University Transportation Research Center - Region 2

Final Report

Optimum Fund Allocations to Rehabilitate Infrastructure

Performing Organization: New Jersey Institute of Technology

March 2014

Sponsor:
University Transportation Research Center - Region 2

University Transportation Research Center - Region 2

The Region 2 University Transportation Research Center (UTRC) is one of ten original University Transportation Centers established in 1987 by the U.S. Congress. These Centers were established with the recognition that transportation plays a key role in the nation's economy and the quality of life of its citizens. University faculty members provide a critical link in resolving our national and regional transportation problems while training the professionals who address our transportation systems and their customers on a daily basis.

The UTRC was established in order to support research, education and the transfer of technology in the field of transportation. The theme of the Center is "Planning and Managing Regional Transportation Systems in a Changing World." Presently, under the direction of Dr. Camille Kamga, the UTRC represents USDOT Region II, including New York, New Jersey, Puerto Rico and the U.S. Virgin Islands. Functioning as a consortium of twelve major Universities throughout the region, UTRC is located at the CUNY Institute for Transportation Systems at The City College of New York, the lead institution of the consortium. The Center, through its consortium, an Agency-Industry Council and its Director and Staff, supports research, education, and technology transfer under its theme. UTRC's three main goals are:

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The research program objectives are (1) to develop a theme based transportation research program that is responsive to the needs of regional transportation organizations and stakeholders, and (2) to conduct that program in cooperation with the partners. The program includes both studies that are identified with research partners of projects targeted to the theme, and targeted, short-term projects. The program develops competitive proposals, which are evaluated to insure the mostresponsive UTRC team conducts the work. The research program is responsive to the UTRC theme: "Planning and Managing Regional Transportation Systems in a Changing World." The complex transportation system of transit and infrastructure, and the rapidly changing environment impacts the nation's largest city and metropolitan area. The New York/New Jersey Metropolitan has over 19 million people, 600,000 businesses and 9 million workers. The Region's intermodal and multimodal systems must serve all customers and stakeholders within the region and globally.Under the current grant, the new research projects and the ongoing research projects concentrate the program efforts on the categories of Transportation Systems Performance and Information Infrastructure to provide needed services to the New Jersey Department of Transportation, New York City Department of Transportation, New York Metropolitan Transportation Council , New York State Department of Transportation, and the New York State Energy and Research Development Authorityand others, all while enhancing the center's theme.

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UTRC-RF Project No: 49997-26-24

Project Date: March 2014

Project Title: Optimum Fund Allocations to Rehabilitate Transportation Infrastructure

Project's Website:
http://www.utrc2.org/research/projects/optimum-fund-allocation

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New Jersey Institute of Technology

Sponsor:

University Transportation Research Center - Region 2, A Regional University Transportation Center sponsored by the U.S. Department of Transportation's Research and Innovative Technology Administration

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1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
UTRC		

4. Title and Subtitle	5. Report Date
Optimum Fund Allocations to Rehabilitate Transport Infrastructure	3/31/2014
	6. Performing Organization Code
7. Author(s)	8. Performing Organization Report No.
Jay N. Meegoda and Shengyan Gao	
9. Performing Organization Name and Address	10. Work Unit No.
Department of Civil and Environmental Engineering New Jersey Institute of Technology University Heights Newark, NJ 07102-1982	11. Contract or Grant No. 49997-26-24
12. Sponsoring Agency Name and Address	13. Type of Report and Period Covered
University Transportation Research Center CCNY, Marshak Building, Suite 910, NY, NY 10031	Final Report September 2012 – March 2014
	14. Sponsoring Agency Code

15. Supplementary Notes

16. Abstract

Transportation Infrastructure in the US is in need of maintenance and rehabilitation. Preservation of road networks at an acceptable level of serviceability subject to the stringent yearly maintenance and rehabilitation (M&R) budgets is a major challenge for State Departments of Transportation (DOTs). Decision-makers are required to develop an optimum financial plan to minimize the total cost of maintenance and rehabilitation for different expected improvement of the road network performance level during a given planning horizon. This research developed a network level budget planning model, a valuable tool for decision makers of DOTs to determine the required minimum network budget and optimal budget allocations. This decision tool can compute the minimum amount of investment needed for a pavement network over a certain planning horizon to achieve specific network level condition state and recommend the best allocation of available budget among competing projects for different treatment strategies of maintenance and rehabilitation each year. This network-level optimization tool integrates a linear programming model and a deterministic roughness progression model which accounts the pavement deterioration. The effect of the uniformity of the annual optimum budget distribution on budget planning is also considered. The flexible pavement condition data from New Jersey Highway Network is used to validate the proposed model. This optimization tool demonstrates its ability to calculate the minimum budget required to achieve a desired level of pavement network condition state and to determine the corresponding optimal treatment scheduling. The developed network-level budget planning model can be used by highway agencies as a decision support tool for network level pavement management.

17. Key Words	18. Distribution Statement
Network-level Optimization, Pavement Management System, roughness progression, Maintenance & Rehabilitation.	No Restrictions.

19. Security Classification (of this report)	20. Security Classification (of this page)	21. No of Pages	22. Price
Unclassified	Unclassified	39	

TECHNICAL REPORT STANDARD TITLE PAGE

Form DOT F 1700.7 (8-69)

ACKNOWLEDGMENTS

Authors would like to thank to the University Transportation Research Center (UTRC) for funding this study. The pavement profile data and traffic data from *Long-Term Pavement Performance (LTPP) Program* are used for developing deterioration of pavement condition state. Authors also acknowledge the New Jersey Department of Transportation for sharing their pavement condition database for validating the proposed network-level budget planning model.

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EXECUTIVE SUMMARY

Infrastructure systems in the US are in need of urgent maintenance and rehabilitations. Preservation of road networks at an acceptable level of serviceability subject to the stringent yearly maintenance and rehabilitation (M&R) budgets is a major challenge for State Departments of Transportation (DOTs). However, budget constraints and other factors have often led to delaying or eliminating the application of these treatments. Such actions are expected to adversely influence the condition and performance and lead to a reduced level of service, early deterioration, and eventually needing costly rehabilitation or replacement. Decision-makers are required to develop an optimum financial plan to minimize the total cost of maintenance and rehabilitation for different expected improvement of the road network performance level with the optimal maintenance and rehabilitation scheduling during a given planning horizon.

In this research, first a pavement performance prediction model is developed for road infrastructure asset management systems. The International Roughness Index (IRI), which is a reasonable measure of the ride comfort perceived by occupants of passenger cars, is used as an indicator of pavement condition level in pavement performance prediction model. The profile data of asphalt pavement data in Long-Term Pavement Performance (LTPP) was selected as the database for developing this model. A quantitative relationship between roughness progression and contributing factors was developed and validated. Those contributing factors included cumulative traffic load, structural number, annual precipitation and freezing index. This quantitative relationship coupled with a reliability analysis based upon the Weibull model is used to estimate the remaining service life of asphalt pavements. Based on the above this research also developed a network level budget planning model, a valuable tool to DOTs' decision makers, to determine the required minimum network budget and optimal budget allocations. This decision tool can compute the minimum amount of investment needed for a pavement network over a certain planning horizon to achieve specific network level condition state and recommend the best allocation of available yearly funds among competing projects for optimal treatment strategies of maintenance and rehabilitation. This network-level optimization tool integrates a linear programming model and a deterministic roughness progression model to estimate the projected future pavement deterioration. The effect of the uniformity of the annual optimum budget distribution on budget planning is also considered. The flexible pavement condition data of New Jersey Highway Network is used to validate this model. This optimization tool demonstrates its capability of calculating the yearly minimum budget required to achieve a desired level of pavement network condition state and corresponding optimal treatment policy. This network-level budget planning model can be used by highway agencies as a decision support tool for network level pavement management.

1 INTRODUCTION

1.1 Background

Over a trillion dollars is invested in the nation's mostly aging infrastructure through various bonds and public funds. Most of that is spent on new construction and replacement of old infrastructure. It can be convincingly argued that it would be more cost effective over the long term to spend a good portion of these investments in taking a proactive course in managing the maintenance processes of the infrastructure rather than waiting and being forced to merely reacting to disruptive incidences.

Various maintenance treatments are employed by transportation agencies to slow deterioration and restore condition of pavements, bridges, culverts, signs and other physical assets. However, budget constraints and other factors have often led to delaying or eliminating the application of these treatments. Such actions are expected to adversely influence the condition and performance and lead to a reduced level of service, early deterioration, and eventually needing costly rehabilitation or replacement. Analytical tools are currently available to quantify the consequences of delayed application of maintenance treatments for highway pavements, bridges, and other assets. However, processes for using these tools to demonstrate the potential savings and performance enhancement resulting from applying maintenance treatments at the right time and also optimum allocation of funds are not readily available. Hence research is needed to develop such process. This information will help highway agencies better assess the economic benefits of maintenance actions and their role in enhancing the level of service of transportation infrastructure. In addition, incorporating these processes in asset management systems would provide a means for optimizing the allocation of resources.

1.2 Objectives

The specific objective of this research was to develop a network level budget planning model as a valuable tool to DOTs’ decision makers to determine the required minimum network level budget and optimal budget allocations of major transportation infrastructure (e.g., pavements both asphalt and concrete and drainage including culverts). This decision tool should be able to compute the minimum amount of investment needed for transportation infrastructure network over a certain planning horizon to achieve specific steady-state network conditions, and to answer which maintenance and rehabilitation (M&R) treatments should be used, where they should be applied, and at what time they should be applied to maximize the available resources and to maintain or improve the current network level average condition state in each analysis year. For the specific objective of network level budget planning, this research developed a flexible pavement deterioration curves to predict the remaining service and deterioration rate.

2 LITERATURE REVIEW

2.1 Introduction

Road asset management systems or pavement management systems (PMS) collects and monitors information on current pavement, forecasts future conditions, and evaluates and prioritizes alternative reconstruction, rehabilitation and maintenance strategies to preserve the pavement system at a predetermined level of performance. Such systems are strongly dependent upon confidence level of the prediction of future pavement condition. Hence Pavement Performance model is a key component of PMS to predict future performance of pavements. It is also a requisite for optimizing the Maintenance, Rehabilitation and Reconstruction (MR&R) policies over a planning horizon. Future performance of pavements depends on existing pavement condition as well as other variables controlling pavement deterioration such as truck traffic volume, climate and pavement structure. By reducing the prediction error of pavement deterioration, state agencies can obtain significant budget savings through timely intervention and accurate planning at different levels (Prozzi and Madanat 2004). At the project level, pavement performance prediction is needed for adequate activity planning and project prioritization. Also at project level it is needed for establishing and designing the necessary corrective actions such as maintenance and rehabilitation. At the network level, pavement performance prediction is essential for rational budget and resource allocation. In this section, the relevant research on prediction of pavement deterioration is first reviewed.

One of the primary goals of PMS is to use the periodically collected statewide condition-monitoring and maintenance-related data to analyze pavement performance and to develop cost effective maintenance and rehabilitation strategies. Some highway agencies determine the annual budget and allocate the available funds among different repair treatments based upon experience or engineering judgment, which is not always an efficient way of managing pavement networks, especially in constrained budgetary environments. Therefore, an effective pavement management system (PMS) that can find the optimal budget planning is a necessity for highway agencies to determine the best maintenance and rehabilitation strategies. Currently available optimization approaches are reviewed and used as the basis of developing such a model to meet our specific objectives.

2.2 Pavement Condition Deterioration Model

The available performance prediction models can be broadly classified into two groups; empirical and mechanistic-empirical models. Those models vary greatly in their comprehensiveness, their ability to predict performance with reasonable accuracy, and input data requirement. In spite of an enormous effort that has been made in the pavement engineering field, it is still difficult to make accurate prediction of pavement life (Molenaar 2003). This is due to the fact that it is very difficult to predict variations of contributing factors that influence the pavement performance. By including variables a roadway pavement endures, such as construction techniques, traffic and weather or aging, the modeling effort becomes even more difficult.

One of the main objectives of our transportation system is to provide a comfortable ride for users. The roadway roughness is a good indicator of ride comfort. It is believed that the general public perceives a good road as one that provides a smooth ride. Studies at the road test sponsored by the American Association of State Highway Officials showed that the subjective evaluation of the pavement based on mean panel ratings was primarily influenced by roughness. Therefore, the development of pavement roughness is a major issue for highway agencies. Roughness is a pavement characteristic reflecting the longitudinal profile along the wheel paths. According to American Society for Testing and Materials (ASTM) Specification E867-82A (1982), roughness is defined as “the deviation of a surface from a true planar surface with characteristic dimensions that affect vehicle dynamics, riding quality, dynamic loads and drainage.” The International Roughness Index (IRI) was developed by the International Road Roughness Experiment held in Brazil in 1982 under the sponsorship of the World Bank to provide a common quantitative basis on which the different measurement of roughness can be compared. The IRI was later adopted as a standard by the Federal Highway Administration (FHWA) in their Highway Performance Monitoring System (HPMS). The IRI summarizes the longitudinal surface profile in the wheel path and is computed from the surface elevation data collected by either a topographic survey or a mechanical profile-meter. It is defined by the average rectified slope (ARS), which is the ratio of the accumulated suspension motion to the distance traveled and obtained from a mathematical model of a standard quarter car traversing a measured profile at a speed of 50 mph (80 km/h). It is expressed in units of inches per mile (m/km).

The popularity of roughness index has increased over the years and now it has become a dominant criterion in describing pavement performance. Its popularity is due to the following reasons. First, as roughness reflects vertical motion of a moving vehicle and reveals both the vehicle’s response and the occupant’s perception of comfort. It not only reflects pavement performance in terms of irregularity but also relates to riding quality.

From the serviceability-performance perspective, roughness was found to be highly correlated with Present Serviceability Rating (PSR) and Present Serviceability Index (PSI). It was shown in the AASHO Road Test that over 90% of PSI was contributed by slope variance of longitudinal profile (Haas et al, 1994). Some States such as South Carolina Department of Transportation (SCDOT) uses the average wheel path roughness to calculate PSI value (Paterson, 1986), where PSI is related to IRI in exponential form ($PSI = 5.0e^{(-0.002841 IRI)}$), where IRI is in inches per mile (in./mi). From an economic viewpoint, roughness is directly related to cost of vehicle operation. A series of studies established the relationship between roughness and vehicle operating costs (e.g., Chesher and Harrison 1987; Archondo-Callao and Faiz, 1994). Also high speed profilers are now available to collect roughness data. The road profile measurements are converted into profile statistics to describe pavement performance and riding quality for practical purposes, which has time-stable and reproducible summary statistics. Lin et al. (2003) developed a correlation between IRI and pavement distress using neural networks and found that the correlation coefficient between IRI and the distress variables was 0.944, which shows that IRI can represent pavement distresses. Thus, it is feasible to use IRI as a pavement performance index. When it is difficult to collect distress measurement and imaging with limited resources, it is reasonable to rely only on the IRI to determine road sections needing maintenance or improvement. Although pavement smoothness has been recognized as one of the important measures of pavement performance, contribution from pavement structure, rehabilitation techniques, climate, traffic levels, layer materials and properties, and pavement distress to changes in pavement smoothness are not well documented.

The progression of roughness with time is a complex phenomenon (Paterson, 1987). Paterson, 1987 showed that composite distress depends on deformation due to traffic loading and rut depth variation, surface defects from spalled cracking, potholes, and patching, and a combination of aging and environmental factors. Madanat et al., 2005 analyzed the roughness data from the PMS database of Washington State and showed that the most relevant predictors of the annual increment in IRI for asphalt concrete pavements and overlays are previous year IRI value, cumulative number of ESALs, base thickness, total thickness of asphalt concrete including all overlays, age of pavement, minimum temperature in the coldest month (average over the life of the pavement) and annual precipitation (average over the life of the pavement). Perera et al., 2001 using LTPP Information Management System (IMS) showed that design and rehabilitation parameters, climatic conditions, traffic levels, material properties, and extent and severity of distress are major factors that cause changes in pavement smoothness. Wen (2011) analyzed the design factors affecting the initial roughness of asphalt pavements. By analyzing the initial international roughness index (IRI) of 442 asphalt pavements constructed from 2000 to 2004, Wen (2011) showed that the significant factors affecting initial pavement roughness were hot mix asphalt (HMA) layer thickness, project location (urban vs. rural), base type, HMA mix classification, and pavement length.

According to Madanat et al. (2005) thicker layers with higher minimum air temperatures and repair activities reduce roughness progression. In addition, the positive correlation on the yearly traffic loading and precipitation coefficients correspond to increase in IRI. The sign of correlation for the time since the last overlay indicates that the rate of roughness progression increases with age; however, the signs of the previous IRI and cumulative ESALs coefficients are negative, suggesting that ΔIRI decreases with increases in the previously recorded IRI and total traffic loading. Observations of Madanat et al. (2005) are counterintuitive; perhaps due to data measurement errors and invalid assumptions in data structure. For instance the many negative values for the ΔIRI found in the dataset are of significant concern, since the roughness should increase with time. In addition, the age of overlay do not necessarily match decreases in IRI values, and information regarding the time of overlays is not provided.

Perera et al. (2001) also obtained rate of change of IRI values of test sections classified according to ranges of all environmental zones, as well as individual environmental zones. It was found that pavements in the wet-freeze zone have the highest potential for change in roughness out of all environmental factors. In dry-freeze zones, higher rates of roughness increase were associated with higher annual precipitation, higher freezing indices and higher amounts of fine in the base layer. In dry no-freeze zones, higher rates of roughness increase were associated with higher mean annual temperatures, higher number of days above 32°C, and higher plastic limits of subgrade soil. In wet freeze zones, higher rates of roughness increase were associated with lower total pavement thicknesses (sum of surface, base and sub-base), lower annual precipitations, lower number of wet days, higher freezing indices and higher amounts of fines in base layer. Pavements located on subgrades with silt contents between 5 and 15 percent also showed high rates of increase of roughness. This is attributed to the frost heave potential of such subgrade soils. In wet no-freeze zones, higher rate of roughness increase were associated with higher number of days above 32°C, higher plasticity index of subgrade soils, higher moisture content of subgrade, higher fine contents of subgrade soils, and for sections with base layers that had higher fine contents (passing No. 200 sieve > 50 percent).

Madanat et al., 2005 developed an Empirical-Mechanistic (E-M) performance model to predict the progression of asphalt concrete pavement roughness using PMS database of Washington State. In this research, linear regression was used to develop a model that predicts incremental roughness progression based on Washington State Pavement

Management System (WSPMS) database of Washington State. Ordinary Least Squares (OLS) regression was employed to find a model that is both statistically significant and explanatory variables are causally linked to roughness progression. However, this model may not be applicable to other states with different topographic, climate regions and traffic data.

By investigating roughness characteristics of GPS sections of LTPP Information Management System (IMS), Perera et al. (1998) found a strong relationship between pavement performance and environmental factors. When roughness progression for test sections in each GPS experiment was plotted for each of the four environmental zones (i.e., wet-freeze, wet no-freeze, dry-freeze, and dry no-freeze), there were distinct trends in roughness progression between four zones. However, in this research insufficient enough data points were used due to the fact that at the time of the publication only limited number of data points was available in the LTPP database. Also there was no statistical analysis in this study to indicate confidence level of these models. According to the analysis by Perera et al. (1998), increase in IRI has an exponential growth as described as $IRI(t) = IRI_0 e^{r_0 t^A / B}$, where, t is the pavement age while, A and B are the growth rate constant and model constant, respectively.

Hence in this research the available profile data in the LTPP database are used to predict the progression of the IRI due to contributing factors such as climatic conditions, traffic levels, and structural properties of pavement. Then the performance model will be used long-term pavement management. Please note that the scope of research is limited to modeling one particular type of pavement from this database specifically original asphalt pavements (i.e. new asphalt pavements without rehabilitation).

2.3 Network Level Budget Planning Model

There are two primary levels of optimization: project level and network level. At project level, PMS determines optimal techniques for repairing specific segments of highway pavements or projects. This level of management involves assessing causes of pavement deterioration, determining potential solutions, assessing effectiveness of alternative repair techniques, and selecting solution and design parameters. Network-level PMS is a set of planning tools and techniques that take into consideration pavement condition and repair work that need to be applied to all highway segments being managed by a transportation agency. At this level the main objectives are to establish network-level repair policies, budget requirements, repair priorities and schedules. In a typical “top down” PMS, network-level decision-making is first performed, followed by project-level decision-making (Mbwana, 2001). The detailed project-level decision-making is essentially guided by the network-level (long-term) strategies while observing budget restrictions and network priorities. Hence, a network-level optimization tool, which could generate the best long-term maintenance and rehabilitation strategies for the entire pavement network is necessary for highway agencies.

The network-level pavement management can be divided into two categories: budget planning and budget allocation (Haas, et al. 1994; Zhang 1996). Budget planning involves prioritization to identify the candidate pavement sections for M&R treatment in each year of the planning horizon in such a way that the required budget is minimized and certain minimal pavement condition states are mainatined. For budget allocation, the M&R benefits are maximized subject to established annual budgets. The development and implementation of efficient and effective network-level optimization modules for budget planning and allocation is one of the major challenges faced by the highway maintenance agencies.

The network-level optimization takes a global view of the entire pavement network, and focuses on the distribution of overall condition state and budget allocations (Huang 2004). The network-level optimization is capable of estimating the total length of pavements to be repaired by the applicable treatments, and determining the budget required to maintain the whole pavement network above a certain acceptable condition state (Bako et al. 1995). The optimization tool should be able to address two critical issues of the decision-maker: (1) determining the minimum budget required to achieve a desired level of pavement network condition, and (2) maximizing the improvements of pavement network condition with a given amount of maintenance and rehabilitation dollars. Previous optimization approaches have, in particular, two essential elements, namely optimization algorithms and pavement performance prediction models. Such elements could vary remarkably depending on the researchers' approach to the problem (De la Garza et al. 2010)

Linear and integer programming are two optimization algorithms utilized by most pavement optimization models. Selecting an appropriate algorithm is important in establishing an efficient optimization tool. Generally, the linear programming model is used at the network-level, and the integer programming model is applied at the project-level (Wang 2011). Linear programming is a powerful mathematical technique for dealing with the problem of "allocating limited resources among competing activities in a best possible way" (Hillier and Lieberman 2010). All functions and constraints of a linear programming model are required to be linear functions. A linear programming model is generally utilized in a "macroscopic" approach for pavement optimization at the network-level (Abaza 2007). Due to the efficient solution algorithms and the rapid progress in computation power, linear programming models can be solved within an acceptable time period even if the problem size is quite large (Hillier and Lieberman 2010).

An accurate and reliable pavement condition prediction model is essential for a pavement optimization model (Akyildiz 2008). There are two main types of prediction models, namely deterministic models and probabilistic models. Currently, the probabilistic model based on the Markov theory is the most frequently used approach (Golabi et al 1982, Chen et al 1996, Li 1998, Abaza 2007, Gao 2010, Wang 2012). The critical component of this type of model is the Markov transition probability matrix. Most optimization models use two transition matrices for each repair treatment: one for condition improvements in the first year the treatment is conducted, and the other for the deterioration trend after the treatment (Gao, et al., 2010). The memory-less property or constant failure rate assumption is a critical assumption of Markov transition which means the transition probabilities from a particular state are only dependent on the current state and not the historical sequence of actions taken to arrive at the current state. As argued by Meegoda and Abdel-Malek (2010), it may not be a reasonable in Markov transition for mechanical components that the failure probability remains unchanged whether one knows the history of deterioration or not. For the deterministic type, the relationship can be, mechanistic, empirical or a combination of both. The mechanistic approach uses fundamental theories of pavement behavior to model deterioration trends. This approach produces models that are more easily transferable to different pavements conditions, but are usually computationally intensive. Empirical models are less structured, relying mostly on statistical analysis of locally observed deterioration trends. Empirical models may not be transferrable to other locations where environmental conditions are different. The combined mechanistic–empirical approach attempts to create models with moderate data requirements and that can be transferred to different pavement conditions with different environmental parameters.

There are studies that have investigated the relationship between deterministic and probabilistic pavement management models. For example, Li et al. (1997) developed a method to convert a deterministic model into a Markov model. Bekheet et al. (2008) compared the performance of a deterministic pavement prediction model and a Markov-based system. Validation was made of both systems against actual measured pavement condition data. The results showed that both systems performed well. This research uses deterministic pavement deterioration model based on the historical data to capture the time variant deterioration rate of pavements. This approach is believed to be more appropriate to site specific road network for which the deterioration rate is easily obtained subject to known weather and traffic conditions. In this research the roughness progression model is developed in the following section, which is also a mechanistic–empirical model, that will be employed to describe the deterioration rate of pavement condition state due to contributing factors such as climatic conditions, traffic levels, and structural properties of the pavement.

Ideally, network-level optimization modules should provide decision support information to answer which M&R treatments should be used, where they should be applied, and at what times they should be applied to maximize the available resources and to maintain or improve the current network level average condition state. This research aims at the development of such a budget-planning approach for network-level pavement management.

2.4 Summary

In summary, the existing literature related to the developing of pavement condition deterioration model and network level budget planning model have been briefly reviewed. According to the review on pavement condition deterioration prediction, IRI was found to be the best and hence selected as the indicator of pavement performance level. This research uses the available profile data in the LTPP database to predict the progression of the IRI due to contributing factors such as climatic conditions, traffic levels, and structural properties of pavement. Then the performance model is applied in the pavement management. According to the review on the network level optimization model, both Linear Programming algorithm and deterministic prediction model for predicting the pavement condition deterioration can be employed in the model development. In addition, there is limited information on the effect of the annual budget distribution on the budget planning, although the budge uncertainty has been considered during budget allocation by Gao, et al. (2010), hence the effect of uniformity of the annual budget on the minimum required budget is also investigated.

3 DEVELOPMENT OF DETERIORATION MODEL FOR PAVEMENT CONDITION

3.1 Introduction

In this section, the development of the pavement condition deterioration model is presented. The methodology of developing the prediction model, the database of historical pavement profile data for the model development and the processing of the data are elaborated first. Following the data processing, the developed model in normalized format is presented. Finally, the application of the model in pavement condition assessment and pavement condition prediction is discussed.

3.2 Methodology

The initiation of pavement distress is highly variable because distresses occur at different times at various locations along a road. Therefore, the time of failure as well as pavement distress should be represented by a probability density function rather than by a point estimate. As regression models can only provide point estimates, Duration models (or Hazard Rate models) are better suited to predict a survival function for the time of failure of an element or system (Madanat et al., 2005). Moreover, point estimate models of the initiation of pavement distress lack the structure, as well as the physical significance, offered by duration models.

Define T as the time (or Cumulative ESALs) to initiate cracking of a pavement, where T is a random variable that takes values in the interval (0, ∞). It has a cumulative distribution *F(t)* and a density function *f(t)* where *F(t)* is given by:

$$F(t) = \int_0^t f(s)ds \qquad (T \leq t) \tag{1}$$

The probability of deterioration after time t is given by the survival function:

$$S(t) = 1 - F(t) \qquad (T \geq t) \tag{2}$$

Define *g(t)* as the probability that a pavement deteriorates in the next small interval, *Δt*, given it lasts at least until time t:

$$g(t) = F(t) \qquad (t \leq T < t + \Delta t) \tag{3}$$

Then instantaneous rate of change of *g(t)*, defined as the Hazard Rate Function, *h(t)*, is given by:

$$h(t) = \lim_{\Delta t \to 0} \frac{g(t)}{\Delta t} \tag{4}$$

The hazard rate quantifies the instantaneous risk that the pavement sections deteriorate at time *t*.

According to Meegoda et al., 2008, the Weibull distribution can be used for reliability modeling since other distributions, e.g., exponential, Rayleigh, and normal are special cases of the Weibull distribution, where time to failure distribution, *f(t)* is given as:

$$f(t) = \alpha\gamma t^{\gamma-1} e^{(-\alpha t^{\gamma})} \tag{5}$$

Where α and γ are positive values and are referred to as the characteristic life and the shape parameters of the distribution, respectively. The hazard rate (or failure rate) function, *h(t)* for the Weibull distribution can be described by:

$$h(t) = \alpha\gamma t^{\gamma-1} \tag{6}$$

When γ >1, the hazard rate is a monotonically increasing function with no upper bound that describes the “wear-out” region i.e., higher condition states with limited remaining service life. The hazard rate becomes constant for γ = 1, which is typical for the design life. When γ <1, the hazard rate is a monotonically decreasing function that describes the early failure-rate region. These failures are typically attributed to manufacturing defects or improper installation and are not considered in this analysis.

The reliability, R(t) for the Weibull distribution is given by:

$$R(t) = e^{-\alpha t^{\gamma}} \tag{7}$$

Where the cumulative distribution function of failure *F(t)*, which is the compliment of *R(t)* is given by:

$$F(t) = 1 - R(t) = 1 - e^{-\alpha t^{\gamma}} \tag{8}$$

The survival function is given by:

$$S(t) = 1 - F(t) = e^{-\alpha t^{\gamma}} \tag{9}$$

The flexibility of the Weibull model enables it to describe the failure rate of real world failure data, and hence will apply in this research to estimate the roughness progression of asphalt pavements.

As road roughness or IRI is chosen as an indicator of a road’s functional performance in this research, IRI value increases with the pavement age and other environmental parameters. Hence the roughness progression can be described by a survival function. The time dependent pavement roughness can be estimated by multiplying Eq. 9 by initial roughness which is expressed in Eq. 10. As there is a positive relation between roughness and age similar to that proposed by Perera et al. (1998), the negative sign in Eq. 9 is replaced by a positive one.

$$IRI = IRI_0 \exp(\alpha t^{\beta}) \tag{10}$$

Where α represents the deterioration rate of the asphalt pavement. As mentioned in the introduction, the main factors contributing to pavement deterioration include cumulative traffic load, structural number, freeze index and annul precipitation. According to mechanistic-empirical models, the relationship to predict rutting in asphalt mixtures, which is based upon a field calibrated statistical analysis of repeated laboratory load permanent deformation tests, is of the form:

$$\frac{\varepsilon_p}{\varepsilon_r} = a_1 T^{a_2} N^{a_3} \tag{11}$$

Where:

ε_p = accumulated plastic strain at N repetitions of load (in/in)

ε_r = resilient strain of the asphalt material as a function of mix properties, temperature and time rate of loading (in/in)

T = temperature (Degree F)

N = number of load repetitions

a_1, a_2, a_3 = non-linear regression coefficients

The power relationship between the strain and affecting factors in Eq. #11 can be incorporated into α value in Eq. #10. Hence, the following form shown in Eq. #12 can be tentatively considered to include all the factors affecting the rate of roughness progression. It should be noted here that to develop the multiplicative relationship among the affecting factors roughness prediction, α should be expressed as the sum of these factors in exponential form as shown in Eq. 10.

$$\alpha = \frac{CL^a}{SN^b} c + FI^d e + AP^f g \tag{12}$$

Where:

CL = cumulative traffic load (KESALs/year)

SN = structural number, which is an appropriate indicator of the pavement characteristics because it combines the layer thickness and materials into one number for each pavement. The SN value is back-calculated using falling weight deflecto-meter readings (Turner-Fairbanks Highway Research Center, 1997).

AP = annul precipitation

a, b, c, d, e, f, g = model parameters

FI = freezing index.

The following equation is used to compute monthly or annual freezing index (Elkins et al., 2011):

$$FI = \sum_{i=1}^{n} (0 - T_i) \tag{13}$$

Where: = freezing index, degrees Celsius (°C) degree-days

T_i = average daily air temperature on day *i*, °C

n = number of days in a specified period with below freezing average daily temperature

i = number of days below freezing

When using this equation, only the days where the average daily temperature is below freezing are used. Therefore, the freezing index is the negative of the sum of all average daily temperatures below 0°C within a given period.

As most of the performance data do not report the initial value, it is preferable to use an incremental model to specify pavement performance. Hence roughness progression between test sections can be evaluated by obtaining a rate of change of roughness for each section as expressed in Eq. 14.

$$\ln IRI_{i+1} - \ln IRI_i = (\alpha_{i+1} t_{i+1}{}^{\beta} - \alpha_i t_i{}^{\beta}) \tag{14}$$

The premise of the above model is the incremental loadings, along with other explanatory variables (e.g.: structural design and the prevailing environmental conditions), that cause incremental changes in condition state of the pavement. In addition to being intuitive from a physical perspective, the incremental model is more appealing because managerial decisions are usually based on incremental predictions. In addition, from the pavement management perspective, an incremental form is beneficial, since condition state data are usually available on a regular basis and predictions are usually only desired for the future time periods.

After obtaining the model parameters, the initial roughness value can be estimated by back calculating the time-sequence IRI values at i_{th} age of the asphalt pavement as follows:

$$IRI_0 = IRI_i / \exp(\alpha t_i{}^{\beta}) \tag{15}$$

3.3 Database Descriptions and Data Processing

The data collected for the Long Term Pavement Performance (LTPP) study was used to investigate the effect of factors described in Eq. #s 10 and 12 to develop a model to predict the roughness propagation. The Long Term Pavement Project Database developed by the Federal Highway Administration had data sought for the above analysis. This program has been collecting data since 1989 at hundreds of sites across the United States. The LTTP database gathers data covering a wide-range of variables and employs precise techniques for data collection. These factors make this database an excellent choice for use in developing pavement models.

The LTPP program consists of two complementary programs, the General Pavement Studies and Specific Pavement Studies. The General Pavement Studies (GPS) is a study of the performance of in-service pavement test sections that were either in their original design phase or in their first overlay phase. Each GPS section is 152 m long. The GPS sections generally represent pavements that incorporate materials and structural designs used in standard engineering practice in the United States. The GPS test sections had been in service for some time when they were accepted into the LTPP program. Roughness data collection at these test sections have been performed at regular intervals after the test sections were accepted into the LTPP program. However, the initial IRI values of these test sections are not known. The SPS experiments were designed to study the effect of specific design features on pavement performance. Each SPS experimental test site consists of multiple test sections, each of which is 152 m long. New pavements were constructed for SPS-1 experiments, and profile data were collected on these pavements after construction. Thereafter, these test sections have been profiled at regular intervals. For these sections, the roughness of the pavement when it was opened to traffic, as well as roughness data collected at approximately annual intervals are available. The data collected for the LTPP program are stored in the LTPP Information Management System (IMS) database. This data can be divided into the following categories: inventory, maintenance, climatic, monitoring, traffic, materials testing, and rehabilitation. For this research, the mean IRI of the test section, which is the average IRI of the left and right wheel paths, was used to characterize the roughness. For a specific test date at a test section, the LTPP database generally has five IRI values that have been obtained from five profile runs. The mean IRI values of these multiple profile runs were averaged to obtain the roughness for that specific test date. It is worth mentioning that during the data processing of IRI, there were some isolated points with extreme variance. As IRI should steadily increase with the time unless there were external interferences, the abnormal breaks in the time-sequence of the IRI data of each section were most likely due to the rehabilitation or maintenance activities or due to the data collection errors. Hence, the maintenance records of the pavement were checked for such data to ensure data reliability. After data from all fifty States were thoroughly investigated, the States having erratic data were filtered to eliminate poor quality data. That resulted in data from 13 States (AL, CA, CT, ID, IL, ME, MT, NM, NY, PA, TX and WY) form GPS database for model development. The SPS data from five States (AL, AR, AZ, FL and IA) were extracted for the model validation. There were wide ranges of each affecting factor to account for the generality of the model with the range of each as follows:

0<pavement age<21, 0.6m/km<International Roughness Index<2.0m/km, 2.5<Structure Number<8, 0<Freezing Index<1172, 220mm<Annual Precipitation<1600mm, and 15KESAL/year<traffic load<900KESAL/year.

Different data quality levels were also defined: Level 1: for the new pavement without maintenance or rehabilitation during data collection; Level 0: for the newly reconstructed/rehabilitated pavement (structural overlay and full depth reclamation) which can be treated as new pavement; level -1: for the pavement with minor but missing maintenance records; and Level -2: for the pavement with unclear record of traffic load. In this research, only the data of the first two levels are used in the model development.

Four environmental zones were considered in this analysis, and they correspond to the four environmental zones that are defined in the LTPP program, which are wet-freeze, wet no-freeze, dry-freeze and dry no-freeze. The boundary between wet and dry regions was taken as 508 mm of annual precipitation, and the boundary between the freezing and non-freezing zones was taken as an annual freezing index of 89°C days. Hence the annual precipitation and freezing index of each section can be obtained. In addition, as the LTPP database has historical as well as monitored traffic data, the cumulative traffic load was estimated as the average of both the historical and monitored traffic data. The historical traffic data in LTPP database refers to data gathered from date a pavement section was opened to traffic to the date when collection begun. The LTPP traffic monitoring data is collected after 1989, which are site specific and are intended to include actual traffic load over each 152 meter (500 foot) long sections.

3.4 Model Development and Validation

As aforementioned, the LTPP program consists of two complementary programs, the General Pavement Studies (GPS) and Specific Pavement Studies (SPS). The data from GPS was be used for model development, the data from SPS was employed for model validation. As the scope of this research was limited to modeling asphalt pavement on granular base that consists only of its original structure (i.e. the pavement was never rehabilitated), only the corresponding data in GPS1 and SPS1 were used in this research and the original roughness data are presented in Figs.1 and 2, respectively.

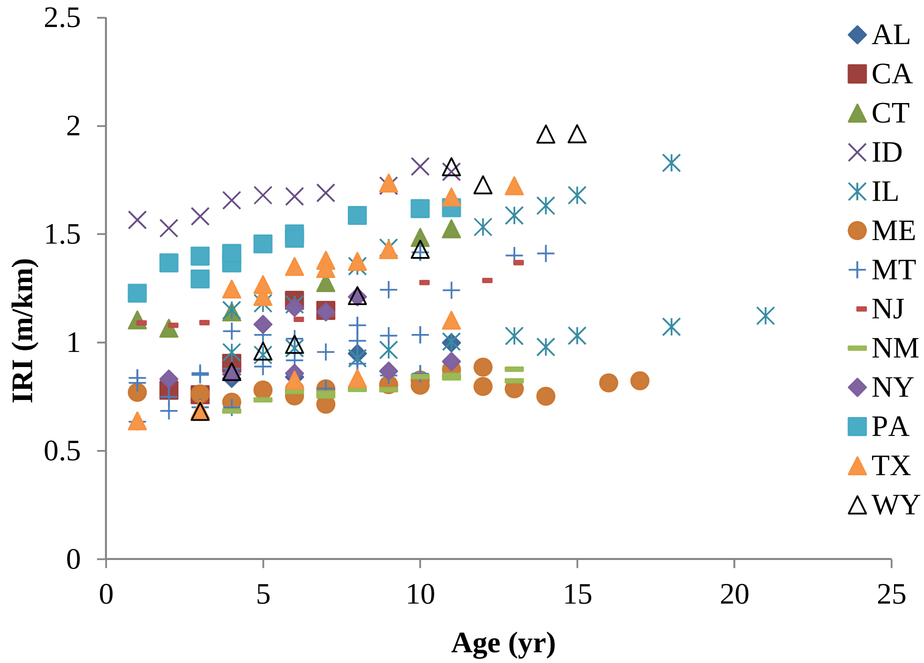

Fig. 1 Selected roughness data from GPS1 sections.

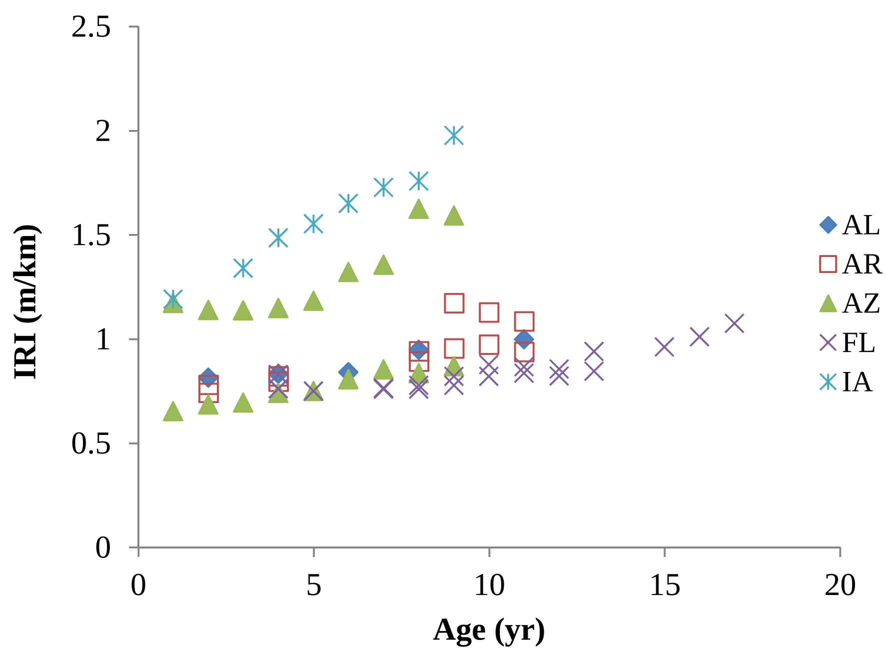

Fig. 2 Selected roughness data from SPS1 sections.

To apply the processed GPS data to equation numbers 12 and 14, a computer optimization program was developed to estimate the eight model parameters. The optimization of the fit was achieved with the function lsqcurvefit based on the Levenberg–Marquardt algorithm in Optimization Toolbox of MATLAB. When using one starting point for the optimization only localized best-fit combination of parameters is found. To obtain global minimum, the range of each parameter is defined and a variety of starting points were used for the optimization. Relative root mean square (RMS) is used to measure the error between the fitted data and the collected data. The fitted model parameters with the minimum RMS was selected as the model parameters and the model can be expressed as:

$$\ln IRI_{i+1} - \ln IRI_i = (\alpha_{i+1} t_{i+1}{}^{0.9715} - \alpha_i t_i{}^{0.9715}) \tag{16}$$

$$\alpha_i = \frac{CL_i{}^{2.6E(-14)}}{SN^{1.992}} 0.4132 + FZI^{3.8086} 2.34E(-14) + AP^{2.53E(-14)} 1.54E(-2) \tag{17}$$

From Eq. 17, it can be observed the pavement deterioration rate varies for different climate regions. In addition, for pavements in same climate region, it deteriorates faster for those with higher traffic loads and lower structure number. All these observations are consistent with the observed pavement deterioration. A comparison of measured and simulated IRI values for the GPS sections are presented in Fig. 3.

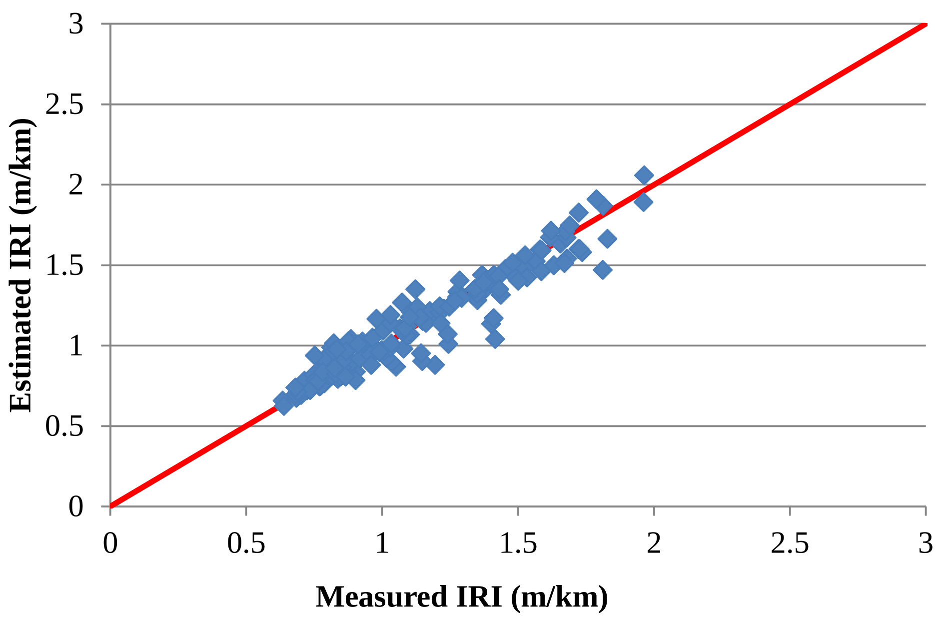

Fig. 3 Comparison of simulated and measured IRI for GPS sections.

Then, pavement roughness data of SPS sections of asphalt pavement with granular base are employed for model validation. The validation result is presented in Fig. 4.

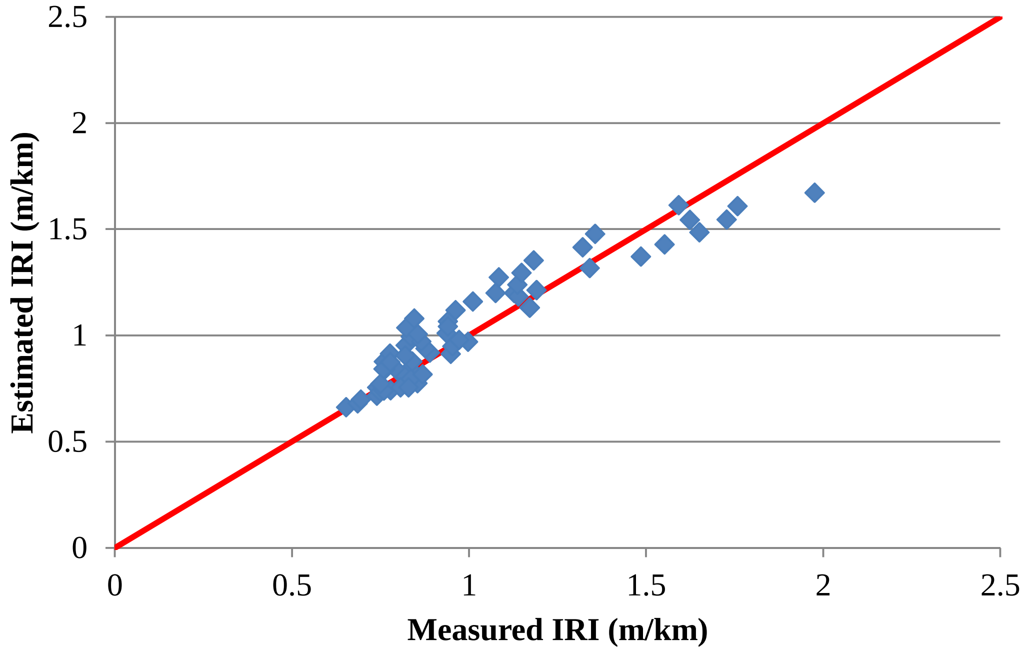

Fig. 4 Validation of the developed IRI model by SPS section data.

3.5 Pavement Condition Assessment and Model Application

To normalize the pavement performance of asphalt pavement with different structural properties, under various traffic loads and weather condition, instead of using pavement age of the of x axis, the ratio of initial IRI to that at time t is plotted. The normalized result is presented in Fig. 5.

Figure 5 shows the variation of ratio of initial IRI and IRI at certain pavement age with the normalized pavement age. From this figure, once the pavement age and the corresponding cumulative traffic load, weather data (freezing index and annual precipitation) and structural data are collected, the ratio of the initial to current IRI value can be determined. If the initial IRI can be provided or estimated based on Eq. 15, the current IRI can be estimated from Fig. 5. Also the y axis can be converted to different condition levels as defined below:

Condition 1: IRI <1.0 m/km (<64in/mile)
Condition 2 : IRI 1.0~1.263 m/km (64~80in/mile)
Condition 3: IRI 1.263~1.8 m/km (80-114 in/mile)
Condition 4 : IRI 1.8~2.352 m/km (115-149 in/mile)
Condition 5 : IRI >2.352 m/km (>150 in/mile)

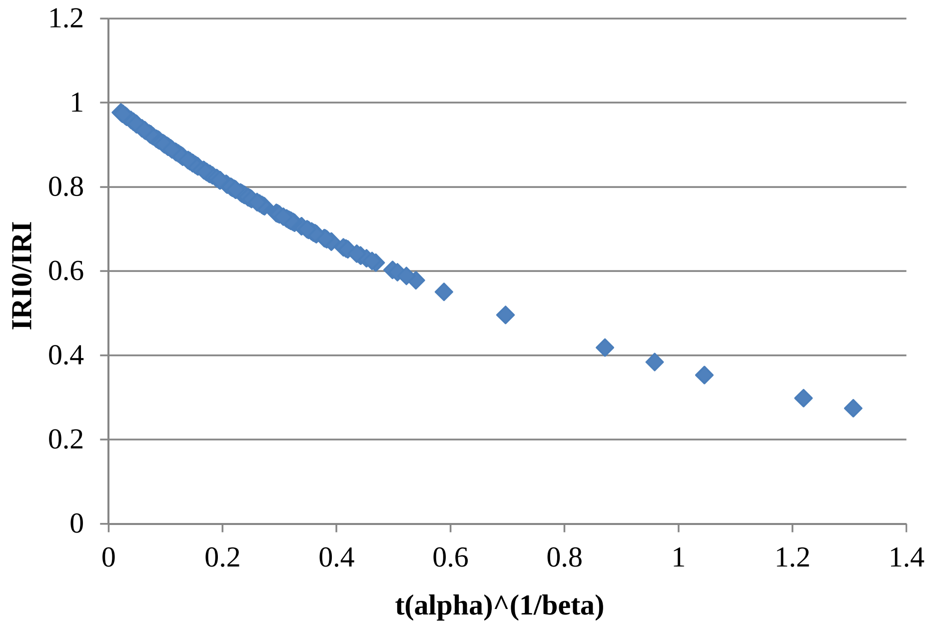

Fig.5 Normalized IRI progression model.

To convert the y axis of Fig. 5 into five condition levels, the typical initial IRI should be determined. Perera et al. (2001) performed an analysis to study the early-age IRI characteristics of SPS-1 projects, where initial IRI values for the SPS-1 projects were obtained at varying times after construction. Therefore, the initial IRI may not necessarily correspond to the IRI that is obtained immediately after construction. Hence the term initial IRI corresponds to early-age IRI. All SPS-1 projects with measured IRI values less than two years after construction were used in this analysis. The cumulative frequency distribution curve shows that 175 mm asphalt concrete (AC) surfaces have lower IRI values than 100 mm AC surfaces. The results show that an IRI value of less than 0.8 m/km for 40 percent of sections with 100 mm AC surface and 55 percent of the sections with 175 mm AC surface. An IRI value of less than 1.0 m/km was obtained for 75 percent of the sections with 100 mm AC surface and 85 percent of the sections with 175 mm AC surface. Table 1 summarizes the average early-age IRI values of asphalt pavements. In this research, typical initial IRI value is taken as 0.9 m/km.

Table 1 Average and standard deviation of early-age IRI: SPS-1

AC Thickness (mm)	IRI (m/km)	
	Average	Std. Dev.
100	0.88	0.21
175	0.82	0.18

The ultimate goal of developing deterioration curve is to assist pavement engineers and decision makers in effectively managing pavement systems. It has been pointed out that pavement design is closely related and sensitive to model accuracy (Small and Winston, 1988). It is known that, one of the major functions in an integrated PMS is to select the cost-effective M&R treatments for each pavement in the network over the analysis period. Pavement maintenance may consist of relatively inexpensive, corrective types of treatments to address specific problems such as localized potholes, or it may take the form of preventive action, such as crack sealing to reduce rate of deterioration. Pavement rehabilitation usually involves a larger amount of investment for extending the pavement life when it has reached some limit of acceptability. The selection of appropriate treatments for long term pavement preservation depends on the current state of the pavement which can be obtained with pavement age, weather data and traffic data. For instance, for an 8 year old asphalt pavement in the dry-non-freeze region with annul precipitation of 400mm, freezing index of 12, structural number of 3.5 and estimated cumulative traffic load of 350 kESALs. The t(alpha)^(1/beta) can be calculated which is equal to 0.4. According to normalized roughness progression curve in Fig.5, the ratio between the current roughness of the pavement and its initial roughness is 0.63. Then, by further knowing the initial roughness the current roughness can be estimated. However, if the initial roughness is not available, two approaches can be employed: 1): using reported IRI data in the past years to predict the initial roughness by Eq. 15; 2): if no reliable data have been stored, 0.9m/km can be considered as the initial IRI value. Then, the condition state can be determined according to the definition given above. In this example, assuming the initial roughness of 0.87m/km the current roughness can be obtained as 0.87/0.63=1.38m/km, and hence the pavement is at condition state 3.

Once the condition level is known, the selection of appropriate pavement treatment can be determined according to the proposed effectiveness of the pavement treatments shown in Table 2. Table 2 is developed based on the performance evaluation of 20 types of rehabilitation and preservation treatments of 180 projects in 6 States for four climatic zones reported in the Long Term Pavement Performance (LTPP) database (Federal Highway Administration, 2010).

Table 2 Suggested Pavement Treatments vs. Condition Levels

Condition State	Description	Treatment	Improved Condition State
1	In excellent pavement condition state, high ride quality without crack	Do nothing	-
2	light to moderate cracking, “raveling”, polishing or flushing with good Ride quality (>70) (or IRI<80) and slight rutting<3mm	1-fog seal	1
		2-chip seal	1
		3-crack sealing	1
3	Light cracking with moderate rutting <12mm, Ride quality<70 (or 80<IRI<114)	4-slurry seal (for light	2
		5-thin overlay (for moderate	1
4	Moderate to severe distress (Centerline joint deterioration, more transverse and multiple cracking Severe alligator cracking), Fair ride quality (with rough IRI (115<IRI<149)), moderate to severe rutting (12mm<rutting < 25 mm)	6-cold-in-place	3
		7-hot-in-place	2
		8-mill and resurfacing	2
		9-micro surfacing	3
5	Extremely more longitudinal & transverse cracks, and patching C/L joint deterioration), Poor IRI (>150), severe rutting (>25mm)	10- structural overlay	1
		11-full depth reclamation	1
		12-whitetopping	1

4 DEVELOPMENT OF NETWORK LEVEL BUDGET PLANNING MODEL

4.1 Introduction

In this section, the formulation of a network-level budget planning model minimizing the total cost of maintenance and rehabilitation of a pavement network over a given planning horizon with a specific pavement condition state at the network level is introduced.

4.2 Model Parameters

To perform the network-level optimization of maintenance & rehabilitation for a road network, the pavement network is divided into three-sub-networks according to the pavement types (flexible, concrete, and composite). Zhang and Murphy (2012) suggested the following treatments for each sub-network:

- Routing maintenance: sealing cracks, patching, pothole repair, level up, etc.
- Preventive maintenance: seal coats (chip seals), thin overlays, micro-surfacing
- Light rehabilitation: 2in.<overlays<3 in., widening pavement and seal coat, base repairs and seal coat, mill, seal and thin overlay
- Medium rehabilitation: 3in.<overlays<5in., mill and inlay (mill and fill), mill, stabilize base and seal, level up and overlay, base repairs and overlay
- Heavy rehabilitation: full pavement reconstruction by adding base and overlay or seal

For each group of pavements, it can have different pavement condition states based on the collected roughness and distress data or other pavement condition scores (*e.g.*, excellent, good, fair, poor and very poor). Usually, the routing maintenance and preventive maintenance can be merged into one work type. Each pavement condition class may be recommended for one of the following five repair treatments (1, Do Nothing; 2, Preventive Maintenance; 3, Light Rehabilitation; 4, Medium rehabilitation; and 5, Heavy Rehabilitation or replacement). To facilitate tracking the improvement of the road network value throughout the planning horizon, the existing performance value of each condition level for each group is introduced (*e.g.*, excellent=0.95; good=0.8; fair=0.65; poor=0.5; and very poor=0.35). Two assumptions are employed to simplify the optimization: (1) the total length of the pavement network remains constant and (2) the pavement types do not change for any pavement section during the analysis period. The general definitions of the sets and parameters used in the model are described as follows:

<u>Sets:</u>

T: set of funding periods (the planning horizon), and T=$\{1, 2, ..., T\}$;

S: set of road groups with similar characteristics and S=$\{1, 2, ..., S\}$;

I: set of pavement condition levels and I=$\{1, 2, ..., I\}$;

M: set of M&R treatments and M=$\{1, 2, ..., M\}$.

<u>Parameters:</u>

A_{si}: existing pavement performance value for road group *s* at condition level *i*;

C_{sm} : unit cost of applying m th treatment to road group s ;

E_{sijm} : the maintenance effectiveness matrix to determine the transition from condition state i to j when the m^{th} treatment is applied to the road group s; 0 for no transition between i and j, 1 for transition from i to j.

L_s : total length of the road group s;

D_{sim} : Deterioration rate for pavement group s at condition level i with treatment m. Here, it is reasonably assumed that condition level i can only drop to the adjacent condition level which is different from that defined in Markov transition probability. In addition, deterioration rate is also considered as a function of treatment for a "perpetual pavement". For Perpetual Pavements, its service life is to be extended by routing maintenance without removing the road structure for reconstruction. Actually, routing maintenance cannot alleviate the deterioration of the sub base. To take into account this, each type of treatment can be assigned a specific deterioration rate.

X^* : the expected percentage of improvement of overall network performance value at the end of the planning horizon;

Y_s^* : the expected percentage of decrease of the worst condition level of group s at the end of the planning horizon.

<u>Decision variables:</u>

X_{tsim} : proportion of the road group s in condition state i that receives the *mth* treatment at year *t;* X_{tsi} : proportion of the road group s in condition state i at year *t,* $\sum_{m=1}^{M} X_{tsim} = X_{tsi}$;

4.3 Model Development

The objective function is to minimize the average M&R cost of the pavement network to achieve a certain condition level goal (note: the inflation and cost of borrowing are not considered in this research):

$$\text{Min } Z = \frac{1}{T}\sum_{t=1}^{T}\sum_{s=1}^{S}\sum_{i=1}^{I}\sum_{m=1}^{M} X_{tsim} L_s C_{sm} \tag{18}$$

Constraints:

$$0 \le X_{tsim} \le 1 \tag{19}$$

$$\sum_{s}\sum_{i}\sum_{m} X_{tsim} = 1 \qquad \forall t \in T \tag{20}$$

$$\sum_{m=1}^{M} X_{1sim} = X_{1si} \qquad \forall s \in S,\ i \in I \tag{21}$$

$$\sum_{m=1}^{M} X_{tsjm} = \sum_{m=1}^{M}\sum_{i=1}^{I}\Big((1-D_{sjm})E_{sijm}X_{(t-1)sim}\Big) + D_{s(j-1)1}X_{(t-1)s(j-1)1} \qquad \forall\, t\geq 2,\ j\in[2,I],$$

$$\sum_{m=1}^{M} X_{tsjm} = \sum_{m=1}^{M}\sum_{i=1}^{I}\Big((1-D_{sjm})E_{sijm}X_{(t-1)sim}\Big) \qquad \forall\ t\geq 2,\ j=1, \qquad (22)$$

$$\sum_{s=1}^{S}\sum_{i=1}^{I}\Big[A_{si}X_{Tsi}L_s - (1+X^*)A_{si}X_{1si}L_s\Big] \geq 0 \qquad (23)$$

$$\sum_{s=1}^{S}(A_{sI}X_{TsI}L_s - (1-Y_s^*)A_{sI}X_{1sI}L_s) \leq 0 \qquad (24)$$

Eqs. 19 and 20 define the range of the decision variable. Eq. 19 ensures that all variables in the optimization are non-negative while Eq.20 ensures the entire pavement network is divided into many proportions and each proportion is represented by a decision variable.

Eq.21 represents the initial condition constraints which pass the values of the current pavement condition state distribution for each pavement group to the optimization model.

Eq.22 defines the transition of performance condition after each treatment which integrates the pavement deterioration. From the second analysis year on, the proportion of pavement group *s* and condition *j* with treatment *m* in year *t* is derived from two parts of pavement at various condition states in year *t-1*: one part is from the deterioration of the part at a better condition level with no new treatment during *t-1* while the other part is those jump to the condition *j* with various treatments during the previous year. For the level 1, as there is no better condition level in the network work, the proportion of level 1 at year *t* is only from those that are improved to condition *j* after treatment *i* during the year *t-1*.

Eq.23 assures the overall network performance level meet the requirement at the end the planning horizon while Eq. 24 represents the constraint for the proportion of network at the worst condition level.

In order to make the optimization model more practical, several sets of optional constraints are also introduced. For instance, during the planning horizon, it may be desirable to ensure the overall network value do not decrease while the proportion of deficient pavements (or deficiency level) do not increase (see Eq. 25 and 26). This requirement is employed to avoid the deferral of treatment to the small proportion of pavements that are in very poor condition state (usually, this proportion of pavement account for only small percentage of the entire road network).

$$\sum_{s=1}^{S}\sum_{i=1}^{I}\left[A_{si}X_{tsi}L_s - A_{si}X_{1si}L_s\right] \geq 0 \qquad \forall\, t \in T \tag{25}$$

$$\sum_{s=1}^{S}(A_{sI}X_{tsI}L_s - A_{sI}X_{1sI}L_s) \leq 0 \qquad \forall\, t \in T \tag{26}$$

In addition, as the objective function is to minimize the average budget over the planning horizon, they may be not evenly distributed based on the optimization results. For instance, most of the budget may need to be spent during the first few years while only the minor maintenance is needed during the following years which may not be the case with the real budget allocations. To solve this issue, a constraint which considers the standard deviation of the annual budget distribution should be adopted as shown in Eq. 27:

$$sqrt\left(\sum_{s=1}^{S}\sum_{i=1}^{I}\sum_{m=1}^{M}(X_{tsim}L_sC_{sm} - \frac{1}{T}\sum_{t=1}^{T}\sum_{s=1}^{S}\sum_{i=1}^{I}\sum_{m=1}^{M}X_{tsim}L_sC_{sm})^2 / T\right) < K\left(\frac{1}{T}\sum_{t=1}^{T}\sum_{s=1}^{S}\sum_{i=1}^{I}\sum_{m=1}^{M}X_{tsim}L_sC_{sm}\right) \tag{27}$$

K is a parameter for characterizing the distributed of the annual budget. In this research, *K* is defined as the standard deviation index and K=0 denotes the uniform distribution of the annual budget.

Experience reveals that some treatments are cost effective only when pavements are in certain condition states. For example, preventive treatment is not effective for the pavements in Fair and Poor conditions, hence, the corresponding decision variables are set to zero to disallow this treatment in other condition states. The suggested treatments are presented in Table 3.

Table 3 Suggested Treatments for Each Condition State

Condition	Do Nothing	Preventive Maintenance	Light Rehabilitation	Medium Rehabilitation	Heavy Rehabilitation
Good	Yes	Yes			
Fair	Yes		Yes	Yes	
Poor	Yes			Yes	Yes
Very Poor	Yes				Yes

5 CASE STUDIES

5.1 Introduction

To demonstrate the applicability of the proposed model, the above model is applied to asphalt pavements in New Jersey to prepare a ten-year maintenance & rehabilitation plan to optimize the maintenance and rehabilitation scheduling for the State Highway System. Different levels of budgets required to maintain or improve the road network value is computed.

5.2 Data Preparation

The total lane mile of asphalt pavement in New Jersey Highway System in 2012 is 4663 miles. The pavement condition is defined at three levels based on the ride quality data and distress data. The ride quality is measured by pavement's roughness (IRI: in/mile) while the distress index SDI reflects the amount of visible surface deterioration of a pavement ranging from 0 (the most distress) to 5(the least distress). The overall condition of asphalt pavement in terms of both distress and ride quality defined at three states: 1-Good: IRI<95 & SDI 3.5; 2-Fair: IRI<95 & 2.4<SDI<3.5 or 95 IRI 170 & SDI>2.4; 3-Poor: IRI>170 or SDI 2.4. The proportion of road group in each condition state is shown in Table 4.

Table 4 Road Network Initial Condition

Groups	1(good)	2(fair)	3(poor)
Group 1	0.273	0.319	0.408

The M&R treatments will be applied at the beginning of each year. However, it is not necessary at the network level to be as detailed as at the project level for programming purposes. Therefore, five simplified M&R treatment levels were assumed as mentioned in the model formulation: no action, preventive maintenance, light rehabilitation, medium rehabilitation and heavy rehabilitation. For pavements with given condition state at any given year, the suggested treatments of the five possible treatments as shown in Table 3 are to be performed. Associated with each M&R activity is a set of effectiveness and costs. The effectiveness of M&R treatments for a pavement section is listed in Table 5. The cost of all types of treatments (Table 6) is selected by considering the research conducted by Zhang and Murphy (2012) and New Jersey Pavement Management System. The selection of the information in Tables 5 and 6 is for the purpose of demonstrating the proposed methodology. When applied to practice, the pavement agencies should choose the appropriate M&R effectiveness and cost based on their experience.

Table 5 Effect of M&R Treatments

M&R Treatment	Condition Before	Condition After
Do Nothing	Good	Good*
	Fair	Fair*
	Poor	Poor*
	Very Poor	Very Poor*
Preventive Maintenance	Good	Good
	Fair	-
	Poor	-
	Very Poor	-
Light Rehabilitation	Good	-
	Fair	Fair
	Poor	-
	Very Poor	-
Medium Rehabilitation	Good	-
	Fair	Good
	Poor	Fair
	Very Poor	-
Heavy Rehabilitation	Good	-
	Fair	-
	Poor	Good
	Very Poor	Good

*: if no action is taken, the pavement deterioration will be considered during the year.

Table 6 Unit Cost of Treatment (based on FY2013 dollars, NJ)

M&R Action	Unit Cost (per lane mile) for Flexible Pavements ($)	Unit Cost (per lane mile) for Rigid Pavements ($)
No Action	0	0
Routine Maintenance	2,000	51,213
Preventive Maintenance	41,256	85,355
Minor Rehabilitation	291,629	364,182
Major Rehabilitation	628,783	926,105

To consider the pavement condition deterioration during the planning horizon, Meegoda and Gao (2014) investigated the progression of pavement roughness over pavement age using data from the Long-Term Pavement Performance (LTPP) database. The developed roughness deterioration curve which is a function of structural number, climate region and traffic load is employed in this research to consider the deterioration of asphalt pavement condition. The progression of roughness of a pavement can be expressed by the following two Eqs.:

$$\ln IRI_{i+1} - \ln IRI_i = (\alpha_{i+1} t_{i+1}^{0.9715} - \alpha_i t_i^{0.9715}) \tag{28}$$

$$\alpha_i = \frac{CL_i^{2.6E(-14)}}{SN^{1.992}} 0.4132 + FZI^{3.8086} 2.34E(-14) + AP^{2.53E(-14)} 1.54E(-2) \tag{29}$$

Where *IRI*, *t*, *CL*, *SN*. *FZI*, *AP* represents the International Roughness Index, pavement age, cumulative traffic load, structure number, freezing index and annual precipitation, respectively. The reliability of the pavement condition for New Jersey can be roughly estimated as $R = \exp(at^{\beta}) = \exp(-0.02745 t^{0.9715})$. Hence, the pavement deterioration rate with known pavement age can be obtained from this equation.

To fully account the deterioration of the last condition level (poor), one additional condition level of ‘very poor’ is added to the current condition level classification. It is considered here that no more deterioration occurs for the added level (very poor) of pavement and only the heavy rehabilitation (reconstruction) or replacement is applicable to this proportion of pavements. The existing performance value of pavement at each condition level is expressed in Table 7.

Table 7 Existing Pavement Performance Value

Group	Condition	Percentage of Performance Value
1	1 (good)	0.9
1	2 (fair)	0.7
1	3 (poor)	0.5
1	4 (very poor)	0.25

5.3 Analysis of Results

With the implementation of the model, the required averaged annual budget for improving the overall network value without considering the constraint of the standard deviation of annual budget distribution can be obtained as shown in Fig. 6.

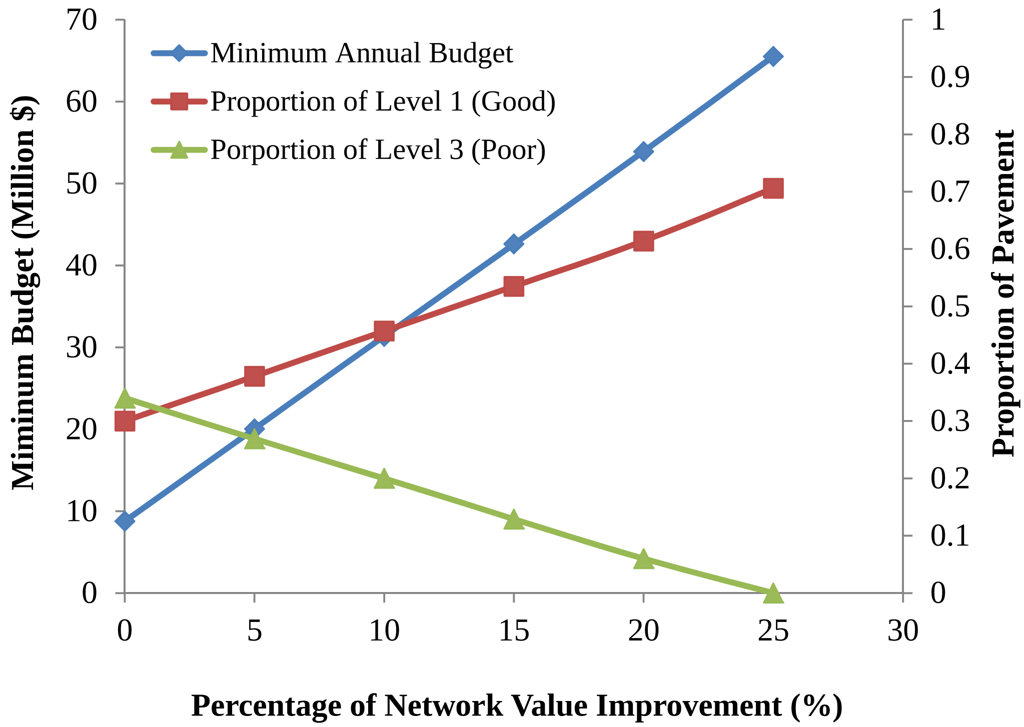

Fig.6 Averaged Annual Budget vs. Percentage of Network Value Improvement.

It can be observed from Fig. 6 that there is an almost linear relationship between the average annual budget and the percentage of improvement of the overall network value. This result shows that the optimization model has found the theoretical minimum budget to improve the network level with the optimal annual budget distribution and maintenance & rehabilitation scheduling. However, if different annual budget distribution is employed in budget allocation, the above correlation between averaged annual budget and percentage of network level improvement will be changed. To further investigate into this problem, sensitivity analysis is conducted for the standard deviation of annual budget distribution for the specific goal of improving the overall network value by 10% and 15% in 10 years, respectively. The computed results are shown in Fig. 7. The recommended treatments for two selected levels of standard deviation are presented in Table 8.

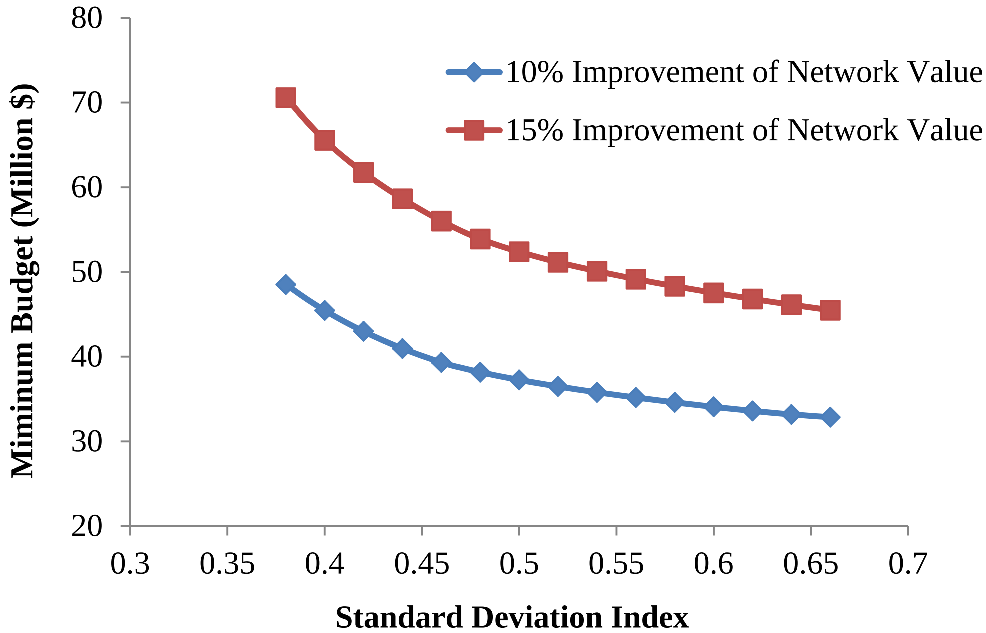

Fig.7 Sensitivity Analysis of Standard Deviation Index of Annual Budget.

Fig. 7 shows the sensitivity analysis to test the impact of different standard deviation of annual budget on the minimum required annual budget to achieve a certain condition target. As shown in Fig.7, there is a trend that the average annual budget decreases by reducing the requirement of the annual budget distribution. This is consistent with our experience in budget allocation that if there is high flexibility of allocating financial resources for different years, lower cost will be needed to improve the overall network condition to a certain level.

To illustrate the effect of budget allocation on total required budget, two budget allocation plans with the same expected overall network value improvement are presented in Table 6. In Table 6(a), as there is a stricter requirement on the annual budget allocation (K=0.4), it needs the average annual budget of $45.5 million which is much higher than that of $34.1 million as shown in Table 6(b) with standard deviation index of 0.6. By comparing the optimized maintenance & rehabilitation scheduling of the two sets of budget allocations, it can be observed that for optimization with standard deviation index of 0.4, its annual budget is allocated more evenly to all the treatments throughout the planning horizon. However, for the reduced constraint of the annual budget distribution (standard deviation index=0.6), most of budget is allocated at the end of the planning horizon to meet the requirement of 10% improvement of overall network value. In addition, no budget is allocated for the preventive maintenance and light rehabilitation, which seems contradictive to our experience on the pavement M&R strategy. However, as the improvement of pavement network value is only required at the end of the time span (10th year) and the correlated cost (e.g., induced traffic delay) is not considered, it may be reasonable to have such M&R schedule on the purpose of minimizing the total cost.

Table 6 Recommended Treatments for Two Levels of Standard Deviation of Annual Budget with 10% Improvement of Network Value in 10 Years

(a) For Standard Deviation Index=0.4 with Averaged Annual Budget=$45.5 Million

Year	Proportion of Pavement with Recommended Treatment (%)				
	No Action	Preventive Maintenance	Light Rehabilitation	Medium Rehabilitation	Heavy Rehabilitation
1st	92.76	3.77	0.42	2.38	0.67
2nd	92.24	3.78	0.42	2.41	1.14
3rd	92.19	3.79	0.42	2.45	1.16
4th	92.13	3.80	0.42	2.48	1.17
5th	92.07	3.81	0.42	2.51	1.19
6th	92.01	3.81	0.42	2.54	1.21
7th	91.96	3.82	0.42	2.58	1.22
8th	91.90	3.83	0.42	2.61	1.24
9th	91.84	3.84	0.42	2.64	1.25
10th	90.11	7.02	1.30	1.00	0.57

(b) For Standard Deviation Index=0.6 with Averaged Annual Budget=$34.1Million

Year	Proportion of Pavement with Recommended Treatment (%)				
	No Action	Preventive Maintenance	Light Rehabilitation	Medium Rehabilitation	Heavy Rehabilitation
1st	98.77	0	0	0.69	0.54
2nd	98.71	0	0	0.68	0.61
3rd	98.26	0	0	0.84	0.90
4th	97.77	0	0	1.00	1.24
5th	97.28	0	0	1.15	1.57
6th	96.54	0	0	1.55	1.91
7th	95.59	0	0	2.17	2.24
8th	94.63	0	0	2.78	2.58
9th	93.68	0	0	3.39	2.93
10th	100	0	0	0	0

6 SUMMARY AND CONCLUSIONS

This research project investigated the time-sequence roughness data of GPS test sections reported in the Long-Term Pavement Performance (LTPP) database to develop a model to predict the progression of pavement roughness over pavement age. The main factors contributing to the pavement deterioration were identified as climate, traffic load and pavement structure. The Weibull distribution was chosen to model the reliability of the roughness progression of asphalt pavements. The developed deterioration curve, which is a function of structural number, climate region and traffic load, was normalized. A five level condition state rating system was proposed. The cost-effectiveness treatment techniques for pavement maintenance was proposed based on estimated condition states.

A network-level budget planning model is developed to optimize how, where and when to resurface and rehabilitate to keep the pavement condition state at least at the current level with the minimum cost. The main outputs of the model consisting of the minimum budget requirements to achieve a given overall pavement network condition and the optimal maintenance and rehabilitation treatment strategy for each year. In this model, both the deterministic pavement deterioration and standard deviation of annual budget distribution were considered. The proposed model was tested in a case study of the asphalt pavement in New Jersey. Through this case study, the model demonstrates its ability to estimate the minimum required annual budget for improving the overall network performance value to a certain expected condition level by optimizing the maintenance and rehabilitation scheduling. In addition, the sensitive analysis was performed to study the effects of standard deviation of the distribution of the annual budget on the budget planning. The results showed that a higher budget is needed with the stricter requirement on the uniformity on the annual budget distribution. This model can help the higher-level decision makers to determine the actual budget for the planning horizon and also determine the optimal time for applying maintenance and rehabilitation treatments.

For the pavement condition deterioration model, further research is needed to consider the variability of model parameters, which may produce uncertainty in the performance predictions. To address this, a probabilistic approach should be followed and the inherent variability of performance predictions should be quantified. The statistical analysis for assessing the reliability of performance predictions may be used to account for the variability of three major items that affect the prediction, which are the model form, the model parameters and the model coefficients (Suherman, et al., 2009).

For the network level optimization model, while applying it the infrastructure system including pavement, bridge and culvert and etc., further improvements should be made to consider an unexpected failure of a component of a vital transportation system (e.g., bridge, culvert, and etc.) which usually creates disruptions and have cascading effects leading not only to havoc and its consequences of inconveniencies.

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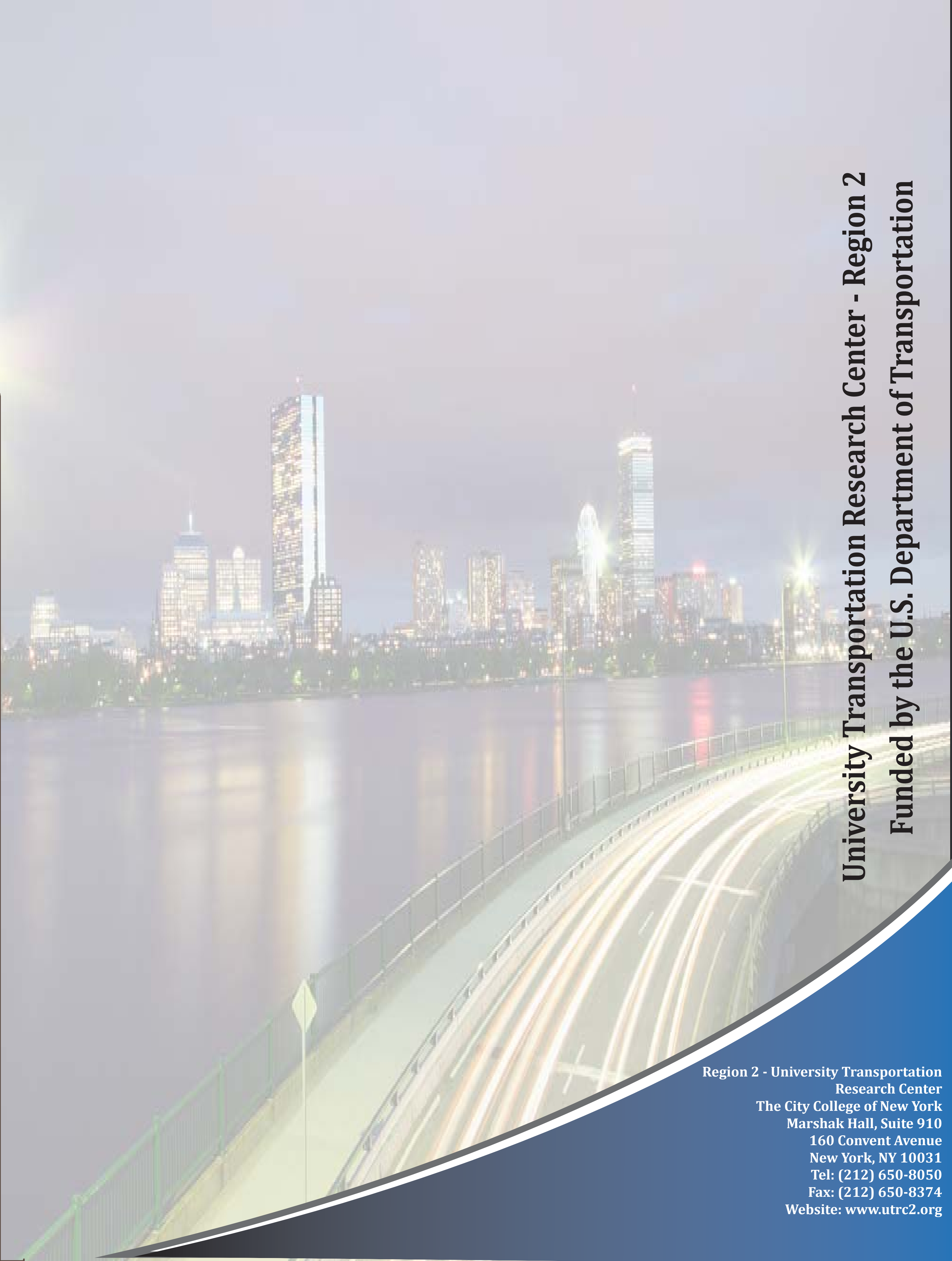

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