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Abstract

This paper presents the development of a pavement design and rehabilitation optimization decision-making framework based on Mechanistic-Empirical (ME) roughness transfer models. The AASHTOWare Pavement ME Design (the software of Pavement ME Design) is used to estimate pavement deterioration based on the combined effects of permanent deformation, fatigue, and thermal cracking. The optimization problem is first formulated into a mixed-integer nonlinear programming model to address the predominant trade-off between agency and user costs. To deal with the complexity associated with the pavement roughness transfer functions in the software and to use the roughness values as input to the optimization framework, a dynamic programming subroutine is developed for determining the optimal rehabilitation timing and asphalt concrete design thickness. An application of the proposed model is demonstrated in a case study. Managerial insights from a series of sensitivity analyses on different unit user cost values and model comparisons are presented.

Keywords

Pavement design, Pavement rehabilitation, Optimization, MEPDG, MINLP, Dynamic programming

Introduction

The increasing passenger and freight traffic demand and the aging pavement infrastructure impose significant challenges on local and state agencies that strive to ensure transportation efficiency and safety. According to the American Society of Civil Engineers (ASCE), federal, state and local governments spend $91 billion annually to maintain and rehabilitate the deteriorated highway pavement infrastructure system of the United States (ASCE 2013). Pavement deterioration is usually the result of a combination of various distresses (e.g., fatigue cracking, thermal cracking, and permanent deformation), each having different severity, extent, and rate of development. It develops overtime and is affected by vehicular loading, the environment, and pavement structure. When part of the pavement system deteriorates as a result of one or more of the aforementioned distresses, the pavement structure needs to be maintained or rehabilitated. Regardless of different pavement treatment methods used, an asphalt concrete (AC) overlay is often needed to address pavement functional and/or structural deficiencies.

As part of the pavement management system, there is a need for optimal timing and a rehabilitation approach to minimize the cost for both the owner (e.g., agency) and users of the road. Agency cost is incurred by the local and state agency for maintaining its serviceability. User cost reflects the quality of service provided to users, such as vehicle operating cost, fuel consumption, vehicle maintenance, driver discomfort, as well as accident cost (Salem and Genaidy 2008). There is a clear trade-off between agency cost and user cost.

Pavement rehabilitation optimization

The problem of optimal pavement rehabilitation planning has been modeled and solved by two major approaches based on (i) optimal control theory for the continuous time and continuous state case, and (ii) mathematical programming or Markov Decision Process (MDP) for discrete time and/or discrete state case. Optimal control theory was initially adopted to analytically obtain closed-form optimal solutions for only a single rehabilitation (Friesz and Fernandez 1979; Fernandez and Friesz 1981; Markow and Balta 1985). Simple multiplicative deterioration factors were used in these studies for roughness development over time. Later, Tsunokawa and Schofer (1994) used a continuous function with linear deterioration rate to approximate the discontinuous pavement condition and were thus able to solve multiple rehabilitation actions. Li and Madanat (2002) further extended this approach by developing a simpler approach based on MDP to solve steady-state problems. Prior to their work, most other studies applied MDP numerically (Golabi et al. 1982; Carnahan et al. 1987). Ouyang and Madanat (2006) derived exact analytical formulas for the optimal rehabilitation timing and thickness of overlay for a single pavement over a specific period of time. An exponential formed, nonlinear deterioration function with respect to time was adopted based on Paterson (1990). More recently, Hajibabai et al. (2014) incorporated these analytical results into joint optimization of transportation network design that involves traffic assignment and pavement rehabilitation. Similar ideas were used to develop models that jointly optimize resurfacing and maintenance planning (Gu et al. 2012), and incorporating budget constraints for a network of pavement facilities (Sathaye and Madanat 2011, 2012).

Mathematical programs are also commonly used to numerically optimize rehabilitation planning. For example, works by Murakami and Turnquist (1985), Al-Subhi et al. (1990) and Jacobs (1992) formulated discrete time mixed-integer mathematical programs, but used unrealistic pavement performance models, such as linear deterioration curves, to make the optimization solvable. Ouyang and Madanat (2004) used anexponential deterioration function and proposed a simple greedy heuristic to solve the problem as a mixed-integer nonlinear program. These research efforts were further extended by incorporating travelers' route choices and the agency's resource allocation decisions (Ouyang 2007).

Pavement Performance Models

Pavement performance models (e.g., those describing the deterioration process and rehabilitation effectiveness) are critical to the rehabilitation planning activities. Functions based on fitting empirical field observations have been used. These functions are usually inaccurate and project-dependent. For example, Ouyang and Madanat (2004) assumed an expontial deterioration function over pavement age. There is only a single parameter to represent the deterioration rate, which cannot capture the site-dependent pavement aging effects, cumulative damage (e.g., fatigue, permanent deformation), and time-varying thermal cracking effects. In fact, several factors that affect deterioration rate, such as material modulus and environment condition, are directly dependent on pavement age; this influences overlay thickness and rehabilitation timing decision (AASHTO 2008). Besides, the simplified deterioration models (e.g., roughness development) are only age-dependent, which does not account for the impacts of traffic load and pavement design. Hence, field-validated mechanistic models would be very helpful in enhancing realism of the optimization framework.

Furthermore, the existing models essentially assume that the rehabilitation level (i.e., thickness of overlay) and pre-rehabilitation pavement condition affect the effectiveness of the rehabilitation action (i.e., roughness reduction), but not the deterioration rate afterwards. This may not be realistic. In several empirical pavement studies, pavement roughness improvement after overlay is found to be independent of (i) the overlay thickness, as long as the overlay thickness exceeds 2 in, or (ii) the pavement condition before rehabilitation (Son and Al-Qadi 2014). Rehabilitation may re-establish the pavement surface roughness value, but the rate of roughness development over time depends on the pavement design and rehabilitation characteristics. This paper emphasizes this distinct feature and highlights the importance of the pavement design, performance models, and rehabilitation characteristics.

To the best of the authors' knowledge, there is still a lack of systematic pavement management methodologies and decision tools that incorporate mechanistic-empirical analysis of pavement response and performance prediction. Hence, this paper attempts to propose a more realistic pavement rehabilitation optimization framework to fill the research gap in systematic pavement management methodology framework based on the advanced pavement ME design approach. A finite horizon, single pavement design and rehabilitation problem is formulated to address the trade-off between agency and user costs. Roughness transfer models, used in the Pavement ME Design, are incorporated into the rehabilitation planning to predict the international roughness index (IRI), which combines structural analysis and pavement responses, and accounts for aging, temperature, water content, speed and other important environmental factors. The high complexity associated with pavement deterioration makes the problem very difficult to solve by conventional mathematical programming approaches. Therefore, to use the Pavement ME Design models embedded in the software, a dynamic programming algorithm is developed using the output of the software (i.e., IRI values) to endogenously determine the optimal asphalt concrete pavement design (i.e., thickness of the AC layer) and resurfacing timings in the planning horizon. This optimization framework is quite general in that it can easily incorporate other pavement performance models as an input to capture more comprehensive pavement deterioration effects. The developed decision-making framework is applied to a case study utilizing a commonly used elastic pavement response model. Managerial insights are then drawn from sensitivity analyses and model comparisons.

The remainder of this paper is organized as follows: Sect. 2 summarizes the IRI transfer functions based on Pavement ME Design models; presents the mathematical formulation of pavement design and rehabilitation planning problem; and proposes a solution approach based on a dynamic programming subroutine. Section 3 presents numerical results from the case study and discusses managerial insights. Section 4 provides conclusions based on this paper and discusses future research directions.

Methodology

Pavement ME Design

The Pavement ME Design is the current prevailing design approach for pavement structures in the United States. The mechanistic part is used to calculate critical pavement responses to traffic loading (e.g., tensile strain at the bottom of the AC and vertical strain on top of the subgrade). The empirical portion links critical pavement responses to pavement distresses based on statistical relations between road structures and field observations. These relations are usually identified as Distresses Prediction Models.

Three distresses (permanent deformation, fatigue cracking, and thermal cracking) are considered in assessing pavement roughness (e.g., in terms of IRI). Permanent deformation and fatigue cracking are estimated using Pavement ME Design empirical models (AASHTO 2008). In this paper, AASHTOWare Pavement ME Design version 1.5 (the software of Pavement ME Design) is used, where distress development that depends on several factors (including AC thickness) and their combined effects are accounted for to estimate the IRI. The detailed IRI transfer functions embedded in the Pavement ME Design software are summarized in Appendix A (AASHTO 2008).

Optimal design and rehabilitation timing decision framework

A single pavement in a finite horizon of discrete time periods $t \in T = \{1,2,\dots,T\}$ is considered. Depending on the resolution of the analysis, the unit of the time period could be year, quarter or month, etc. In this problem, two types of decision variables are considered: design asphalt concrete thickness $h \in \mathcal{H}$, where $\mathcal{H}$ is a finite set of possible design thickness values, and rehabilitation timing, $\mathbf{r}(h) = \{r_t(h)\}^T \in \{0,1\}^T$, where $r_t = 1$ indicates that there should be a rehabilitation activity during time period t, or rt=0. Each rehabilitation activity could be completed at the beginning of a time period, and the rehabilitation duration is negligible. The roughness development, in terms of IRI, is based on the Pavement ME Design model as presented in Appendix A. We apply these models using the Pavement ME Design software, which outputs the IRI value at the end of each time period. For the purpose of maintaining consistent highway geometry and profile before and after rehabilitation, the thicknesses of pavement layers are assumed to be constant in this study. As such, each resurfacing removes a certain thickness of the AC surface layer and then repaves a layer with the same thickness on the top. In this case, under any AC design thickness $h \in \mathcal{H}$, the rehabilitation timing r(h) can be decided. Son and Al-Qadi (2014) found that right after rehabilitation, the pavement condition could be improved up to 80–100 %. For analysis simplicity, initial IRI is assumed to be fully (100 %) recovered if the top 2 in the AC layer is resurfaced. Although this approach can incorporate the complex pavement deterioration factors including traffic and environment, it does not capture the historic damages in pavement sublayers because current software cannot account for impacts of rehabilitation activities on pavement sublayer deterioration. If this feature becomes available in the future, it can be easily incorporated in the optimization framework. So in this paper, we assume that IRI is renewed right after each rehabilitation activities, and the subsequent IRI development is computed in the software. Figure 1 shows an example of roughness development trajectory under rehabilitation.

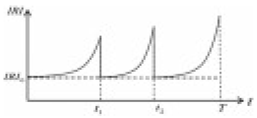

Fig. 1. An example of roughness development trajectory under rehabilitation

The user cost per unit time (primarily due to excessive fuel consumption and driving discomfort) is assumed to be proportional to the pavement roughness (Ouyang and Madanat 2006). In discrete time, the user cost incurred during each time period is calculated based on the roughness at the end of the time period. This assumption is made for simplicity. The user cost is over-estimated. For more accurate results, the average IRI between the beginning and the end of each time period can be used. The traffic load $\mathbf{f} = \{f_t\}^T$ on the pavement facility is given throughout the planning horizon (e.g., from traffic demand forecasts). In practice, the agency cost for construction and rehabilitation is often in proportion to the thickness of the overlay; the unit agency cost is denoted as m ($/in/mi/lane).

The key parameters and decision variables related to the optimization model are summarized as follows:

$$\textbf{InputParameters}: t \in \mathcal{T} = \{1,2,\ldots,T\}\text{:discrete time period,}$$
$$IRI_0\text{: initial } IRI, \mathbf{f} = \{f_t\}^T\text{: total traffic during each time interval}$$
$$t\text{(e.g., \textbackslash, veh/lane/year)}, u\text{:unit user cost (e.g., } \$/veh/mi/\text{IRI)}, m:$$
$$\text{unit agency cost for a unit thickness of overlay (e.g., \$/in/mi/lane)}, i\text{:interest rate.}$$

$$\textbf{DecisionVariables}:$$
$$h \in \mathcal{H}\text{:design AC layer thickness at the beginning of the planning horizon,}$$
$$r_t(h) \in \{0,1\}: rehabilitation\text{decision at time}t\text{under design AC thinkness}h,$$
$$l_t(h) \in \mathcal{T}\text{:last rehabilitation time before time}t\text{under design AC thinkness}h,$$
$$F(h,t)\text{:the lowest cost until the end of time interval}t\text{under design AC thickness}h,$$
$$IRI_t(h, l_t(h))\text{:the IRI at time}t\text{since the latest rehabilitation}l$$
$$(t)\text{under design AC thickness}h.$$

$IRI_t(h, l_t(h))$ represents the IRI value outputs from the pavement performance models (e.g., the ME roughness transfer functions used in this paper) at each time $t \in \mathcal{T}$ as a function of design AC thickness $h \in \mathcal{H}$ and last rehabilitation timing $l_t(h)$. It can be computed as a three- dimensional parameter in the software.

The net present value of the life-cycle cost consists of agency and user costs over the planning horizon; therefore, the optimization problem can be formulated as follows:

(1)

$$\min_{h \in \mathcal{H}, l(h), r(h)} mh + \sum_{t=1}^{T} (u/(1+i)^t) f_t IRI_t(h, l_t(h)) + \sum_{t=2}^{T} (2m/(1+i)^{t-1}) r_t(h)$$

subject to

(2)

$$l_1(h) = 1, \qquad \forall h \in \mathcal{H}$$

(3)

$$r_1(h) = 1, \qquad \forall h \in \mathcal{H}$$

(4)

$$l_t(h) \leq l_{t+1}(h), \qquad \forall t \in \mathcal{T}\backslash\{T\}, h \in \mathcal{H}$$

(5)

$$l_t(h) = \max_{0 \leq t' \leq t} \{r_t(h) \times t'\}, \qquad \forall t \in \mathcal{T}, h \in \mathcal{H}$$

(6)

$$0 \leq l_t(h) \leq T, \qquad \forall t \in \mathcal{T}, h \in \mathcal{H}$$

(7)

$$r_t(h) \in \{0,1\}, \qquad \forall t \in \mathcal{T}, h \in \mathcal{H}.$$

The objective function (1) minimizes the total relevant cost including the agency cost for initial constructing and, subsequently, pavement rehabilitation, and the total user cost throughout the planning horizon over the solution space $h \in \mathcal{H}$. Constraints (2) and (3) specify the initial conditions, i.e., building a new pavement at $t = 1$. Constraints (4) and (5) stipulate that the latest rehabilitation timing $l_t(h)$ by time t should be the largest time index during $[1, t]$ with a rehabilitation action. Constraints (6) and (7) define the solution spaces of the decision variables.

The optimization problem (1)–(7) is essentially a mixed integer nonlinear program (MINLP). Due to the high dimension of the discrete solution space (i.e., the binary variable $\mathbf{r}(h) = \{r_t(h)\}^T \in \{0,1\}^T$) and highly nonlinear roughness transfer models, exact optimal solutions are very difficult to find by conventional algorithms or existing solvers. However, by virtue of the recursive and decomposable nature of the rehabilitation problem, we manage to develop a dynamic program-based solution approach to find exact optimal solutions to problem (1)–(7) with reduced computations.

Dynamic programming solution approach

For any $h \in \mathcal{H}$, the forward Bellman equation is used to solve the optimization problem (1)–(7). $F(h, t)$ is defined as the minimal cost up to t conditional on h, and then it can be computed by step-wise optimization as follows:

(8)

$$\pi_1(t,h) = \min_{1\le\tilde{t}\le t-1} \overbrace{\left\{ \begin{array}{c} \underbrace{\min_{2\le\tilde{t}\le t-1} \left\{ F(\tilde{t}-1) + \sum_{k=\tilde{t}}^{t} (u/(1+\imath)^k) f_k IRI_k(h,\tilde{t}) + 2m/(1+\imath)^{\tilde{t}-1} \right\}}_{2\le\tilde{t}\le t-1}, \\ \underbrace{\sum_{k=1}^{t} (u/(1+\imath)^k) f_k IRI_k(h,1) + hm}_{\tilde{t}=1} \end{array} \right\}}^{\text{no \, rehabilitate \, at\,} t, \text{ and \, roughness \, continues \, to \, develop \, since \, the \, last \, rehabilitation \, at} \tilde{t}}, \forall t \ge 2$$

(9)

$$\pi_2(t,h) = \overbrace{F(t-1) + (u/(1+\imath)^t) \cdot f_t \cdot IRI_t(h,t) + 2m/(1+\imath)^{t-1}}^{\text{rehabilitate \, at \, time} t\text{,\,\,and \, roughness \, is \, renewed}}, \forall t \ge 2$$

(10)

$$F(t,h) = \min\{\pi_1(t,h), \pi_2(t,h)\}, \forall t \ge 2,$$

where $\pi_1(t,h)$ is the minimal total cost up tot if we choose not to rehabilitate at t, and $\pi_2(t,h)$ is the same cost if we choose to rehabilitate at t. Based on the result from (8), $r_t^*(h)$ and $l_t^*(h)$ at period t are updated as follows:

If $\pi_1(t,h) \le \pi_2(t,h)$,then$r_t^*(h) = 0$, and

$$l_t^*(h) = \underset{1\le\tilde{t}\le t-1}{\text{argmin}} \left\{ \begin{array}{c} F(\tilde{t}-1,h) + \sum_{k=\tilde{t}}^{t} \big(u/(1+i)^k\big) f_k IRI_k(h,\tilde{t}) + 2m/(1+i)^{\tilde{t}-1}, \\ \sum_{k=1}^{t} \big(u/(1+i)^k\big) f_k IRI_k(h,1) + hm \end{array} \right\}$$

If $\pi_1(t,h) > \pi_2(t,h)$, then $r_t^*(h) = 1$, and, $l_t^*(h) = t$

It is not difficult to prove the correctness of the recursive formulas (8)–(10) based on the principle of optimality. If it is supposed that $F(\tilde{t},h)$ stores the optimal total cost up to time $\tilde{t} = 1, \dots, t-1$, then, given the decision at t (whether to rehabilitate or not), the optimal total cost up to t must be the minimum of two options. If the pavement is rehabilitated at t, the optimal total cost up to t equals the optimal total cost by $t-1$, plus the agency cost at time t and the user cost during period t. If the pavement is not rehabilitated at t, the roughness during period t depends on its last rehabilitation time $\tilde{t}$, which may occur between time 1 and $t-1$. Thus, the optimal cost is the minimum among all possible scenarios when the last rehabilitation occurs at time $\tilde{t} = 1, \dots, t-1$. The cost in scenario $\tilde{t}$ is formulated by the following:

$$F(\tilde{t}-1,h) + \sum_{k=\tilde{t}}^{t} (u/(1+i))^k f_k IRI_k(h,\tilde{t}) + 2m/(1+i)^{\tilde{t}-1}, \forall\tilde{t} = 2, \dots, t-1, \text{ and}$$

$$\sum_{k=1}^{t} \big(u/(1+i)^k\big) f_k IRI_k(h,1) + hm, \text{for}, \tilde{t} = 1$$

After $F(T,h)$ is computed for each $h \in \mathcal{H}$, the optimal value of h can be found by sorting the set of $F(T,h)$ values. The complete algorithm framework is summarized as follows:


Step 1. Initialization for all $h \in H$,

construct a new pavement with h design HMA thickness

user cost

agency cost

Step 2. Starting from any h, update $F(t,h)$, for $t = 2,...,T$.

Step 2.1. Start from $t = 2$, compute $F(t,h)$ based on Bellman equation (10).

Step 2.2. Update the optimal solutions:

Step 2.3. Let $t = t+1$ and go back to Step 2.1.

Step 3. Repeat Step 2 for all $h \in H$.

Step 4. Find the optimal cost $F^* = \min_{h \in H} F(T,h)$, the optimal thickness of asphalt concrete layer $h^* = \arg\min F(T,h)$, and the optimal rehabilitation plan $\xi_t^*(h^*)$ and $\zeta_t^*(h^*)$.

Case study

Data preparation

A 20-year planning horizon, i.e., $t \in T = \{1,2, \ldots, 20\}$, is assumed for a two-lane highway segment. The Total Average Annual Daily Truck Traffic (AADTT) is assumed to be 4,500 with a 3 % annual growth rate (AASHTO 2008) Assuming that AADTT is 15 % of the Average Annual Daily Traffic (AADT) (AASHTO 2008), the total annual traffic in the first year can be calculated, i.e., f_1 = 5,480,000 veh/lane/year, and, correspondingly, the traffic in subsequent years by simply multiplying the growth factor. The annual interest rate is assumed to be $i = 5\,\%$.

A three-layered flexible pavement is considered: AC, base, and subgrade layers. The AC thickness is assumed to vary between 2 in and 10 in, with 0.5 in increments, i.e., $\mathcal{H} = \{2, 2.5, \ldots, 10\}$. AC material cost is assumed to be 70$/ton. By assuming 12 ft lane width and 145 lb/ft^3 material density, the unit agency cost is estimated as $27912/in/mi/lane. Thickness and elasticity modulus of base are 10 in and 29,000 psi (200 MPa), respectively. Elasticity modulus of subgrade is 10,152 psi (70 MPa). Design speed is assumed to be 60 mph speed.1 All distress and IRI are calculated based on 90 % reliability.

Traffic loads are considered as load spectra based on the default traffic distribution and parameters in Pavement ME Design software (AASHTOW are, 2014) Tire pressure is assumed to be 105 psi. Temperature profile, average annual freezing index, and average annual precipitation are determined based on the weather conditions in Champaign, IL. For other parameters, default values in the software are used (see Table 1).

Table 1. Parameters used in distress models

Initial IRI (IRI_0)	**63 (in/mile)**
Percent plasticity index of soil	29 (for A-2-7 soil type)
Average annual freezing index, F°-days	1,256.2
Average annual precipitation or rainfall	37 in
Percent air voids in the HMA mixture	7 %
Effective asphalt content by volume	11.6 %
Ground water table	10 ft

Besides, several existing studies estimate the user costs for different types of vehicles or under different roughness conditions, such as Islam and Buttlar (2012). However, the accurate quantification of this cost based on existing approaches or data is very difficult. For example, it is difficult to judge how an individual would value delay in travel time or estimate the accident cost resulting from fatal or property damage on account of increased congestion(Salem and Genaidy 2008). Islam and Buttlar (2012) estimated additional user cost resulting from increased pavement roughness compared with that in new conditions, mainly accounting for fuel, repair and maintenance, depreciation and tire costs, which ranges from 0.00003 to 0.0003 $/veh/mi/IRI depending on roughness levels. Considering the variability of user cost, a sensitivity analysis of user cost will be performed to evaluate its impact on optimal pavement rehabilitation decisions.

Numerical results

In the case study, a set of numerical results are obtained based on the elastic model. Besides, a sensitivity analysis is performed to show the impact of the unit user costu, which is key to the trade-off between user cost and agency cost, in the range from 1.0E–5 to 1.0E–3 $/vehicle/mi/IRI. The solution algorithm is coded in MATLAB. All cases can be solved instantly (less than 1 s). Figure 2 displays the optimal IRI trajectories under different values of unit user cost for the elastic models under 100 % reliability.

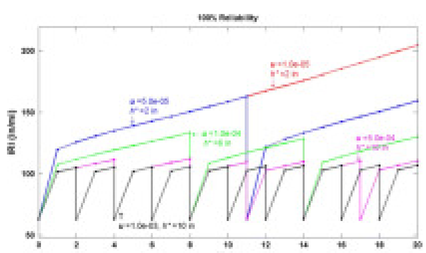

Fig. 2. Optimal IRI trajectories under 100 % reliability

As shown in Fig. 2, when unit user cost increases, the optimal AC layer design becomes thicker and the average optimal rehabilitation cycle becomes shorter. Also, note that for each case, the cycle between two consecutive rehabilitation activities tends to become shorter as pavement ages. For example, when $u = 0.0001$ $/veh/mi/IRI, the cycle lengths of the two rehabilitation activities are 8, 6, and 6 years, respectively. Similarly, when $u = 0.0005$ $/veh/mi/IRI, the lengths of the first two and the last four rehabilitation cycles are 4 and 3 years, respectively. Two factors have likely contributed to this phenomenon—the growing traffic load and the annual interest rate—so that more frequent rehabilitations become necessary. It can also be seen through the experiments that the optimal design and rehabilitation plans are not very sensitive to the user cost, except when u varies dramatically (e.g., the 100 % increment between each two adjacent levels of u as the five levels of user cost in this analysis). Table 2 summarizes the optimal itemized costs and decisions of the sensitivity analysis, which shows the agency cost increases with u as a result of the increasing weight of the user cost.

Table 2. Optimal costs and solutions under different u and pavement response approaches

***u*(\$/vehicle/mi/IRI)**	**Optimal Costs and Solutions**				

	Total cost ($/mi)	Agency cost ($/mi)	User cost ($/mi)	*h**	Number of rehabilitation activities
0.00001	194,875	55,824	139,051	2	0
0.00005	706,401	88,463	617,938	2	1
0.0001	1,279,453	233,451	1,046,003	6	2
0.0005	5,093,369	448,020	4,645,348	10	5
0.001	9,679,815	597,442	9,082,373	10	9

The proposed decision framework is further compared with the one in Ouyang and Madanat (2006). Same parameter values are used in these two models. Ouyang and Madanat (2006) adopted a simplified pavement deterioration model in exponential form and, under this model, a nice threshold structure of the optimal solution is proven (i.e., rehabilitation is conducted only when the roughness reaches a threshold). Furthermore, their model determines the rehabilitation intensity (in terms of roughness reduction) while it neglects the optimal design thickness. Figure 3 shows the optimal IRI trajectories under two different user cost values.

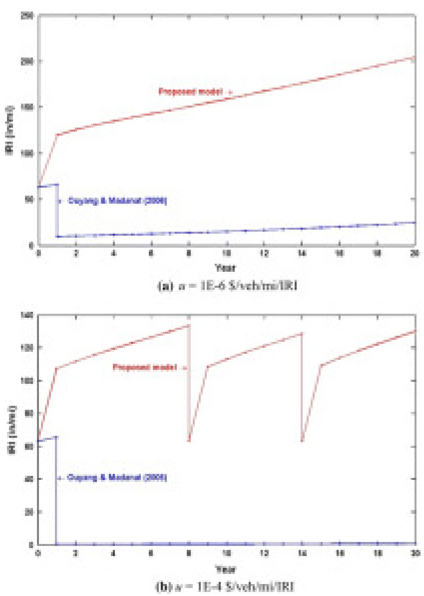

Fig. 3. Comparison of optimal IRI trajectories from the model in Ouyang and Madanat (2006) and from this study under two different *u* values

However, their optimal rehabilitation trajectories are very different from those obtained by the proposed model with Pavement ME Design. In their result, the roughness is reduced to much lower than the initial roughness in the second year. This behavior could be due to two reasons: (i) the threshold structure of the optimal solution heavily depends on the simplified form of the exponential deterioration model; and (ii) the assumption that the roughness reduction effectiveness of a rehabilitation activity is dependent on the thickness of overlay, but the deterioration rate remains the same during the planning horizon. Therefore, this study shows that pavement deterioration and rehabilitation effectiveness models significantly affect optimal rehabilitation plans. Hence, it is important to incorporate realistic conditions and empirically calibrated pavement models into the optimization framework.

Conclusion

This paper integrates pavement design and rehabilitation decision making for a single facility in finite horizon based on Pavement ME Design. The trade-off between agency and use costs is mainly considered to determine optimal rehabilitation timings and design thickness of the AC layer. A dynamic programing algorithm is developed to solve the highly challenging problem. A case study with realistic data demonstrates the application of the proposed methodology, where managerial insights are drawn from a series of sensitivity analyses on unit user cost. It is also found that the proposed mechanistic-empirical approach leads to pavement design and rehabilitation plans that differ significantly from those in the literature.

This paper focuses on the development of a generalized framework for optimum rehabilitation planning. Hence, a few simplifying assumptions, such as constant unit user cost, were made. In reality, the unit user cost u resulting from pavement roughness is dependent on the types of vehicles. The user cost as a variable parameter should be incorporated in a future study. Furthermore, the impacts of sublayer damage are not considered in the decision making framework because of the limitation of the Pavement ME Design software, which affects the IRI values used in the optimization model. More comprehensive pavement performance models and empirical data can be easily incorporated when available. Besides, the methodology developed in this paper is a building block for future research problems at network levels. The modeling framework can be further generalized into a systematic sustainable infrastructure management framework that includes life-cycle analyses on energy consumption and emissions to determine optimal design and rehabilitation. To maximize social welfare, recyclable materials in the construction and/or rehabilitation processes can also be included.

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Appendix A

Pavement ME Design

Three distresses (permanent deformation, fatigue cracking, and thermal cracking) are considered for pavement roughness (e.g., in terms of IRI) assessment. Permanent deformation and fatigue cracking are estimated using Pavement ME Design empirical models (AASHTO 2008). Distresses' development depends on several factors (including AC thickness) and their effects are accounted for to estimate the IRI, as summarized below.

International Roughness Index

IRI is estimated considering the permanent deformation, fatigue cracking, and thermal cracking. IRI equation for new flexible pavements and AC overlays of flexible pavements can be written as follows:

$$\text{IRI} = \text{IRI}_0 + 0.0150\text{SF}' \times \text{Age} + 0.4(\text{FC}^{\text{total}}) + 0.008(\text{TC}) + 40(\text{RD}^{\text{total}}),$$

where IRI_0 is the Initial IRI,SF′ the Site factor parameter related to percent plasticity of soil, average annual freezing, and average annual precipitation or rainfall, Age the Pavement age (year), FC^{total} the Area of total fatigue cracking (combined alligator and longitudinal), TC the Thermal cracking, and RD^{total} is the Ruth depth.

Permanent deformation

The permanent deformation presented in Pavement ME Design is based on incremental damage. The permanent deformation is estimated for each analysis period and for each layer of the pavement structure. The total permanent deformation in the pavement structure is the summation of the permanent deformation in the AC layer and the permanent deformation in the unbound materials.

Total permanent deformation for a typical flexible pavement with AC and base layers includes those from asphalt concrete and those from unbound materials, as follows:

$$\text{RD}^{\text{total}} = \text{RD}^{\text{ac}} + \text{RD}^{\text{base}}.$$

The total permanent deformation in asphalt concrete is given as follows:

$$\text{RD}^{\text{ac}} = B_1^{\text{ac}} \varepsilon_r n^{B_2^{\text{ac}}} T^{B_3^{\text{ac}}}$$

where ε_r is the Resilient or elastic strain calculated by the structural response model at the mid-depth of each AC layer/sublayer (in/in),n the Number of axle-load repetitions, *T* the Mix or pavement temperatures (F°), $B_1^{\text{ac}}, B_2^{\text{ac}}, B_3^{\text{ac}}$ the Parameters related to global and local calibration factors, $C_1 = -0.1039 \times h^2 + 2.4868 \times h - 17.342$, $C_2 = 0.0172 \times h^2 - 1.7331 \times h + 27.428$, depth the analysis depth, and h is the AC layer thickness.

The permanent deformation in unbound materials can be calculated as

$$\text{RD}^{\text{base}} = B^{\text{base}} \varepsilon_v h_{\text{soil}} \left(\frac{\varepsilon_0}{\varepsilon_r}\right) e^{-\left(\frac{\rho}{n}\right)^{\beta}},$$

where B^{base} is the Parameter related to global and local calibration factors, ε_v the Average vertical resilient or elastic strain in the layer as obtained from the primary response model, h_{soil} the Thickness of the soil layer, ε_r is Resilient strain imposed in laboratory test, and $\varepsilon_0, \rho, \beta$ is the material properties.

Fatigue cracking

This study considers two of Pavement ME Design fatigue cracking models: alligator cracking and longitudinal cracking. The Pavement ME Design assumes that longitudinal cracks are caused by fatigue damage on the surface of the AC layer, while alligator cracks are assumed to initiate as a result of fatigue damage at the bottom of AC. The total fatigue cracking is as follows:

$$FC^{total} = FC^{top-down} + FC^{bottom-up}.$$

Fatigue damage estimation in Pavement ME Design is stated by Miner's Law as follows:

$$D = \sum_{i=1}^{T} \frac{\text{traffic}_i}{N_i},$$

where D is the Damage,traffici the Actual traffic for period i, and N_i is the Allowable number of axle-load applications for period i.

The prediction model for allowable number of axle-load application for fatigue cracking:

$N_f = K_1 C_H \varepsilon_{\text{tensile}}^{K_2} E^{K_3}$,, where K_1, K_2, K_3 is the Parameters related to global and local calibration factors, $\varepsilon_{\text{tensile}}$ the Tensile strain at critical locations (in/in), from structural response model, E the Dynamic modulus of the HMA measured in compression (psi), and C_H is the Thickness correction term, where $C_H = \left[0.000398 + \frac{0.003602}{1+e^{11.02-3.49\times h}}\right]^{-1}$ for alligator cracking and $C_H = \left[0.01 + \frac{12}{1+e^{15.676-2.8186\times h}}\right]^{-1}$ for longitudinal cracking.

Final fatigue cracking estimation model using fatigue damage for alligator cracking (% of total lane area) is as follows:

$$\text{FC}^{\text{bottom - up}} = 100\left(1 + e^{C_2' - 2C_2' \times log_{10}(D^{\text{bottom - up}} \times 100)}\right)^{-1},$$

Where

$C_2' = -2.40874 - 39.748 \times (1+h)^{-2.856}$, while for longitudinal cracking (feet/mile), the formula is

$$\text{FC}^{\text{top - down}} = 10560\left(1 + e^{7-3.5\times log_{10}(D^{\text{top - down}} \times 100)}\right)^{-1}.$$

Thermal cracking

The amount of thermal cracking, which occurs on pavement surface, is predicted by the following formula:

$$\text{TC} = \beta_{f1} \times N\left[\frac{log(C_d/h_{ac})}{\sigma_d}\right],$$

where TC is the Observed amount of thermal cracking (feet/500 feet), β_{f1} the Calibration factor, $N[z]$ the Standard normal distribution evaluated at (z), σ_d the Standard deviation of log of the depth of cracks in pavement, C_d the Crack depth, and h_{ac} the Thickness of asphalt layer.

Paris law is used to predict the amount of crack propagation induced by a given thermal cooling cycle.

$$\Delta C = A \Delta K^n,$$

where

ΔC = Change in crack depth due to a cooling cycle,

ΔK = Change in the stress intensity factor due to a cooling cycle, and

A, n = Fracture parameters for the asphalt mixture.

In the design guide, a simplified equation derived based on finite element analysis is used to compute stress intensity factor, *K*.

$$K = \sigma_{\text{tip}}[0.45 + 1.99(C_0)^{0.56}],$$

where σ_{tip} is the Far-field stress at depth of crack tip, and C_0 the Current crack length.

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[1] Although Pavement ME Design has quite comprehensive empirical models which link the mechanistic pavement response to distress and IRI by considering aging, climate conditions, and traffic load spectra, it does not take speed variations into account. It is simply a user-defined value. It is one of the limitations of Pavement ME Design models and it is considered as such in this paper.