An Optimum Design Approach for Flexible Pavements

Article *in* International Journal of Pavement Engineering · March 2003

DOI: 10.1080/1029843031000087464

2 authors:

Khaled Abaza
Birzeit University

Sameer Abu-Eisheh
An-Najah National University

The International Journal of Pavement Engineering, Vol. 4 (1) March 2003, pp. 1–11

Spon Press
Taylor & Francis Group

An Optimum Design Approach for Flexible Pavements

KHALED A. ABAZA[a,*] and SAMEER A. ABU-EISHEH[b,†]

[a]*Department of Civil Engineering, Birzeit University, P.O. Box 14, Birzeit, West Bank, Palestine;* [b]*Department of Civil Engineering, An-Najah National University, P.O. Box 7, Nablus, West Bank, Palestine*

(Received 4 January 2002; In final form 5 November 2002)

An optimum approach for the design of flexible pavements has been developed which utilizes the anticipated performance of pavement and its life-cycle cost. The optimum approach developed has been applied to the design method recommended by the American Association of State Highway and Transportation Officials (AASHTO) for the design of flexible pavements. Pavement performance, defined using the initial and terminal serviceability indices, is a major design parameter that directly affects future pavement condition, initial construction cost and maintenance and added user costs. The optimum design is the one associated with the optimum terminal serviceability index and corresponds to the most cost-effective design. Cost-effectiveness is defined using a parameter called pavement life-cycle disutility which is the ratio of the pavement life-cycle cost to the pavement life-cycle performance identified by the area under the corresponding performance curve. The optimum pavement design is the one associated with the minimum pavement life-cycle disutility value and yields the optimum terminal serviceability index. The optimum terminal serviceability index value replaces the general AASHTO design index recommendations of 2.0 and 2.5 for minor and major roads, respectively. A performance curve is generated for a particular pavement structure using an incremental solution of the AASHTO basic design equation. It is shown that pavements should be designed for higher terminal serviceability index values than currently recommended.

Keywords: Flexible pavement design; Pavement serviceability; Pavement maintenance; Optimum pavement design; Pavement life-cycle cost

INTRODUCTION

The American Association of State Highway and Transportation Officials (AASHTO) design method of flexible pavements is probably the most widely used design method not only in the United States of America but also worldwide. The AASHTO Guide for Design of Pavement Structures (AASHTO, 1993) is based on several parameters that account for traffic loads, materials properties, drainage and environmental conditions, reliability and prediction variations and performance trends. Pavement performance is a major design parameter and is mainly defined based on the terminal serviceability index (P_t) value in the presence of the other specified design parameters. AASHTO recommends 2.0 and 2.5 terminal serviceability index values for minor low volume roads and major high volume roads, respectively. The AASHTO recommendations aim to minimize added user cost as the pavement reaches an advanced stage of deterioration with major roads being largely affected due to their high traffic volumes. Practitioners using the AASHTO design method have long used these recommended values without really questioning their applicability to the local conditions being considered. The recommended values as related to traffic conditions are very narrow and not flexible enough to respond to wide traffic variations. In addition, there is no guarantee that the recommended values would yield the most cost-effective design for the prevailing conditions.

Therefore, the need to develop an optimum design approach for flexible pavements that is performance-based and cost-effective is highly desirable. Performance-based design has been proposed by researchers (Yoder and Witczak, 1975; Haung, 1993) but the methodology to achieve that has not been developed partially due to the time needed to solve a design problem. However, with the presence of high-speed personal computers, the computational time no longer presents a drawback. The approach developed here replaces the traditional design procedure outlined by AASHTO, with a simple and effective one that

*Corresponding author. Tel.: +972-2-298-2108. Fax: +972-2-298-2984. E-mail: kabaza@birzeit.edu
†Tel.: +972-9-238-1113. Fax: +972-9-238-3280. E-mail: sameeraa@najah.edu

ISSN 1029-8436 print/ISSN 1477-268X online
DOI: 10.1080/1029843031000087464

basically aims to maximize pavement life-cycle performance while minimizing pavement life-cycle cost. This objective has been accomplished by introducing a new parameter called pavement life-cycle disutility that is defined as the ratio of the pavement life-cycle cost to the area falling under the corresponding life-cycle performance curve. In other words, pavement life-cycle disutility is the monetary cost per unit area of pavement performance. The only variable used in the optimization process is the terminal serviceability index. The optimum design is the most cost-effective one as indicated by the lowest pavement life-cycle disutility value and yields the optimum terminal serviceability index for the given design conditions.

The traditional AASHTO design approach applies all specified parameters to obtain a measure of the required pavement structural strength through an index known as the Structural number (SN). The SN is then converted to pavement layers' thicknesses according to the relative strength of used materials as represented by layers' coefficients. The basic design equation used for flexible pavements is as follows:

$$\log W_{80} = Z_R S_o + 9.36 \log (\mathrm{SN} + 1) + \frac{\log \left[\frac{\Delta \mathrm{PSI}}{4.2 - 1.5}\right]}{0.40 + \frac{1094}{(\mathrm{SN}+1)^{5.19}}} + 2.32 \log (M_R) - 8.27 \quad (1)$$

where, W_{80} is the number of 80 kN equivalent single axle load (ESAL) applications estimated for a selected design period and design lane; Z_R the standard normal deviate for a specified reliability level; S_o the combined standard error of the traffic prediction and performance prediction; ΔPSI the difference between the initial or present serviceability index (P_o) and the terminal serviceability index (P_t); P_t the terminal serviceability index value that indicates the end of the pavement performance period and would prompt an agency action; SN the design structural number indicative of the total required pavement thickness and M_R is the subgrade design resilient modulus in pound per square inch.

Once the design SN is determined from Eq. (1), it is then converted to layers' thicknesses using Eq. (2). The designer needs to select an appropriate number of pavement layers as Eq. (2) allows for a maximum of three layers in the pavement structure.

$$\mathrm{SN} = a_1 D_1 + a_2 m_2 D_2 + a_3 m_3 D_3 \quad (2)$$

where, a_1, a_2 and a_3 are the layers' relative strength coefficients; m_2 and m_3 are the layers' drainage coefficients; and D_1, D_2 and D_3 are the layers' thicknesses in inches for surface, base and subbase, respectively. Equation (2) provides a large number of feasible solutions in terms of layers' thicknesses, however, any selected solution must satisfy the AASHTO recommended minimum thickness requirements for surface and base layers (AASHTO, 1993). The layers' thicknesses are sequentially calculated using Eq. (3).

$$D_1 = \frac{\mathrm{SN}_1}{a_1}, \quad D_2 = \frac{\mathrm{SN}_2 - a_1 D_1}{a_2 m_2}, \quad D_3 = \frac{\mathrm{SN} - a_1 D_1 - a_2 m_2 D_2}{a_3 m_3} \quad (3)$$

where, SN_1 is the structural number obtained from Eq. (1) using the base resilient modulus and $\mathrm{SN}_2$is the structural number obtained from Eq. (1) using the subbase resilient modulus.

METHODOLOGY

The optimum approach developed for the design of flexible pavements still applies the general approach recommended by AASHTO, but adds to it the performance-based feature discussed earlier with the intent of yielding an optimum design considered to be the most cost-effective. Two major requirements are needed to apply the developed optimum approach. The first requirement is the estimation of the pavement life-cycle cost associated with a particular pavement structure, which includes initial construction cost, maintenance cost and added user cost. The second requirement is the estimation of the area under the corresponding life-cycle performance curve. A pavement performance curve is generated for a particular pavement structure using an incremental solution of the AASHTO basic design equation. Pavement life-cycle disutility is simply calculated as the ratio of pavement life-cycle cost to the area under the corresponding life-cycle performance curve. Pavement life-cycle disutility for a given design is determined using varying values of the terminal serviceability index (P_t). The optimum design is the one associated with the minimum pavement life-cycle disutility value and yields the optimum terminal serviceability index.

Pavement Life-cycle Cost

The cost items included in the pavement life-cycle are mainly the initial construction cost, maintenance cost and added user cost. Other cost items such as traffic control costs and added shoulder borrow costs can be considered as well. The initial construction cost is estimated for the designed pavement structure by calculating the cost for each pavement layer individually based on prevailing market prices. The maintenance cost considered in this approach is the cost of routine maintenance estimated from the files of the transportation agencies. Routine maintenance includes maintenance activities such as crack sealing and pothole patching necessary to maintain safe road operating conditions. Routine maintenance can have a considerable impact on pavement life cycle cost. It is assumed to add very little to the pavement design life; therefore, it has no significant contribution to pavement performance (Yoder and Witczak, 1975; Abaza and Ashur, 1999). Added user cost is the additional cost incurred by the road users as a result of the deteriorating pavement condition. It includes both added vehicle running cost and added travel time cost, and it is directly related to the serviced traffic volume for a given pavement condition. Routine maintenance and added user costs are inversely related. The more maintenance work is performed, and thus more maintenance cost is incurred, the less will be the added user cost.

The pavement life-cycle cost associated with a particular pavement design needs to be estimated as a present sum for the purpose of making appropriate economical evaluations. The initial construction cost is already estimated as a present cost per square meter. The routine maintenance and added user costs are typically estimated on an annual basis and need to be converted to a present sum using economic principles. While their annual costs are variable over time, they are typically averaged out to obtain corresponding uniform annual costs per square meter. The pavement life-cycle cost is then estimated using Eq. (4),

$$P_{\mathrm{LC}} = P_{\mathrm{IC}} + \left[\frac{\mathrm{AC}_{\mathrm{RM}} + \mathrm{AC}_{\mathrm{AU}}}{\mathrm{CR}}\right] \tag{4}$$

where,

$$\mathrm{CR} = \left[\frac{r(1+r)^T}{(1+r)^T - 1}\right]$$

and P_{LC} is the present pavement life-cycle cost per square meter; P_{IC} the present pavement initial construction cost per square meter; $\mathrm{AC}_{\mathrm{RM}}$ the uniform annual routine maintenance cost per square meter; $\mathrm{AC}_{\mathrm{AU}}$ the uniform annual added user cost per square meter; CR the capitol recovery factor converting a uniform annual sum to a present one; r the uniform annual interest rate; and T is the pavement design life in years.

Potential pavement design alternatives can be evaluated using the life-cycle cost (P_{LC}) as the sole indicator with the design associated with the least cost selected. But, such an approach is not considered cost-effective because pavement life-cycle performance has not been accounted for in this evaluation.

Pavement Performance Prediction Model

A procedure that applies an incremental analysis of the AASHTO basic design equation has been developed (Abaza *et al.*, 2001) to construct flexible pavement performance curves. The procedure provides a simple tool to predict the pavement performance condition at any given future time. This procedure can be used in the absence of actual pavement performance condition data. The two main parameters defining performance are the Present serviceability index (PSI) and 80 kN ESAL applications. In the design mode and after all related parameters are estimated, Eq. (1) is solved for the design SN using a trial and error approach.

The approach used to define a pavement performance curve as a function of the present serviceability index and 80 kN ESAL applications or service time is based on the direct use of Eq. (1). The incremental 80 kN ESAL applications $(W_{80})_i$ are calculated by specifying varying values of the incremental change in the present serviceability index ($\Delta\mathrm{PSI}_i$). The incremental change in the present serviceability index is defined as the difference between the initial serviceability index (P_o) and the present serviceability index (PSI_i). The present serviceability index (PSI_i) is varied between its assigned initial value and its terminal one. Equation (1) is used to determine the incremental 80 kN ESAL applications $(W_{80})_i$ for a specified incremental change in the present serviceability index ($\Delta\mathrm{PSI}_i$).

Figure 1 shows the basic concept by which the difference between two successive data points can be used to construct a pavement performance curve. The estimated incremental change in load applications ($\Delta(W_{80})_{i,i+1}$) can then be converted into an equivalent incremental service time interval ($\Delta T_{i,i+1}$) using Eq. (5). The assumption made in establishing Eq. (5) is that the 80 kN ESAL applications increase linearly with time. A computer system has been designed using visual basic programming language with one of its main functions being the solving of the mathematical algorithm presented below:

$$\Delta T_{i,i+1} = \frac{\Delta(W_{80})_{i,i+1}}{(W_{80})_T} T \tag{5}$$

where, $\Delta(W_{80})_{i,i+1} = (W_{80})_{i+1} - (W_{80})_i,\ i = 1, 2, \ldots, n$; $(W_{80})_i = F(\Delta\mathrm{PSI}_i,\ \mathrm{SN},\ M_{\mathrm{R}},\ Z_{\mathrm{R}},\ S_o)$ from Eq. (1); $(W_{80})_{i+1} = F(\Delta\mathrm{PSI}_{i+1},\ \mathrm{SN},\ M_{\mathrm{R}},\ Z_{\mathrm{R}},\ S_o)$ from Eq. (1);

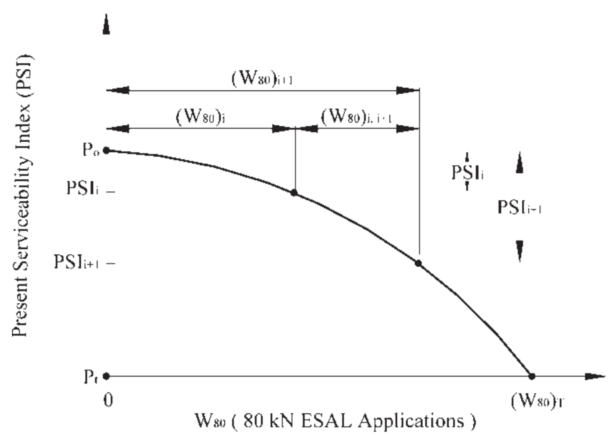

FIGURE 1 Basic pavement performance curve.

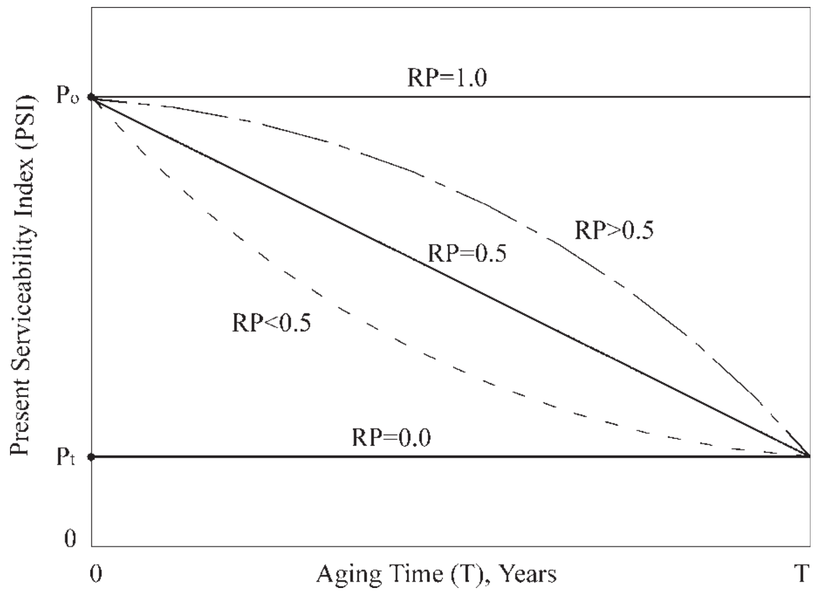

FIGURE 2 Performance curves with typical RP values.

$(W_{80})_T = \sum_{i=1}^{n} \Delta (W_{80})_{i,i+1}$ where $(W_{80})_T$ is the total number of 80 kN ESAL applications estimated over a design service life of T years; $\mathrm{SN} = F((W_{80})_T, \Delta\mathrm{PSI}, Z_\mathrm{R}, S_o, M_\mathrm{R})$ from Eq. (1); $T = \sum_{i=1}^{n} \Delta T_{i,i+1}$; and $N_{T_{i+1}} = \sum_i \Delta\ (W_{80})_{i,i+1} = (W_{80})_{i+1}, N_{T_1} = 0.0$ where $N_{T_{i+1}}$ is the cumulative number of 80 kN ESAL applications estimated over a design service life of T_{i+1} years.

Also, $T_{i+1} = \sum_i \Delta T_{i,i+1},\ T_1 = 0.0$ where T_{i+1} is the cumulative service time in years associated with the cumulative 80 kN ESAL applications ($N_{T_{i+1}}$).

In addition,

$$\Delta\mathrm{PSI}_i = P_o - \mathrm{PSI}_i$$

$$\mathrm{PSI}_i = P_o - (i-1)\Delta P, \quad i = 1, 2, \ldots, n+1$$

$$n = \frac{P_o - P_t}{\Delta P}$$

ΔP is the specified incremental change in the PSI value used to generate $(n+1)$ data points to be used in the construction of a particular pavement performance curve. It must be specified either as a tenth or hundredth of a point to ensure n will be an integer. In the computer system, one hundredth of a point has been specified with the corresponding computer time being very small. A performance curve is then constructed by plotting the present serviceability index (PSI_i) versus the cumulative aging time (T_{i+1}).

Relative Performance

Evaluation of pavement design alternatives can be made using a newly introduced indicator called relative performance (RP). Performance is defined as the integral of the present serviceability index versus aging time or cumulative 80 kN ESAL applications curve. Therefore, the area falling under the curve is by definition an indication of performance (Yoder and Witczak, 1975; Haung, 1993). RP is defined as the ratio of the area corresponding to a particular curve to that of a perfect performance curve. A perfect performance curve is the one represented by a hypothetical horizontal straight line indicating constant PSI value over time. The maximum theoretical value of RP is unity. Figure 2 shows typical performance trends along with their corresponding RP values as obtained when evaluating design alternatives with the same terminal serviceability index values. The pavement design that provides the best performance is the one associated with the highest RP value. Pavement life-cycle RP is mathematically stated by Eq. (6) when evaluating design alternatives with variable terminal serviceability index values,

$$A_\mathrm{LC} = \sum_{i=1}^{n} A_{i,i+1}$$

and

$$A_{i,i+1} = \left[\frac{1}{2}(\mathrm{PSI}_i + \mathrm{PSI}_{i+1}) - P_t\right]\Delta T_{i+1}$$

$$\mathrm{RP}_\mathrm{LC} = \frac{A_\mathrm{LC}}{(P_o - P_t)T} \tag{6}$$

where, RP_LC is the pavement life-cycle relative performance; A_LC the pavement life-cycle area representing the area under the performance curve and $A_{i,i+1}$ is a trapezoidal strip area bounded by two curve points.

The terminal serviceability index (P_t) will be assumed to be 1.5 in Eq. (6) since this value is the minimum permissible one as required by the AASHTO design method. It is with respect to this minimum value that the life-cycle RP and area are estimated when evaluating pavement design alternatives with variable terminal serviceability index values. Evaluation of pavement design alternatives based merely on life-cycle RP (RP_{LC}) is not considered an effective approach since it does not take into account the pavement life-cycle cost (P_{LC}) associated with each investigated design alternative. An effective approach is one that considers both pavement life-cycle performance and cost as will be presented in the subsequent subsection.

Pavement Life-cycle Disutility

The pavement life-cycle disutility is a newly introduced parameter identified as a means to replace both pavement life-cycle RP (RP_{LC}) and life-cycle cost (P_{LC}) with an effective single indicator used in evaluating potential pavement design alternatives. The pavement life-cycle disutility is defined as the ratio of life-cycle cost to life-cycle performance represented by the area falling under the performance curve. It simply assigns a monetary value to pavement performance and provides an effective mechanism by which potential design alternatives can be evaluated. The optimum pavement design is the one associated with the minimum pavement life-cycle disutility value. The pavement life-cycle disutility is simply calculated using Eq. (7). An equivalent alternative to using the life-cycle area in Eq. (7) is to use the life-cycle RP,

$$U_{LC} = \frac{P_{LC}}{A_{LC}} \quad \text{or} \quad U_{LC} = \frac{P_{LC}}{RP_{LC}} \tag{7}$$

where, U_{LC} is the pavement life-cycle disutility per unit area under the performance curve (US Dollars (USD)/m^2/year, or USD/m^2 if RP is used); P_{LC} the pavement life-cycle present worth cost (USD/m^2) obtained from Eq. (4); A_{LC} the pavement life-cycle area (year) under the performance curve obtained from Eq. (6); and RP_{LC} is the pavement life-cycle relative performance (unitless) as obtained from Eq. (6).

The developed approach for the design of flexible pavements is based on minimizing the pavement life-cycle disutility value with the terminal serviceability index (P_t) being the only considered variable. Other parameters required to be used in the AASHTO design method have to be fixed. The optimization process is simultaneously performed using selected practical terminal serviceability index values that are uniformly increased in the search for an optimum solution. The terminal serviceability index search values typically range from a 1.5 minimum to a maximum value defined to be equal to the initial serviceability index (P_o). An incremental uniform increase of 0.5 in the terminal serviceability index search value is considered adequate. However, with the use of high speed personal computers, a 0.1 incremental increase in the search value can effectively be deployed.

SAMPLE PRESENTATION

To illustrate the suggested optimum design approach, a sample problem is presented. The data requirements mainly consist of three types, namely: (1) data needed to generate the performance curves associated with the terminal serviceability index search values, (2) data needed to design a pavement structural section using the traditional AASHTO design method and (3) data needed to estimate the pavement life-cycle cost and determine the optimum design. The input and output data for the sample presentation is provided below for three traffic loading levels.

Performance Data

The following values have been assigned to various performance input parameters as required by the mathematical algorithm presented in Eq. (5):

$(W_{80})_T = 1.0 \times 10^6$, 5.0×10^6 and 10.0×10^6 for low, medium and high traffic loading levels, respectively, $M_R = 63$ MPa (9000 psi), $T = 20$ years, $P_o = 4.5$, $S_o = 0.35$, $Z_R = -1.645$ and $\Delta P = 0.01$.

Six distinct performance curves have been generated for the six different terminal serviceability index search values as shown in Fig. 3 for the low traffic loading level. The terminal search value for the sample presentation starts with 1.5 and ends with 4.0 using a 0.5 incremental increase as shown in Fig. 3 with all six curves having the same 4.5 initial serviceability index value. The design SN for each case is estimated using Eq. (1) prior to solving Eq. (5) with the corresponding values as provided in Table I. The pavement life-cycle area (A_{LC}) falling under each performance curve is then calculated along with the corresponding life-cycle RP value (RP_{LC}) using Eq. (6) with the results provided in Table II.

Pavement Design Data

The practitioner needs to select the number of layers to be included in a particular flexible pavement structure. A two-layer pavement structure has been selected for the sample presentation. The materials properties of the pavement structure as indicated by the layers' relative strength coefficients and resilient modules need to be specified. A high-stability asphalt-mix ($a_1 = 0.44$) and crushed limestone aggregates ($a_2 = 0.14$) have been selected. The selected aggregate layer coefficient corresponds to approximately 280 MPa (40,000 psi) resilient modulus value which is needed to determine the asphalt surface layer thickness. The layers' drainage coefficients are assumed to be unity (i.e. $m_2 = m_3 = 1.0$). The SN representing the asphalt layer strength requirement (SN_1) is estimated from Eq. (1) and then Eq. (3) is used to determine the corresponding thickness (D_1) and the thickness of the aggregate base layer (D_2). The calculated design thicknesses have been converted from inches to centimeters with D_1 rounded to half centimeter and D_2

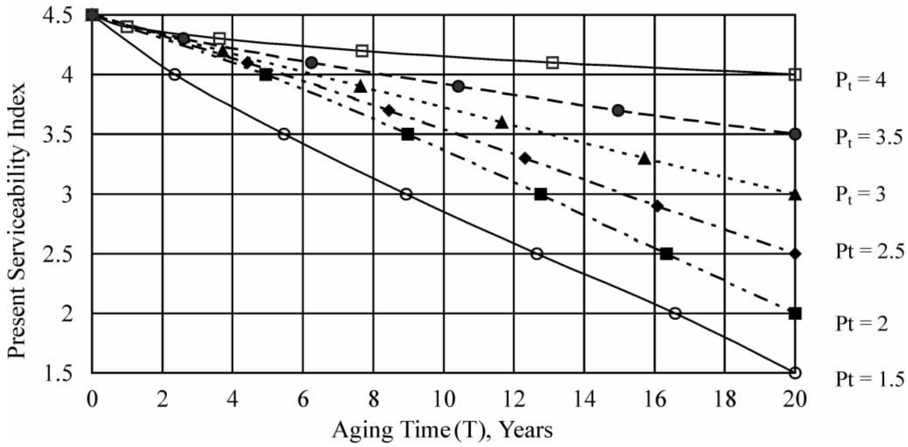

FIGURE 3 Generated pavement performance curves for low loading level ($(W_{80})_T = 1.0 \times 10^6$).

rounded to a full centimeter as provided in Table I for the low traffic loading level.

Pavement Life-cycle Cost

The life-cycle cost associated with each investigated pavement design needs to be estimated for the selection of the optimum design. There are three cost elements to be considered as outlined in the "Methodology" section. The initial construction cost of the pavement structural section is estimated from prevailing market prices and the actual costs of similar works. The initial construction cost in this sample presentation is estimated for each design by considering the unit cost of each layer. The unit cost for the asphalt layer is estimated at USD 90/m^3 and the unit cost for the aggregate base at USD 25/m^3 on the basis of market rates. The routine maintenance and added user costs are directly related to each other. In the presence of active routine maintenance program, the added user cost is significantly eliminated. A rational procedure to estimate routine maintenance cost as a function of the design SN and terminal serviceability index (P_t) is proposed by Eq. (8).

$$AC_{RM_{P_t}} = \left(\frac{SN_{1.5}}{SN_{P_t}}\right)^n AC_{RM_{1.5}} \tag{8}$$

where, $AC_{RM_{P_t}}$ is the uniform annual routine maintenance cost (USD/m^2) associated with a particular P_t value; $SN_{1.5}$ the design structural number associated with $P_t = 1.5$; SN_{P_t} the design structural number associated with a particular P_t value; $AC_{RM_{1.5}}$ the uniform annual routine maintenance cost (USD/m^2) associated with $P_t = 1.5$ and n is an appropriate power value, typically 3 or 4, that would recognize the exponential trend between pavement maintenance cost and pavement distress condition. A value of 4 has been used in the sample presentation.

Equation (8) provides a logical and systematic mechanism to estimate routine maintenance cost for inclusion in the pavement life-cycle cost (P_{LC}) to perform the necessary evaluations and comparisons when considering the same traffic loading level and subgrade design resilient modulus. The uniform annual routine maintenance cost (when $P_t = 1.5$) is estimated at USD 0.50/m^2 for the low loading level ($(W_{80})_T = 1.0 \times 10^6$), USD 1.00/m^2 for the medium loading level ($(W_{80})_T = 5.0 \times 10^6$) and USD 1.50/m^2 for the high loading level ($(W_{80})_T = 10.0 \times 10^6$). The uniform annual routine maintenance costs at other specified terminal serviceability index search values are calculated using Eq. (8), and are provided in Tables II–IV for the low, medium and high loading levels, respectively. Both the pavement initial

TABLE I Pavement layer thickness calculations for low loading level ($(W_{80})_T = 1.0 \times 10^6$)

Case No.	P_t	SN	SN_1	D_1(in.)*	D_2 (in.)*	D_1 (cm)†	D_2 (cm)†
1	1.5	2.996	1.780	4.05	8.67	10.5	22
2	2.0	3.061	1.790	4.07	9.07	10.5	23
3	2.5	3.154	1.801	4.09	9.67	10.5	25
4	3.0	3.305	1.818	4.13	10.63	10.5	27
5	3.5	3.610	1.843	4.19	12.62	11.0	32
6	4.0	4.633	1.892	4.30	19.58	11.0	50

* Computed values using Eq. (3) in inches.
† Rounded design values in centimeters.

TABLE II Sample optimum pavement design calculations for low loading level ($(W_{80})_T = 1.0 \times 10^6$)

Case No.	P_t	SN	AC_{RM} (USD/m^2)	P_{IC}* (USD/m^2)	P_{LC}† (USD/m^2)	A_{LC} (year)	RP_{LC}	U_{LC}* (USD/m^2/year)	U_{LC}† (USD/m^2/year)
1	1.5	2.996	0.50	14.95	20.69	28.00	0.467	0.534	0.739
2	2.0	3.061	0.46	15.20	20.48	36.84	0.614	0.413	0.556
3	2.5	3.154	0.41	15.70	20.40	40.72	0.679	0.386	0.501
4	3.0	3.305	0.34	16.20	20.10	44.70	0.745	0.362‡	0.450
5	3.5	3.610	0.24	17.90	20.65	48.85	0.814	0.366	0.423‡
6	4.0	4.633	0.09	22.40	23.43	53.00	0.883	0.423	0.442

* Excluding routine maintenance cost (Case No. 4 is the optimum).
† Including routine maintenance cost (Case No. 5 is the optimum).
‡ Optimum pavement life-cycle disutility values.

construction cost (P_{IC}) and life-cycle cost (P_{LC}) are provided in Tables II–IV with the added user cost eliminated. The pavement life-cycle cost (P_{LC}) has been computed using Eq. (4) for 20 years design life and 6% uniform annual interest rate.

Optimum Pavement Design

The optimum pavement design is the one associated with the lowest pavement life-cycle disutility value (U_{LC}) determined as the ratio of the pavement life-cycle cost (P_{LC}) to the pavement life-cycle area (A_{LC}). Table II provides the life-cycle disutility values for the six design cases investigated with the design corresponding to a 3.0 terminal serviceability index (P_t) value being the optimum one (Case No. 4) when excluding routine maintenance costs. The optimum terminal serviceability index increased to 3.5 when routine maintenance cost was included (Case No. 5). The optimum terminal serviceability index is in disagreement with the AASHTO recommendation of 2.0 for low traffic loading. Table II also shows that the optimum design is not the one associated with the highest life-cycle RP value (RP_{LC}) and it is neither the one associated with the lowest life-cycle cost value (P_{LC}).

Tables III and IV provide similar results for the medium and high loading levels, respectively, with all other input data unchanged except for the cost of routine maintenance. The optimum design, for both loading levels, is the one represented by Case No. 5 that corresponds to a 3.5 terminal serviceability index value when excluding routine maintenance cost. The optimum design remains Case No. 5 for medium loading, and becomes Case No. 6 for high loading, which corresponds to a 4.0 terminal serviceability index value, when routine maintenance cost was included. Again, these optimum terminal values are higher than the recommended AASHTO value of 2.5 for major high volume roads. The pavement life-cycle costs (P_{LC}) are directly proportional to the traffic loading level as one would expect.

Figures 3 and 4 show the performance curves associated with the low and high traffic loading levels, respectively. Figure 4 shows the high traffic performance curves to be inferior (i.e. higher deterioration rates) to the ones associated with the low traffic loading level (Fig. 3) especially for lower terminal serviceability index values. This provides another justification for the AASHTO recommendation of a higher terminal serviceability index in the case of high traffic loading condition. The resulting pavement life-cycle areas (A_{LC}) are therefore lower and the pavement life-cycle disutility values are higher for the designs corresponding to the high traffic loading level.

Generally, the optimum pavement life-cycle disutility value becomes higher when routine maintenance and added user costs are added to the pavement life-cycle cost resulting in even a higher optimum terminal serviceability index value as previously indicated. Routine maintenance cost is inversely proportional to the terminal serviceability index value as stated by Eq. (8) whereas initial construction cost is directly proportional. The pavement life-cycle costs (P_{LC}) generally become higher but with a minimum low value, observable in Tables II–IV, resulting

TABLE III Sample optimum pavement design calculations for medium loading level ($(W_{80})_T = 5.0 \times 10^6$)

Case No.	P_t	SN	AC_{RM} (USD/m^2)	P_{IC}* (USD/m^2)	P_{LC}† (USD/m^2)	A_{LC} (year)	RP_{LC}	U_{LC}* (USD/m^2/year)	U_{LC}† (USD/m^2/year)
1	1.5	3.717	1.00	18.40	29.87	25.72	0.429	0.715	1.161
2	2.0	3.845	0.87	19.02	29.00	32.84	0.547	0.579	0.883
3	2.5	4.031	0.72	19.98	28.24	37.57	0.626	0.532	0.752
4	3.0	4.332	0.54	21.35	27.54	42.46	0.708	0.503	0.649
5	3.5	4.898	0.33	23.85	27.63	47.61	0.794	0.501‡	0.580‡
6	4.0	6.224	0.13	30.20	31.69	52.76	0.879	0.572	0.601

* Excluding routine maintenance cost (Case No. 5 is the optimum).
† Including routine maintenance cost (Case No. 5 is the optimum).
‡ Optimum pavement life-cycle disutility values.

TABLE IV Sample optimum pavement design calculations for high loading level ($(W_{80})_T = 10.0 \times 10^6$)

Case No.	P_t	SN	AC_{RM} (USD/m^2)	P_{IC}* (USD/m^2)	P_{LC}† (USD/m^2)	A_{LC} (year)	RP_{LC}	U_{LC}* (USD/m^2/year)	U_{LC}† (USD/m^2/year)
1	1.5	4.067	1.50	20.50	37.71	23.43	0.390	0.875	1.609
2	2.0	4.228	1.28	21.00	35.68	28.84	0.481	0.728	1.237
3	2.5	4.459	1.04	22.20	34.13	34.42	0.574	0.645	0.992
4	3.0	4.823	0.76	23.90	32.62	40.22	0.670	0.594	0.811
5	3.5	5.470	0.46	26.85	32.13	46.37	0.773	0.579‡	0.693
6	4.0	6.886	0.18	33.35	35.41	52.52	0.875	0.635	0.674‡

* Excluding routine maintenance cost (Case No. 5 is the optimum).
† Including routine maintenance cost (Case No. 6 is the optimum).
‡ Optimum pavement life-cycle disutility values.

in higher life-cycle disutility and terminal serviceability index values as the pavement life-cycle areas remain unchanged for the same loading level and subgrade design resilient modulus. Therefore, the resulting higher optimum serviceability index strongly supports the design and construction of higher quality pavements as they are definitely cost-effective as demonstrated in this sample presentation.

Sensitivity Analysis

In this section, the impact of variable subgrade resilient modulus values, traffic loading levels, initial construction cost levels and the inclusion of one major rehabilitation cycle on optimum solutions is investigated.

The impact of the subgrade design resilient modulus on the optimum results is investigated using five different resilient modulus values that range from 21 MPa (3000 psi) to 105 MPa (15,000 psi) with 21 MPa (3000 psi) incremental increase. Table V summarizes the optimum pavement life-cycle disutility values in relation to their corresponding optimum terminal serviceability index values for the three previously considered traffic loading levels, namely: 1.0×10^6, 5.0×10^6 and 10.0×10^6. For the case of low traffic loading, four optimum designs have resulted in a 3.0 corresponding terminal serviceability index value with one design resulted in a 3.5 optimum terminal value which is the one associated with a 21 MPa (3000 psi) resilient modulus value. All five optimum designs associated with the medium and high loading levels have resulted in a 3.5 terminal serviceability index value when considering only initial construction cost. Thus, the optimum terminal serviceability indices obtained, based on a 4.5 initial serviceability index value, appear to be independent of the subgrade design resilient modulus.

The impact of the initial construction cost levels on optimum life-cycle disutility is investigated using six combinations of cost levels as provided in Table VI. The resulting optimum terminal serviceability index values remained unchanged for all three loading levels when including routine maintenance costs. The corresponding values are 3.5 for low and medium loading levels, and 4.0 for high loading level. As would be expected, the life-cycle disutility values are directly proportional to the initial construction cost levels.

The impact of incorporating one major rehabilitation treatment into the life-cycle is also investigated. The assumed rehabilitation treatment, applied when reaching the terminal serviceability, consists of partial or full removal of the existing asphaltic layer (D_1) and replacement with new material. The required thickness

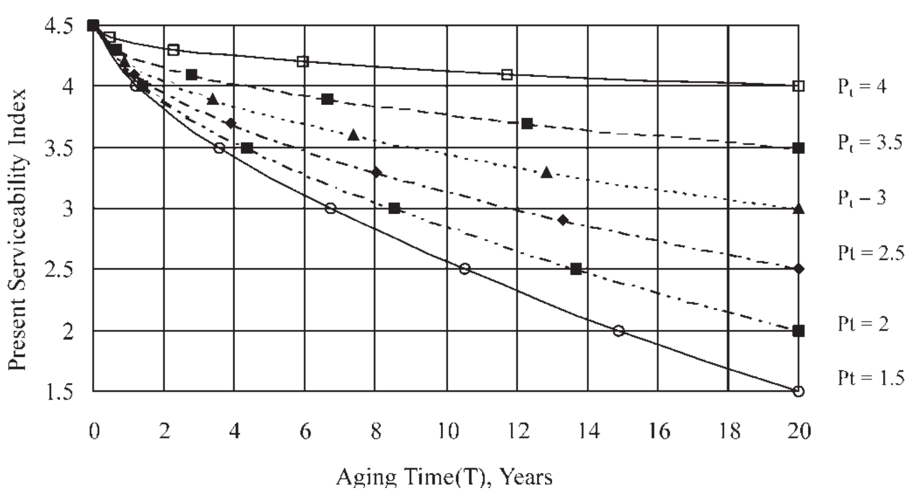

FIGURE 4 Generated pavement performance curves for high loading level ($(W_{80})_T = 10.0 \times 10^6$).

TABLE V Impact of subgrade resilient modulus on pavement life-cycle disutility values

Resilient modulus (M_R) MPa	Load level	Terminal serviceability index (P_t) 1.5	2.0	2.5	3.0	3.5	4.0
21 MPa (3000 psi)	L	0.899	0.749	0.652	0.599	0.592*	0.640
	M	1.192	0.982	0.844	0.774	0.752*	0.821
	H	1.440	1.138	0.955	0.851	0.809*	0.865
42 MPa (6000 psi)	L	0.597	0.523	0.479	0.448*	0.452	0.508
	M	0.834	0.714	0.644	0.604	0.594*	0.665
	H	1.075	0.867	0.759	0.693	0.659*	0.714
63 MPa (9000 psi)	L	0.534	0.413	0.386	0.362*	0.366	0.423
	M	0.715	0.579	0.532	0.503	0.501*	0.572
	H	0.875	0.728	0.645	0.594	0.579*	0.635
84 MPa (12,000 psi)	L	0.372	0.348	0.321	0.314*	0.316	0.350
	M	0.540	0.500	0.456	0.443	0.441*	0.499
	H	0.732	0.638	0.573	0.534	0.521*	0.584
105 MPa (15,000 psi)	L	0.326	0.303	0.289	0.276*	0.278	0.294
	M	0.473	0.436	0.412	0.396	0.394*	0.438
	H	0.642	0.566	0.519	0.487	0.474*	0.541

L, Low; M, Medium; H, High.
* Optimum pavement life-cycle disutility values.

(t) of the new asphaltic material is estimated from Eq. (9), which provides a logical and systematic mechanism to estimate thickness as a proportion of the existing asphaltic layer thickness (D_1).

$$t = D_1\left[1 - \left(\frac{P_t}{P_o}\right)^n\right]G_\mathrm{f} \tag{9}$$

A traffic growth factor (G_f) has been applied in Eq. (9), and assumed to be 1.2 in the sample presentation. The power (n) is again introduced to account for the exponential trend between pavement rehabilitation cost and pavement distress condition. A value of 3 or 4 is recommended for (n) with 3 being used in the sample presentation. The present value of life-cycle rehabilitation cost (P_R) is calculated by assuming USD 150/m^3 present cost rate (removal plus replacement), 3% annual inflation rate and 6% annual interest rate. The resulting net present rehabilitation cost rate is about USD 85/m^3.

The present life-cycle rehabilitation cost (P_R) is provided in Table VII for the three considered loading levels. The corresponding life-cycle cost (P_{LC}) includes initial construction cost, routine maintenance cost (40 years analysis period), and the cost of one rehabilitation cycle. The life-cycle disutility values are obtained assuming that the applied rehabilitation treatment would provide new performance curves identical to those shown in Figs. 3 and 4; therefore, the life-cycle area (A_{LC}) for a particular terminal serviceability index value is twice the area under one performance curve. Table VII shows that

TABLE VI Impact of construction cost rates on optimum life-cycle disutility values

Initial construction cost rates (USD/m^3) Surface	Base	Load level	Terminal serviceability index (P_t) 1.5	2.0	2.5	3.0	3.5	4.0
120	30	L	0.891	0.673	0.609	0.550	0.523*	0.552
		M	1.367	1.048	0.902	0.789	0.716*	0.750
		H	1.842	1.440	1.166	0.969	0.847	0.846*
110	30	L	0.853	0.644	0.583	0.527	0.501*	0.531
		M	1.315	1.007	0.865	0.756	0.686*	0.721
		H	1.779	1.388	1.122	0.931	0.812	0.810*
100	25	L	0.776	0.584	0.527	0.473	0.445*	0.463
		M	1.262	0.925	0.789	0.682	0.609*	0.630
		H	1.657	1.285	1.029	0.844	0.725	0.712*
90	25	L	0.739	0.556	0.501	0.450	0.423*	0.442
		M	1.161	0.883	0.752	0.649	0.580*	0.601
		H	1.609	1.237	0.992	0.811	0.693	0.674*
80	20	L	0.662	0.491	0.430	0.396	0.367*	0.374
		M	1.060	0.801	0.675	0.574	0.504*	0.510
		H	1.473	1.130	0.893	0.718	0.602	0.577*
70	20	L	0.625	0.468	0.419	0.372	0.345*	0.353
		M	1.008	0.759	0.638	0.541	0.474*	0.480
		H	1.410	1.078	0.849	0.680	0.567	0.541*

L, Low; M, Medium; H, High.
* Optimum pavement life-cycle disutility values.

TABLE VII Impact of one rehabilitation cycle on optimum life-cycle disutility values

	Low loading			Medium loading			High loading		
P_t	P_R (USD/m^2)	P_{LC} (USD/m^2)	U_{LC} (USD/m^2/year)	P_R (USD/m^2)	P_{LC} (USD/m^2)	U_{LC} (USD/m^2/year)	P_R (USD/m^2)	P_{LC} (USD/m^2)	U_{LC} (USD/m^2/year)
1.5	10.31	32.78	0.585	13.26	46.70	0.908	14.44	57.50	1.227
2.0	9.77	31.89	0.433	12.56	44.66	0.680	13.86	54.11	0.938
2.5	8.87	30.74	0.377	11.83	42.64	0.567	12.84	50.68	0.736
3.0	7.54	28.85	0.322	10.05	39.52	0.465	11.20	46.53	0.578
3.5	5.94	27.45	0.281	7.56	36.37	0.382	8.86	42.63	0.460
4.0	3.34	27.09	0.256*	4.71	36.86	0.349*	5.77	41.83	0.398*

* Optimum pavement life-cycle disutility values.

the life-cycle disutility values have all decreased when compared to the corresponding values provided in Tables II–IV. It also shows that the optimum terminal serviceability index values increased to 4.0 for all three loading levels.

The foregoing analysis provides strong support for not using the general AASHTO recommendations of 2.0 and 2.5 terminal serviceability index values for low and high traffic loading conditions, respectively. Instead, the presented approach should be used to obtain the most cost-effective design that yields the corresponding optimum terminal serviceability index value. It can also be concluded that as the traffic loading level increases, the optimum terminal serviceability index increases as indicated by the three analyzed loading levels. This conclusion is in agreement with the AASHTO recommendation of a higher terminal serviceability index value in the case of a high loading condition.

CONCLUSIONS AND RECOMMENDATIONS

A simple and cost-effective optimum design approach based on the traditional AASHTO design method for flexible pavements has been presented. The data requirements for the optimum approach are very similar to those of the traditional one. However, additional data are needed for the pavement life-cycle performance and cost estimation. The pavement life-cycle performance is identified by the area under the performance curve generated using the presented prediction model with minimal data requirement. The pavement life-cycle cost can be estimated using solely the initial construction cost as the first attempt to obtain an optimum design. Pavement routine maintenance cost is definitely a major cost element that should be considered unless the highway agency is not really planning on doing much of it during the design life. This would have serious impact on the added user cost, which is unfortunately neglected from the consideration of many highway agencies, especially in developing countries. The presented optimum design approach has considered the impact of one major rehabilitation cycle. Major rehabilitation has significantly affected the pavement life-cycle performance and cost, which resulted in reduced optimum life-cycle disutility values. Application of multiple cycles of rehabilitation has been considered in a separate research paper with the intent of establishing an optimum pavement rehabilitation program (Abaza, 2002).

The results obtained from the sample presentation have yielded optimum pavement designs that are cost-effective as indicated by the lowest pavement life-cycle disutility values. The optimum pavement design simply means paying less money for better pavements, which is the essence of pavement management. The results have also indicated a significant disagreement with the AASHTO recommendations for selecting a design terminal serviceability index value in relation to the traffic loading condition. The AASHTO recommends 2.0 and 2.5 for low and high traffic loading conditions, respectively. The presented sample design results have indicated optimum 3.0 and 3.5 terminal serviceability index values for low and high traffic loading levels, respectively, when considering only initial construction cost. These optimum terminal index values become even higher when routine maintenance and major rehabilitation costs are included.

The significant increase in the value of the design terminal serviceability index clearly supports the design and construction of better quality pavements in relation to the AASHTO recommendations since they are indeed cost-effective as supported by the sample results. Therefore, it is recommended that for any flexible pavement design, the presented optimum design approach be applied using a 0.5 incremental increase in the terminal serviceability index search value. A computer system has been designed that can effectively be applied to reach optimal solutions using a 0.1 incremental increase in the search value with the corresponding computer time being very small. Finally, the obtained results are in agreement with the AASHTO recommendation of using higher terminal serviceability index for higher traffic loading conditions.

Acknowledgements

The authors wish to acknowledge the assistance of Eng. Fawz Kobari and Eng. Salim Abu-Thaher in the production of the presented figures.

References

Abaza, K.A. (2002) An optimum flexible pavement life-cycle analysis model, *J. Transportation Eng.* 128, pp. 542–549, ASCE.

Abaza, K.A. and Ashur, S.A. (1999) “An optimum decision policy for management of pavement maintenance and rehabilitation”, *Transportation Research Record* **1655** (TRB, Washington, DC), pp 8–15.

Abaza, K.A., Ashur, S.A., Abu-Eisheh, S.A. and Rabay’a, A. (2001) “An optimum macroscopic system for management of pavement rehabilitation”, *Journal of Transportation Engineering* **127**(6), 493–500, ASCE.

American Association of State Highway and Transportation Officials (1993) *AASHTO Guide for Design of Pavement Structures*, Washington, DC.

Haung, Y.H. (1993) *Pavement analysis and design* (Prentice Hall, Englewood Cliffs, NJ).

Yoder, E.J. and Witczak, M.W. (1975) *Principles of pavement design*, 2nd Ed. (Wiley, New York).