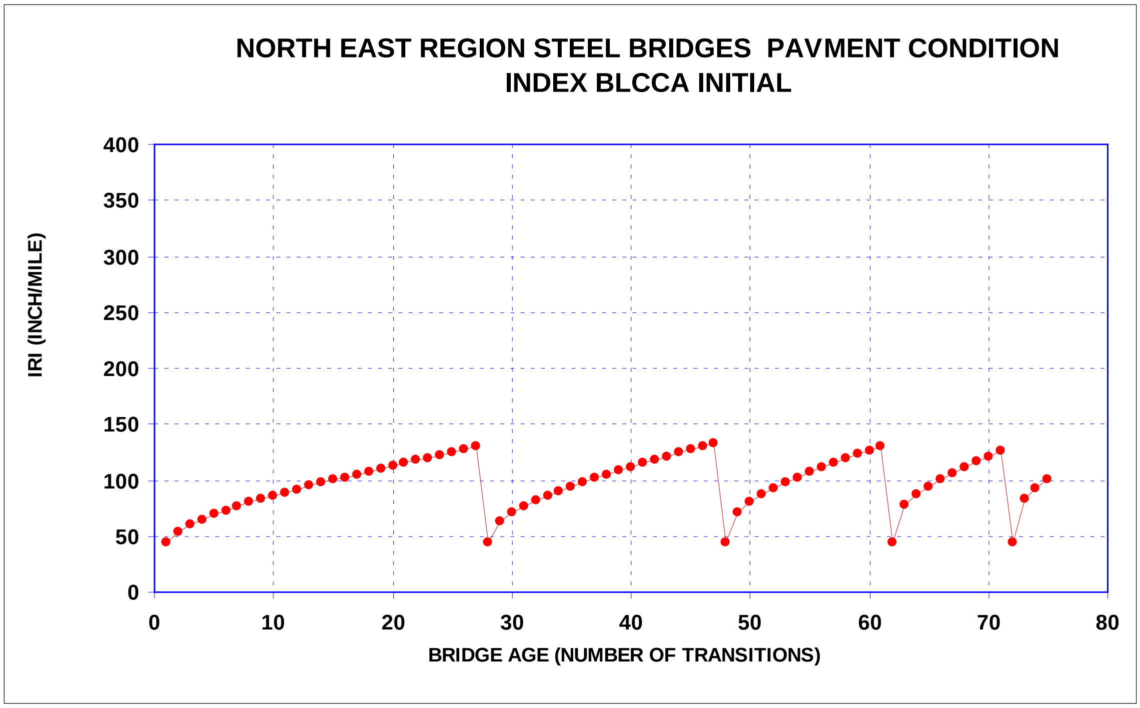

Figure 4.13: Deterioration Curve of Initial Management Strategy for Bridge Pavement

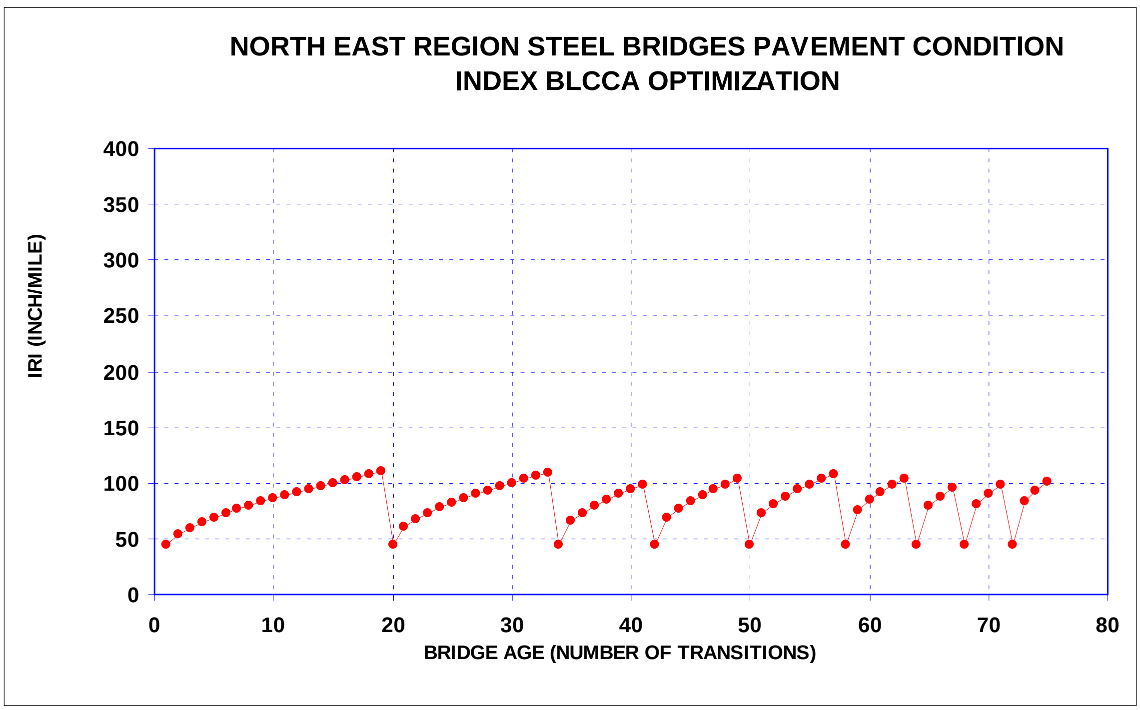

Figure 4.14: Deterioration Curve of Optimized Management Strategy for Bridge Pavement

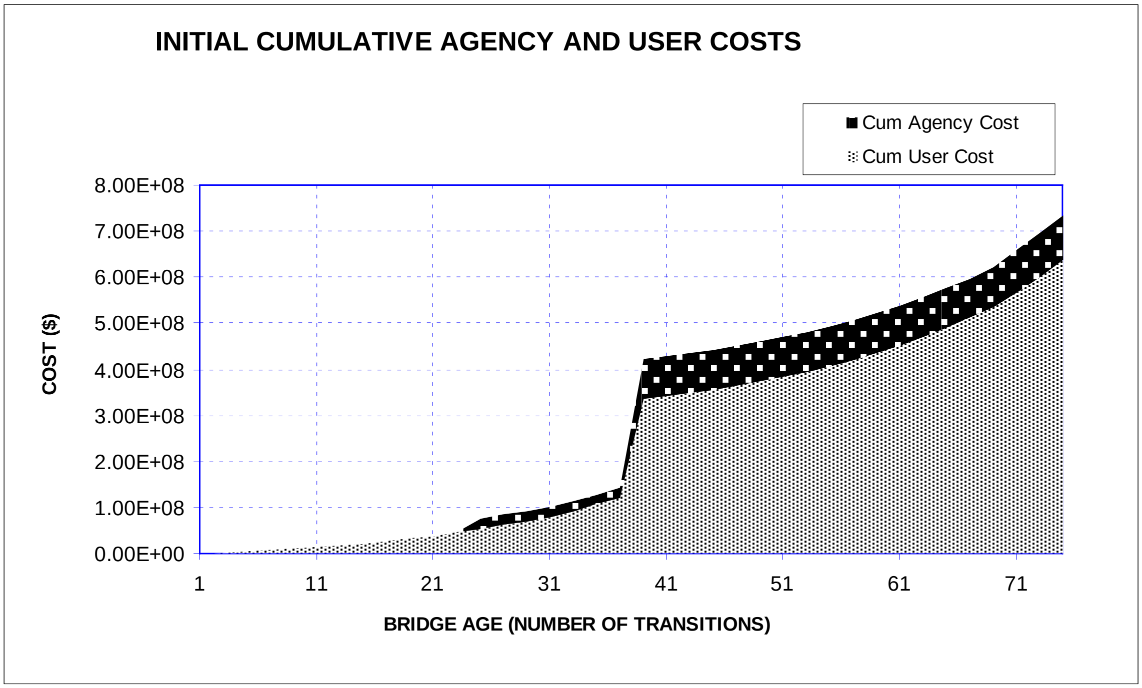

Figure 4.15: Initial Cumulative Agency and User Costs

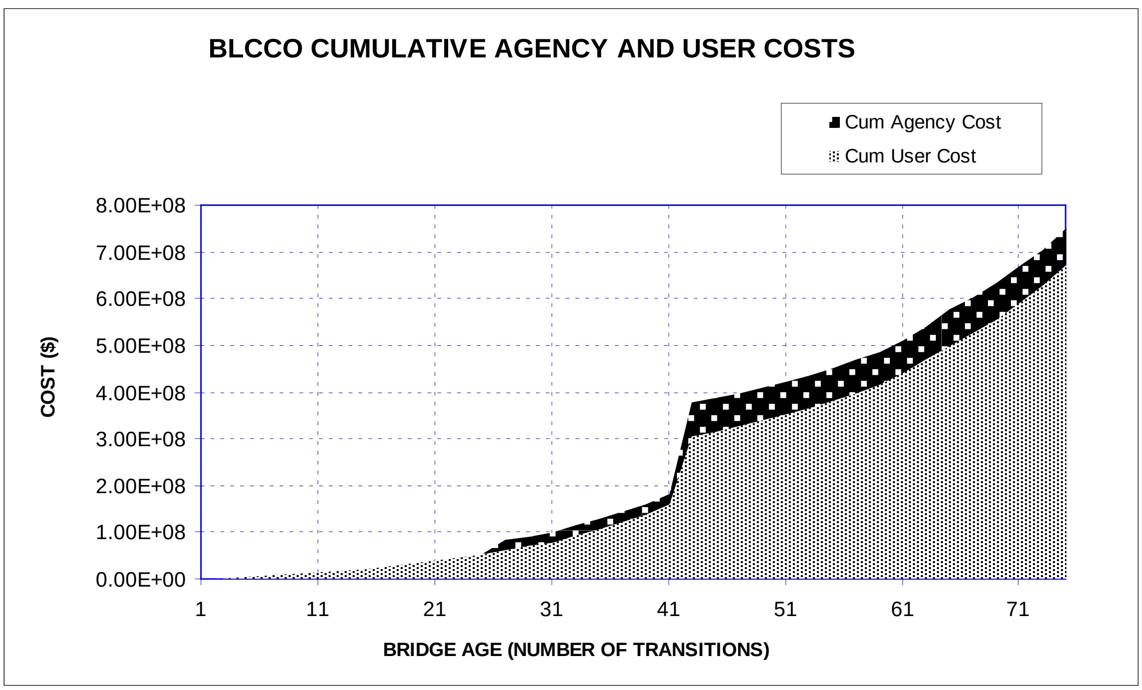

Figure 4.16: Optimized Cumulative Agency and User Costs

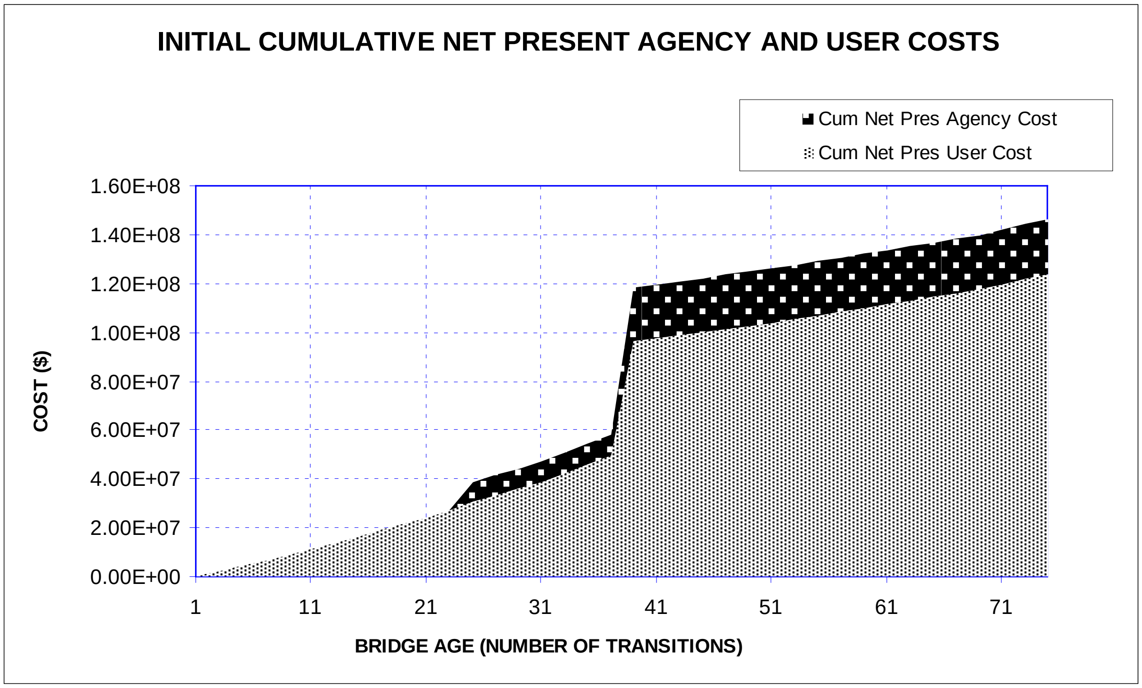

Figure 4.17: Initial Cumulative Net Present Agency and User Costs

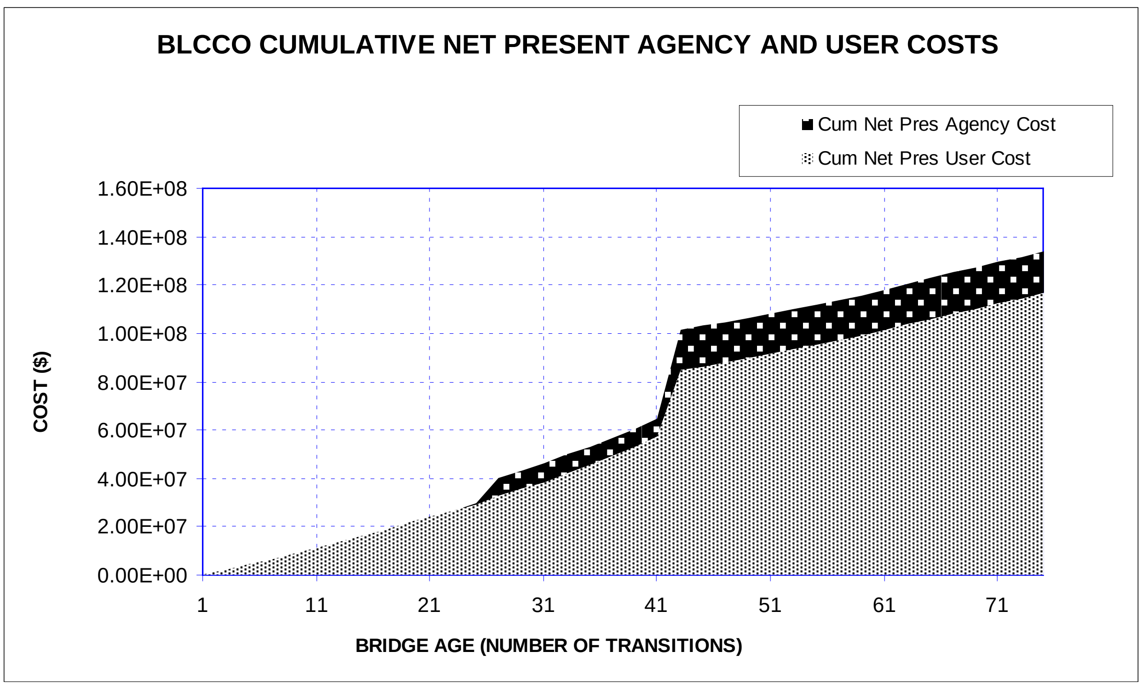

Figure 4.18: Optimized Cumulative Net Present Agency and User Costs

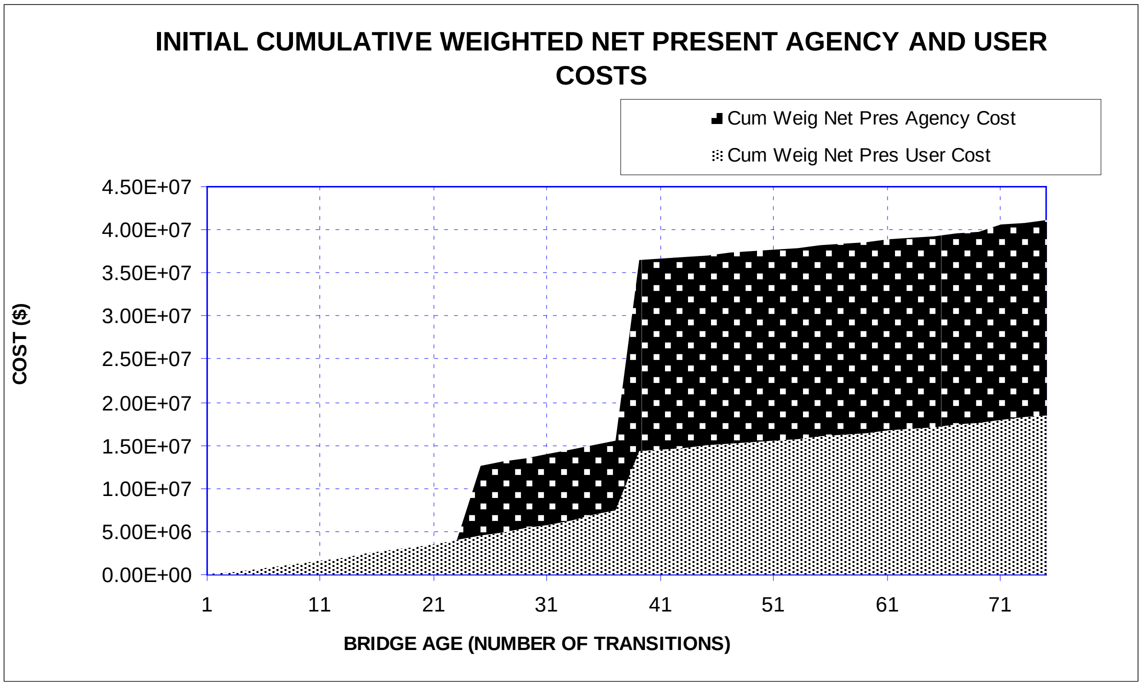

Figure 4.19: Initial Cumulative Net Present Weighted Agency and User Costs

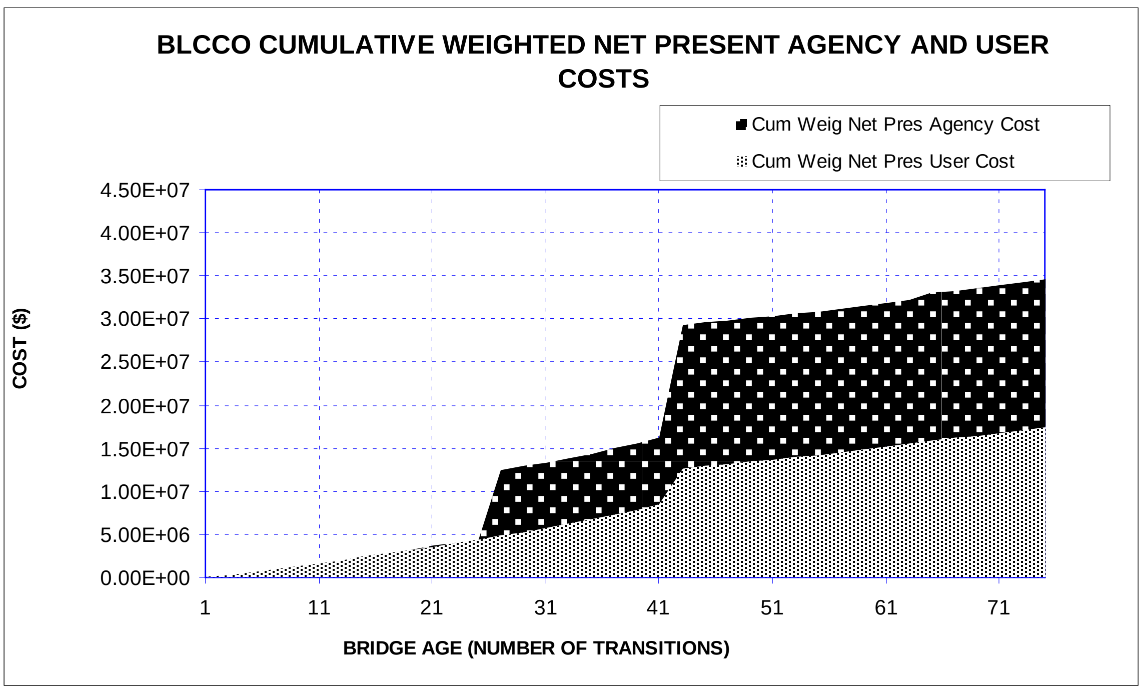

Figure 4.20: Optimized Cumulative Net Present Weighted Agency and User Costs

Chapter Five

PROBABILISTIC BRIDGE LIFE CYCLE COST OPTIMIZATION (BLCCOM-p) & SENSITIVITY ANALYSIS

5.1. Introduction

A comprehensive BLCCO methodology was described in detail in Chapter 4. This methodology is a deterministic one. For the optimization, it makes point estimates and it does not take the uncertainty of the parameters into account. In this chapter, however, a probabilistic bridge life cycle cost optimization model (BLCCOM-p) is developed. In order to tackle the uncertainties, Monte Carlo simulation is used as a risk analysis technique and applied within the genetic algorithm to reach the optima. Both the Monte Carlo simulation and the genetic algorithm are the parts of the software Risk Optimizer which facilitates applying them in the optimization. The reason for preparing a probabilistic optimization is to change the uncertainty of the deterministic method to risk in the stochastic method. In a deterministic method, the analysis parameters must rely on the estimates, which are always uncertain. Therefore, a deterministic approach relies on the outcomes due to the estimations and disregards the potential variability in the estimated parameters. However, in stochastic methods, uncertain parameters are represented as probability distributions. Thus, a stochastic BLCCOM computes the life cycle cost as a probabilistic distribution.

First of all, the extra steps in the probabilistic approach are described here. Later, a case study illustrating the BLCCOM-p is presented. Discount rate, average daily traffic, cost weight factor and several user cost values are used as the uncertain parameters. Their distributions are obtained from the previous studies.

After the case study, sensitivity analysis of the parameters is performed and the effect of the most sensitive parameters on the model is further investigated. Additionally, the effect of unexpected events on the deterioration model is investigated using the overall model.

Again, Ozbay and Javad (2003, 2004, 2005, 2006, and 2007) formed the foundation of the BLCCOM-p developed in this dissertation.

5.2. Optimization Formulations

5.2.1. Decision Variables & Constraints

The decision variables and the constraints are the same as mentioned in the deterministic BLCCO process given in Section 4.4 of Chapter 4.

5.2.2. Objective Function

BLCCOM-p uses the same objective function which is used in the deterministic BLCCOM. The objective function formulation is shown again for the ease of reference.

$$WNPV_I(T) = \sum_t^T \frac{\omega_1\{AC_t(\alpha_t) - SV_t\} + \omega_2\{TAVOC_t(IRI_t) + ACC_t(IRI_t) + UORC(\delta_t) + UC_t(\alpha_t)\}}{(1+r)^t} \qquad \text{Eq. 5.1}$$

where:

α : an integer representing the type of rehabilitation activity to be scheduled at year t and equal to zero if no rehabilitation activity planned for pavement, deck, superstructure and substructure.

ω : the weight of the cost in decision making process.

δ : operating rating factor of the bridge.

r : discount rate (decimals)

$WNPV_i(T)$: the weighted net present economic worth of the life cycle strategy in

$project_i$ due to the agency and user costs over a planning period of T years.

IRI_t : a function modeling the deterioration of pavement roughness

AC_t : the agency cost in year t depending on construction, rehabilitation or reconstruction activity.

SV_t : the salvage value at the end of the service life (T).

$TAVOC_t$ & ACC_t : the total annual vehicle operating costs and the total annual accident costs in year t respectively which are functions of the pavement roughness in that year.

$UORC_t$: user cost (detour) due to operating rating of the bridge in that year.

UC_t : the user work zone costs during the rehabilitation activities for pavement, deck, superstructure and substructure.

BLCCOM-p works using the probabilistic distributions of the parameters instead of using their deterministic values. Using these probabilistic distributions, Monte Carlo simulations are performed to calculate the objective function. The mean of the objective function that is constructed by the probability distributions and the Monte Carlo iterations measures the fitness of the solution in genetic algorithm process and using the fitness of each solution and the constraints, the optimal result is reached.

Figure 5.1 demonstrates genetic algorithm as an optimization technique in BLCCOM-p.

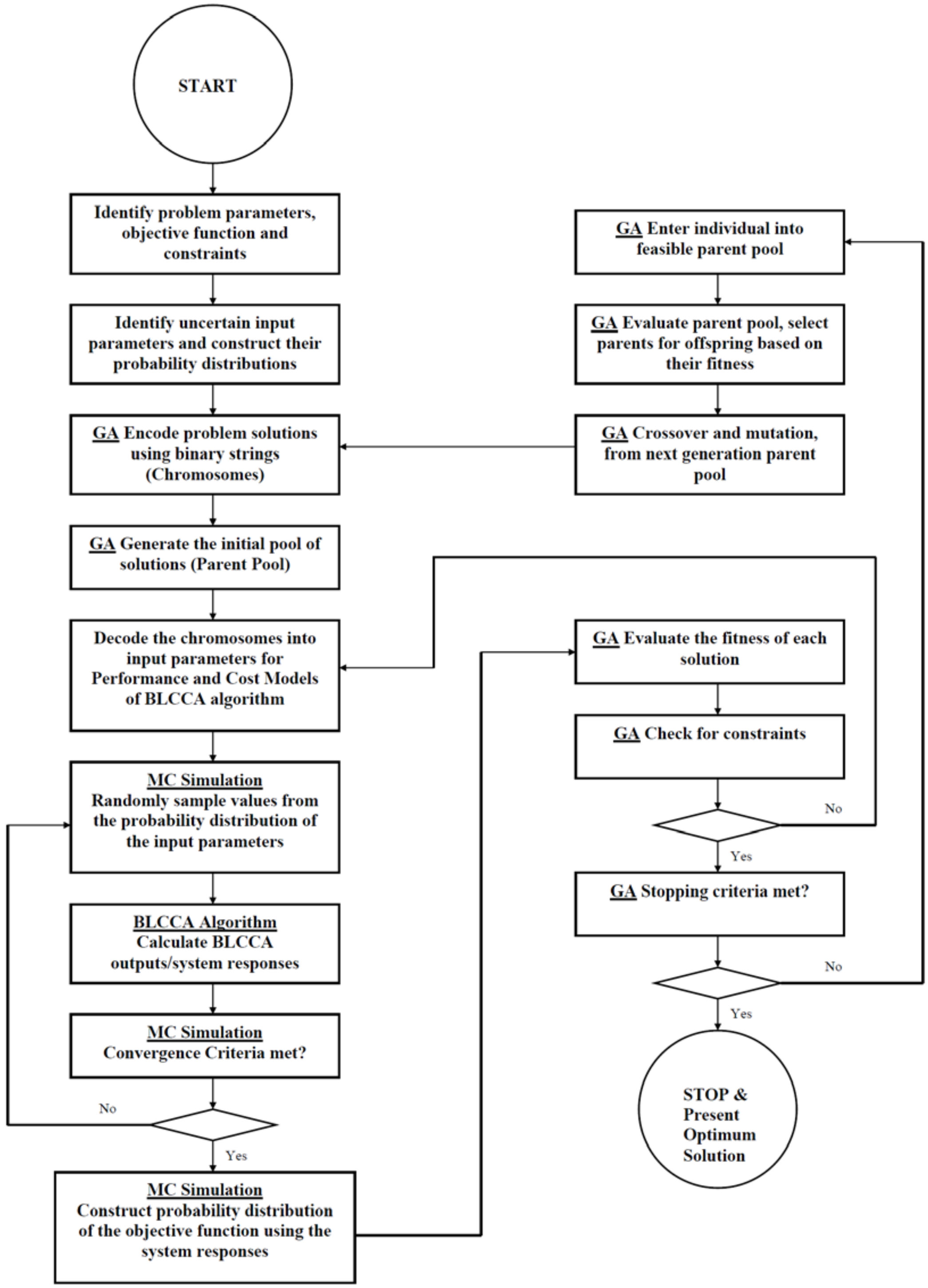

Figure 5.1 : Chart of BLCCOM-p
(Modified from Ozbay and Javad (2003) LCCO-p Chart)

5.2.3. Steps in Implementing BLCCOM-p

Step A: Initial Input Entry – Determine uncertain input parameters such as discount rate, traffic characteristic and deterioration characteristic. Prepare their probability distributions. Choose genetic algorithm operators.

Step B: GA Initialization- generate a pool of initial solutions (parent pool): for each solution assign a value for 4 different "α" valuess (pavement, deck, superstructure, substructure) for all t values randomly (each solution represents a life cycle strategy and it is always better to prepare a logical parent pool for a quick and accurate optimization process).

Step C: BLCCA Algorithm

For each solution in the parent pool

Step C-1: Monte Carlo Simulation – Using random sampling from the probability distributions of the uncertain parameters calculate.

Step C-1-1: Bridge Condition – For all t, using traffic equations and non linear optimization with Markov Chain calculate the condition of the bridge members. (Eqs 3.1 to 3.12).

Step C-1-1-1: Updating facility condition – If α >0 (for pavement) then, update IRI_t otherwise calculate roughness level using performance models (Eqs. 3.9 to 3.12).

Step C-1-1-2: Updating facility condition – If α >0 (for deck) then, update condition rating using proper repair vector (Eq. 3.7); otherwise calculate condition rating by using transition matrix (Eq. 3.1).

Step C-1-1-3: Updating facility condition – If $\alpha > 0$ (for substructure) then, update condition rating using a proper repair vector (Eq. 3.7); otherwise calculate condition rating by using transition matrix (Eq. 3.1).

Step C-1-1-4: Updating facility condition – If $\alpha > 0$ (for superstructure) then, update condition rating using a proper repair vector (Eq. 3.7); otherwise calculate the condition rating by using transition matrix (Eq. 3.1). Also no matter what α value is, update the operating rating of the bridge (Eq. 3.8).

Step C-1-1-5: Cost Estimation – If $\alpha > 0$ for pavement, deck, superstructure or substructure, calculate the agency costs for the repair or replacement activity and user work zone costs (Eqs. 4.1, 4.2, 4.4, 4.5 & 4.6).

Step C-1-1-6: Cost estimation – Calculate total annual accident cost and also calculate the annual inspection and third party costs for the bridge (Eqs. 4.3, 4.7 & 4.8).

Step C-1-1-7: Discounting – Calculate the present value of the total cost (user costs and agency costs) (Eq. 4.11).

Step C-1-2: Objective function calculation – Record system response for the iteration and calculate the objective function (Eq. 5.1).

Step C-2: Fitness of Solution – Construct probability distribution of the objective function from the system response in each Monte Carlo iterations.

Step C-3: Legitimacy of Solution – Check for constraints (Eqs. 4.9 to 4.13). If constraints are violated, disregard the solution. Otherwise, keep the solution in the parent pool.

Step D: Stopping Rule – Repeat steps F, G, H then C until the preset stopping criteria is met.

Step E: Optimum Solution – Stop and present the optimum life cycle strategy that yields the minimum mean value for the objective function.

Step F: GA Selection of parent offspring – Evaluate the valid solutions based on their fitness, and select the best two solutions as the parents for the offspring.

Step G: GA Next pool generation – Generate the next parent pool of solutions.

Step H: GA Operations Performance – GA operations of cross over and mutation on the parent pool of the next generation and go back to Step C.

5.3. Implementation of Genetic Algorithm in BLCCOM-p

In order to validate the effectiveness of the suggested algorithm, results from a case study are presented here. The bridge No. 1100070 in NJ over Jacobs Creek is used for the case study.

5.3.1. Bridge No. 1100070 Properties from the NBI Database

Figure 5.2 shows the location and below is the information about the bridge No. 1100070 from the NBI database.

Name: Penngtn-Titusvl Rd over Jacobs Creek

Structure number: 1100070

Location: 0.8 mi east of co 579 jct

Purpose: Carries two-lane highway over waterway

Route classification: Local (Rural) [09]

Length of largest span: 58.1 ft. [17.7 m]

Total length: 65.9 ft. [20.1 m]

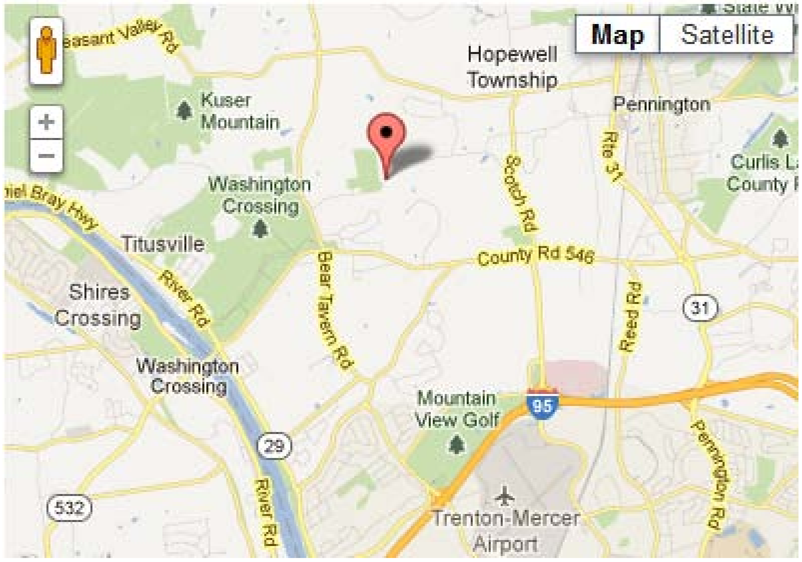

Figure 5.2 : Location of Bridge No. 1100070 in NJ

Roadway width between curbs: 24.3 ft. [7.4 m]

Deck width edge-to-edge: 30.2 ft. [9.2 m]

Skew angle: 53°

Owner: County Highway Agency [02]

Year built: 1963

Historic significance: Bridge is not eligible for the National Register of Historic Places [5]

Design load: Other or Unknown [0]

Main span material: Prestressed concrete [5]

Main span design: Box beam or girders - Multiple [05]

Deck type: Concrete Cast-in-Place [1]

Number of main spans: 1

Wearing surface: Monolithic Concrete (concurrently placed with structural deck) [1]

Recommended work: Replacement of bridge or other structure because of substandard load carrying capacity or substantial bridge roadway geometry. [31]

Estimated cost of work: $1,199,000

May 2009 Inspection

Average daily traffic: 1,300 [as of 2009]

Truck traffic: 3% of total traffic

Deck condition: Fair [5 out of 9]

Superstructure condition: Serious [3 out of 9]

Substructure condition: Satisfactory [6 out of 9]

Structural appraisal: Basically intolerable requiring high priority of replacement [2]

Deck geometry appraisal: Meets minimum tolerable limits to be left in place as is [4]

Water adequacy appraisal: Better than present minimum criteria [7]

Roadway alignment appraisal: Meets minimum tolerable limits to be left in place as is [4]

Channel protection: Bank protection is being eroded. River control devices and/or embankment have major damage. Trees and rush restrict the channel. [5]

Scour condition: Bridge foundations determined to be stable for the assessed or calculated scour condition. [8]

Inventory rating: 9.0 tons [8.2 metric tons]

Operating rating: 39.9 tons [36.3 metric tons]

Evaluation: Structurally deficient [1]

Sufficiency rating: 18.4

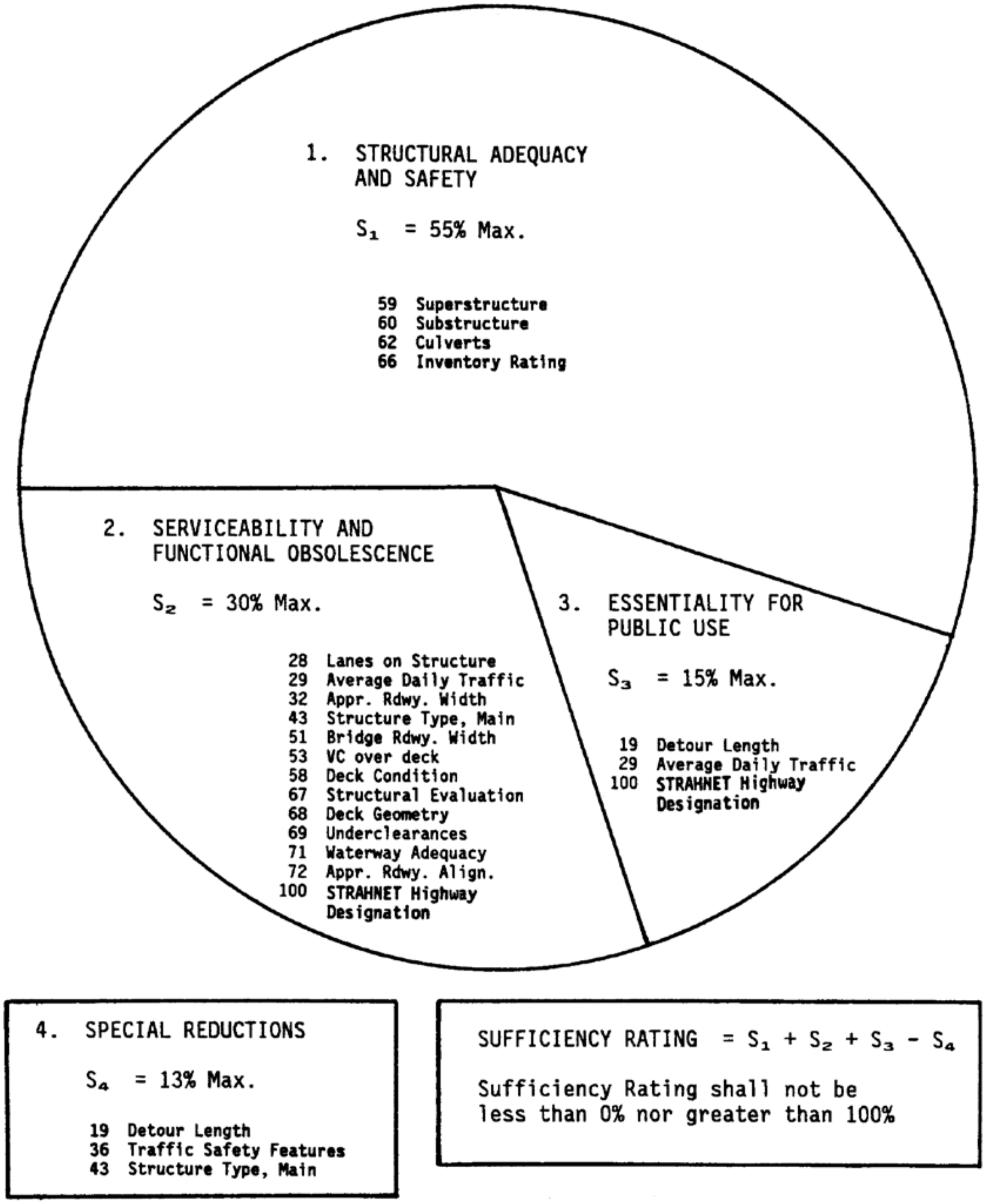

Figure 5.3: Summary of Sufficiency Rating Factors (Report No. FHWA-PD-96-001, 1995)

5.3.2. New Jersey Department of Transportation Consideration for the Bridge

This bridge is a 50 years old bridge in New Jersey. Its deck, superstructure, substructure condition ratings, operating rating and sufficiency rating clearly shows that the bridge is in a bad condition. Among these, sufficiency rating is the parameter which NJDOT uses with the highest weight on bridge project ranking criteria. Sufficiency rating is a method of evaluating highway bridge data by calculating four separate factors to obtain a numeric value which is indicative of the bridge sufficiency to remain in service. The result of this method is a percentage in which 100 percent would represent an entirely sufficient bridge and zero percent would represent an entirely insufficient or deficient bridge. The summary of sufficiency rating factor is shown in Figure 5.3.

For the bridge No. 1100070 the sufficiency rating is:

S1 = 0 S2 = 0.06 S3 = 0.124 S4 = 0 **SR = 18.4%**

For the sufficiency rating equations and calculations please see Appendix A4.

In order to have a better idea on the urgency of repair or replacement of the bridge, NJDOT Bridge Project Ranking Criteria is shown below:

	Criteria	Weight (W)	Scoring (S)
A	Average Daily Traffic (Item 29) (If Item 102=1 multiply ADT by 2)	20%	0 to 30,000 = 0 30,001 to 60,000 = 0.25 60,001 to 90,000 = 0.5 90,001 to 120,000 = 0.75 Greater than 120,000 = 1.0
B	Functional Class (Item 26)	10%	Interstate/Freeways (01,11,12) = 1.0 Arterials (02,06,14,16) = 0.67 Collectors (07,08,17) = 0.33 Locals (09, 19) = 0
C	Deck (Item 58)	10%	3 or 4 = 1.00 5 or 6 = 0.5 >6 = 0.00
D	Sufficiency Rating	60%	(100 - SR) / 100

Table 5.1: NJDOT Bridge Project Ranking Criteria

Final Score = $\dfrac{((S_A \times W_A) + (S_B \times W_B) + (S_C \times W_C) + (S_D \times W_D)) \times 1000}{4}$

The higher the score, the higher the ranking. The maximum score is 250.

Priority 1: >200, 2: 150-199, 3: 100-149, 4: 50-99, 5: <50

After ranking, the list of the bridges is reviewed by an engineering group to further refine the list based on engineering judgment. At this point, the bridge management program, BLCCO-p model which is developed in this research can be used by the authorities very effectively not only to understand the bridge's existing condition, but also to figure out the possible repair-reconstruction activities for the remaining life of the bridge. Furthermore, this bridge management program would consider the user cost aspect of the projects and consider the uncertainties.

The final score for the bridge No. 1100070 is 135, Priority 3 (ADT=1300, Functional Class=Local, Deck=5 and SR=18.4%). Accordingly, the bridge does not need an urgent replacement. This will be further investigated with BLCCO-p model in the following sections.

5.3.3. Optimization Model Input Parameters

The optimization input parameters are determined the same way explained in the case study in Chapter 4. The uncertainties are dealt with by reducing them into risks, which is done by associating each uncertain variable with a probability distribution in the algorithm. This way, for the final result, it is be possible to estimate the outcomes in terms of probabilistic ranges. All the deterministic and probabilistic parameters with their distributions are shown in Table 5.2.

BRIDGE #1100070 CHARACTERISTICS		
Bridge Length =	65.90	feet
Bridge Width =	30.20	feet
ADT (total) =	1300	veh/day
% ADT (Commercial) =	3	
Length Betw Det Loc=	0.5	mile
Speed =	45	mile / hour
Work Zone Speed =	25	mile / hour
Detour Length =	3	mile
Detour Speed =	25	mile / hour

VEHICLE SPEED RESTRICTIONS		
Condition Rating 5 =	50	mile / hour
Condition Rating 4 =	45	mile / hour
Condition Rating 3 =	35	mile / hour

OPERATING RATING PARAMETERS	
Intercept "C" =	1.903
X Variable "b" =	7.043

GENETIC ALGORITHM PARAMETERS	
Population Size =	50
Cross Over Rate =	50%
Mutation Rate =	5%

PAVEMENT CHARACTERISTICS		
$IRI_{t=0}$ =	45	in/mile
AC_{visc} =	1600	Poise
AC_{void} =	6	% volume
AC_{thick} =	6	in
B_{thick} =	14	in
DGT =	9	Days
FI =	925	F^o/day
D =	0.5	
L =	0.9	

ECONOMIC PARAMETERS		
Period =	25	years

DECK COST INPUT		
Minor Repair Cost =	200	$\$/ft^2$
Major Repair Cost =	600	$\$/ft^2$
Replacement Cost =	1000	$\$/ft^2$
Minor Repair Time =	40	ft^2/hour
Major Repair Time =	10	ft^2/hour
Replacement Time =	5	ft^2/hour

SUPERSTRUCTURE COST INPUT		
# of Girders =	5	
Minor Repair Time =	16	ft/hour
Major Repair Time =	4	ft/hour
Replacement Time =	1	ft/hour

SUBSTRUCTURE COST INPUT		
Minor Repair Cost =	20	$\$/ft^3$
Major Repair Cost =	150	$\$/ft^3$
Volume of Piers + Abutments=	1200	ft^3

PAVEMENT COST INPUT		
Replacement Cost =	65.71	$\$/yd^2$

BRIDGE MARKOV CHAIN DETERIORATION VECTORS										
P_{deck} =	0.727	0.855	0.951	0.949	0.954	0.778	0.645	0.504	0.424	1.000
$P_{superstr}$ =	0.831	0.906	0.944	0.925	0.908	0.581	0.757	0.497	0.375	1.000
P_{substr} =	0.766	0.873	0.956	0.949	0.939	0.857	0.717	0.644	0.586	1.000

Table 5.2 : Input Parameters for the BLCCO-p Model

PARAMETER NAME	DISTRIBUTION
Traffic Growth Rate (%)	Triangular (0.5, 2.5, 4.5)
Discount Rate (r)	Triangular (0.02, 0.04, 0.06)
User Cost Weight (%)	Triangular (10%, 15%, %20)
Super Structure Replacement Cost ($/ft)	Normal (5000, 500)
Super Structure Major Repair Cost ($/ft)	Normal (3000, 300)
Super Structure Minor Repair Cost ($/ft)	Normal (1000, 100)
Passenger Vehicle Operating Cost ($/hr)	Normal (5, 1)
Commercial Vehicle Operating Cost ($/hr)	Normal (30, 5)
Superstructure Deterioration	Normal (n_t, 1.05)

Table 5.2 (continued): Input Parameters for the BLCCO-p Model

"Traffic Growth Rate" and "Discount Rate" distributions are obtained from Jawad et al. (2007). "User Cost Weight" distribution is determined using common sense during the optimization process. The range which leads to the better optimization is chosen. Repair and replacement costs are obtained using the information from the New Jersey Department of Transportation and a normal distribution is assumed with a standard deviation of 10% of the means. "Vehicle Operating Costs" are obtained from the US Department of Transportation, Federal Highway Administration from a report finalized in 2000 "Federal Highway Cost Allocation Study" FHWA (2000). Reasonable assumptions were made for their distributions and standard deviations due to data unavailability. . Distribution for superstructure deterioration is calculated by analyzing the NBI data and preparing deterioration models at the mean, upper and lower limits of the NBI data for every transition. Northeast region, concrete bridges are used to set up the model for this case study, to simulate the bridge deterioration.

5.3.4. Developing the Life Cycle Cost Management Plan for the Model

The bridge is 50 years old, therefore the life cycle cost management plan for the remaining life of the Bridge No. 1100070 is prepared for a 25-years period. The facts that the bridge is in Priority 3 state for replacement and the ADT is low are considered and lower condition rating constraints for the serviceability are assigned.

For the deck:

- It is assumed that the deck condition will be maintained at a minimum condition rating of 3 and up as a serviceability constraint
- It is assumed that there can be up to 2 repairs for the remaining life of the bridge.
- The activities are assigned the following codes:

  1 : Do nothing, 2 : Minor Repair, 3 : Major Repair, 4 : Replace

For the superstructure:

- It is assumed that the superstructure condition will be maintained at a minimum condition rating of 3 and up as a serviceability constraint
- It is assumed that there can be up to 2 repairs throughout the life span of the bridge.
- The activities are assigned the following codes:

  1 : Do nothing, 2 : Minor Repair, 3 : Major Repair, 4 : Replace

For the substructure:

- It is assumed that the substructure condition will be maintained at a minimum condition rating of 3 and up as a serviceability constraint
- It is assumed that there can be up to 1 repair throughout the life span of the bridge.
- The activities are assigned the following codes:

  1 : Do nothing, 2 : Minor Repair, 3 : Major Repair

For the pavement:

- It is assumed that the road condition index will be maintained at a minimum value of 80 and down as a serviceability constraint

- It is assumed that there can be up to 5 repairs throughout the life span of the bridge.
- The activities are assigned the following codes:

1 : Do nothing, 2 : Replace

Year	Deck Condition	Deck	Super Condition	Super	Sub Condition	Sub	Pave. Cond.	Pavement
50	4	1	4	1	5	1	77	1
51	4	1	4	1	5	1	78	2
52	4	1	4	1	5	1	18	1
53	4	1	4	1	4	1	30	1
54	4	1	3	1	4	1	33	1
55	4	1	3	1	4	1	36	1
56	4	1	3	1	4	1	39	1
57	4	1	3	1	4	1	41	1
58	4	1	3	2	4	1	43	2
59	4	1	5	1	4	1	18	1
60	4	1	5	1	4	1	31	1
61	4	1	5	1	4	1	35	1
62	3	1	5	1	4	1	38	1
63	3	1	5	1	4	1	40	1
64	3	1	5	1	4	1	42	2
65	3	1	4	1	4	1	18	1
66	3	1	4	1	3	1	32	1
67	3	2	4	1	3	1	36	1
68	5	1	4	1	3	1	39	2
69	5	1	4	1	3	1	18	1
70	5	1	4	1	3	1	33	1
71	5	1	3	1	3	1	37	1
72	5	1	3	1	3	1	40	1
73	5	1	3	1	3	1	43	1
74	5	1	3	1	3	1	45	1
75	5	1	3	1	3	1	47	1

Table 5.3: Probabilistic Optimized Remaining Life Repair and Reconstruction Plan for Bridge No. 1100070 (BLCCOM-p)

5.3.5. Results

BLCCO-p is performed using the Risk Optimizer software. The resulting optimized bridge life span management plan is shown in Table 5.3. As can be seen, the optimization scheduled the repair activities for the remaining 25 years in the life of the bridge. The effect of the repair-replacement activities on the bridge components deterioration curves are shown in Figures 5.4 through 5.7 for the optimized bridge management strategy.

The objective function which is “the weighted net present economic worth of the life cycle strategy in project due to the agency and the users over a planning period of 25 years” is minimized by the optimization process. Since this is a probabilistic optimization, the results are given in terms of ranges with certain probabilities, rather than discrete values or single lines in the graphs.

In order to show how the uncertain parameters affect the optimization model, the cumulative total, weighted and discounted costs for 25 years due to the BLCCO-p model are graphed as shown in Figures 5.8 through 5.10. For clarity, the graphs are not shown

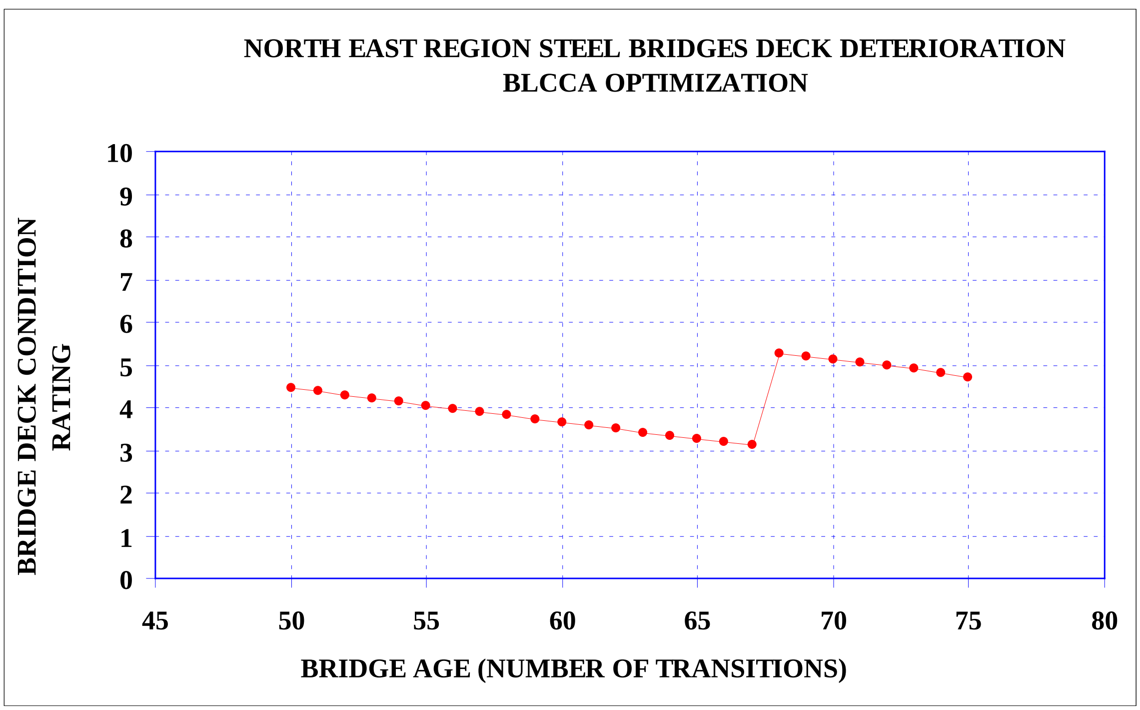

Figure 5.4: Deterioration Curve of Optimized Management Strategy for Bridge Deck

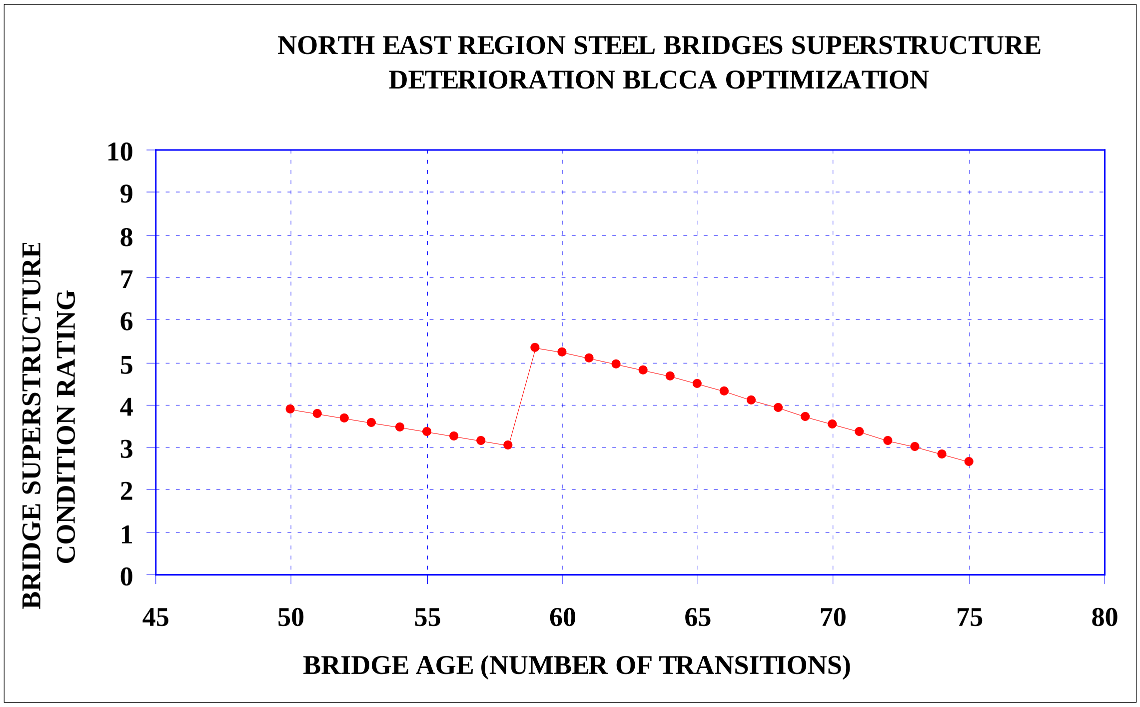

Figure 5.5: Deterioration Curve of Optimized Management Strategy for Bridge Superstructure

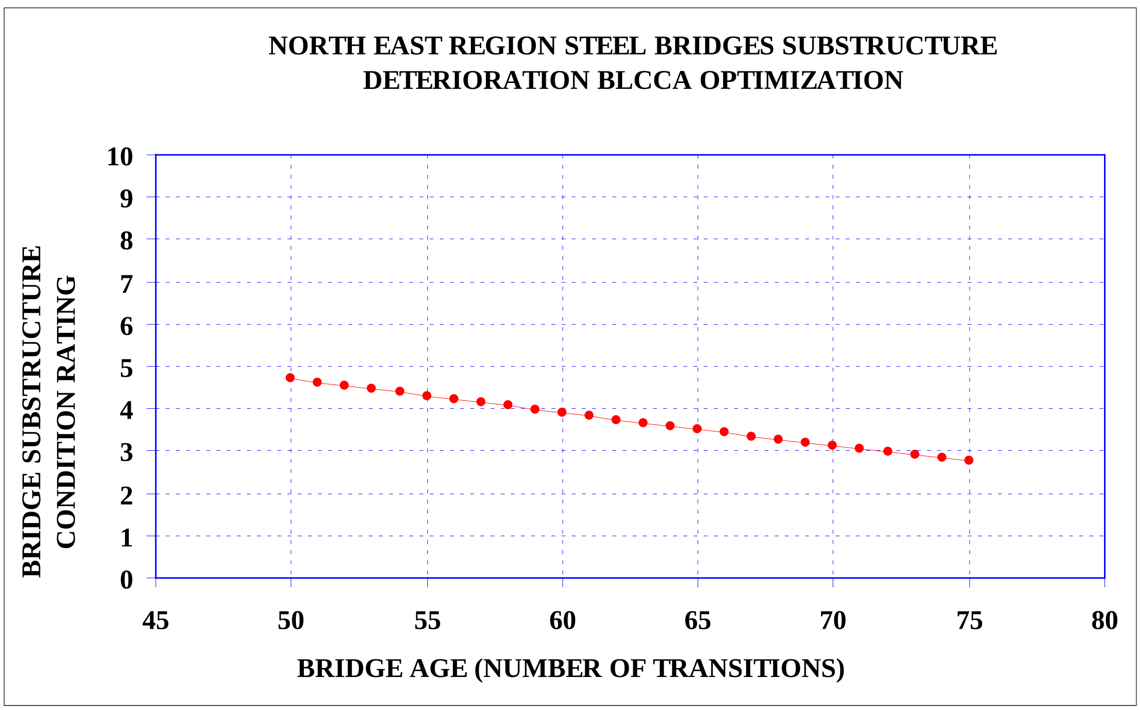

Figure 5.6: Deterioration Curve of Optimized Management Strategy for Bridge Substructure

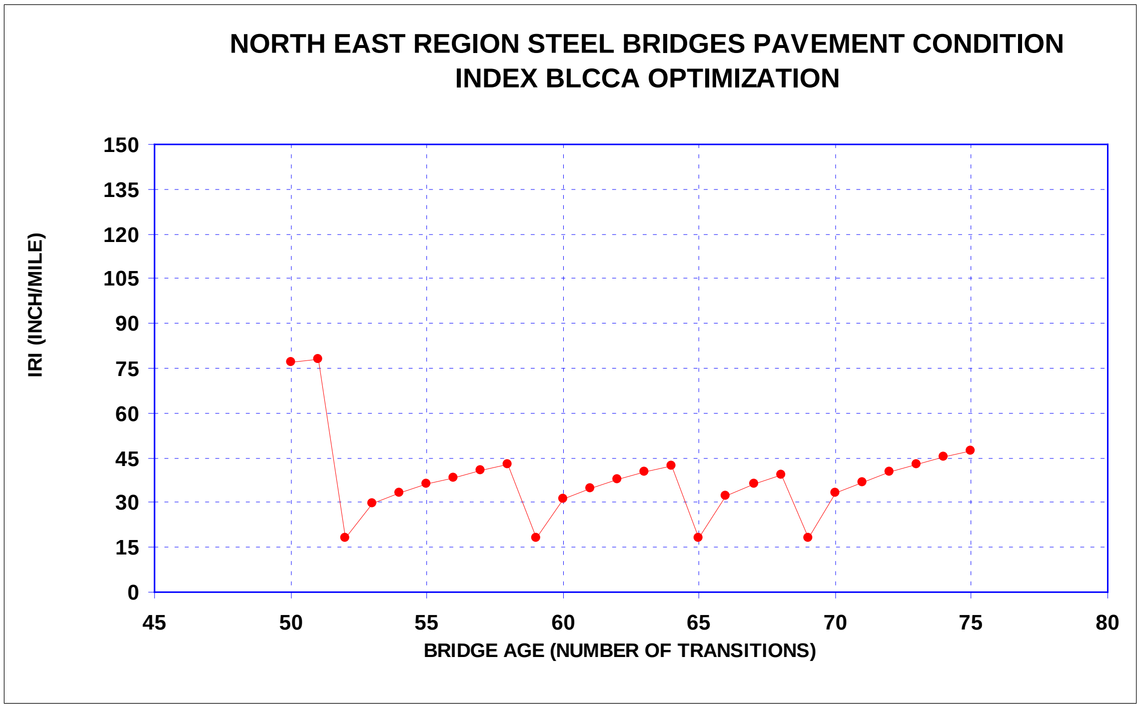

Figure 5.7: Deterioration Curve of Optimized Management Strategy for Bridge Pavement

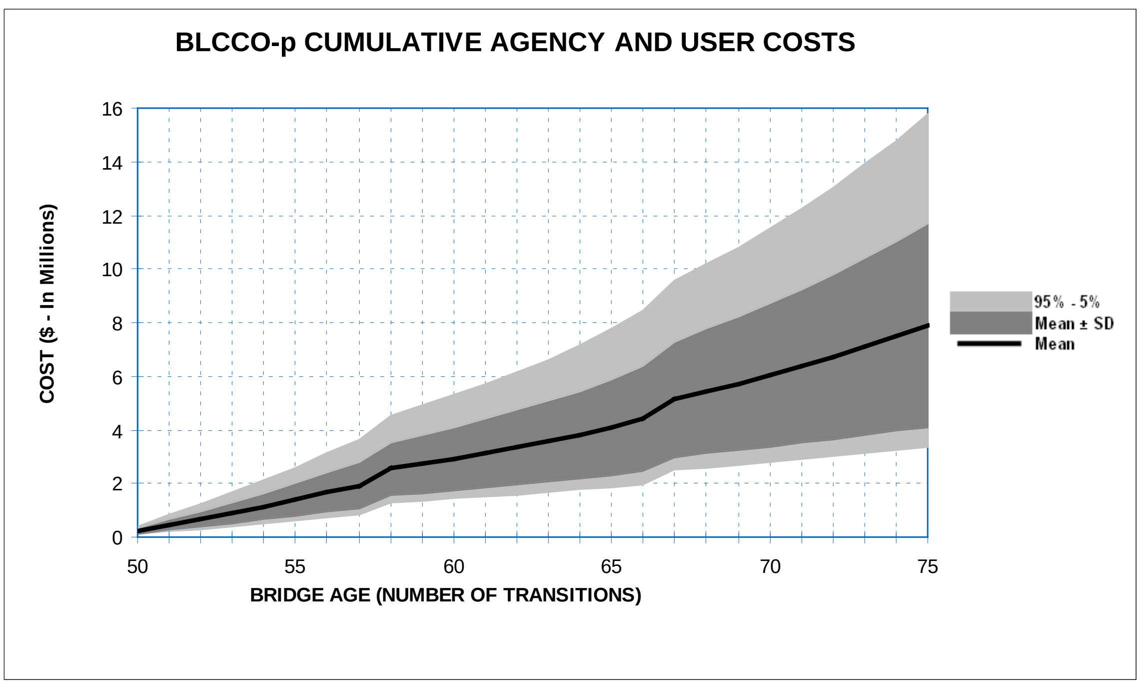

Figure 5.8: Cumulative Total Cost According to BLCCO-p

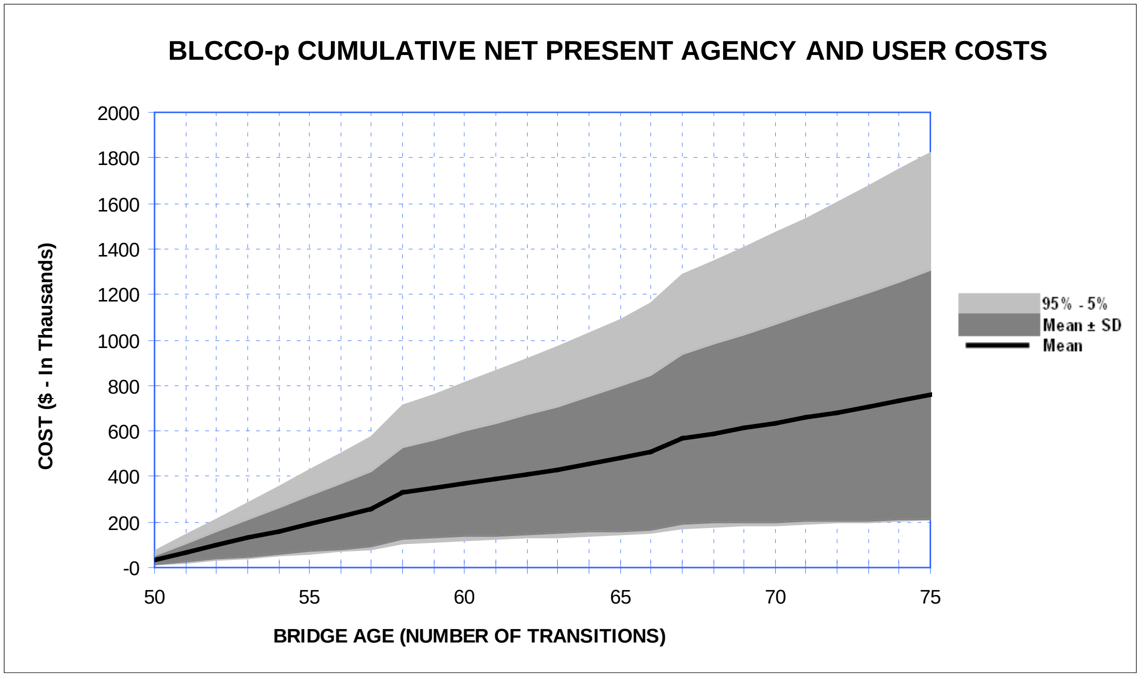

Figure 5.9: Cumulative Total Net Present Cost According to BLCCO-p

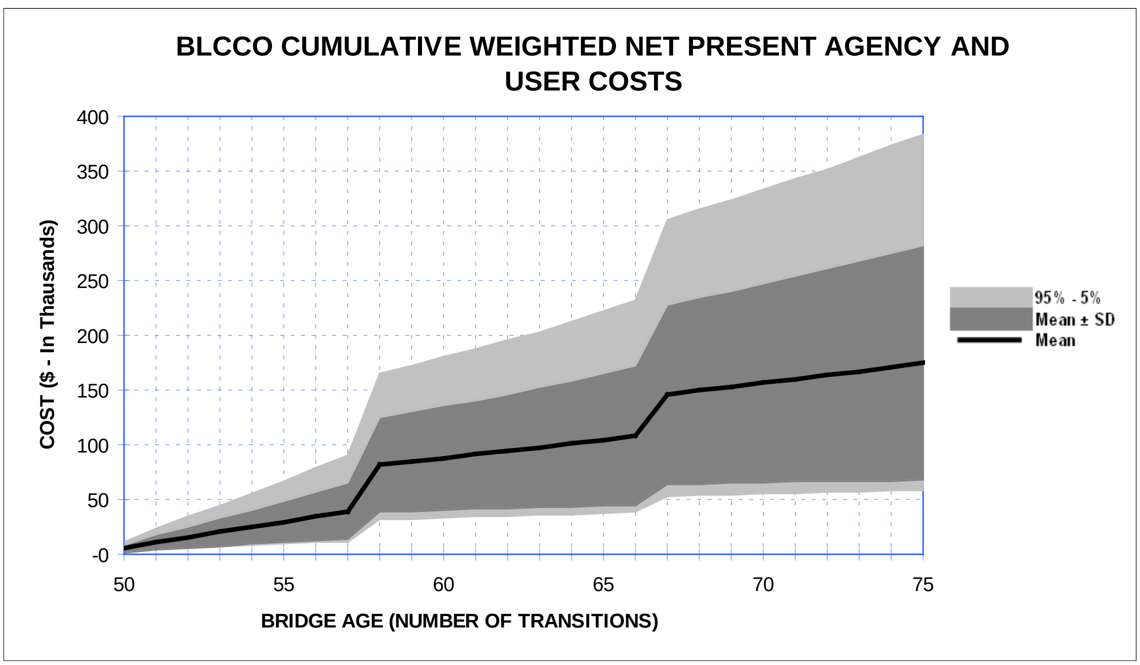

Figure 5.10: Cumulative Total Weighted Net Present Cost According to BLCCO-p

in the same scale but rather given in their appropriate scales. In these graphs, the black thick line represents the mean values surrounded by two margins which are ±1 standard deviation (shown in dark grey color) and 90% confidence interval (i.e. %5 to % 95, in light grey color).

As it can be seen clearly from the Cumulative Total Weighted Net Present Cost according to BLCCO-p, the need for major actions on the bridge in years 58 and 67, results in peaks in the total cost values in those years. The action in year 58 is due to superstructure repair and the action in year 67 is due to deck repair, which can also be seen in Table 5.3.

The graphs also show that, as the years pass, the variations in the total cost increase. This increased variance is due to the uncertainty of the variables used in the optimization process (i.e. discount rate, average daily traffic growth rate, superstructure deterioration rate, etc.).

Resultant dollar values of the 25 year period optimization process for Agency, User and Total costs are also presented in Table 5.4. The results are presented with the minimum and maximum values, 90% confidence interval (95%-5%), Mean±SD and the Mean values.

According to the NBI database, it is recommended that the bridge needs to be replaced. The cost of this operation is estimated to be $1,199,000. This estimated value takes into account that the replacement will be done in a very short time, therefore, no discount factors are used. No user cost factor or uncertainties are considered either. According to the BLCCO-p model developed here, the bridge replacement seems to be not necessary and the bridge could be maintained with the minimum required condition ratings applying several repairs for the remaining life time of 25 years. The NBI database

	Min	5%	Mean-SD	Mean	Mean+SD	95%	Max
User Cost	1,703,996	2,606,845	3,355,238	7,150,274	10,945,309	15,020,880	23,209,240
Agency Cost	627,183	688,006	709,219	741,795	774,371	795,678	847,140
Total Cost	2,331,179	3,294,851	4,064,457	7,892,069	11,719,679	15,816,558	24,056,380
Net Present User Cost	69,904	172,929	162,425	686,170	1,209,916	1,700,651	4,537,686
Net Present Agency Cost	17,270	28,534	35,595	71,965	108,336	146,699	217,431
Net Present Total Cost	87,174	201,463	198,020	758,136	1,318,252	1,847,350	4,755,117
Weighted Net Present User Cost	10,505	24,647	22,465	102,867	183,269	259,407	703,189
Weighted Net Present Agency Cost	17,270	28,534	35,595	71,965	108,336	146,699	217,431
Weighted Net Present Total Cost (WNPV)	27,775	53,181	58,060	174,833	291,605	406,105	920,620
Lognormal Distribution for WNPV	13,716	60,240	61,465	175,512	289,559	389,615	∞

Table 5.4: Numeric 25 Year Optimization Values for Agency, User and Total Cost in US Dollars

recommended bridge replacement cost can be compared with “Agency Cost” item in Table 5.4 (Please note that the objective function is “Weighted Net Present Total Cost” in the table). The optimal solution, however, which is obtained by running the probabilistic optimization algorithm in conjunction with GA, yields an optimal total agency cost with a 90% confidence interval of $688,006 to $795,678 with a mean value of $741,795. This is 38% lower than the recommendation given in the NBI database.

Turning back to the objective function ‘WNPV’, which is the parameter that is actually being optimized given by Eq. 16, the probability distribution of this optimum value with a mean of $174,833 is presented in Figure 5.11. As shown, a LogNormal distribution (Lognorm”161796, 114047” Shift=+13716) fits perfectly to the data, with a mean value of $175,512, which is quite close to the actual mean value. By plotting the probability or cumulative distributions of the WNPV, it is easier to quantify the optimization alternatives' probabilities.

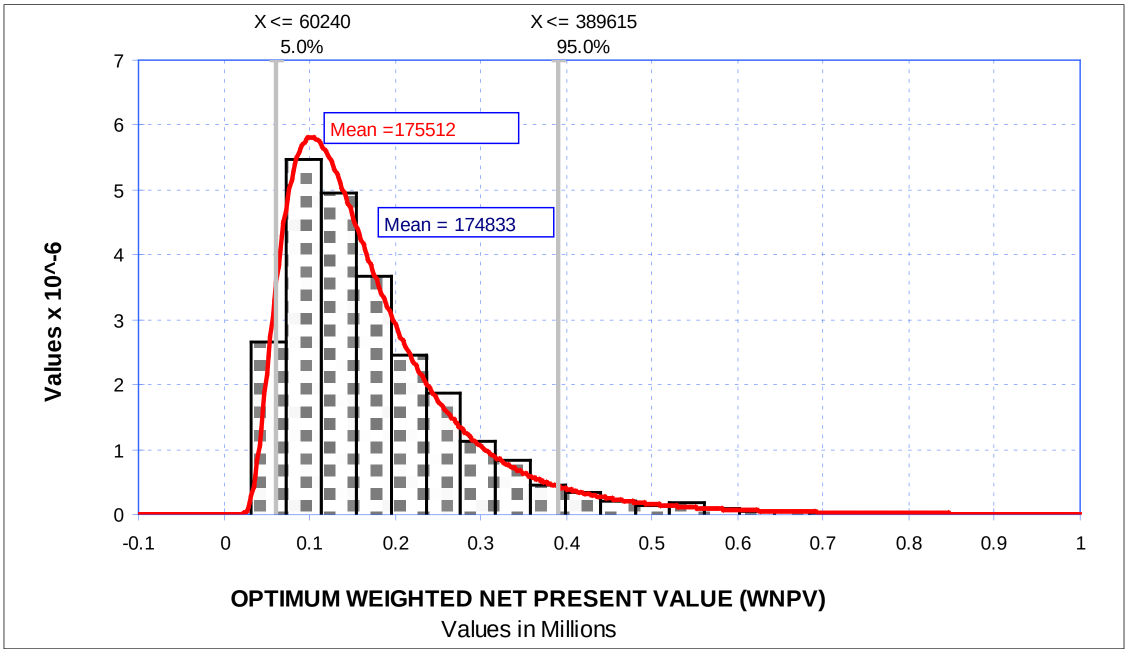

Figure 5.11: Probability distribution of optimal WNPV.

5.4. Sensitivity Analysis

Good modeling practice requires that the modeler provides an evaluation of the confidence in the model, possibly assessing the uncertainties associated with the modeling process and with the outcome of the model itself. Uncertainty and Sensitivity Analysis offer valid tools for characterizing the uncertainty associated with a model. Uncertainty analysis quantifies the uncertainty in the outcome of a model. Sensitivity Analysis has the complementary role of ordering, by importance, the strength and the relevance of the inputs in determining the variations in the output. In models involving many uncertain input variables, sensitivity analysis is an essential ingredient of model building and quality assurance.

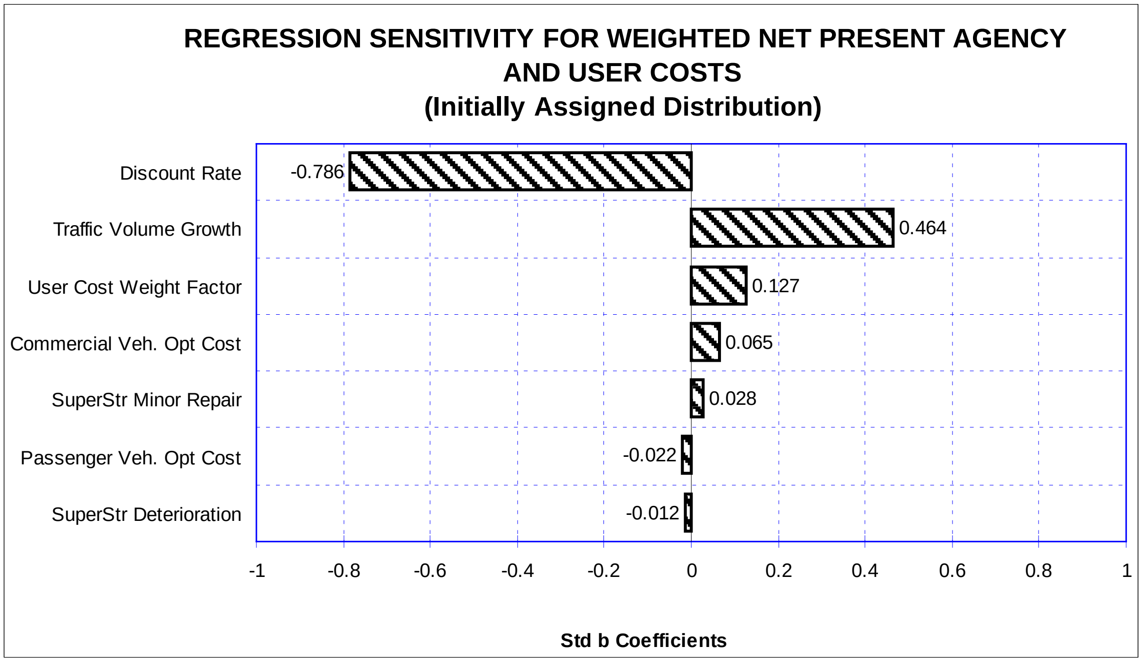

Figure 5.12: Tornado graph for the regression sensitivity coefficients in the BLCCO-p

Sensitivity analyses are carried out in order to systematically examine the effect of a set of variables given previously, on the WNPV. This is done simply by estimating the correlation coefficients between the input parameters and presenting them in a tornado graph.

As it is seen in the tornado graph (Figure 5.12), the input parameters which have the most significant effect of adding the variability to the objective function are the discount rate, traffic volume growth rate and the user cost weight factor.

In order to observe if the most sensitive parameters will still be the same ones when their distributions change, the most sensitive parameters determined above are assigned different distributions as shown in Table 5.5.

PARAMETER NAME	DISTRIBUTION
Traffic Growth Rate (%)	Normal(2.5, 1.5, Truncate(0, 5))
Discount Rate (r)	Normal(0.04, 0.02, Truncate(0, 0.12))
User Cost Weight (%)	Normal(0.15, 0.05, Truncate(0, 0.3))

Table 5.5 : New Distributions for the Most Sensitive Probabilistic Parameters (Dist-2)

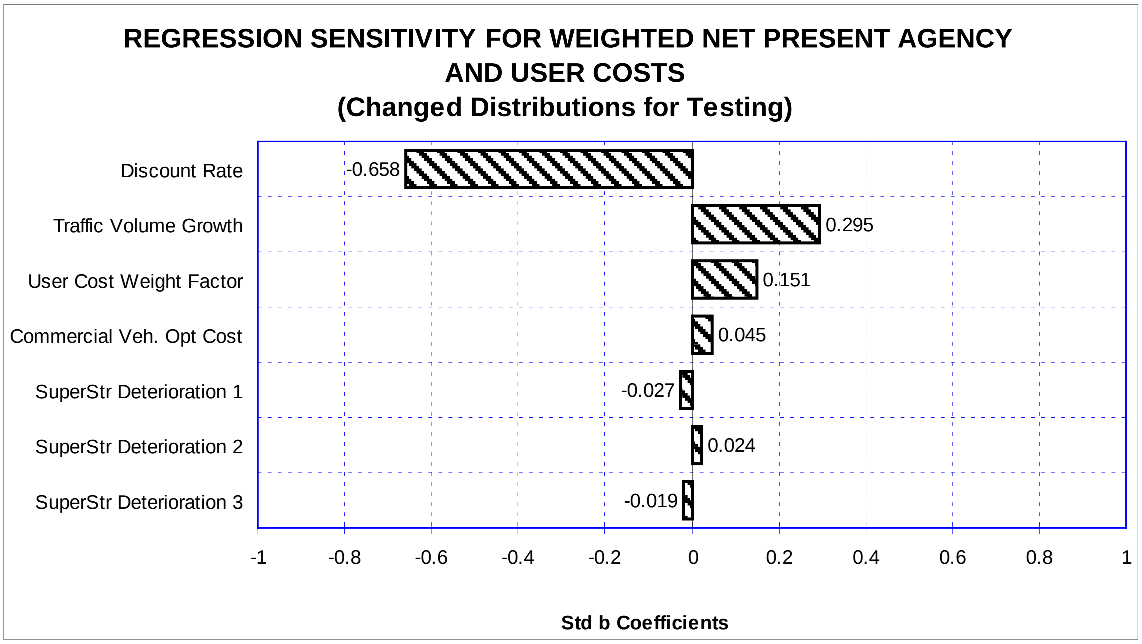

Figure 5.13: Tornado graph for the regression sensitivity coefficients in the BLCCO-p (Dist-2)

Comparison of the two graphs in Figures 5.12 and 5.13 indicates that, even when the uncertain parameters are assigned different probability distributions, their importance in the model does not change significantly. In the next section, these sensitive uncertain parameters will be investigated further and a parametric study will be performed to see the quantitative effects of these parameters on the objective function.

5.5. Parametric Study for the Most Sensitive Parameters

Parametric studies allow researchers to nominate parameters for evaluation, define the parameter range and analyze the results of each parameter variation.

A parametric study requires the following:

- Design Objective (BLCCO)
- Parameters nominated for use in the simulation (Determined from sensitivity study)

- Parameter ranges identified (Shown in the figures below)
- Optimization criteria specified (WNPV)
- Various configurations generated (Genetic Algorithm, Monte Carlo Simulation)

In addition to the most sensitive three parameters, namely the traffic growth rate, discount rate and user cost weight, the occurrence probability of an unexpected event is also investigated in the parametric study. In order to give a better understanding of how these parameters affect the part of the optimization process during different stages, the results are shown for the user cost, agency cost and total cost for the costs, net present costs and weighted net present costs. Weighted net present cost represents the objective function (WNPV). Figures 5.14 through 5.17 show the graphs for parametric study performed here. For all the cases, the life cycle management strategies differed slightly, but the overall serviceability of the bridge was maintained at a steady level.

For the traffic volume growth parametric study, the growth rate is investigated from 0% to 5% increases, yearly. Since the serviceability condition rating is set at very low numbers, the change in weighted net present agency cost is minimal and since the increased vehicle volumes means higher user costs, the increase in the user cost and the total cost is very steep (both polynomial and exponential trend line fits).

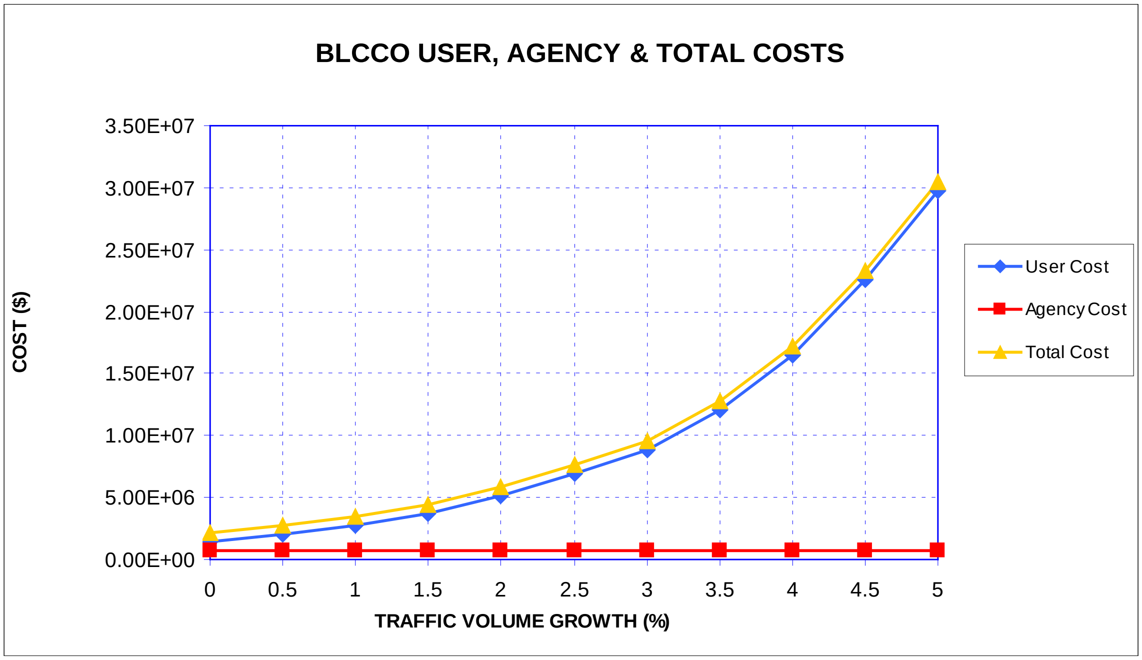

Figure 5.14a: Effect of Traffic Volume Growth on Cost

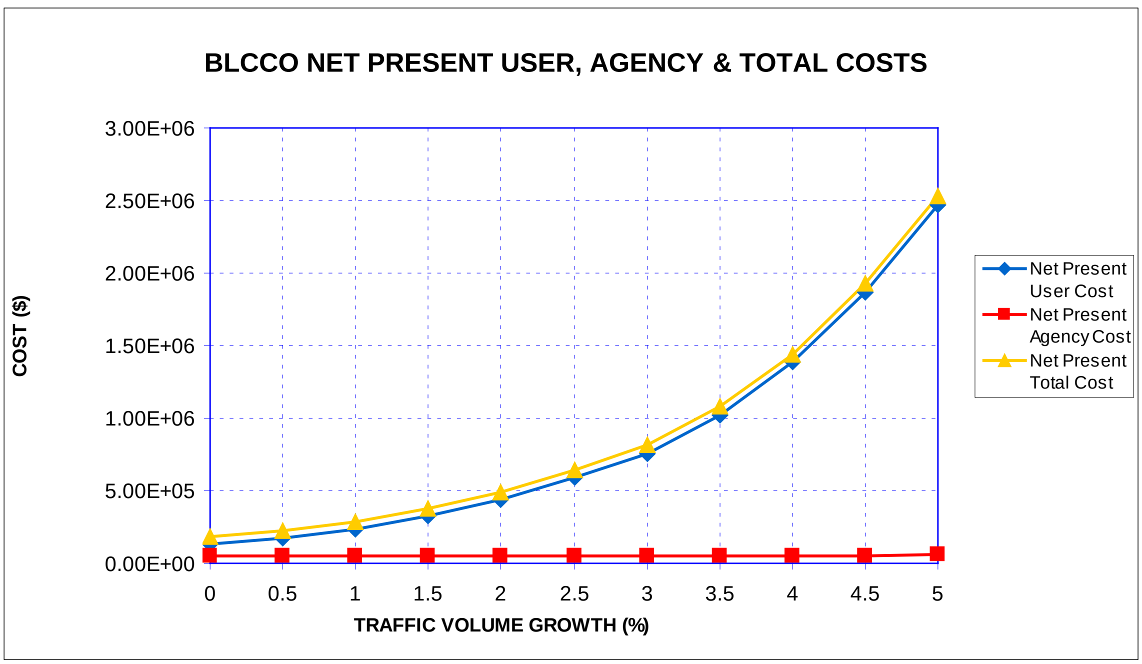

Figure 5.14b: Effect of Traffic Volume Growth on Net Present Cost

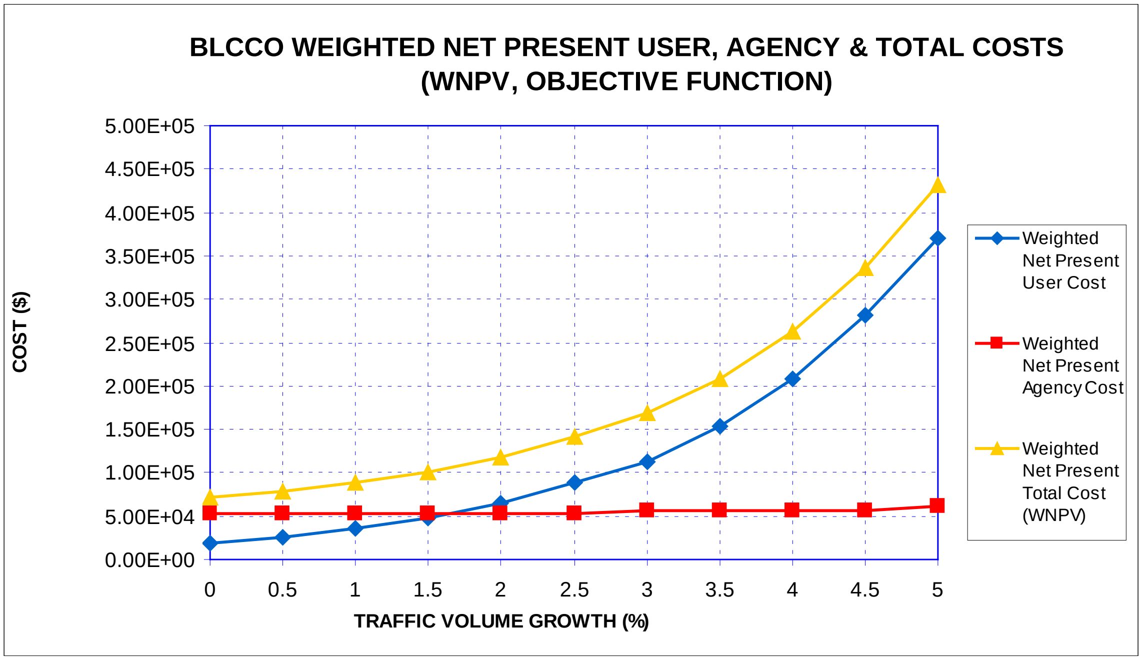

Figure 5.14c: Effect of Traffic Volume Growth on Weighted Net Present Cost

For the discount rate parametric study, the rate is investigated from 0.02 to 0.16. All the cost values decrease exponentially as the discount rate decreases.

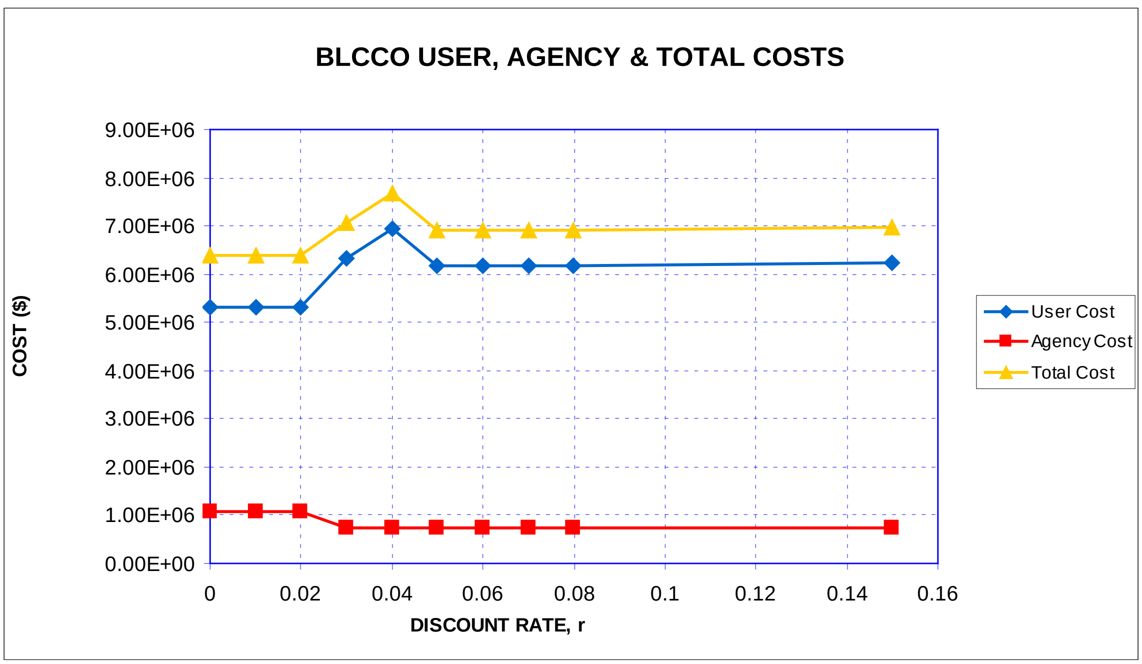

Figure 5.15a: Effect of Discount Rate on Cost

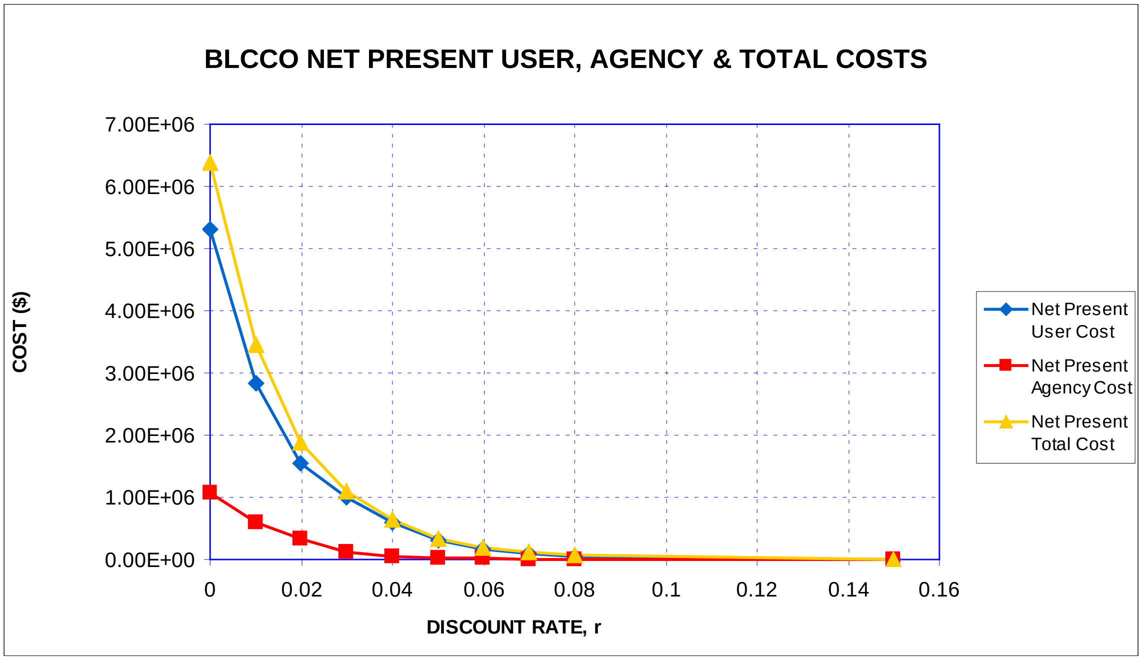

Figure 5.15b: Effect of Discount Rate on Net Present Cost

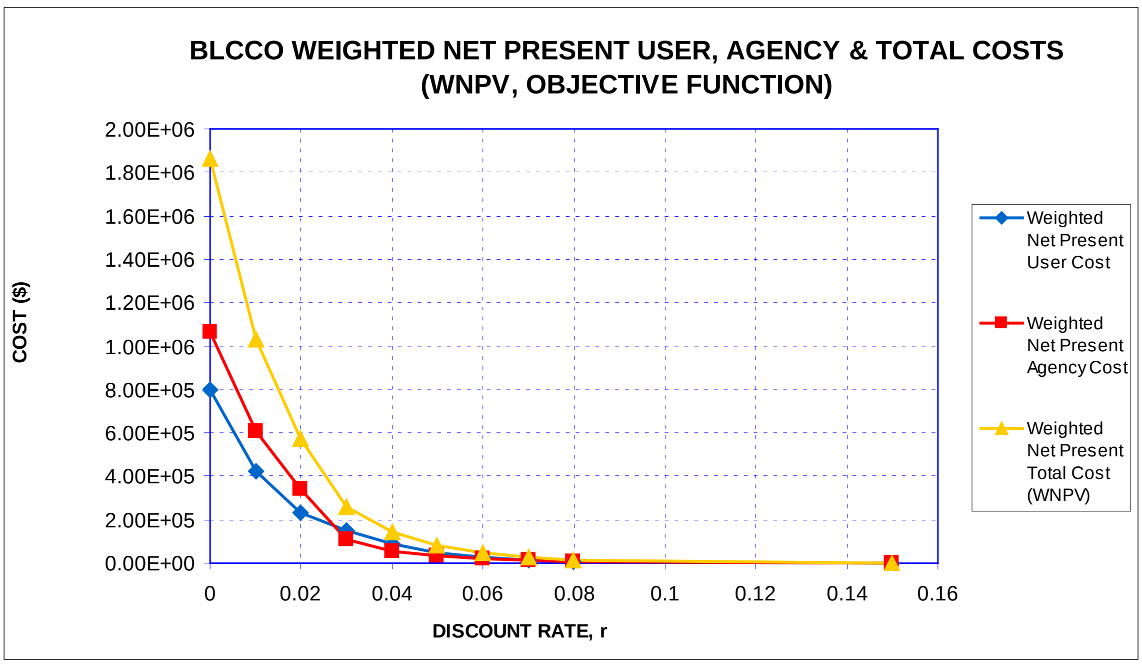

Figure 5.15c: Effect of Discount Rate on Weighted Net Present Cost

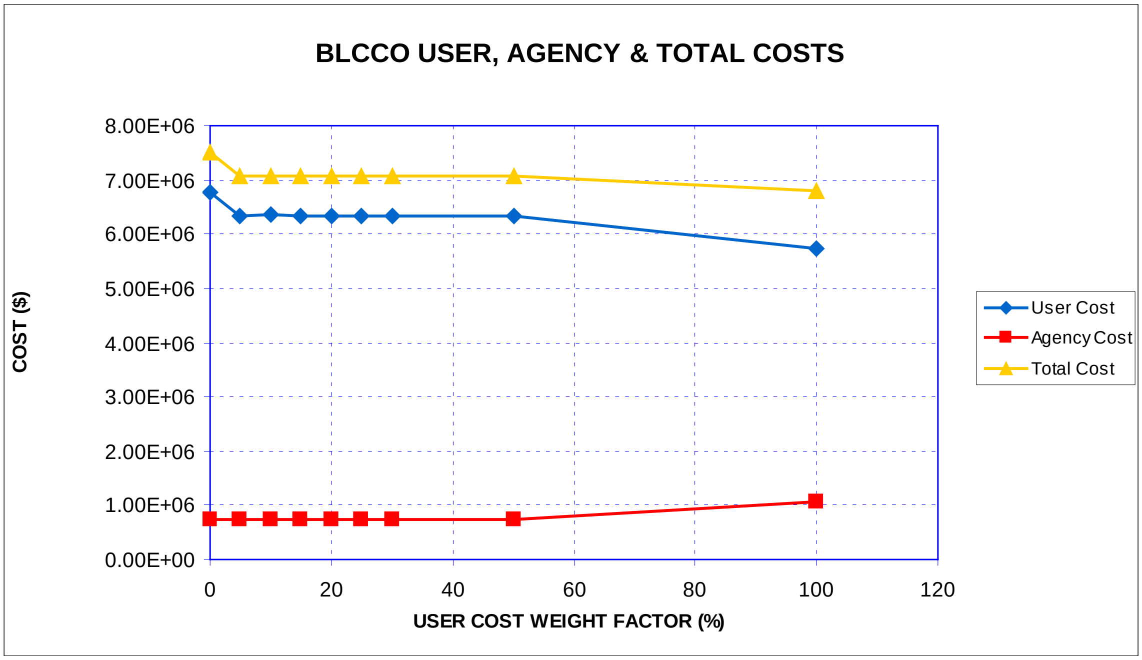

Figure 5.16a: Effect of User Cost Weight on Cost

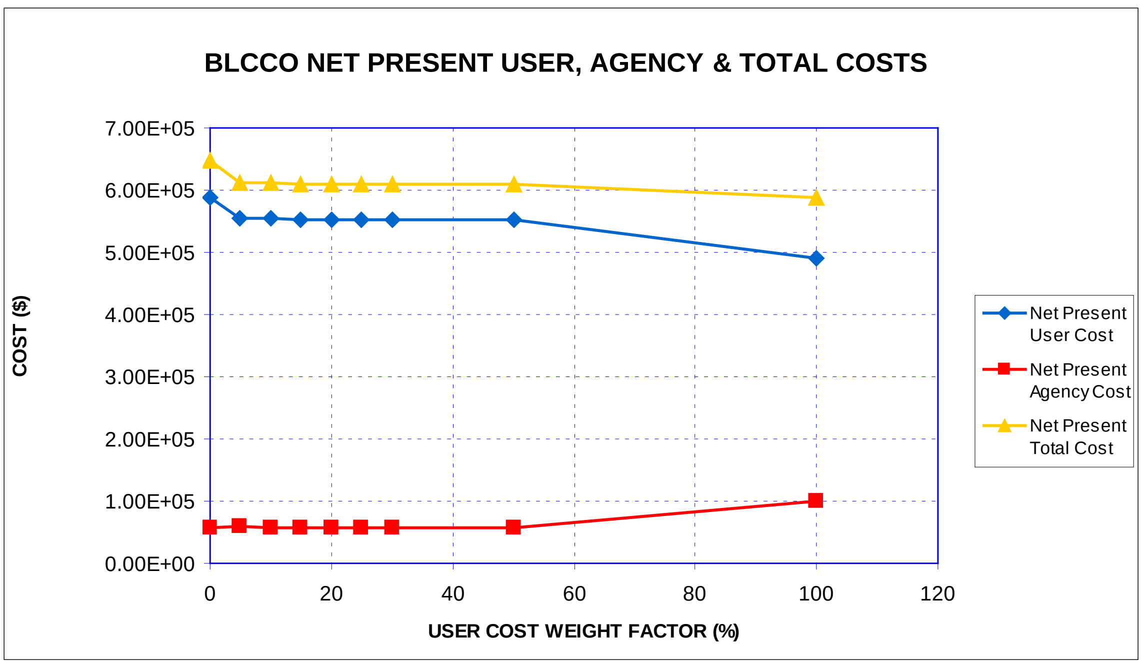

Figure 5.16b: Effect of User Cost Weight on Net Present Cost

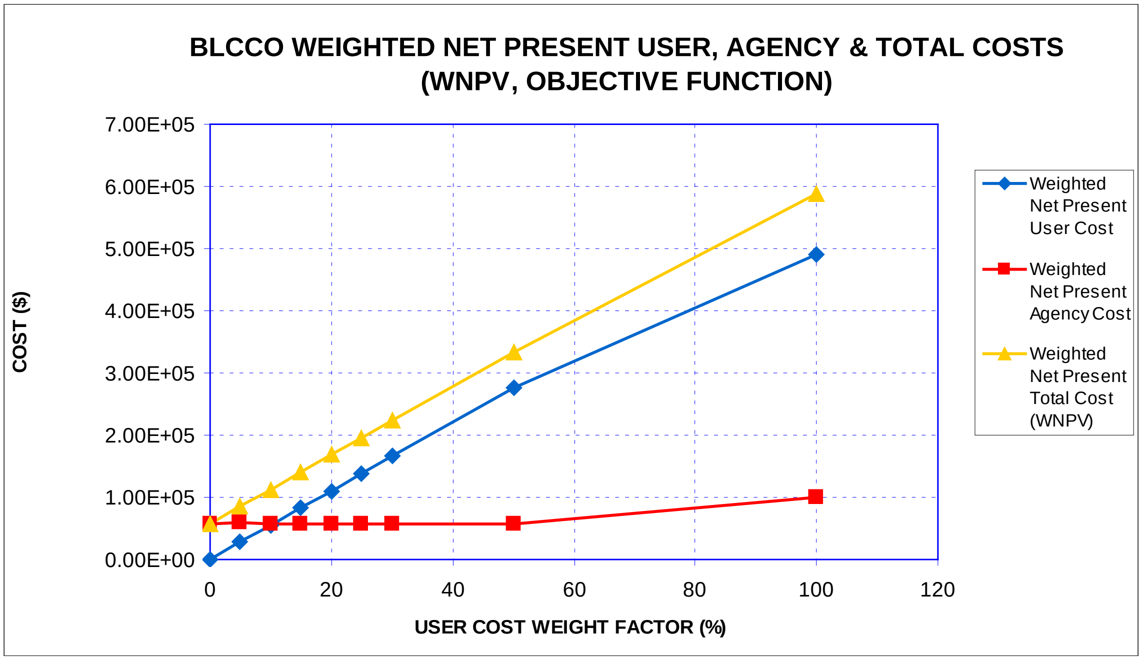

Figure 5.16c: Effect of User Cost Weight on Weighted Net Present Cost

For the user cost weight factor parametric study, the factor is investigated from 0% to 100%. Increasing the weight of the user cost, automatically increases the user cost and this triggers the repair of the bridge, which increases the agency cost as well. As a result, the total cost increases.

For the unexpected events, the probability of having the event is investigated from 0% to 5%. Increasing the probability of unexpected events increases the weighted net present user cost, and this triggers the repair of the bridge, which increases the agency cost as well. As a result, the total cost increases.

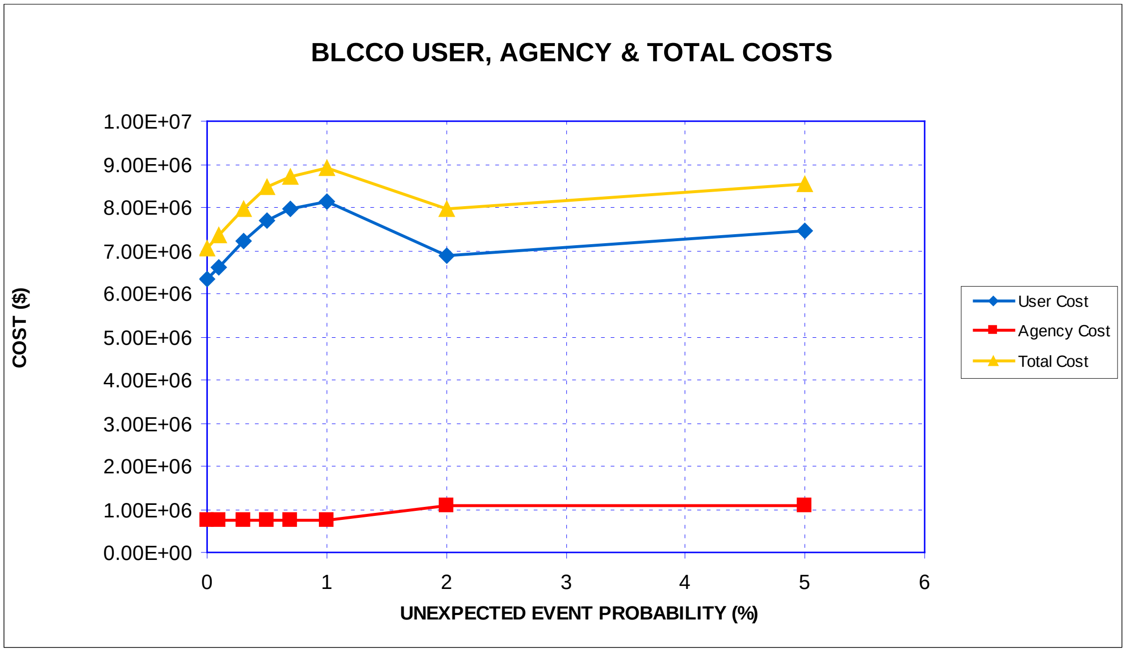

Figure 5.17a: Effect of Occurrence Probability of Unexpected Event on Cost

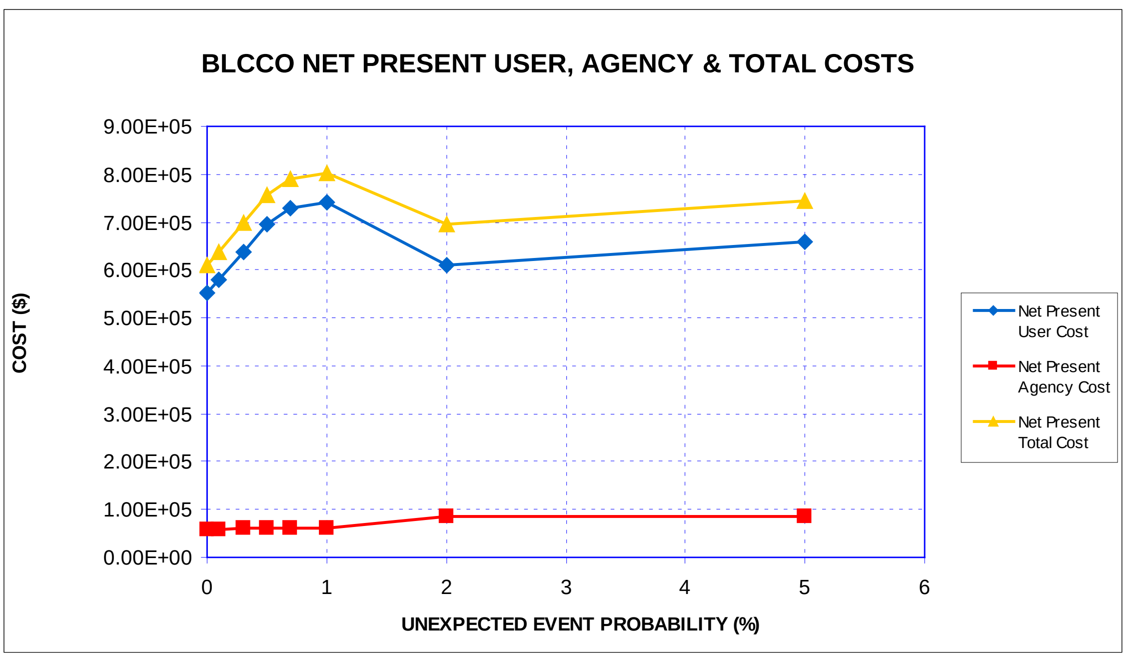

Figure 5.17b: Effect of Occurrence Probability of Unexpected Event on Net Present Cost

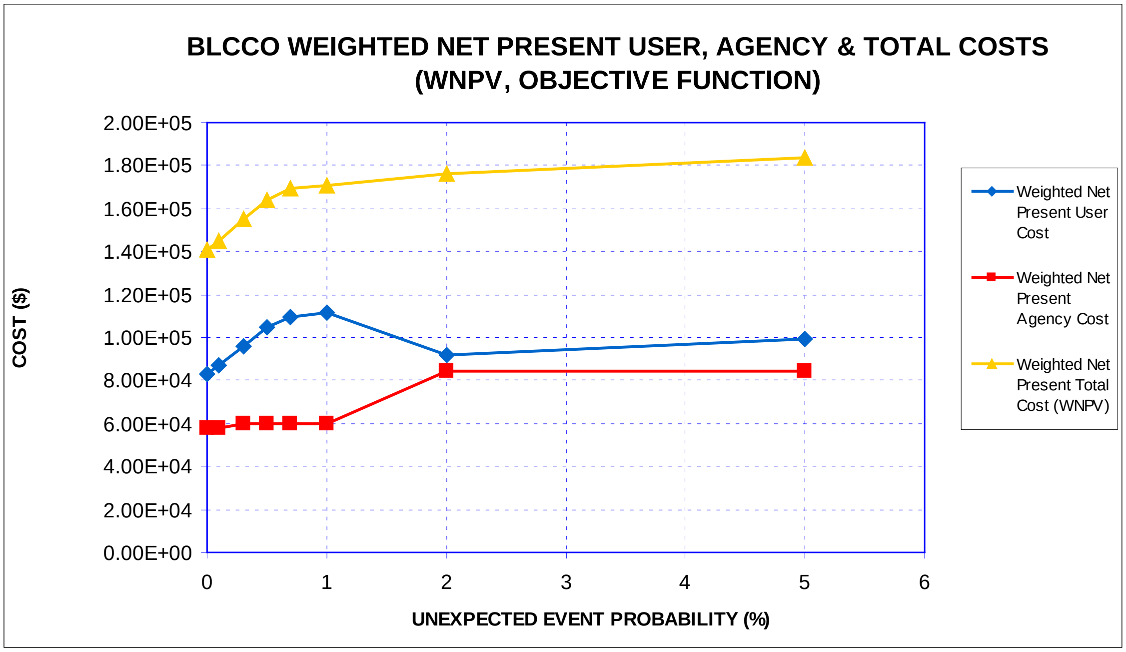

Figure 5.17c: Effect of Occurrence Probability of Unexpected Event on Weighted Net Present Cost

Chapter Six

CONCLUSION

6.1. Research Summary

Given that 25% of the nation's bridges are structurally deficient or functionally obsolete according to the US NBI database's 2011 figures, the US faces a considerable infrastructure problem. Replacing or repairing these bridges requires millions of dollars, and BLCCO algorithms are considered to be invaluable tools for allocating relatively scarce public resources efficiently and in a cost-effective manner.

FHWA promotes Life-Cycle Cost Analysis (LCCA) as an engineering economic analysis tool that allows transportation officials to quantify the differential costs of alternative investment options for a given project. LCCA can be used to study either new construction projects or to examine preservation strategies for existing transportation assets.

In the simplest terms, the time between a bridge's construction and its replacement or removal from service is called its service life. The sequence of actions and events and their outcomes–e.g., construction, usage, aging, damage, repair, renewal– that lead to the end of the service life and the condition of the bridge during its life compose the life cycle. Authorities must make decisions about what management strategy to follow, what materials and designs to use, what repairs to make and when they should be made, based on their expectations about what the subsequent costs and outcomes will be. LCCA is a set of economic principles and computational procedures for comparing initial and future costs to arrive at the most economical strategy for ensuring that a bridge will provide the services for which it was intended. (NCHRP 12-43, 2003)

Considering these, in order to prepare an efficient and powerful BLCCO model, below items are considered in this research:

- Deterioration model of the major bridge parts (superstructure, deck, substructure and wearing surface of the bridge and approaches)
- Rehabilitation, repairs or reconstruction activities for the bridge
- The agency and the user costs for the lifetime of the bridge.

Detailed bridge deterioration curves are prepared for bridge superstructure, deck, substructure and wearing surface of the bridge and approaches. Bridge condition ratings are used as a basis to simulate the bridge parts deterioration where a numeric ranking system from *0* to *9* is used (*0* represents "Failed Condition" and *9* represents "Excellent Condition"). The NBI database is used to gather the condition rating data of the bridges nationwide (603,309 bridges). Markov Chain Matrices are used to perform selective bridge deterioration simulation. First 30 years of the bridges are simulated by Markov Chain using bridge condition ratings and the remaining 45 year of the bridges are simulated by Markov Chain Matrices extrapolating the initial 30 years. The results are verified by comparing the simulated models with the actual condition rating data for individual bridges. A pavement deterioration model, which was used in previous studies, is adapted to complete the whole bridge deterioration system.

Several different parameters are considered for the deterioration curves. These are;

- Material type. Steel (S) or Prestressed (PS) Superstructure).

- Length (L). Bridges less than 20m and bridges longer than 20m.
- Average Daily Traffic (ADT). ADT<10000 and ADT>10000.
- Climatic Regions in USA (Central, East North Central, Northeast, Northwest, South, Southeast, Southwest, West, West North Central).

In view of these parameters, a total of 72 deterioration curves are formed for bridge deck, substructure and superstructure. They are presented in a table format. The deterioration curve database covers bridges nationwide and presented in Appendix A2. Bridge repair Markov Chain Matrices are prepared as well.

Several supplementary improvements for the deterioration model are also presented. They can be improved and be great future research topics.

- A Bayesian Updating methodology is introduced. This methodology facilitates updating the deterioration models when new and reliable bridge deterioration data is available.
- The unexpected events such as seismic, scour, terrorist attacks, hurricane etc. are incorporated into the deterioration model so is in BLCCO model.
- A methodology for obtaining a bridge reliability index from the SI&A data in NBI database is developed.

A BLCCO methodology is developed and explained item by item in detail and with flowcharts. The cost model in the methodology is described, these are:

- Vehicle Operation Cost (PVOC) Model due to Pavement Condition (Prancl,2000)
- Accident Cost Model (PACC) due to Pavement Condition (Prancl,2000)
- Delay Time Cost During Pavement Work Zone Operations (PUWZC) (Jawad,2003)

- User Costs Due To Operating Rating of the Bridge (UORC)
- Vehicle Operation Cost (BVOC) Model due to Bridge Rehabilitation Activity (Roychoudhury,2001)
- Accident Cost (BACC) Model due to Bridge Rehabilitation Activity (Roychoudhury,2001)
- Third Party Cost (BTPC) Model due to Bridge Rehabilitation Activity (Roychoudhury,2001)

The genetic algorithm which is used as an optimization tool is also explained in detail. Finally bridge No. 18G0702 in NJ over Raritan River is used for the study to show how the optimization methodology works. The results are presented to show the advantages of using BLCCO algorithm.

Furthermore, methodology showing how to deal with the uncertainties in the optimization process and forming BLCCO-p is presented. To tackle the uncertainties Monte Carlo simulation is used and applied within the genetic algorithm to reach the optima. The reason for preparing a probabilistic optimization is to change the uncertainty of the deterministic method to risk in stochastic method. In a deterministic method, the analysis parameters must rely on the estimates, which are always uncertain. Therefore, a deterministic approach relies on the outcomes due to the estimations and disregards the potential variability of the estimated parameters. However in stochastic method, uncertain parameters are represented as probability distributions therefore a stochastic BLCCOM computes the life cycle cost as a probabilistic distribution.

The bridge No. 1100070 in NJ over Jacobs Creek is used for the study to explain the optimization methodology. Comparison of the budget allocated for the bridge by

NJDOT is compared with the BLCCO-p results and the schedule and cost superiority of using BLCCO-p algorithm is emphasized.

The uncertain parameters used are:

- Traffic Growth Rate (%)
- Discount Rate (r)
- User Cost Weight (%)
- Super Structure Replacement Cost ($/ft)
- Super Structure Major Repair Cost ($/ft)
- Super Structure Minor Repair Cost ($/ft)
- Passenger Vehicle Operating Cost ($/hr)
- Commercial Vehicle Operating Cost ($/hr)
- Superstructure Deterioration

Sensitivity analyses are carried out in order to systematically examine the effect of the above variables on the objective function which is explained in Chapter 5. This is done by simply by estimating the correlation coefficients between the input parameters and presenting them in a tornado graph. The most sensitive variables are determined and a parametric study is performed. The probability of unexpected events is also included in the parametric study. The results are presented with detailed graphs.

This chapter presents the summary, conclusions and the possible future research.

6.2. Conclusions

In this research a comprehensive probabilistic BLCCO methodology is developed. Bridge data from the National Bridge Inventory Database is used to develop deterioration curves. This methodology takes into consideration all the structural components of the bridge and predicts the agency and user costs of rehabilitating, repairing, and reconstructing the bridges more accurately. Within the framework of this approach, Genetic Algorithm for cost optimization, Markov-Chain for deterioration modeling, and Monte-Carlo Simulation for dealing with uncertainties are used. The effectiveness of the proposed methodology is evaluated by using bridge data obtained from NBI database. Moreover, the results from an in-depth, risk and sensitivity analysis and parametric study are presented.

The main contributions of this research to the state of the art are:

- Improving and perfecting a previously used deterioration model using Markov Chain Matrixes and NBI bridge database that considers several different parameters; material type, length, average daily traffic, climatic regions in the US.
- In view of these parameters, developing 72 deterioration curves for bridge deck, substructure and superstructure. They are presented in an easy-to-read table format.
- Presenting a methodology to update the bridge deterioration curves using Bayesian methodology when more reliable data exists
- Incorporating unexpected events (seismic, scour, terrorist attacks, hurricanes) into the deterioration model of BLCCO.
- Showing a methodology to combine both reliability index and bridge condition index to be used in deterioration models.
- Presenting a BLCCO methodology which reflects the real life conditions as closely as possible.

- Presenting a BLCCO-p methodology which converts, uncertainties into risk and the costs are represented as probabilistic ranges. The probabilistic optimization resulted in a better understanding of the bridge life cycle cost, showing final cost as a range rather than a single value. This probabilistic approach is superior to the deterministic methods as it reflects the real-world cases to a greater extent and gives the agencies a better view for how to manage their resources. The probabilistic approach described in this research makes it easier to get a clear idea about the big picture (i.e. the bridge as a system) regarding its lifetime cost requirements by conducting the BLCCO at relatively macro-levels of detail (i.e. analyzing bridge components separately), consequently producing more realistic and reliable solutions.

6.3. Future Research

Although the research performed in this dissertation is very thorough and detailed, there is still plenty of areas left for the future research endeavors. They can be outlined as shown below:

- Improving the reliability methodology that can be used for the deterioration model in the BLCCA, this study can be further improved by developing the reliability indices for steel superstructure bridges and formulating the reliability based deterioration curves and integrating them into the BLCCO. Developing the reliability indices for all the bridges can be done by preparing charts for the average weight and the capacity of bridges according to their lengths or it can be really easy if this information being included in the NBI database starting with the next cycle of inspections of the bridges. Since a bridge database is readily available, it would be better to use it or improve it rather than trying to focus on individual bridges.

- BLCCO-p can be applied to a group of bridges. This approach is a highly effective optimization algorithm that can be incorporated into a network-level bridge management system for either their life time or a specific time period (i.e. 5 to 20 year budget management of 20 bridges in a network).

BLCCO-p can be applied to different types of structures or infrastructure systems.

APPENDIX A1 – MARKOV CHAIN MATRIX FOR DETERIORATION MODEL

Table A1.1: 2009 Northeast Region Steel Superstructure Bridges Observed Superstructure Condition Rating Distribution for Bridge Ages (75 Transitions)

	BRIDGE CONDITION RATING									
BRIDGE AGE (TRANSITIONS)	**9**	**8**	**7**	**6**	**5**	**4**	**3**	**2**	**1**	**0**
1	0.71	0.25	0.03	0.00	0.01	0.00	0.00	0.00	0.00	0.00
2	0.76	0.20	0.02	0.02	0.00	0.00	0.00	0.00	0.00	0.00
3	0.52	0.38	0.08	0.02	0.00	0.00	0.00	0.00	0.00	0.00
4	0.51	0.40	0.09	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5	0.42	0.52	0.07	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6	0.55	0.36	0.06	0.02	0.01	0.00	0.00	0.00	0.00	0.00
7	0.38	0.46	0.16	0.00	0.00	0.00	0.00	0.00	0.00	0.00
8	0.37	0.50	0.12	0.01	0.00	0.00	0.00	0.00	0.00	0.00
9	0.37	0.39	0.22	0.01	0.01	0.00	0.00	0.00	0.00	0.00
10	0.34	0.51	0.12	0.02	0.01	0.00	0.01	0.00	0.00	0.00
11	0.37	0.43	0.17	0.03	0.00	0.00	0.00	0.00	0.00	0.00
12	0.31	0.46	0.16	0.04	0.03	0.00	0.00	0.00	0.00	0.00
13	0.25	0.44	0.23	0.03	0.04	0.00	0.00	0.00	0.00	0.00
14	0.18	0.41	0.34	0.05	0.01	0.00	0.01	0.00	0.00	0.01
15	0.12	0.53	0.28	0.05	0.01	0.00	0.00	0.00	0.00	0.00
16	0.17	0.48	0.31	0.02	0.02	0.00	0.00	0.00	0.00	0.00
17	0.12	0.41	0.40	0.05	0.01	0.01	0.00	0.00	0.00	0.00
18	0.13	0.45	0.34	0.06	0.01	0.01	0.00	0.00	0.00	0.00
19	0.09	0.42	0.38	0.06	0.04	0.00	0.00	0.00	0.00	0.00
20	0.09	0.43	0.40	0.05	0.02	0.00	0.00	0.00	0.00	0.00
21	0.11	0.36	0.38	0.13	0.02	0.01	0.00	0.00	0.00	0.00
22	0.07	0.36	0.45	0.06	0.05	0.02	0.00	0.00	0.00	0.00
23	0.06	0.23	0.51	0.15	0.04	0.01	0.00	0.00	0.00	0.00
24	0.03	0.32	0.44	0.10	0.10	0.01	0.00	0.00	0.00	0.00
25	0.03	0.31	0.39	0.17	0.08	0.02	0.00	0.00	0.00	0.01
26	0.05	0.30	0.35	0.18	0.09	0.00	0.00	0.00	0.00	0.03
27	0.03	0.33	0.36	0.19	0.08	0.01	0.00	0.00	0.00	0.00
28	0.03	0.34	0.38	0.19	0.05	0.01	0.00	0.00	0.00	0.00
29	0.06	0.28	0.40	0.17	0.07	0.02	0.00	0.00	0.00	0.00
30	0.05	0.28	0.42	0.12	0.10	0.02	0.00	0.00	0.00	0.00
31	0.00	0.20	0.50	0.18	0.09	0.01	0.02	0.00	0.00	0.00
32	0.02	0.22	0.39	0.27	0.07	0.02	0.01	0.00	0.00	0.01
33	0.00	0.20	0.43	0.23	0.08	0.04	0.01	0.00	0.00	0.00
34	0.01	0.16	0.39	0.27	0.13	0.02	0.02	0.00	0.00	0.00
35	0.03	0.16	0.44	0.27	0.07	0.02	0.01	0.00	0.00	0.00
36	0.03	0.17	0.43	0.26	0.08	0.02	0.00	0.00	0.00	0.00
37	0.01	0.19	0.35	0.24	0.16	0.03	0.01	0.00	0.00	0.00
38	0.03	0.24	0.39	0.20	0.11	0.03	0.00	0.00	0.00	0.00
39	0.02	0.11	0.41	0.32	0.10	0.03	0.00	0.00	0.00	0.00
40	0.04	0.16	0.36	0.27	0.12	0.04	0.00	0.00	0.00	0.00

	41	0.02	0.19	0.43	0.24	0.09	0.02	0.00	0.00	0.00	0.00
	42	0.01	0.15	0.36	0.29	0.16	0.03	0.00	0.00	0.00	0.00
	43	0.00	0.10	0.38	0.33	0.14	0.04	0.01	0.00	0.00	0.00
	44	0.01	0.11	0.34	0.32	0.16	0.06	0.00	0.00	0.00	0.00
	45	0.01	0.10	0.35	0.32	0.17	0.04	0.01	0.00	0.00	0.00
	46	0.02	0.10	0.34	0.30	0.17	0.05	0.02	0.00	0.00	0.00
	47	0.01	0.09	0.34	0.33	0.20	0.03	0.00	0.00	0.00	0.00
	48	0.01	0.09	0.40	0.31	0.14	0.04	0.00	0.00	0.00	0.00
	49	0.02	0.12	0.34	0.28	0.18	0.05	0.01	0.00	0.00	0.00
	50	0.02	0.09	0.33	0.30	0.20	0.05	0.02	0.00	0.00	0.00
	51	0.02	0.05	0.32	0.38	0.17	0.04	0.02	0.00	0.00	0.00
	52	0.02	0.07	0.42	0.30	0.17	0.02	0.01	0.00	0.00	0.00
	53	0.02	0.10	0.32	0.31	0.17	0.06	0.01	0.00	0.00	0.00
	54	0.03	0.06	0.26	0.37	0.20	0.06	0.01	0.00	0.00	0.00
	55	0.01	0.04	0.28	0.36	0.23	0.06	0.01	0.00	0.00	0.00
	56	0.02	0.05	0.26	0.40	0.21	0.06	0.01	0.00	0.00	0.00
	57	0.01	0.07	0.23	0.29	0.33	0.07	0.01	0.00	0.00	0.00
	58	0.01	0.05	0.25	0.32	0.26	0.10	0.01	0.00	0.00	0.00
	59	0.01	0.05	0.27	0.35	0.22	0.08	0.02	0.00	0.00	0.00
	60	0.02	0.06	0.25	0.29	0.23	0.13	0.00	0.00	0.00	0.00
	61	0.01	0.07	0.25	0.27	0.28	0.08	0.04	0.00	0.00	0.00
	62	0.04	0.07	0.19	0.27	0.27	0.14	0.02	0.00	0.00	0.00
	63	0.00	0.05	0.15	0.32	0.35	0.09	0.05	0.00	0.00	0.00
	64	0.02	0.03	0.15	0.26	0.31	0.12	0.09	0.02	0.00	0.00
	65	0.04	0.04	0.12	0.27	0.35	0.19	0.00	0.00	0.00	0.00
	66	0.02	0.07	0.12	0.22	0.12	0.29	0.12	0.02	0.00	0.00
	67	0.06	0.02	0.18	0.32	0.22	0.15	0.05	0.01	0.00	0.00
	68	0.03	0.05	0.15	0.23	0.36	0.14	0.03	0.00	0.00	0.00
	69	0.01	0.05	0.11	0.28	0.33	0.16	0.05	0.01	0.00	0.00
	70	0.01	0.05	0.20	0.31	0.27	0.13	0.02	0.00	0.00	0.00
	71	0.01	0.05	0.14	0.29	0.28	0.19	0.03	0.00	0.00	0.00
	72	0.02	0.05	0.13	0.23	0.33	0.19	0.05	0.00	0.00	0.00
	73	0.03	0.03	0.17	0.31	0.29	0.13	0.04	0.01	0.00	0.00
	74	0.02	0.05	0.12	0.31	0.30	0.16	0.03	0.01	0.00	0.00
	75	0.00	0.03	0.12	0.30	0.28	0.20	0.07	0.00	0.00	0.01

Table A1.2: Markov Chain Matrixes (Only 12 Transitions are Shown)

1

0.8834	0.1166	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
0.0000	0.9331	0.0669	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
0.0000	0.0000	0.9619	0.0381	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
0.0000	0.0000	0.0000	0.8396	0.1604	0.0000	0.0000	0.0000	0.0000	0.0000
0.0000	0.0000	0.0000	0.0000	0.7448	0.2552	0.0000	0.0000	0.0000	0.0000
0.0000	0.0000	0.0000	0.0000	0.0000	0.3989	0.6011	0.0000	0.0000	0.0000
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.3403	0.6597	0.0000	0.0000
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.2790	0.7210	0.0000
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.2167	0.7833
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	1.0000

2

0.7804	0.2118	0.0078	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
0.0000	0.8707	0.1268	0.0025	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
0.0000	0.0000	0.9253	0.0685	0.0061	0.0000	0.0000	0.0000	0.0000	0.0000
0.0000	0.0000	0.0000	0.7049	0.2542	0.0409	0.0000	0.0000	0.0000	0.0000
0.0000	0.0000	0.0000	0.0000	0.5548	0.2918	0.1534	0.0000	0.0000	0.0000
0.0000	0.0000	0.0000	0.0000	0.0000	0.1591	0.4443	0.3965	0.0000	0.0000
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.1158	0.4085	0.4756	0.0000
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0778	0.3574	0.5648
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0469	0.9531
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	1.0000

3

0.6895	0.2886	0.0217	0.0003	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
0.0000	0.8125	0.1802	0.0070	0.0004	0.0000	0.0000	0.0000	0.0000	0.0000
0.0000	0.0000	0.8901	0.0928	0.0155	0.0016	0.0000	0.0000	0.0000	0.0000
0.0000	0.0000	0.0000	0.5918	0.3024	0.0812	0.0246	0.0000	0.0000	0.0000
0.0000	0.0000	0.0000	0.0000	0.4132	0.2580	0.2276	0.1012	0.0000	0.0000
0.0000	0.0000	0.0000	0.0000	0.0000	0.0635	0.2469	0.4038	0.2859	0.0000
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0394	0.1904	0.3976	0.3726
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0217	0.1335	0.8447
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0102	0.9898
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	1.0000

4

0.6091	0.3496	0.0401	0.0011	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
0.0000	0.7581	0.2277	0.0127	0.0014	0.0001	0.0000	0.0000	0.0000	0.0000
0.0000	0.0000	0.8563	0.1118	0.0265	0.0046	0.0009	0.0000	0.0000	0.0000
0.0000	0.0000	0.0000	0.4969	0.3202	0.1095	0.0572	0.0162	0.0000	0.0000
0.0000	0.0000	0.0000	0.0000	0.3078	0.2083	0.2325	0.1784	0.0730	0.0000
0.0000	0.0000	0.0000	0.0000	0.0000	0.0253	0.1222	0.2755	0.3531	0.2240
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0134	0.0791	0.2234	0.6841
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0061	0.0446	0.9494
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0022	0.9978
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	1.0000

5

0.5381	0.3973	0.0620	0.0024	0.0002	0.0000	0.0000	0.0000	0.0000	0.0000
0.0000	0.7074	0.2697	0.0193	0.0031	0.0004	0.0001	0.0000	0.0000	0.0000
0.0000	0.0000	0.8237	0.1264	0.0376	0.0086	0.0031	0.0006	0.0000	0.0000
0.0000	0.0000	0.0000	0.4172	0.3182	0.1254	0.0853	0.0422	0.0117	0.0000
0.0000	0.0000	0.0000	0.0000	0.2293	0.1616	0.2044	0.2032	0.1444	0.0571
0.0000	0.0000	0.0000	0.0000	0.0000	0.0101	0.0568	0.1574	0.2751	0.5005
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0046	0.0309	0.1054	0.8591
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0017	0.0140	0.9843
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0005	0.9995
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	1.0000

6

0.4754	0.4334	0.0862	0.0044	0.0005	0.0001	0.0000	0.0000	0.0000	0.0000
0.0000	0.6601	0.3068	0.0265	0.0054	0.0010	0.0003	0.0000	0.0000	0.0000
0.0000	0.0000	0.7923	0.1375	0.0483	0.0130	0.0062	0.0022	0.0004	0.0000
0.0000	0.0000	0.0000	0.3503	0.3039	0.1312	0.1044	0.0681	0.0330	0.0092
0.0000	0.0000	0.0000	0.0000	0.1708	0.1230	0.1667	0.1915	0.1778	0.1703
0.0000	0.0000	0.0000	0.0000	0.0000	0.0040	0.0254	0.0814	0.1731	0.7161
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0016	0.0116	0.0451	0.9417
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0005	0.0043	0.9953
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0001	0.9999
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	1.0000

7

0.4199	0.4598	0.1119	0.0070	0.0011	0.0002	0.0000	0.0000	0.0000	0.0000
0.0000	0.6159	0.3392	0.0339	0.0083	0.0018	0.0007	0.0002	0.0000	0.0000
0.0000	0.0000	0.7622	0.1456	0.0580	0.0175	0.0099	0.0047	0.0017	0.0003
0.0000	0.0000	0.0000	0.2941	0.2826	0.1299	0.1144	0.0879	0.0562	0.0350
0.0000	0.0000	0.0000	0.0000	0.1272	0.0926	0.1307	0.1634	0.1766	0.3095
0.0000	0.0000	0.0000	0.0000	0.0000	0.0016	0.0111	0.0395	0.0962	0.8517
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0005	0.0043	0.0182	0.9770
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0001	0.0013	0.9986
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	1.0000
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	1.0000

8

0.3710	0.4780	0.1384	0.0101	0.0019	0.0003	0.0001	0.0000	0.0000	0.0000
0.0000	0.5747	0.3675	0.0414	0.0116	0.0028	0.0013	0.0005	0.0001	0.0000
0.0000	0.0000	0.7332	0.1512	0.0666	0.0218	0.0139	0.0079	0.0038	0.0017
0.0000	0.0000	0.0000	0.2469	0.2576	0.1239	0.1170	0.1000	0.0755	0.0790
0.0000	0.0000	0.0000	0.0000	0.0947	0.0694	0.1001	0.1318	0.1561	0.4479
0.0000	0.0000	0.0000	0.0000	0.0000	0.0006	0.0047	0.0183	0.0493	0.9270
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0002	0.0015	0.0070	0.9913
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0004	0.9996
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	1.0000
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	1.0000

9

0.3277	0.4893	0.1651	0.0138	0.0031	0.0006	0.0002	0.0001	0.0000	0.0000
0.0000	0.5363	0.3920	0.0487	0.0153	0.0041	0.0021	0.0010	0.0004	0.0001
0.0000	0.0000	0.7053	0.1549	0.0738	0.0257	0.0178	0.0114	0.0065	0.0046
0.0000	0.0000	0.0000	0.2073	0.2315	0.1152	0.1143	0.1051	0.0884	0.1382
0.0000	0.0000	0.0000	0.0000	0.0706	0.0519	0.0758	0.1028	0.1288	0.5701
0.0000	0.0000	0.0000	0.0000	0.0000	0.0003	0.0020	0.0082	0.0239	0.9656
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0001	0.0005	0.0026	0.9968
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0001	0.9999
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	1.0000
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	1.0000

10

0.2895	0.4948	0.1916	0.0178	0.0045	0.0010	0.0005	0.0002	0.0001	0.0000
0.0000	0.5004	0.4130	0.0558	0.0192	0.0055	0.0032	0.0017	0.0008	0.0004
0.0000	0.0000	0.6785	0.1569	0.0798	0.0291	0.0215	0.0149	0.0096	0.0097
0.0000	0.0000	0.0000	0.1740	0.2057	0.1050	0.1081	0.1047	0.0949	0.2075
0.0000	0.0000	0.0000	0.0000	0.0526	0.0387	0.0570	0.0787	0.1021	0.6710
0.0000	0.0000	0.0000	0.0000	0.0000	0.0001	0.0008	0.0036	0.0111	0.9843
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0002	0.0010	0.9988
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	1.0000
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	1.0000
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	1.0000

11

0.2558	0.4954	0.2174	0.0223	0.0062	0.0016	0.0008	0.0004	0.0001	0.0001
0.0000	0.4669	0.4307	0.0626	0.0233	0.0071	0.0044	0.0026	0.0014	0.0011
0.0000	0.0000	0.6526	0.1575	0.0846	0.0320	0.0248	0.0184	0.0129	0.0172
0.0000	0.0000	0.0000	0.1461	0.1811	0.0944	0.0999	0.1005	0.0961	0.2819
0.0000	0.0000	0.0000	0.0000	0.0391	0.0288	0.0426	0.0595	0.0788	0.7510
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0003	0.0016	0.0050	0.9930
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0001	0.0003	0.9996
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	1.0000
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	1.0000
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	1.0000

12

0.2260	0.4921	0.2423	0.0270	0.0082	0.0022	0.0012	0.0006	0.0003	0.0002
0.0000	0.4357	0.4456	0.0689	0.0274	0.0088	0.0058	0.0036	0.0021	0.0021
0.0000	0.0000	0.6278	0.1571	0.0883	0.0343	0.0277	0.0215	0.0160	0.0273
0.0000	0.0000	0.0000	0.1227	0.1584	0.0839	0.0907	0.0940	0.0933	0.3571
0.0000	0.0000	0.0000	0.0000	0.0292	0.0215	0.0319	0.0447	0.0600	0.8127
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0001	0.0007	0.0022	0.9970
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0001	0.9998
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	1.0000
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	1.0000
0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	1.0000

Table A1.3: 2009 Northeast Region Steel Superstructure Bridges Markov Chain Simulated Superstructure Condition Rating Distribution for Bridge Ages (75 Transitions) (Simulation is done for the initial 30 transitions and it is extrapolated to 75 transitions)

		BRIDGE CONDITION RATING									
		9	**8**	**7**	**6**	**5**	**4**	**3**	**2**	**1**	**0**
BRIDGE AGE (TRANSITIONS)	**1**	0.88	0.12	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	2	0.78	0.21	0.01	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	3	0.69	0.29	0.02	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	4	0.61	0.35	0.04	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	5	0.54	0.40	0.06	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	6	0.48	0.43	0.09	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	7	0.42	0.46	0.11	0.01	0.00	0.00	0.00	0.00	0.00	0.00
	8	0.37	0.48	0.14	0.01	0.00	0.00	0.00	0.00	0.00	0.00
	9	0.33	0.49	0.17	0.01	0.00	0.00	0.00	0.00	0.00	0.00
	10	0.29	0.49	0.19	0.02	0.00	0.00	0.00	0.00	0.00	0.00
	11	0.26	0.50	0.22	0.02	0.01	0.00	0.00	0.00	0.00	0.00
	12	0.23	0.49	0.24	0.03	0.01	0.00	0.00	0.00	0.00	0.00
	13	0.20	0.49	0.27	0.03	0.01	0.00	0.00	0.00	0.00	0.00
	14	0.18	0.48	0.29	0.04	0.01	0.00	0.00	0.00	0.00	0.00
	15	0.16	0.47	0.31	0.04	0.02	0.00	0.00	0.00	0.00	0.00
	16	0.14	0.45	0.33	0.05	0.02	0.01	0.00	0.00	0.00	0.00
	17	0.12	0.44	0.35	0.05	0.02	0.01	0.00	0.00	0.00	0.00
	18	0.11	0.42	0.36	0.06	0.02	0.01	0.01	0.00	0.00	0.01
	19	0.09	0.41	0.38	0.06	0.03	0.01	0.01	0.01	0.00	0.01
	20	0.08	0.39	0.39	0.07	0.03	0.01	0.01	0.01	0.00	0.01
	21	0.07	0.37	0.40	0.07	0.03	0.01	0.01	0.01	0.01	0.01
	22	0.07	0.36	0.41	0.07	0.04	0.01	0.01	0.01	0.01	0.02
	23	0.06	0.34	0.42	0.08	0.04	0.01	0.01	0.01	0.01	0.02
	24	0.05	0.33	0.43	0.08	0.04	0.02	0.01	0.01	0.01	0.03
	25	0.05	0.31	0.43	0.08	0.04	0.02	0.01	0.01	0.01	0.03
	26	0.04	0.29	0.44	0.09	0.05	0.02	0.01	0.01	0.01	0.04
	27	0.04	0.28	0.44	0.09	0.05	0.02	0.02	0.01	0.01	0.05
	28	0.03	0.26	0.44	0.09	0.05	0.02	0.02	0.01	0.01	0.06
	29	0.03	0.25	0.44	0.09	0.05	0.02	0.02	0.01	0.01	0.07
	30	0.02	0.24	0.44	0.10	0.05	0.02	0.02	0.02	0.01	0.08
	31	0.02	0.22	0.44	0.10	0.06	0.02	0.02	0.02	0.01	0.09
	32	0.02	0.21	0.44	0.10	0.06	0.02	0.02	0.02	0.02	0.10
	33	0.02	0.20	0.44	0.10	0.06	0.02	0.02	0.02	0.02	0.11
	34	0.01	0.19	0.43	0.10	0.06	0.02	0.02	0.02	0.02	0.12
	35	0.01	0.18	0.43	0.10	0.06	0.02	0.02	0.02	0.02	0.14
	36	0.01	0.17	0.42	0.10	0.06	0.03	0.02	0.02	0.02	0.15
	37	0.01	0.16	0.42	0.10	0.06	0.03	0.02	0.02	0.02	0.16
	38	0.01	0.15	0.41	0.10	0.06	0.03	0.02	0.02	0.02	0.18
	39	0.01	0.14	0.41	0.10	0.06	0.03	0.02	0.02	0.02	0.19
	40	0.01	0.13	0.40	0.10	0.06	0.03	0.02	0.02	0.02	0.21

	41	0.01	0.12	0.40	0.10	0.06	0.03	0.02	0.02	0.02	0.22
	42	0.01	0.12	0.39	0.10	0.06	0.03	0.02	0.02	0.02	0.24
	43	0.00	0.11	0.38	0.10	0.06	0.03	0.02	0.02	0.02	0.25
	44	0.00	0.10	0.37	0.10	0.06	0.03	0.02	0.02	0.02	0.27
	45	0.00	0.10	0.37	0.09	0.06	0.03	0.02	0.02	0.02	0.28
	46	0.00	0.09	0.36	0.09	0.06	0.03	0.02	0.02	0.02	0.30
	47	0.00	0.08	0.35	0.09	0.06	0.03	0.02	0.02	0.02	0.32
	48	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00
	49	0.00	0.07	0.34	0.09	0.06	0.03	0.02	0.02	0.02	0.35
	50	0.00	0.07	0.33	0.09	0.06	0.03	0.02	0.02	0.02	0.36
	51	0.00	0.06	0.32	0.09	0.06	0.03	0.02	0.02	0.02	0.38
	52	0.00	0.06	0.31	0.08	0.06	0.02	0.02	0.02	0.02	0.40
	53	0.00	0.06	0.30	0.08	0.06	0.02	0.02	0.02	0.02	0.41
	54	0.00	0.05	0.30	0.08	0.05	0.02	0.02	0.02	0.02	0.43
	55	0.00	0.05	0.29	0.08	0.05	0.02	0.02	0.02	0.02	0.44
	56	0.00	0.05	0.28	0.08	0.05	0.02	0.02	0.02	0.02	0.46
	57	0.00	0.04	0.27	0.08	0.05	0.02	0.02	0.02	0.02	0.47
	58	0.00	0.04	0.27	0.07	0.05	0.02	0.02	0.02	0.02	0.49
	59	0.00	0.04	0.26	0.07	0.05	0.02	0.02	0.02	0.02	0.50
	60	0.00	0.04	0.25	0.07	0.05	0.02	0.02	0.02	0.02	0.51
	61	0.00	0.03	0.24	0.07	0.05	0.02	0.02	0.02	0.02	0.53
	62	0.00	0.03	0.24	0.07	0.05	0.02	0.02	0.02	0.02	0.54
	63	0.00	0.03	0.23	0.07	0.05	0.02	0.02	0.02	0.02	0.56
	64	0.00	0.03	0.22	0.06	0.04	0.02	0.02	0.02	0.02	0.57
	65	0.00	0.03	0.22	0.06	0.04	0.02	0.02	0.02	0.02	0.58
	66	0.00	0.02	0.21	0.06	0.04	0.02	0.02	0.02	0.02	0.59
	67	0.00	0.02	0.20	0.06	0.04	0.02	0.02	0.02	0.02	0.61
	68	0.00	0.02	0.20	0.06	0.04	0.02	0.02	0.02	0.02	0.62
	69	0.00	0.02	0.19	0.06	0.04	0.02	0.02	0.02	0.01	0.63
	70	0.00	0.02	0.19	0.05	0.04	0.02	0.02	0.02	0.01	0.64
	71	0.00	0.02	0.18	0.05	0.04	0.02	0.02	0.01	0.01	0.65
	72	0.00	0.02	0.17	0.05	0.04	0.02	0.02	0.01	0.01	0.66
	73	0.00	0.01	0.17	0.05	0.03	0.02	0.01	0.01	0.01	0.68
	74	0.00	0.01	0.16	0.05	0.03	0.02	0.01	0.01	0.01	0.69
	75	0.00	0.01	0.16	0.05	0.03	0.01	0.01	0.01	0.01	0.70

APPENDIX A2 – NATIONWIDE BRIDGE DECK, SUBSTRUCTURE & SUPERSTRUCTURE DETERIORATION MARKOV VECTORS

Table A2.1: Nationwide Bridge Deck Deterioration Markov Vectors

Region	Material	Length	ADT	P(9,9)	P(8,8)	P(7,7)	P(6,6)	P(5,5)	P(4,4)	P(3,3)	P(2,2)	P(1,1)	P(0,0)
Northeast	PS	>20	>10000	0.6977	0.8206	0.9505	0.9611	0.9199	0.7312	0.4785	0.4276	0.3603	1.0000
	PS	>20	<10000	0.7272	0.8556	0.9511	0.9492	0.9541	0.7782	0.6453	0.5048	0.4241	1.0000
	PS	<20	>10000	0.5843	0.8437	0.9516	0.9642	0.9312	0.7975	0.6282	0.4829	0.3941	1.0000
	PS	<20	<10000	0.7999	0.8722	0.9567	0.9480	0.9652	0.7035	0.6156	0.4663	0.3365	1.0000
	S	>20	>10000	0.8033	0.8503	0.9455	0.9439	0.9475	0.8305	0.7373	0.6377	0.5608	1.0000
	S	>20	<10000	0.8352	0.8719	0.9462	0.9427	0.9367	0.8801	0.8024	0.6820	0.6248	1.0000
	S	<20	>10000	0.7825	0.8262	0.9461	0.9452	0.9532	0.8829	0.6515	0.5240	0.3127	1.0000
	S	<20	<10000	0.8079	0.8592	0.9340	0.9464	0.9523	0.9318	0.8683	0.8445	0.8023	1.0000
Southeast	PS	>20	>10000	0.5390	0.8546	0.9801	0.9378	0.9182	0.6802	0.5989	0.5524	0.3146	1.0000
	PS	>20	<10000	0.6432	0.9125	0.9833	0.9749	0.6140	0.5238	0.4610	0.4050	0.3204	1.0000
	PS	<20	>10000	0.4594	0.8265	0.9808	0.9465	0.8595	0.6949	0.5740	0.4956	0.2582	1.0000
	PS	<20	<10000	0.5889	0.9360	0.9828	0.9707	0.9649	0.5652	0.4878	0.4320	0.3271	1.0000
	S	>20	>10000	0.6472	0.8267	0.9657	0.9700	0.9794	0.9147	0.6437	0.5968	0.5785	1.0000
	S	>20	<10000	0.6392	0.8565	0.9674	0.9692	0.8678	0.7023	0.5510	0.5023	0.3187	1.0000
	S	<20	>10000	0.6823	0.8016	0.9639	0.9671	0.9406	0.8649	0.6625	0.5773	0.3848	1.0000
	S	<20	<10000	0.5625	0.9184	0.9555	0.9525	0.9475	0.8651	0.7094	0.7292	0.8395	1.0000
Central	PS	>20	>10000	0.7616	0.8690	0.9664	0.9511	0.9137	0.7689	0.5380	0.4860	0.3471	1.0000
	PS	>20	<10000	0.8118	0.9212	0.9680	0.9493	0.7832	0.8130	0.6080	0.5246	0.4544	1.0000
	PS	<20	>10000	0.7179	0.8766	0.9631	0.9319	0.8790	0.7630	0.6031	0.5136	0.4332	1.0000
	PS	<20	<10000	0.8269	0.9381	0.9617	0.9471	0.8257	0.6412	0.5336	0.4722	0.4206	1.0000
	S	>20	>10000	0.7356	0.7955	0.9404	0.9537	0.8838	0.8411	0.7995	0.5622	0.4928	1.0000
	S	>20	<10000	0.8205	0.8819	0.9443	0.9524	0.8908	0.8343	0.6986	0.7092	0.5570	1.0000
	S	<20	>10000	0.7563	0.8234	0.9407	0.9402	0.9344	0.9482	0.7658	0.5398	0.4312	1.0000
	S	<20	<10000	0.8394	0.9019	0.9310	0.9497	0.9543	0.9288	0.6006	0.4834	0.4317	1.0000

Table A2.1 (continued): Nationwide Bridge Deck Deterioration Markov Vectors

Region	Material	Length	ADT	P(9,9)	P(8,8)	P(7,7)	P(6,6)	P(5,5)	P(4,4)	P(3,3)	P(2,2)	P(1,1)	P(0,0)
East North Central	PS	>20	>10000	0.7243	0.8478	0.9753	0.8210	0.3413	0.3052	0.2384	0.1589	0.0754	1.0000
	PS	>20	<10000	0.7983	0.9284	0.9739	0.9451	0.8802	0.6687	0.5067	0.4532	0.3593	1.0000
	PS	<20	>10000	0.7079	0.9034	0.9505	0.9512	0.9441	0.6729	0.6271	0.6908	0.6380	1.0000
	PS	<20	<10000	0.8191	0.9404	0.9674	0.9614	0.9632	0.5797	0.5066	0.4408	0.3785	1.0000
	S	>20	>10000	0.7000	0.8542	0.9442	0.9644	0.9575	0.9101	0.8514	0.6311	0.4954	1.0000
	S	>20	<10000	0.7420	0.8709	0.9525	0.9554	0.9527	0.8877	0.7805	0.6255	0.4577	1.0000
	S	<20	>10000	0.7000	0.8780	0.9388	0.8836	0.9549	0.9788	0.5494	0.4531	0.3392	1.0000
	S	<20	<10000	0.7889	0.8879	0.9346	0.9405	0.9530	0.8906	0.8311	0.7143	0.6723	1.0000
South	PS	>20	>10000	0.2129	0.8725	0.9784	0.9104	0.6574	0.7000	0.6738	0.6032	0.2880	1.0000
	PS	>20	<10000	0.5924	0.9207	0.9794	0.9056	0.6190	0.6589	0.6534	0.6179	0.5291	1.0000
	PS	<20	>10000	0.3870	0.8882	0.9705	0.9472	0.7278	0.6097	0.5525	0.4810	0.4108	1.0000
	PS	<20	<10000	0.6296	0.9417	0.9741	0.9277	0.7810	0.5677	0.5148	0.4541	0.3098	1.0000
	S	>20	>10000	0.4038	0.8741	0.9736	0.9661	0.8365	0.6211	0.6920	0.5792	0.2945	1.0000
	S	>20	<10000	0.5243	0.9014	0.9692	0.9250	0.8652	0.7300	0.5709	0.5134	0.3280	1.0000
	S	<20	>10000	0.5026	0.8440	0.9782	0.9471	0.9356	0.5894	0.4936	0.4207	0.2767	1.0000
	S	<20	<10000	0.3138	0.8540	0.9617	0.9544	0.9401	0.8515	0.7807	0.7056	0.5468	1.0000
West North Central	PS	>20	>10000	0.8502	0.8720	0.9426	0.9382	0.9137	0.5307	0.6526	0.6328	0.5957	1.0000
	PS	>20	<10000	0.8773	0.9273	0.9727	0.9435	0.8797	0.5458	0.4998	0.4251	0.3251	1.0000
	PS	<20	>10000	0.7106	0.8842	0.9519	0.9139	0.8156	0.7344	0.6152	0.6656	0.7543	1.0000
	PS	<20	<10000	0.8768	0.9003	0.9587	0.9138	0.9311	0.6342	0.5717	0.5072	0.4534	1.0000
	S	>20	>10000	0.8343	0.9319	0.8791	0.9389	0.9334	0.8027	0.6627	1.0000	0.6795	1.0000
	S	>20	<10000	0.8952	0.9072	0.9234	0.9404	0.9412	0.8349	0.7784	0.8146	0.6243	1.0000
	S	<20	>10000	0.7883	0.8937	0.8055	0.8600	0.9621	0.1297	0.3221	0.9343	0.6036	1.0000
	S	<20	<10000	0.9060	0.8994	0.9145	0.8749	0.9666	0.7879	0.8190	0.6714	0.4955	1.0000

Table A2.1 (continued): Nationwide Bridge Deck Deterioration Markov Vectors

Region	Material	Length	ADT	P(9,9)	P(8,8)	P(7,7)	P(6,6)	P(5,5)	P(4,4)	P(3,3)	P(2,2)	P(1,1)	P(0,0)
Southwest	PS	>20	>10000	0.0006	0.8607	0.9752	0.9710	0.7201	0.5688	0.4961	0.4148	0.2841	1.0000
	PS	>20	<10000	0.0736	0.8855	0.9790	0.9654	0.8590	0.5989	0.8124	0.5606	0.2604	1.0000
	PS	<20	>10000	0.5903	0.7500	0.9679	0.9702	0.7631	0.6410	0.6830	0.5926	0.4672	1.0000
	PS	<20	<10000	0.2666	0.8907	0.9777	0.9711	0.9091	0.7167	0.5920	0.5042	0.3044	1.0000
	S	>20	>10000	0.4185	0.7515	0.9667	0.9351	0.9671	0.9247	0.5922	0.4845	0.3411	1.0000
	S	>20	<10000	0.2885	0.8409	0.9695	0.9674	0.9026	0.8184	0.8948	0.5457	0.4338	1.0000
	S	<20	>10000	0.0000	0.3664	0.9591	0.9682	0.9123	0.5587	0.6431	0.7337	0.4362	1.0000
	S	<20	<10000	0.2671	0.8628	0.9552	0.9611	0.9664	0.6257	0.5159	0.4301	0.3043	1.0000
Northwest	PS	>20	>10000	0.2623	0.6188	0.9643	0.9814	0.7971	0.6527	0.5502	0.4536	0.2980	1.0000
	PS	>20	<10000	0.3814	0.9248	0.9792	0.9636	0.8096	0.5880	0.5604	0.5031	0.2905	1.0000
	PS	<20	>10000	0.4276	0.8512	0.9505	0.9585	0.8125	0.5863	0.6337	0.7835	0.8402	1.0000
	PS	<20	<10000	0.2913	0.9527	0.9862	0.9938	0.6328	0.5437	0.4705	0.4101	0.3393	1.0000
	S	>20	>10000	0.4558	0.7423	0.9345	0.9751	0.9160	0.7435	0.5527	0.4893	0.3448	1.0000
	S	>20	<10000	0.0018	0.8298	0.9537	0.9602	0.9210	0.6175	0.5191	0.4144	0.2602	1.0000
	S	<20	>10000	0.8780	0.8424	0.9233	0.9507	0.8975	0.8501	0.8310	0.8644	0.9150	1.0000
	S	<20	<10000	0.6376	0.8571	0.9500	0.9532	0.9363	0.8318	0.5647	0.7348	0.5079	1.0000
West	PS	>20	>10000	0.4831	0.1227	0.9567	0.9425	0.9249	0.8926	0.8448	0.8013	0.3293	1.0000
	PS	>20	<10000	0.4708	0.0352	0.9603	0.9457	0.9187	0.9194	0.8113	0.5617	0.3129	1.0000
	PS	<20	>10000	0.4734	0.5319	0.9463	0.9332	0.8994	0.8586	0.8532	0.5554	0.4513	1.0000
	PS	<20	<10000	0.4719	0.0000	0.9743	0.9309	0.9606	0.8706	0.7691	0.5255	0.3754	1.0000
	S	>20	>10000	0.0000	0.4629	0.9345	0.9228	0.9365	0.8302	0.9160	0.7726	0.8631	1.0000
	S	>20	<10000	0.3009	0.5147	0.9602	0.9424	0.9493	0.9120	0.7164	0.5782	0.4547	1.0000
	S	<20	>10000	0.8741	0.8336	0.9536	0.9411	0.9104	0.9568	0.8938	0.8865	0.8992	1.0000
	S	<20	<10000	0.2432	0.0000	0.9778	0.9427	0.8387	0.8653	0.6517	0.5263	0.3485	1.0000

Table A2.2: Nationwide Bridge Substructure Deterioration Markov Vectors

Region	Material	Length	ADT	P(9,9)	P(8,8)	P(7,7)	P(6,6)	P(5,5)	P(4,4)	P(3,3)	P(2,2)	P(1,1)	P(0,0)
Northeast	PS	>20	>10000	0.7190	0.8050	0.9449	0.9495	0.9526	0.7024	0.5581	0.4791	0.4083	1.0000
	PS	>20	<10000	0.7669	0.8730	0.9568	0.9499	0.9395	0.8573	0.7174	0.6442	0.5867	1.0000
	PS	<20	>10000	0.6810	0.8141	0.9480	0.9550	0.9495	0.8083	0.6518	0.5354	0.4173	1.0000
	PS	<20	<10000	0.8115	0.8613	0.9606	0.9544	0.9556	0.8298	0.6381	0.5502	0.4303	1.0000
	S	>20	>10000	0.7926	0.8288	0.9342	0.9369	0.9448	0.8836	0.7464	0.6251	0.3283	1.0000
	S	>20	<10000	0.8213	0.8568	0.9419	0.9300	0.9338	0.8869	0.7692	0.7046	0.5074	1.0000
	S	<20	>10000	0.5540	0.8105	0.9341	0.9433	0.9463	0.9069	0.8147	0.6740	0.5271	1.0000
	S	<20	<10000	0.7289	0.7849	0.9149	0.9398	0.9545	0.9382	0.8420	0.8758	0.5853	1.0000
Southeast	PS	>20	>10000	0.5296	0.9152	0.9804	0.9195	0.7916	0.7010	0.6241	0.5772	0.3160	1.0000
	PS	>20	<10000	0.6452	0.9342	0.9803	0.9737	0.8536	0.5601	0.4830	0.4297	0.3465	1.0000
	PS	<20	>10000	0.4462	0.8938	0.9812	0.9404	0.8291	0.7015	0.5837	0.5082	0.3005	1.0000
	PS	<20	<10000	0.5295	0.9403	0.9726	0.9076	0.7849	0.4474	0.3600	0.3097	0.2203	1.0000
	S	>20	>10000	0.7095	0.8801	0.9641	0.9613	0.8715	0.7204	0.6066	0.4417	0.2772	1.0000
	S	>20	<10000	0.6872	0.9042	0.9685	0.9535	0.8581	0.5946	0.5175	0.4482	0.3756	1.0000
	S	<20	>10000	0.4313	0.8210	0.9604	0.9657	0.9277	0.8182	0.7821	0.6681	0.5442	1.0000
	S	<20	<10000	0.4989	0.8925	0.9469	0.9464	0.9391	0.9234	0.8350	0.7010	0.6045	1.0000
Central	PS	>20	>10000	0.8112	0.9076	0.9687	0.9706	0.9104	0.6761	0.4995	0.4381	0.3633	1.0000
	PS	>20	<10000	0.8595	0.9428	0.9685	0.9498	0.9518	0.5725	0.4934	0.4387	0.3471	1.0000
	PS	<20	>10000	0.7857	0.8998	0.9661	0.9561	0.9591	0.7743	0.6896	0.7550	0.5614	1.0000
	PS	<20	<10000	0.8500	0.9448	0.9563	0.9426	0.9259	0.6214	0.5022	0.4432	0.3437	1.0000
	S	>20	>10000	0.7883	0.8574	0.9597	0.9712	0.9449	0.8156	0.5405	0.3574	0.1859	1.0000
	S	>20	<10000	0.8532	0.9071	0.9550	0.9581	0.9222	0.7909	0.6484	0.5587	0.5020	1.0000
	S	<20	>10000	0.7392	0.8355	0.9568	0.9662	0.8879	0.8071	0.7642	0.8310	0.7115	1.0000
	S	<20	<10000	0.8410	0.8913	0.9283	0.9369	0.9340	0.9187	0.7198	0.6264	0.5624	1.0000

Table A2.2 (continued): Nationwide Bridge Substructure Deterioration Markov Vectors

Region	Material	Length	ADT	P(9,9)	P(8,8)	P(7,7)	P(6,6)	P(5,5)	P(4,4)	P(3,3)	P(2,2)	P(1,1)	P(0,0)
East North Central	PS	>20	>10000	0.7855	0.9051	0.9783	0.9280	0.7416	0.5981	0.5277	0.4710	0.3589	1.0000
	PS	>20	<10000	0.8427	0.9550	0.9809	0.9614	0.8426	0.5890	0.5136	0.4550	0.4011	1.0000
	PS	<20	>10000	0.7452	0.8988	0.9637	0.9427	0.9558	0.7870	0.8224	0.6582	0.5923	1.0000
	PS	<20	<10000	0.8363	0.9491	0.9596	0.9375	0.9337	0.6319	0.5198	0.4545	0.3757	1.0000
	S	>20	>10000	0.7978	0.8033	0.9689	0.9616	0.9424	0.9029	0.5553	0.3622	0.1889	1.0000
	S	>20	<10000	0.8006	0.8830	0.9631	0.9454	0.9252	0.8602	0.7930	0.6561	0.5978	1.0000
	S	<20	>10000	0.0000	0.7979	0.9589	0.9466	0.9399	0.8838	0.8121	0.6181	0.3214	1.0000
	S	<20	<10000	0.8130	0.8840	0.9243	0.9291	0.9441	0.9311	0.8953	0.7839	0.7353	1.0000
South	PS	>20	>10000	0.2249	0.8281	0.9802	0.9334	0.7559	0.6857	0.6544	0.5854	0.4641	1.0000
	PS	>20	<10000	0.5370	0.9115	0.9747	0.9155	0.8002	0.6687	0.5822	0.5129	0.3006	1.0000
	PS	<20	>10000	0.2126	0.8504	0.9752	0.9449	0.7947	0.6899	0.6925	0.6087	0.2838	1.0000
	PS	<20	<10000	0.5888	0.9405	0.9696	0.9188	0.7459	0.6838	0.5782	0.4815	0.3906	1.0000
	S	>20	>10000	0.7143	0.8892	0.9615	0.9452	0.8410	0.7570	0.5919	0.4283	0.2653	1.0000
	S	>20	<10000	0.6453	0.9351	0.9628	0.9505	0.9181	0.8247	0.7431	0.6629	0.5731	1.0000
	S	<20	>10000	0.5777	0.8706	0.9537	0.9576	0.9261	0.9140	0.8211	0.7723	0.7318	1.0000
	S	<20	<10000	0.3336	0.8449	0.9398	0.9495	0.9433	0.8924	0.8175	0.8529	0.8799	1.0000
West North Central	PS	>20	>10000	0.8973	0.8512	0.9562	0.6830	0.9572	0.4135	0.3661	0.2998	0.2040	1.0000
	PS	>20	<10000	0.8945	0.9252	0.9719	0.9447	0.8901	0.5028	0.4984	0.4301	0.3823	1.0000
	PS	<20	>10000	0.0520	0.9222	0.9765	0.8473	0.7746	0.7293	0.6439	0.6875	0.5701	1.0000
	PS	<20	<10000	0.8999	0.9224	0.9324	0.9154	0.8541	0.8149	0.7250	0.6061	0.5486	1.0000
	S	>20	>10000	0.8647	0.9215	0.9445	0.9355	0.8379	0.7028	0.5636	0.4055	0.2454	1.0000
	S	>20	<10000	0.9017	0.8970	0.9676	0.9473	0.8951	0.7806	0.8174	0.6903	0.5607	1.0000
	S	<20	>10000	0.7925	0.8932	0.9128	0.9388	0.9352	0.3282	0.5506	0.6619	0.7905	1.0000
	S	<20	<10000	0.9202	0.9176	0.9550	0.9391	0.9511	0.9192	0.7627	0.6261	0.5653	1.0000

Table A2.2 (continued): Nationwide Bridge Substructure Deterioration Markov Vectors

Region	Material	Length	ADT	P(9,9)	P(8,8)	P(7,7)	P(6,6)	P(5,5)	P(4,4)	P(3,3)	P(2,2)	P(1,1)	P(0,0)
Southwest	PS	>20	>10000	0.0467	0.9086	0.9805	0.9424	0.7708	0.8055	0.5743	0.4986	0.2721	1.0000
	PS	>20	<10000	0.0032	0.9288	0.9825	0.9492	0.6283	0.5429	0.4750	0.3985	0.4334	1.0000
	PS	<20	>10000	0.5690	0.8476	0.9559	0.9514	0.8351	0.7281	0.6060	0.5049	0.3865	1.0000
	PS	<20	<10000	0.3565	0.9026	0.9741	0.9305	0.9088	0.7408	0.5652	0.4810	0.2789	1.0000
	S	>20	>10000	0.5648	0.9022	0.9826	0.9513	0.9125	0.5666	0.5042	0.4333	0.3068	1.0000
	S	>20	<10000	0.2181	0.9084	0.9697	0.9579	0.9431	0.8211	0.5306	0.4672	0.2766	1.0000
	S	<20	>10000	0.0000	0.0000	0.9686	0.9728	0.8744	0.5378	0.6056	0.6936	0.8116	1.0000
	S	<20	<10000	0.0000	0.8391	0.9592	0.9427	0.9173	0.9361	0.5590	0.4406	0.3188	1.0000
Northwest	PS	>20	>10000	0.2066	0.8619	0.9844	0.9587	0.8253	0.5866	0.5229	0.4523	0.2912	1.0000
	PS	>20	<10000	0.3373	0.9350	0.9807	0.9657	0.8368	0.6176	0.4954	0.4536	0.2885	1.0000
	PS	<20	>10000	0.3857	0.9048	0.9759	0.9535	0.8514	0.5514	0.5019	0.4506	0.3012	1.0000
	PS	<20	<10000	0.2375	0.9435	0.9696	0.9347	0.8972	0.4159	0.3731	0.3224	0.2424	1.0000
	S	>20	>10000	0.3243	0.8154	0.9443	0.9569	0.8736	1.0000	0.5523	0.4936	0.4067	1.0000
	S	>20	<10000	0.0000	0.8549	0.9592	0.9203	0.9176	0.8437	0.9963	0.4947	0.3040	1.0000
	S	<20	>10000	0.8770	0.8561	0.8901	0.9526	0.8651	0.8680	0.8330	0.8996	0.9157	1.0000
	S	<20	<10000	0.6267	0.8476	0.9357	0.9456	0.9414	0.8375	0.7615	0.8616	0.6661	1.0000
West	PS	>20	>10000	0.4261	0.1591	0.9943	0.9781	0.7549	0.6646	0.6215	0.5378	0.4531	1.0000
	PS	>20	<10000	0.3678	0.0000	0.9945	0.9722	0.8888	0.7432	0.6312	0.5245	0.4334	1.0000
	PS	<20	>10000	0.5984	0.6468	0.9919	0.9877	0.6950	0.5656	0.5305	0.4377	0.3177	1.0000
	PS	<20	<10000	0.2139	0.4751	0.9903	0.9679	0.7995	0.8342	0.7782	0.6518	0.3704	1.0000
	S	>20	>10000	0.6158	0.7028	0.9915	0.9684	0.9076	0.6726	0.4987	0.3317	0.1412	1.0000
	S	>20	<10000	0.3055	0.8142	0.9885	0.9705	0.9400	0.9198	0.5917	0.5021	0.3699	1.0000
	S	<20	>10000	0.5586	0.0000	0.9846	0.9668	0.9053	0.9715	0.7310	0.6095	0.4109	1.0000
	S	<20	<10000	0.3333	0.3612	0.9769	0.9494	0.9377	0.8794	0.8892	0.7040	0.8704	1.0000

Table A2.3: Nationwide Bridge Superstructure Deterioration Markov Vectors

Region	Material	Length	ADT	P(9,9)	P(8,8)	P(7,7)	P(6,6)	P(5,5)	P(4,4)	P(3,3)	P(2,2)	P(1,1)	P(0,0)
Northeast	PS	>20	>10000	0.6929	0.8956	0.9282	0.9277	0.9168	0.6639	0.3886	0.3559	0.2906	1.0000
	PS	>20	<10000	0.8316	0.9068	0.9449	0.9250	0.9080	0.5819	0.7573	0.4979	0.3754	1.0000
	PS	<20	>10000	0.7260	0.9104	0.9339	0.9192	0.9330	0.8183	0.4488	0.3783	0.2789	1.0000
	PS	<20	<10000	0.8483	0.9293	0.9465	0.8661	0.8715	0.6883	0.9390	0.4963	0.4076	1.0000
	S	>20	>10000	0.8388	0.8921	0.9275	0.9049	0.9011	0.7941	0.6819	0.5553	0.4878	1.0000
	S	>20	<10000	0.8709	0.9153	0.9344	0.8938	0.8902	0.8253	0.8068	0.6796	0.6240	1.0000
	S	<20	>10000	0.8319	0.8320	0.9392	0.9512	0.8758	0.7821	0.7632	0.5469	0.3285	1.0000
	S	<20	<10000	0.8182	0.8712	0.9120	0.9159	0.9318	0.9114	0.8286	0.8009	0.7154	1.0000
Southeast	PS	>20	>10000	0.5889	0.9450	0.9808	0.9056	0.8119	0.5738	0.5126	0.4502	0.3415	1.0000
	PS	>20	<10000	0.6642	0.9604	0.9875	0.9523	0.8654	0.5399	0.4873	0.4379	0.3447	1.0000
	PS	<20	>10000	0.4079	0.9296	0.9806	0.9224	0.8021	0.6051	0.5010	0.4309	0.3707	1.0000
	PS	<20	<10000	0.5844	0.9555	0.9789	0.8969	0.5902	0.4647	0.3853	0.3123	0.2292	1.0000
	S	>20	>10000	0.6649	0.9280	0.9686	0.9397	0.8855	0.5739	0.4618	0.4006	0.2829	1.0000
	S	>20	<10000	0.8744	0.9206	0.9428	0.9186	0.9059	0.8558	0.8123	0.7920	0.7378	1.0000
	S	<20	>10000	0.8346	0.8391	0.9454	0.9489	0.9103	0.8278	0.7379	0.6065	0.4883	1.0000
	S	<20	<10000	0.8182	0.8712	0.9120	0.9159	0.9318	0.9114	0.8286	0.8009	0.7154	1.0000
Central	PS	>20	>10000	0.7813	0.9207	0.9569	0.9505	0.8909	0.4096	0.3380	0.2734	0.2082	1.0000
	PS	>20	<10000	0.8387	0.9448	0.9600	0.9290	0.8249	0.8183	0.6242	0.5517	0.4653	1.0000
	PS	<20	>10000	0.7749	0.9186	0.9611	0.9247	0.7977	0.8607	0.6004	0.5050	0.4409	1.0000
	PS	<20	<10000	0.8394	0.9493	0.9617	0.9448	0.8847	0.6201	0.4938	0.4307	0.3239	1.0000
	S	>20	>10000	0.7985	0.8721	0.9559	0.9506	0.8698	0.7420	0.5516	0.4797	0.4135	1.0000
	S	>20	<10000	0.7779	0.9219	0.9519	0.9617	0.8729	0.7560	0.6293	0.4827	0.1917	1.0000
	S	<20	>10000	0.7930	0.8514	0.9438	0.9718	0.9312	0.8702	0.5391	0.4282	0.3119	1.0000
	S	<20	<10000	0.8058	0.8980	0.9262	0.9413	0.9307	0.9229	0.8230	0.6193	0.5331	1.0000

Table A2.3 (continued): Nationwide Bridge Superstructure Deterioration Markov Vectors

Region	Material	Length	ADT	P(9,9)	P(8,8)	P(7,7)	P(6,6)	P(5,5)	P(4,4)	P(3,3)	P(2,2)	P(1,1)	P(0,0)
East North Central	PS	>20	>10000	0.8068	0.9497	0.9789	0.8985	0.7423	0.6012	0.5260	0.4758	0.3749	1.0000
	PS	>20	<10000	0.8583	0.9681	0.9772	0.8494	0.6338	0.5114	0.4439	0.3812	0.3253	1.0000
	PS	<20	>10000	0.7726	0.9552	0.9551	0.9029	0.8897	0.8432	0.5853	0.4755	0.3314	1.0000
	PS	<20	<10000	0.8534	0.9646	0.9624	0.9131	0.9372	0.5550	0.4689	0.4005	0.3238	1.0000
	S	>20	>10000	0.7948	0.8530	0.9570	0.9381	0.9111	0.8024	0.5558	0.5094	0.4285	1.0000
	S	>20	<10000	0.8168	0.8902	0.9583	0.9390	0.9277	0.8354	0.7831	0.6452	0.5938	1.0000
	S	<20	>10000	0.6452	0.8465	0.9550	0.9502	0.9356	0.7339	0.8522	0.7131	0.6125	1.0000
	S	<20	<10000	0.7940	0.8480	0.9329	0.9365	0.9373	0.8514	0.8206	0.7886	0.8310	1.0000
South	PS	>20	>10000	0.4140	0.9529	0.9676	0.9453	0.7564	0.5237	0.4522	0.3778	0.2878	1.0000
	PS	>20	<10000	0.6478	0.9762	0.9555	0.8413	0.5377	0.4071	0.3494	0.2877	0.1949	1.0000
	PS	<20	>10000	0.4816	0.9618	0.9714	0.9281	0.8013	0.5849	0.5612	0.4675	0.3647	1.0000
	PS	<20	<10000	0.6690	0.9809	0.9633	0.8555	0.6957	0.4957	0.4711	0.4056	0.2765	1.0000
	S	>20	>10000	0.4474	0.9354	0.9568	0.9642	0.8955	0.6677	0.5450	0.4542	0.3585	1.0000
	S	>20	<10000	0.3329	0.9234	0.9311	0.9786	0.8667	0.7400	0.6182	0.4851	0.2385	1.0000
	S	<20	>10000	0.4916	0.8767	0.9531	0.9652	0.9798	0.7711	0.5551	0.4726	0.3848	1.0000
	S	<20	<10000	0.3208	0.8291	0.9408	0.9618	0.9516	0.8796	0.7733	0.8496	0.8505	1.0000
West North Central	PS	>20	>10000	0.9143	0.9608	0.9412	0.4276	0.7306	0.3681	0.3303	0.2857	0.2330	1.0000
	PS	>20	<10000	0.8950	0.9692	0.9772	0.9590	0.6442	0.5691	0.5104	0.4566	0.3910	1.0000
	PS	<20	>10000	0.0000	0.9653	0.9807	0.9116	0.6175	0.6107	0.4746	0.4775	0.3300	1.0000
	PS	<20	<10000	0.9034	0.9671	0.9486	0.8998	0.8525	0.6244	0.6113	0.6206	0.5991	1.0000
	S	>20	>10000	0.9134	0.9106	0.9243	0.8789	0.8803	0.7535	0.7091	0.4707	0.3677	1.0000
	S	>20	<10000	0.9167	0.9016	0.9335	0.9432	0.8597	0.8284	0.6706	0.6423	0.5257	1.0000
	S	<20	>10000	0.8461	0.8686	0.9214	0.9904	1.0000	0.7274	0.4595	0.4593	0.4593	1.0000
	S	<20	<10000	0.9212	0.9322	0.9448	0.9494	0.9531	0.8939	0.6645	0.5500	0.5078	1.0000

Table A2.3 (continued): Nationwide Bridge Superstructure Deterioration Markov Vectors

Region	Material	Length	ADT	P(9,9)	P(8,8)	P(7,7)	P(6,6)	P(5,5)	P(4,4)	P(3,3)	P(2,2)	P(1,1)	P(0,0)
Southwest	PS	>20	>10000	0.0499	0.9490	0.9805	0.8905	0.6938	0.6755	0.5741	0.4728	0.3599	1.0000
	PS	>20	<10000	0.2762	0.9632	0.9860	0.9343	0.7772	0.6116	0.5212	0.4450	0.2785	1.0000
	PS	<20	>10000	0.6170	0.8680	0.9686	0.9557	0.9557	0.8557	0.6636	0.5425	0.4173	1.0000
	PS	<20	<10000	0.3267	0.9367	0.9743	0.9556	0.8937	0.6820	0.5523	0.4439	0.3356	1.0000
	S	>20	>10000	0.5693	0.9117	0.9635	0.9286	0.8119	0.7399	0.9864	0.4696	0.2822	1.0000
	S	>20	<10000	0.4548	0.9226	0.9717	0.9643	0.9616	0.5737	0.5076	0.4244	0.3425	1.0000
	S	<20	>10000	0.6889	0.9011	0.9702	0.9495	0.9593	0.4111	0.5164	0.6558	0.4507	1.0000
	S	<20	<10000	0.3015	0.7748	0.9680	0.9650	0.9304	0.8503	0.6978	0.5734	0.4502	1.0000
Northwest	PS	>20	>10000	0.2269	0.8602	0.9785	0.9638	0.9269	0.5641	0.4968	0.4304	0.2794	1.0000
	PS	>20	<10000	0.3530	0.9444	0.9840	0.9646	0.7957	0.5812	0.5051	0.4495	0.3303	1.0000
	PS	<20	>10000	0.4042	0.9131	0.9760	0.9368	0.9430	0.6045	0.5764	0.5615	0.4022	1.0000
	PS	<20	<10000	0.3528	0.9572	0.9779	0.9568	0.8385	0.5445	0.4338	0.3665	0.2793	1.0000
	S	>20	>10000	0.2975	0.8576	0.9314	0.9509	0.8904	0.8863	0.9272	0.5756	0.4841	1.0000
	S	>20	<10000	0.3953	0.8593	0.9475	0.9460	0.9023	0.8384	0.7318	0.5076	0.3947	1.0000
	S	<20	>10000	0.8857	0.8689	0.9118	0.9520	0.8947	0.8318	0.8318	0.8651	0.9151	1.0000
	S	<20	<10000	0.6587	0.8611	0.9443	0.9604	0.9256	0.8284	0.7494	0.7081	0.5739	1.0000
West	PS	>20	>10000	0.1124	0.9833	0.9663	0.9026	0.6279	0.5389	0.4558	0.3991	0.2170	1.0000
	PS	>20	<10000	0.0000	0.9879	0.9737	0.8225	0.5836	0.4860	0.4163	0.3604	0.2255	1.0000
	PS	<20	>10000	0.4126	0.9308	0.9825	0.9199	0.8075	0.7146	0.5556	0.4943	0.4656	1.0000
	PS	<20	<10000	0.6759	0.9534	0.9712	0.9025	0.8147	0.7953	0.7494	0.6801	0.5809	1.0000
	S	>20	>10000	0.3402	0.8647	0.9518	0.9674	0.9113	0.7527	0.7075	0.8076	0.7554	1.0000
	S	>20	<10000	0.2732	0.8680	0.9525	0.9516	0.9226	0.8016	0.8419	0.6869	0.5496	1.0000
	S	<20	>10000	0.7834	0.0000	0.9634	0.9731	0.9502	0.7811	0.9303	0.9046	0.6038	1.0000
	S	<20	<10000	0.2407	0.0000	0.9303	0.9557	0.9613	0.7743	0.9119	0.8157	0.5786	1.0000

Table A2.4: Northeast Region Bridge Deck, Substructure and Superstructure Mean, +SD, -SD Deterioration Markov Vectors

Region	Part	Material	Length	ADT	Vector	P(9,9)	P(8,8)	P(7,7)	P(6,6)	P(5,5)	P(4,4)	P(3,3)	P(2,2)	P(1,1)	P(0,0)
NORTHEAST	SUPERSTRUCTURE	PS	>20	>10000	Mean	0.6929	0.8956	0.9282	0.9277	0.9168	0.6639	0.3886	0.3559	0.2906	1.0000
					+SD	0.9017	0.9682	0.8856	0.8174	0.7257	0.6086	0.4821	0.3297	0.1917	1.0000
					-SD	0.2429	0.6158	0.9192	0.9256	0.8904	0.8432	0.8170	0.5036	0.2378	1.0000
		PS	>20	<10000	Mean	0.8316	0.9068	0.9449	0.9250	0.9080	0.5819	0.7573	0.4979	0.3754	1.0000
					+SD	0.9626	0.9599	0.8706	0.8066	0.7188	0.6089	0.4782	0.3333	0.1854	1.0000
					-SD	0.1931	0.6425	0.9516	0.9125	0.8324	0.9033	0.7676	0.4680	0.2251	1.0000
		PS	<20	>10000	Mean	0.7260	0.9104	0.9339	0.9192	0.9330	0.8183	0.4488	0.3783	0.2789	1.0000
					+SD	0.9319	0.9435	0.9516	0.8533	0.7769	0.6682	0.5365	0.3855	0.2041	1.0000
					-SD	0.3612	0.6497	0.9142	0.9367	0.9159	0.8929	0.6894	0.4280	0.2102	1.0000
		PS	<20	<10000	Mean	0.8483	0.9293	0.9465	0.8661	0.8715	0.6883	0.9390	0.4963	0.4076	1.0000
					+SD	0.9771	0.9343	0.8673	0.7972	0.7083	0.5962	0.4703	0.3245	0.1818	1.0000
					-SD	0.0000	0.8614	0.9112	0.8686	0.9194	0.9510	0.6978	0.4454	0.2191	1.0000
		S	>20	>10000	Mean	0.8388	0.8921	0.9275	0.9049	0.9011	0.7941	0.6819	0.5553	0.4878	1.0000
					+SD	0.9650	0.9205	0.8786	0.7929	0.6939	0.5843	0.4545	0.3092	0.1833	1.0000
					-SD	0.0000	0.5370	0.9290	0.9114	0.8939	0.8600	0.8409	0.5057	0.2360	1.0000
		S	>20	<10000	Mean	0.8709	0.9153	0.9344	0.8938	0.8902	0.8253	0.8068	0.6796	0.6240	1.0000
					+SD	0.9737	0.9296	0.8800	0.7957	0.7006	0.5822	0.4520	0.3024	0.1758	1.0000
					-SD	0.0113	0.5988	0.9336	0.9189	0.8960	0.8733	0.8454	0.5102	0.2430	1.0000
		S	<20	>10000	Mean	0.8319	0.8320	0.9392	0.9512	0.8758	0.7821	0.7632	0.5469	0.3285	1.0000
					+SD	0.9319	0.9473	0.9474	0.8640	0.7597	0.6570	0.5230	0.3684	0.1980	1.0000
					-SD	0.0000	0.5942	0.9166	0.9498	0.9351	0.6145	0.5638	0.4066	0.2043	1.0000
		S	<20	<10000	Mean	0.8182	0.8712	0.9120	0.9159	0.9318	0.9114	0.8286	0.8009	0.7154	1.0000
					+SD	0.9778	0.9419	0.6798	0.5147	0.4239	0.3065	0.1611	0.0000	0.1806	1.0000
					-SD	0.0000	0.3272	0.8397	0.9325	0.9276	0.9080	0.8755	0.5420	0.2492	1.0000

Table A2.4 (continued): Northeast Region Bridge Deck, Substructure and Superstructure Mean, +SD, -SD Deterioration Markov Vectors

Region	Part	Material	Length	ADT	Vector	P(9,9)	P(8,8)	P(7,7)	P(6,6)	P(5,5)	P(4,4)	P(3,3)	P(2,2)	P(1,1)	P(0,0)
NORTHEAST	DECK	PS	>20	>10000	Mean	0.6977	0.8206	0.9505	0.9611	0.9199	0.7312	0.4785	0.4276	0.3603	1.0000
					+SD	0.8873	0.9183	0.9603	0.9528	0.8944	0.7695	0.6399	0.4965	0.1872	1.0000
					-SD	0.0000	0.5897	0.9056	0.9570	0.8742	0.8920	0.9236	0.5477	0.2538	1.0000
		PS	>20	<10000	Mean	0.7272	0.8556	0.9511	0.9492	0.9541	0.7782	0.6453	0.5048	0.4241	1.0000
					+SD	0.9192	0.9244	0.9621	0.9608	0.9008	0.7813	0.6520	0.5091	0.1885	1.0000
					-SD	0.0567	0.5586	0.9062	0.9515	0.9064	0.9406	0.7947	0.4879	0.2351	1.0000
		PS	<20	>10000	Mean	0.5843	0.8437	0.9516	0.9642	0.9312	0.7975	0.6282	0.4829	0.3941	1.0000
					+SD	0.8582	0.9443	0.9605	0.9498	0.8907	0.7732	0.6361	0.4877	0.1953	1.0000
					-SD	0.0076	0.4801	0.8822	0.9741	0.8628	0.8796	0.8824	0.5325	0.2491	1.0000
		PS	<20	<10000	Mean	0.7999	0.8722	0.9567	0.9480	0.9652	0.7035	0.6156	0.4663	0.3365	1.0000
					+SD	0.9400	0.9289	0.9609	0.9674	0.8890	0.7763	0.6512	0.5106	0.1843	1.0000
					-SD	0.0000	0.8383	0.8474	0.9610	0.9660	0.7725	0.5761	0.3881	0.1997	1.0000
		S	>20	>10000	Mean	0.8033	0.8503	0.9455	0.9439	0.9475	0.8305	0.7373	0.6377	0.5608	1.0000
					+SD	0.9219	0.9773	0.9074	0.8169	0.7267	0.6208	0.4940	0.3483	0.1824	1.0000
					-SD	0.0000	0.6678	0.8721	0.9592	0.9289	0.9339	0.6838	0.4334	0.2159	1.0000
		S	>20	<10000	Mean	0.8352	0.8719	0.9462	0.9427	0.9367	0.8801	0.8024	0.6820	0.6248	1.0000
					+SD	0.9538	0.9822	0.7767	0.6559	0.5661	0.4479	0.3086	0.1485	0.1826	1.0000
					-SD	0.0000	0.6686	0.9180	0.9396	0.9374	0.9286	0.7026	0.4434	0.2182	1.0000
		S	<20	>10000	Mean	0.7825	0.8262	0.9461	0.9452	0.9532	0.8829	0.6515	0.5240	0.3127	1.0000
					+SD	0.9309	0.9043	0.9768	0.9125	0.7266	0.6436	0.5256	0.3746	0.2000	1.0000
					-SD	0.3256	0.7076	0.7511	0.9676	0.9281	0.8903	0.8712	0.6181	0.2833	1.0000
		S	<20	<10000	Mean	0.8079	0.8592	0.9340	0.9464	0.9523	0.9318	0.8683	0.8445	0.8023	1.0000
					+SD	0.9490	0.9858	0.7522	0.4808	0.4098	0.3012	0.1612	0.0000	0.1826	1.0000
					-SD	0.0000	0.3102	0.8941	0.9081	0.9639	0.9592	0.6706	0.4320	0.2168	1.0000

Table A2.4 (continued): Northeast Region Bridge Deck, Substructure and Superstructure Mean, +SD, -SD Deterioration Markov Vectors

Region	Part	Material	Length	ADT	Vector	P(9,9)	P(8,8)	P(7,7)	P(6,6)	P(5,5)	P(4,4)	P(3,3)	P(2,2)	P(1,1)	P(0,0)
NORTHEAST	SUBSTRUCTURE	PS	>20	>10000	Mean	0.7190	0.8050	0.9449	0.9495	0.9526	0.7024	0.5581	0.4791	0.4083	1.0000
					+SD	0.9238	0.8549	0.9398	0.9605	0.9296	0.7880	0.6591	0.5147	0.1834	1.0000
					-SD	0.0000	0.5214	0.8479	0.9753	0.8714	0.8148	0.8080	0.5372	0.2555	1.0000
		PS	>20	<10000	Mean	0.7669	0.8730	0.9568	0.9499	0.9395	0.8573	0.7174	0.6442	0.5867	1.0000
					+SD	0.9377	0.9191	0.9580	0.9619	0.8767	0.7630	0.6360	0.4939	0.1882	1.0000
					-SD	0.2200	0.5879	0.9116	0.9612	0.9234	0.9437	0.7318	0.4532	0.2237	1.0000
		PS	<20	>10000	Mean	0.6810	0.8141	0.9480	0.9550	0.9495	0.8083	0.6518	0.5354	0.4173	1.0000
					+SD	0.8913	0.9077	0.9754	0.9347	0.7975	0.6687	0.5325	0.3752	0.2005	1.0000
					-SD	0.2644	0.4727	0.8473	0.9664	0.9343	0.8810	0.8803	0.7111	0.3066	1.0000
		PS	<20	<10000	Mean	0.8115	0.8613	0.9606	0.9544	0.9556	0.8298	0.6381	0.5502	0.4303	1.0000
					+SD	0.9534	0.8323	0.9853	0.8881	0.7619	0.6527	0.5216	0.3662	0.2140	1.0000
					-SD	0.3849	0.6750	0.8834	0.9753	0.9089	0.9309	0.8126	0.4969	0.2370	1.0000
		S	>20	>10000	Mean	0.7926	0.8288	0.9342	0.9369	0.9448	0.8836	0.7464	0.6251	0.3283	1.0000
					+SD	0.9360	0.9251	0.9025	0.8905	0.9550	0.8555	0.6786	0.4781	0.2523	1.0000
					-SD	0.4937	0.5859	0.8433	0.9358	0.9589	0.9134	0.6784	0.4449	0.2307	1.0000
		S	>20	<10000	Mean	0.8213	0.8568	0.9419	0.9300	0.9338	0.8869	0.7692	0.7046	0.5074	1.0000
					+SD	0.9586	0.9764	0.7022	0.5570	0.4613	0.3360	0.1906	0.0294	0.1811	1.0000
					-SD	0.0000	0.8563	0.3880	0.9536	0.9385	0.9150	0.6769	0.4300	0.2153	1.0000
		S	<20	>10000	Mean	0.5540	0.8105	0.9341	0.9433	0.9463	0.9069	0.8147	0.6740	0.5271	1.0000
					+SD	0.8875	0.8912	0.9719	0.9431	0.8017	0.6784	0.5340	0.3751	0.2027	1.0000
					-SD	0.0000	0.2152	0.8100	0.9476	0.9403	0.9334	0.8812	0.5745	0.2587	1.0000
		S	<20	<10000	Mean	0.7289	0.7849	0.9149	0.9398	0.9545	0.9382	0.8420	0.8758	0.5853	1.0000
					+SD	0.9357	0.9855	0.0000	0.0000	0.0412	0.0593	0.0243	0.0000	0.1367	1.0000
					-SD	0.0000	0.0000	0.7248	0.8971	0.9664	0.9244	0.9286	0.5155	0.2423	1.0000

APPENDIX A3 – BRIDGE β CALCULATIONS

The calculations of β values for bridges with different length and different condition ratings.

	Span (m)	Average NSHRP-368 Table B-2	Span (ft)	COV NSHRP-368 Figure B-11	Sum of LL	Count of LL & OR	Sum of OR	Mean LL	SD LL	Mean to Nom ratio DL (Nowak,1979)	COV DL (Nowak, 1979)	Mean DL	SD DL	Mean OR	SD OR	Mean NDL	SD NDL	β	Mean Age
BRIDGE CONDITION STATE 9	3.048	0.62	10	0.2	41.61	2.00	104.00	20.81	4.16	1.03	0.04	7.6684	0.31	52.00	11.17	7.45	0.30	2.595949	7.00
	6.096	0.71	20	0.21	140.61	6.00	499.00	23.44	4.92	1.03	0.04	15.337	0.61	83.17	20.53	14.89	0.60	2.806385	25.50
	9.144	0.74	30	0.22	646.29	27.00	2041.10	23.94	5.27	1.03	0.04	23.005	0.92	75.60	22.20	22.34	0.89	2.231506	15.59
	12.192	0.75	40	0.25	530.72	22.00	1689.90	24.12	6.03	1.03	0.04	47.434	1.90	76.81	17.31	46.05	1.84	2.770531	12.32
	15.24	0.72	50	0.35	726.57	31.00	2233.00	23.44	8.20	1.03	0.04	71.863	2.87	72.03	16.80	69.77	2.79	2.432247	12.45
	18.288	0.72	60	0.36	678.43	29.00	1821.90	23.39	8.42	1.03	0.04	96.292	3.85	62.82	21.47	93.49	3.74	1.546852	10.66
	21.336	0.74	70	0.37	387.11	16.00	1078.10	24.19	8.95	1.03	0.04	140.48	5.62	67.38	20.40	136.38	5.46	1.655539	11.50
	24.384	0.77	80	0.39	249.69	10.00	849.50	24.97	9.74	1.03	0.04	184.66	7.39	84.95	12.63	179.28	7.17	2.876857	14.00
	27.432	0.79	90	0.4	179.10	7.00	564.70	25.59	10.23	1.03	0.04	239.57	9.58	80.67	17.44	232.60	9.30	1.984871	18.00
	30.48	0.82	100	0.41	133.34	5.00	301.00	26.67	10.93	1.03	0.04	294.49	11.78	60.20	22.79	285.91	11.44	0.827984	9.20
	33.528	0.84	110	0.42	108.59	4.00	335.40	27.15	11.40	1.03	0.04	366.69	14.67	83.85	18.54	356.01	14.24	1.54107	4.25
	36.576	0.85	120	0.43	27.43	0.00	0.00	0.00	0.00	1.03	0.04	438.89	17.56	0.00	0.00	426.11	17.04	0	0.00
	39.624	0.86	130	0.43	0.00	0.00	0.00	0.00	0.00	1.03	0.04	562.41	22.50	0.00	0.00	546.03	21.84	0	0.00
	42.672	0.86	140	0.44	0.00	0.00	0.00	0.00	0.00	1.03	0.04	685.93	27.44	0.00	0.00	665.95	26.64	0	0.00

Table A3.1: 2009, HS20 Rated, Prestressed, Northeast Region Bridges NBI Processed Data to Obtain the β and Mean Bridge Ages for Different Length of Bridges (3.048m to 42.67m) for Bridge Condition Rating of 9.

	Span (m)	Average NSHRP-368 Table B-2	Span (ft)	COV NSHRP-368 Figure B-11	Sum of LL	Count of LL & OR	Sum of OR	Mean LL	SD LL	Mean to Nom ratio DL (Nowak,1979)	COV DL (Nowak, 1979)	Mean DL	SD DL	Mean OR	SD OR	Mean NDL	SD NDL	β	Mean Age
BRIDGE CONDITION STATE 8	3.048	0.62	10	0.2	63.00	3.00	132.30	21.00	4.20	1.03	0.04	7.6684	0.31	44.10	0.00	7.45	0.30	5.419425	9.33
	6.096	0.71	20	0.21	442.91	19.00	1423.10	23.31	4.90	1.03	0.04	15.337	0.61	74.90	19.96	14.89	0.60	2.486223	16.95
	9.144	0.74	30	0.22	1483.96	62.00	4875.40	23.93	5.27	1.03	0.04	23.005	0.92	78.64	15.98	22.34	0.89	3.201807	20.61
	12.192	0.75	40	0.25	2123.22	88.00	6367.10	24.13	6.03	1.03	0.04	47.434	1.90	72.35	16.13	46.05	1.84	2.689006	17.84
	15.24	0.72	50	0.35	1968.31	84.00	5914.90	23.43	8.20	1.03	0.04	71.863	2.87	70.42	15.85	69.77	2.79	2.454074	16.80
	18.288	0.72	60	0.36	1685.72	72.00	4861.30	23.41	8.43	1.03	0.04	96.292	3.85	67.52	18.62	93.49	3.74	1.954019	19.01
	21.336	0.74	70	0.37	1419.69	59.00	4234.10	24.06	8.90	1.03	0.04	140.48	5.62	71.76	15.37	136.38	5.46	2.246742	18.25
	24.384	0.77	80	0.39	1416.55	57.00	4416.90	24.85	9.69	1.03	0.04	184.66	7.39	77.49	15.01	179.28	7.17	2.291488	17.47
	27.432	0.79	90	0.4	794.34	31.00	2346.80	25.62	10.25	1.03	0.04	239.57	9.58	75.70	14.76	232.60	9.30	1.924782	20.84
	30.48	0.82	100	0.41	821.01	31.00	2273.20	26.48	10.86	1.03	0.04	294.49	11.78	73.33	18.15	285.91	11.44	1.429118	17.26
	33.528	0.84	110	0.42	407.07	15.00	1011.60	27.14	11.40	1.03	0.04	366.69	14.67	67.44	21.36	356.01	14.24	0.934785	12.33
	36.576	0.85	120	0.43	109.93	4.00	313.00	27.48	11.82	1.03	0.04	438.89	17.56	78.25	24.28	426.11	17.04	1.042314	9.75
	39.624	0.86	130	0.43	83.59	3.00	206.80	27.86	11.98	1.03	0.04	562.41	22.50	68.93	18.53	546.03	21.84	0.643934	26.33
	42.672	0.86	140	0.44	0.00	0.00	0.00	0.00	0.00	1.03	0.04	685.93	27.44	0.00	0.00	665.95	26.64	0	0.00

Table A3.2: 2009, HS20 Rated, Prestressed, Northeast Region Bridges NBI Processed Data to Obtain the β and Mean Bridge Ages for Different Length of Bridges (3.048m to 42.67m) for Bridge Condition Rating of 8.

	Span (m)	Average NSHRP-368 Table B-2	Span (ft)	COV NSHRP-368 Figure B-11	Sum of LL	Count of LL & OR	Sum of OR	Mean LL	SD LL	Mean to Nom ratio DL (Nowak,1979)	COV DL (Nowak, 1979)	Mean DL	SD DL	Mean OR	SD OR	Mean NDL	SD NDL	β	Mean Age
BRIDGE CONDITION STATE 7	3.048	0.62	10	0.2	0.00	0.00	0.00	0.00	0.00	1.03	0.04	7.6684	0.31	0.00	0.00	7.45	0.30	0	0.00
	6.096	0.71	20	0.21	857.77	37.00	2556.50	23.18	4.87	1.03	0.04	15.337	0.61	69.09	19.19	14.89	0.60	2.294791	23.16
	9.144	0.74	30	0.22	1795.17	75.00	5148.70	23.94	5.27	1.03	0.04	23.005	0.92	68.65	15.03	22.34	0.89	2.756245	25.25
	12.192	0.75	40	0.25	3429.38	142.00	10144.70	24.15	6.04	1.03	0.04	47.434	1.90	71.44	15.40	46.05	1.84	2.740901	26.35
	15.24	0.72	50	0.35	2346.19	100.00	6708.60	23.46	8.21	1.03	0.04	71.863	2.87	67.09	15.30	69.77	2.79	2.330278	27.01
	18.288	0.72	60	0.36	2904.40	124.00	8739.10	23.42	8.43	1.03	0.04	96.292	3.85	70.48	16.89	93.49	3.74	2.254892	26.18
	21.336	0.74	70	0.37	1584.13	66.00	4596.40	24.00	8.88	1.03	0.04	140.48	5.62	69.64	17.13	136.38	5.46	1.995607	24.05
	24.384	0.77	80	0.39	1964.27	79.00	5785.60	24.86	9.70	1.03	0.04	184.66	7.39	73.24	17.35	179.28	7.17	1.921025	23.01
	27.432	0.79	90	0.4	1077.85	42.00	3201.40	25.66	10.27	1.03	0.04	239.57	9.58	76.22	15.68	232.60	9.30	1.893876	22.67
	30.48	0.82	100	0.41	1090.51	41.00	3246.50	26.60	10.91	1.03	0.04	294.49	11.78	79.18	17.88	285.91	11.44	1.653871	23.49
	33.528	0.84	110	0.42	271.29	10.00	776.50	27.13	11.39	1.03	0.04	366.69	14.67	77.65	16.82	356.01	14.24	1.382245	24.70
	36.576	0.85	120	0.43	247.88	9.00	586.30	27.54	11.84	1.03	0.04	438.89	17.56	65.14	26.80	426.11	17.04	0.650191	23.56
	39.624	0.86	130	0.43	27.86	1.00	89.80	27.86	11.98	1.03	0.04	562.41	22.50	89.80	0.00	546.03	21.84	1.357264	16.00
	42.672	0.86	140	0.44	110.91	4.00	266.10	27.73	12.20	1.03	0.04	685.93	27.44	66.53	27.39	665.95	26.64	0.387231	13.75

Table A3.3: 2009, HS20 Rated, Prestressed, Northeast Region Bridges NBI Processed Data to Obtain the β and Mean Bridge Ages for Different Length of Bridges (3.048m to 42.67m) for Bridge Condition Rating of 7.

	Span (m)	Average NSHRP-368 Table B-2	Span (ft)	COV NSHRP-368 Figure B-11	Sum of LL	Count of LL & OR	Sum of OR	Mean LL	SD LL	Mean to Nom ratio DL (Nowak,1979)	COV DL (Nowak, 1979)	Mean DL	SD DL	Mean OR	SD OR	Mean NDL	SD NDL	β	Mean Age
BRIDGE CONDITION STATE 6	3.048	0.62	10	0.2	0.00	0.00	0.00	0.00	0.00	1.03	0.04	7.6684	0.31	0.00	0.00	7.45	0.30	0	0.00
	6.096	0.71	20	0.21	416.30	18.00	1175.40	23.13	4.86	1.03	0.04	15.337	0.61	65.30	16.96	14.89	0.60	2.362254	33.56
	9.144	0.74	30	0.22	980.31	41.00	2803.10	23.91	5.26	1.03	0.04	23.005	0.92	68.37	16.68	22.34	0.89	2.496955	32.85
	12.192	0.75	40	0.25	1738.31	72.00	4827.90	24.14	6.04	1.03	0.04	47.434	1.90	67.05	17.88	46.05	1.84	2.178911	32.61
	15.24	0.72	50	0.35	985.42	42.00	2882.40	23.46	8.21	1.03	0.04	71.863	2.87	68.63	14.21	69.77	2.79	2.549594	29.67
	18.288	0.72	60	0.36	1428.26	61.00	4392.80	23.41	8.43	1.03	0.04	96.292	3.85	72.01	15.49	93.49	3.74	2.48413	31.30
	21.336	0.74	70	0.37	1176.32	49.00	3373.30	24.01	8.88	1.03	0.04	140.48	5.62	68.84	18.58	136.38	5.46	1.848989	31.02
	24.384	0.77	80	0.39	970.20	39.00	2846.70	24.88	9.70	1.03	0.04	184.66	7.39	72.99	16.78	179.28	7.17	1.947178	28.03
	27.432	0.79	90	0.4	795.30	31.00	2329.30	25.65	10.26	1.03	0.04	239.57	9.58	75.14	18.73	232.60	9.30	1.687571	31.00
	30.48	0.82	100	0.41	692.07	26.00	2025.60	26.62	10.91	1.03	0.04	294.49	11.78	77.91	20.19	285.91	11.44	1.513564	25.85
	33.528	0.84	110	0.42	108.72	4.00	341.00	27.18	11.42	1.03	0.04	366.69	14.67	85.25	19.76	356.01	14.24	1.546866	28.50
	36.576	0.85	120	0.43	55.12	2.00	148.90	27.56	11.85	1.03	0.04	438.89	17.56	74.45	35.99	426.11	17.04	0.75618	18.50
	39.624	0.86	130	0.43	27.86	1.00	99.90	27.86	11.98	1.03	0.04	562.41	22.50	99.90	0.00	546.03	21.84	1.658071	30.00
	42.672	0.86	140	0.44	0.00	0.00	0.00	0.00	0.00	1.03	0.04	685.93	27.44	0.00	0.00	665.95	26.64	0	0.00

Table A3.4: 2009, HS20 Rated, Prestressed, Northeast Region Bridges NBI Processed Data to Obtain the β and Mean Bridge Ages for Different Length of Bridges (3.048m to 42.67m) for Bridge Condition Rating of 6.

	Span (m)	Average NSHRP-368 Table B-2	Span (ft)	COV NSHRP-368 Figure B-11	Sum of LL	Count of LL & OR	Sum of OR	Mean LL	SD LL	Mean to Nom ratio DL (Nowak,1979)	COV DL (Nowak, 1979)	Mean DL	SD DL	Mean OR	SD OR	Mean NDL	SD NDL	β	Mean Age
BRIDGE CONDITION STATE 5	3.048	0.62	10	0.2	21.29	1.00	75.30	21.29	4.26	1.03	0.04	7.6684	0.31	75.30	0.00	7.45	0.30	12.57216	47.00
	6.096	0.71	20	0.21	278.55	12.00	749.50	23.21	4.87	1.03	0.04	15.337	0.61	62.46	16.26	14.89	0.60	2.282272	37.92
	9.144	0.74	30	0.22	572.59	24.00	1725.70	23.86	5.25	1.03	0.04	23.005	0.92	71.90	18.75	22.34	0.89	2.428047	39.42
	12.192	0.75	40	0.25	603.58	25.00	1825.70	24.14	6.04	1.03	0.04	47.434	1.90	73.03	18.20	46.05	1.84	2.45374	40.52
	15.24	0.72	50	0.35	681.35	29.00	1823.20	23.49	8.22	1.03	0.04	71.863	2.87	62.87	18.97	69.77	2.79	1.770314	39.93
	18.288	0.72	60	0.36	584.94	25.00	1574.10	23.40	8.42	1.03	0.04	96.292	3.85	62.96	15.56	93.49	3.74	1.988049	35.44
	21.336	0.74	70	0.37	624.34	26.00	1725.60	24.01	8.88	1.03	0.04	140.48	5.62	66.37	16.39	136.38	5.46	1.89221	36.46
	24.384	0.77	80	0.39	450.64	18.00	1222.90	25.04	9.76	1.03	0.04	184.66	7.39	67.94	18.26	179.28	7.17	1.62278	29.11
	27.432	0.79	90	0.4	153.56	6.00	507.50	25.59	10.24	1.03	0.04	239.57	9.58	84.58	15.63	232.60	9.30	2.264718	37.33
	30.48	0.82	100	0.41	397.47	15.00	1222.90	26.50	10.86	1.03	0.04	294.49	11.78	81.53	12.97	285.91	11.44	1.970413	30.33
	33.528	0.84	110	0.42	54.04	2.00	189.70	27.02	11.35	1.03	0.04	366.69	14.67	94.85	7.14	356.01	14.24	2.337644	20.00
	36.576	0.85	120	0.43	27.56	1.00	74.10	27.56	11.85	1.03	0.04	438.89	17.56	74.10	0.00	426.11	17.04	1.241455	49.00
	39.624	0.86	130	0.43	0.00	0.00	0.00	0.00	0.00	1.03	0.04	562.41	22.50	0.00	0.00	546.03	21.84	0	0.00
	42.672	0.86	140	0.44	0.00	0.00	0.00	0.00	0.00	1.03	0.04	685.93	27.44	0.00	0.00	665.95	26.64	0	0.00

Table A3.5: 2009, HS20 Rated, Prestressed, Northeast Region Bridges NBI Processed Data to Obtain the β and Mean Bridge Ages for Different Length of Bridges (3.048m to 42.67m) for Bridge Condition Rating of 5.

	Span (m)	Average NSHRP-368 Table B-2	Span (ft)	COV NSHRP-368 Figure B-11	Sum of LL	Count of LL & OR	Sum of OR	Mean LL	SD LL	Mean to Nom ratio DL (Nowak,1979)	COV DL (Nowak, 1979)	Mean DL	SD DL	Mean OR	SD OR	Mean NDL	SD NDL	β	Mean Age
BRIDGE CONDITION STATE 4	3.048	0.62	10	0.2	0.00	0.00	0.00	0.00	0.00	1.03	0.04	7.6684	0.31	0.00	0.00	7.45	0.30	0	0.00
	6.096	0.71	20	0.21	46.11	2.00	111.90	23.05	4.84	1.03	0.04	15.337	0.61	55.95	9.83	14.89	0.60	2.95279	46.00
	9.144	0.74	30	0.22	191.14	8.00	502.10	23.89	5.26	1.03	0.04	23.005	0.92	62.76	23.52	22.34	0.89	1.583044	41.00
	12.192	0.75	40	0.25	169.04	7.00	422.90	24.15	6.04	1.03	0.04	47.434	1.90	60.41	15.16	46.05	1.84	2.109812	37.29
	15.24	0.72	50	0.35	117.46	5.00	310.40	23.49	8.22	1.03	0.04	71.863	2.87	62.08	23.52	69.77	2.79	1.446078	43.60
	18.288	0.72	60	0.36	211.03	9.00	547.80	23.45	8.44	1.03	0.04	96.292	3.85	60.87	20.01	93.49	3.74	1.547085	32.44
	21.336	0.74	70	0.37	96.69	4.00	278.70	24.17	8.94	1.03	0.04	140.48	5.62	69.68	25.25	136.38	5.46	1.48368	41.25
	24.384	0.77	80	0.39	74.92	3.00	131.60	24.97	9.74	1.03	0.04	184.66	7.39	43.87	9.68	179.28	7.17	0.787453	39.67
	27.432	0.79	90	0.4	50.73	2.00	159.90	25.37	10.15	1.03	0.04	239.57	9.58	79.95	28.21	232.60	9.30	1.45039	39.50
	30.48	0.82	100	0.41	26.16	1.00	62.10	26.16	10.73	1.03	0.04	294.49	11.78	62.10	0.00	285.91	11.44	1.395283	34.00
	33.528	0.84	110	0.42	0.00	0.00	0.00	0.00	0.00	1.03	0.04	366.69	14.67	0.00	0.00	356.01	14.24	0	0.00
	36.576	0.85	120	0.43	0.00	0.00	0.00	0.00	0.00	1.03	0.04	438.89	17.56	0.00	0.00	426.11	17.04	0	0.00
	39.624	0.86	130	0.43	0.00	0.00	0.00	0.00	0.00	1.03	0.04	562.41	22.50	0.00	0.00	546.03	21.84	0	0.00
	42.672	0.86	140	0.44	0.00	0.00	0.00	0.00	0.00	1.03	0.04	685.93	27.44	0.00	0.00	665.95	26.64	0	0.00

Table A3.6: 2009, HS20 Rated, Prestressed, Northeast Region Bridges NBI Processed Data to Obtain the β and Mean Bridge Ages for Different Length of Bridges (3.048m to 42.67m) for Bridge Condition Rating of 4.

	Span (m)	Average NSHRP-368 Table B-2	Span (ft)	COV NSHRP-368 Figure B-11	Sum of LL	Count of LL & OR	Sum of OR	Mean LL	SD LL	Mean to Nom ratio DL (Nowak,1979)	COV DL (Nowak, 1979)	Mean DL	SD DL	Mean OR	SD OR	Mean NDL	SD NDL	β	Mean Age
BRIDGE CONDITION STATE 3	3.048	0.62	10	0.2	0.00	0.00	0.00	0.00	0.00	1.03	0.04	7.6684	0.31	0.00	0.00	7.45	0.30	0	0.00
	6.096	0.71	20	0.21	0.00	0.00	0.00	0.00	0.00	1.03	0.04	15.337	0.61	0.00	0.00	14.89	0.60	0	0.00
	9.144	0.74	30	0.22	143.26	6.00	310.70	23.88	5.25	1.03	0.04	23.005	0.92	51.78	22.63	22.34	0.89	1.170713	49.00
	12.192	0.75	40	0.25	0.00	0.00	0.00	0.00	0.00	1.03	0.04	47.434	1.90	0.00	0.00	46.05	1.84	0	0.00
	15.24	0.72	50	0.35	46.67	2.00	98.00	23.33	8.17	1.03	0.04	71.863	2.87	49.00	0.00	69.77	2.79	2.591294	45.50
	18.288	0.72	60	0.36	70.24	3.00	191.50	23.41	8.43	1.03	0.04	96.292	3.85	63.83	31.40	93.49	3.74	1.141568	36.00
	21.336	0.74	70	0.37	0.00	0.00	0.00	0.00	0.00	1.03	0.04	140.48	5.62	0.00	0.00	136.38	5.46	0	0.00
	24.384	0.77	80	0.39	24.76	1.00	54.40	24.76	9.66	1.03	0.04	184.66	7.39	54.40	0.00	179.28	7.17	1.71868	42.00
	27.432	0.79	90	0.4	0.00	0.00	0.00	0.00	0.00	1.03	0.04	239.57	9.58	0.00	0.00	232.60	9.30	0	0.00
	30.48	0.82	100	0.41	0.00	0.00	0.00	0.00	0.00	1.03	0.04	294.49	11.78	0.00	0.00	285.91	11.44	0	0.00
	33.528	0.84	110	0.42	0.00	0.00	0.00	0.00	0.00	1.03	0.04	366.69	14.67	0.00	0.00	356.01	14.24	0	0.00
	36.576	0.85	120	0.43	0.00	0.00	0.00	0.00	0.00	1.03	0.04	438.89	17.56	0.00	0.00	426.11	17.04	0	0.00
	39.624	0.86	130	0.43	0.00	0.00	0.00	0.00	0.00	1.03	0.04	562.41	22.50	0.00	0.00	546.03	21.84	0	0.00
	42.672	0.86	140	0.44	0.00	0.00	0.00	0.00	0.00	1.03	0.04	685.93	27.44	0.00	0.00	665.95	26.64	0	0.00

Table A3.7: 2009, HS20 Rated, Prestressed, Northeast Region Bridges NBI Processed Data to Obtain the β and Mean Bridge Ages for Different Length of Bridges (3.048m to 42.67m) for Bridge Condition Rating of 3.

APPENDIX A4 – SUFFICIENCY RATING CALCULATIONS

Sufficiency rating equations and calculations for the bridge #1100070. For the item numbers referred below, please see the National Bridge Inventory Record Format at the end of the Appendix 4.

1. Structural Adequacy and Safety (55% maximum)

   a. Only the lowest rating code of Item 59, 60, or 62 applies.

      If Item 59 (Superstructure Rating) or

      Item 60 (Substructure Rating) is ≤ 2 then A = 55%

      = 3 A = 40%

      = 4 A = 25%

      = 5 A = 10%

      If Item 59 and Item 60 = N and

      Item 62 (Culvert Rating) is ≤ 2 then A = 55%

      = 3 A = 40%

      = 4 A = 25%

      = 5 A = 10%

b. Reduction for Load Capacity:

   Calculate using the following formulas where

   IR is the Inventory Rating (MS Loading) in tons

   $B = (32.4 - IR)^{1.5} \times 0.3254$

   or

If (32.4 - IR) $\leq$ 0, then B = 0

"B" shall not be less than 0% nor greater than 55%.

S_1 = 55 - (A + B)

S_1 shall not be less than 0% nor greater than 55%.

1						
	a	Superstructure Rating	3	A	=	0.4
		Substructure Rating	6			
	b	Inventory Rating	8.2	B	=	0.38738
				S1	=	**0**

2. Serviceability and Functional Obsolescence (30% maximum)

a. Rating Reductions (13% maximum)

If #58 (Deck Condition) is $\leq$ 3 then A = 5%

= 4 A = 3%

= 5 A = 1%

If #67 (Structural Evaluation) is $\leq$ 3 then B = 4%

= 4 B = 2%

= 5 B = 1%

If #68 (Deck Geometry) is $\leq$ 3 then C = 4%

= 4 C = 2%

= 5 C = 1%

If #69 (Underclearances) is ≤ 3 then D = 4%

= 4 D = 2%

= 5 D = 1%

If #71 (Waterway Adequacy) is ≤ 3 then E = 4%

= 4 E = 2%

= 5 E = 1%

If #72 (Approach Road Alignment) is ≤ 3 then F = 4%

= 4 F = 2%

= 5 F = 1%

J = (A + B + C + D + E + F)

J shall not be less than 0% nor greater than 13%.

b. Width of Roadway Insufficiency (15% maximum)

Use the sections that apply:

(1) applies to all bridges;

(2) applies to 1-lane bridges only;

(3) applies to 2 or more lane bridges;

(4) applies to all <u>except</u> 1-lane bridges.

Also determine X and Y:

$$X\ (ADT/Lane) = \frac{\text{Item 29 (ADT)}}{\text{first 2 digits of \#28 (Lanes)}}$$

$$Y\ (Width/Lane)^{*} = \frac{\text{Item 51 (Bridge Rdwy. Width)}}{\text{first 2 digits of \#28 (Lanes)}}$$

*A value of 10.9 Meters will be substituted when item 51 is coded 0000 or not numeric.

(1) Use when the last 2 digits of #43 (Structure Type) are

not equal to 19 (Culvert):

If(#51 + 0.6 meters) < #32 (Approach Roadway Width) G = 5%

(2) For 1-lane bridges only, use Figure 3 or the following:

If the first 2 digits of #28 (Lanes) are equal to 01 and

$Y < 4.3$ then $H = 15\%$

$Y \geq 4.3 < 5.5$ $H = 15\left[\frac{5.5 - Y}{1.2}\right]\%$

$Y \geq 5.5$ $H = 0\%$

(3) For 2 or more lane bridges. If these limits apply, do not continue on to (4) as no lane width reductions are allowed.

If the first 2 digits of #28 = 02 and $Y \geq 4.9$, $H = 0\%$

If the first 2 digits of #28 = 03 and $Y \geq 4.6$, $H = 0\%$

If the first 2 digits of #28 = 04 and $Y \geq 4.3$, $H = 0\%$

If the first 2 digits of #28 $\geq$ 05 and $Y \geq 3.7$ $H = 0\%$

(4) For all <u>except</u> 1-lane bridges,use Figure 3 or the following:

If $Y < 2.7$ and $X > 50$ then $H = 15\%$

$Y < 2.7$ and $X \leq 50$ $H = 7.5\%$

$Y \geq 2.7$ and $X \leq 50$ $H = 0\%$

If $X > 50$ but ≤ 125 and

$Y < 3.0$ then $H = 15\%$

$Y \geq 3.0 < 4.0$ $H = 15(4\text{-}Y)\%$

$Y \geq 4.0$ $H = 0\%$

If $X > 125$ but ≤ 375 and

$Y < 3.4$ then $H = 15\%$

$Y \geq 3.4 < 4.3$ $H = 15(4.3\text{-}Y)\%$

$Y \geq 4.3$ $H = 0\%$

If $X > 375$ but ≤ 1350 and

$Y < 3.7$ then $H = 15\%$

$Y \geq 3.7 < 4.9$ $H = 15\left[\frac{4.9 - Y}{1.2}\right]\%$

$Y \geq 4.9$ $H = 0\%$

If $X > 1350$ and

$Y < 4.6$ then $H = 15\%$

$Y \geq 4.6 < 4.9$ $H = 15\left[\frac{4.9 - Y}{0.3}\right]\%$

$Y \geq 4.9$ $H = 0\%$

G + H shall not be less than 0% nor greater than 15%.

c. Vertical Clearance Insufficiency - (2% maximum)

If #100 (STRAHNET Highway Designation) > 0 and

#53 (VC over Deck) $\geq$ 4.87 then I = 0%

#53 < 4.87 I = 2%

If #100 = O and

#53 $\geq$ 4.26 then I = 0%

#53 < 4.26 I = 2%

S_2 = 30 - [J + (G + H) + I]

S_2 shall not be less than 0% nor greater than 30%.

2						
	a	Deck Condition	5	A	=	0.01
		Structural Evaluation	2	B	=	0.04
		Deck Geometry	4	C	=	0.02
		Underclearances	N	D	=	0
		Waterway Adequacy	7	E	=	0
		App Road Alignment	4	F	=	0.02
				J	=	**0.09**
	b	Lanes	2	X	=	650
		ADT	1300	Y	=	3.7
		Road Width	7.4			
	(1)	Structure Type	505			5
		App Roadway Width	5.8	G	=	0
		Road Width	7.4			
	(2)			H	=	0
	(3)			H	=	0.00001
				H	=	0.00001
				H	=	0.00001
				H	=	0.00001
	(4)			H	=	0
				H	=	0
				H	=	0
				H	=	0.15
				H	=	0
			G	H	=	**0.15**
	c	Highway Designation	0	I	=	0
		VC over Deck	9999	I	=	0
				I	=	**0**
				S2	**=**	**0.06**

3. Essentiality for Public Use (15% maximum)

a. Determine:

$K = (S_1 + S_2) / 85$

b. Calculate:

$$A \quad 15\left[\frac{\#29(ADT)x\#19(DetourLength)}{320{,}000xK}\right] =$$

"A" shall not be less than 0% nor greater than 15%.

c. STRAHNET Highway Designation:

If #100 is > 0 then B = 2%

If #100 = 0 then B = 0%

S_3 = 15 - (A + B)

S_3 shall not be less than 0% nor greater than 15%.

3						
	a			K	=	0.07059
	b	ADT	1300	A	=	0.0259
		Detour Length	3			
	c	Highway Designation	0	B	=	0
				S3	**=**	**0.1241**

4. Special Reductions (Use only when $S_1 + S_2 + S_3 \geq 50$)

a. Detour Length Reduction, use Figure 4 or the following:

$A = (\#19)^4 \times (7.9 \times 10^{-9})$

"A" shall not be less than 0% nor greater than 5%.

b. If the 2nd and 3rd digits of #43 (Structure Type, Main) are equal to 10, 12, 13, 14, 15, 16, or 17; then

B = 5%

c. If 2 digits of #36 (Traffic Safety Features) = 0 C = 1%

If 3 digits of #36 = 0 C = 2%

If 4 digits of #36 = 0 C = 3%

S_4 = A + B + C

S_4 shall not be less than 0% nor greater than 13%.

4						
	a	Detour Length	3	A	=	0
	b	Structure Type	505			5
				B	=	0
	c	Traffic Safety	4	4 zeros		
				C	=	0
				S4	**=**	**0**

Sufficiency Rating = S_1 + S_2 + S_3 - S_4

The Rating shall not be less than 0% nor greater than 100%.

SR =	**S1**	**+**	**S2**	**+**	**S3**	**+**	**S4**	**=**	0.1841

National Bridge Inventory Record Format

With the conversion to metric and the addition of new items it is required to expand the size of the NBI record to 432 characters. The following format will be use to submit data to the FHWA.

ITEM NO	ITEM NAME	ITEM POSITION	ITEM LENGTH/TYPE
1	State Code	1 - 3	3/N
8	Structure Number	4 - 18	15/AN
5	Inventory Route	19 - 27	9/AN
5A	Record Type	19	1/AN
5B	Route Signing Prefix	20	1/N
5C	Designated Level of Service	21	1/N
5D	Route Number	22 - 26	5/AN
5E	Directional Suffix	27	1/N
2	Highway Agency District	28 - 29	2/AN
3	County (Parish) Code	30 - 32	3/N
4	Place Code	33 - 37	5/N
6	Features Intersected	38 - 62	25/AN
6A	Features Intersected	38 - 61	24/AN
6B	Critical Facility Indicator	62	1/AN
7	Facility Carried By Structure	63 - 80	18/AN
9	Location	81 - 105	25/AN
10	Inventory Rte, Min Vert Clearance	106 - 109	4/N
11	Kilometerpoint	110 - 116	7/N
12	Base Highway Network	117	1/N
13	Inventory Route, Subroute Number	118 - 129	12/AN
13A	LRS Inventory Route	118 - 127	10/AN
13B	Subroute Number	128 - 129	2/N
16	Latitude	130 - 137	8/N
17	Longitude	138 - 146	9/N
19	Bypass/Detour Length	147 - 149	3/N
20	Toll	150	1/N
21	Maintenance Responsibility	151 - 152	2/N
22	Owner	153 - 154	2/N
26	Functional Class Of Inventory Rte.	155 - 156	2/N
27	Year Built	157 - 160	4/N
28	Lanes On/Under Structure	161 - 164	4/N
28A	Lanes On Structure	161 - 162	2/N
28B	Lanes Under Structure	163 - 164	2/N
29	Average Daily Traffic	165 - 170	6/N
30	Year Of Average Daily Traffic	171 - 174	4/N
31	Design Load	175	1/N
32	Approach Roadway Width	176 - 179	4/N
33	Bridge Median	180	1/N
34	Skew	181 - 182	2/N
35	Structure Flared	183	1/N
36	Traffic Safety Features	184 - 187	4/AN
36A	Bridge Railings	184	1/AN
36B	Transitions	185	1/AN
36C	Approach Guardrail	186	1/AN
36D	Approach Guardrail Ends	187	1/AN
37	Historical significance	188	1/N
38	Navigation Control	189	1/AN
39	Navigation Vertical Clearance	190 - 193	4/N
40	Navigation Horizontal Clearance	194 - 198	5/N
41	Structure Open/Posted/Closed	199	1/AN

ITEM NO	ITEM NAME	ITEM POSITION	ITEM LENGTH/TYPE
42	Type Of Service	200 - 201	2/N
42A	Type of Service On Bridge	200	1/N
42B	Type of Service Under Bridge	201	1/N
43	Structure Type, Main	202 - 204	3/N
43A	Kind of Material/Design	202	1/N
43B	Type of Design/Construction	203 - 204	2/N
44	Structure Type, Approach Spans	205 - 207	3/N
44A	Kind of Material/Design	205	1/N
44B	Type of Design/Construction	206 - 207	2/N
45	Number Of Spans In Main Unit	208 - 210	3/N
46	Number Of Approach Spans	211 - 214	4/N
47	Inventory Rte Total Horz Clearance	215 - 217	3/N
48	Length Of Maximum Span	218 - 222	5/N
49	Structure Length	223 - 228	6/N
50	Curb/Sidewalk Widths	229 - 234	6/N
50A	Left Curb/Sidewalk Width	229 - 231	3/N
50B	Right Curb/Sidewalk Width	232 - 234	3/N
51	Bridge Roadway Width C-To-C	235 - 238	4/N
52	Deck Width, Out-To-Out	239 - 242	4/N
53	Min Vert Clear Over Br Roadway	243 - 246	4/N
54	Minimum Vertical Underclearance	247 - 251	5/AN
54A	Reference Feature	247	1/AN
54B	Minimum Vertical Underclearance	248 - 251	4/N
55	Min Lateral Underclear On Right	252 - 255	4/AN
55A	Reference Feature	252	1/AN
55B	Minimum Lateral Underclearance	253 - 255	3/N
56	Min Lateral Underclear On Left	256 - 258	3/N
58	Deck	259	1/AN
59	Superstructure	260	1/AN
60	Substructure	261	1/AN
61	Channel/Channel Protection	262	1/AN
62	Culverts	263	1/AN
63	Method Used To Determine OR	264	1/N
64	Operating Rating	265 - 267	3/N
65	Method Used To Determine IR	268	1/N
66	Inventory Rating	269 - 271	3/N
67	Structural Evaluation	272	1/AN
68	Deck Geometry	273	1/AN
69	Underclear, Vertical & Horizontal	274	1/AN
70	Bridge Posting	275	1/N
71	Waterway Adequacy	276	1/AN
72	Approach Roadway Alignment	277	1/AN
75	Type of Work	278 - 280	3/N
75A	Type of Work Proposed	278 - 279	2/N
75B	Work Done By	280	1/AN
76	Length Of Structure Improvement	281 - 286	6/N
90	Inspection Date	287 - 290	4/N
91	Designated Inspection Frequency	291 - 292	2/N
92	Critical Feature Inspection	293 - 301	9/AN
92A	Fracture Critical Details	293 - 295	3/AN
92B	Underwater Inspection	296 - 298	3/AN
92C	Other Special Inspection	299 - 301	3/AN
93	Critical Feature Inspection Dates	302 - 313	12/AN
93A	Fracture Critical Details Date	302 - 305	4/AN
93B	Underwater Inspection Date	306 - 309	4/AN
93C	Other Special Inspection Date	310 - 313	4/AN

ITEM NO	ITEM NAME	ITEM POSITION	ITEM LENGTH/TYPE
94	Bridge Improvement Cost	314 - 319	6/N
95	Roadway Improvement Cost	320 - 325	6/N
96	Total Project Cost	326 - 331	6/N
97	Year Of Improvement Cost Est	332 - 335	4/N
98	Border Bridge	336 - 340	5/AN
98A	Neighboring State Code	336 - 338	3/AN
98B	Percent Responsibility	339 - 340	2/N
99	Border Bridge Structure Number	341 - 355	15/AN
100	STRAHNET Highway Designation	356	1/N
101	Parallel Structure Designation	357	1/AN
102	Direction Of Traffic	358	1/N
103	Temporary Structure Designation	359	1/AN
104	Highway System Of Inventory RT	360	1/N
105	Federal Lands Highways	361	1/N
106	Year Reconstructed	362 - 365	4/N
107	Deck Structure Type	366	1/AN
108	Wearing Surface/Protective System	367 - 369	3/AN
108A	Type of Wearing Surface	367	1/AN
108B	Type of Membrane	368	1/AN
108C	Deck Protection	369	1/AN
109	AVERAGE DAILY TRUCK TRAFFIC	370 - 371	2/N
110	DESIGNATED NATIONAL NETWORK	372	1/N
111	PIER/ABUTMENT PROTECTION	373	1/N
112	NBIS BRIDGE LENGTH	374	1/AN
113	SCOUR CRITICAL BRIDGES	375	1/AN
114	FUTURE AVERAGE DAILY TRAFFIC	376 - 381	6/N
115	YEAR OF FUTURE AVG DAILY TR	382 - 385	4/N
116	MINIMUM NAVIGATION VERTICAL CLEARANCE VERTICAL LIFT BRIDGE	386 - 389	4/N
---	Washington Headquarters Use	392 - 426	
	STATUS	427	
n/a	Asterisk Field in SR	428	1/AN
SR	SUFFICIENCY RATING (select from last 4 positions only)	429 - 432	4/N

Status field: 1=Structurally Deficient; 2=Functionally Obsolete; 0=Not Deficient; N=Not Applicable

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