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Evolution Analysis of Asphalt Pavement Performance in Its Life Cycle: Case Study in Qinghai–Tibet Highway

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Abstract: Owing to the combination of bitumen aging, traffic loading, and environmental factors, the performance of asphalt will gradually deteriorate with time. However, characterizing the deterioration is still challenging. Aiming to reveal how the performance of asphalt pavement deteriorates with time, the AASHTO design equation was applied to investigate the evolution trend of pavement performance by adopting a reliability method in terms of freeze–thaw cycles. It was found that the combination of the rate of evolution and curvature could identify the abrupt change points and significant variation stages. Risk analysis was introduced to provide a novel method to evaluate the pavement performance evolution by identifying the change of the hazard rate and the cumulative hazard rate. It was found that the evolution curve of asphalt pavement strength reliability could be divided linearly during its life cycle, which can be extended to any n-stage linear deterioration model according to the actual situation. Moreover, reliability levels for pavement strength were also proposed in this research according to the integrated pavement travel and structure performance.

Keywords: asphalt pavement; evolution; freeze–thaw cycles; reliability method; risk analysis

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1. Introduction

Due to the aging of asphalt binders and environmental effects, such as lights, thermals, and UV, combined with repeated loading, the performance of asphalt pavement will inevitably deteriorate [1–4]. Transportation, especially highways, contributes a lot to the development of economy, society, and communities. The pavements should provide a reasonably smooth riding surface and adequate surface friction, which is normally the texture that is conducive to safety, have a sufficient structural capacity to protect the subgrade, as well as provide a waterproof function to prevent the underlying support layers from becoming saturated [5–7]. However, pavements gradually deteriorate under the combined effect of traffic and environmental loads. Generally, its deterioration rate is relatively slow in the first 1–10 years after construction, and becomes faster thereafter [8,9].

Asphalt pavement's performance and serviceability are prone to deteriorating compared with other infrastructures, such as railway tracks, since it is sensitive to temperature and moisture. Thermal cracks, moisture damages, and other low-temperature distresses are the key issues for asphalt pavement in cold regions [10–12]. Numerous studies have shown that pavement's performance and serviceability are significantly influenced by freeze–thaw (F-T) cycles, which accelerate the deterioration and damage rate of pavement [13–15]. The F-T cycles result in stripping and potholes in asphalt mixtures, which causes premature distress and accelerates the degradation of the pavement load capacity under repeated vehicle loads [13,14,16].

Highway agencies are interested in developing performance indicators that can be used to predict pavement deterioration trends and optimum maintenance plans that can be applied to enhance pavement performance. Since preservation treatments can be more cost-effective than rehabilitation or reconstruction, most highway agencies now implement sophisticated pavement management systems by developing curves to track and predict the future conditions of pavements. The consensus in improving pavement management systems is to incorporate a performance-based funding policy, where management decisions are based on pavement performance and the performance measures are quantifiable and outcome-driven. The challenge for highway agencies exists in the pavement performance analysis and decision making, while also obtaining accurate attenuation trends of pavement performance and acquiring the levels of uncertainty during its life cycle [17,18].

Life-cycle assessment is defined as the compilation and evaluation of the inputs, outputs, and the potential environmental impacts of a product system throughout its life cycle [19]. Due to its enormous environmental and economic impacts, there is a growing interest in accurately quantifying the life-cycle performance of pavements. Currently, agencies all over the world (especially in North America) are striving to extend the knowledge of sustainable highway practices beyond construction and into pavement preservation and maintenance [20].

Reliability models are probabilistic models which predict the failure probability of a given system. The limit-state function for asphalt pavements is typically defined as the difference between the designed load applications and the traffic prediction. The supply of a pavement is a criterion that can withstand certain loads before failing, and the number of loads applied is called the demand [21,22]. If there is a clear definition of a failure event and the consequences of that failure, reliability models can be effectively used to predict the performance and lifetime of pavements [21,22].

The frequent and harsh F-T cycles have a significant impact on asphalt pavement performance in cold regions. In these regions, the low temperature increases the stiffness of asphalt binders, leading to poorer elastic recovering abilities and thereby increasing the risk of the thermal cracking of the asphalt pavement. Considering these limitations, this paper applies a reliability method to analyze the asphalt pavement (AASHTO) design equation based on the variation of asphalt pavement performance under F-T cycles [22,23].

However, it is still challenging to characterize the deterioration of the performance of asphalt pavement, especially for the asphalt pavement in extreme weather, such as in the Qinghai–Tibet Plateau. Moreover, appropriate predictions of pavement performance deterioration are beneficial for maintenance planning; however, the relative research in Qinghai–Tibet is still very limited.

In this paper, the American Association of State Highway and Transportation Officials (AASHTO) asphalt pavement design equation was applied to study the evolution trend in the pavement life cycle. Compression properties of asphalt concrete under F-T cycles were considered to reflect the natural environmental influence in cold regions. The objectives of this research are to establish the asphalt pavement performance evolution model by applying a reliability method based on the AASHTO design equation. In addition, the reliability method is used to illustrate the impact of uncertainty on pavement performance during its life cycle, and a novel assessment method is proposed to reveal the pavement performance deterioration. The scope of the paper is confined to the consideration of the variability associated with the response and utilization of pavements.

2. Pavement Parameters and Reliability Method

2.1. Pavement Parameters

The research objective of the paper concerns a typical highway in the Qinghai–Tibet cold region. Such pavement is divided into three layers: the surface, base, and sub-base. The surface layer is made of asphalt concrete (i.e., AC): 5 cm AC-13 + 5 cm AC-20 + 7 cm AC-25. The base layer consists of a 1 cm crushed stone seal coat and a 20 cm 4% cement-stabilized crushed stone. The sub-base layer is composed of a 20 cm 5% cement-stabilized gravel. Under the sub-base is a 20 cm gravel cushion layer. Usually, the 1 cm (0.39 in) crushed stone seal coat is considered as a part of the base layer. Most pavement parameters in this paper come from specifications and other references [21,22]. The equivalent resilient modulus (MR) for the base and sub-base are 600 MPa (87,000 psi) and 207 MPa (30,000 psi), respectively. The resilient modulus for the gravel cushion layer is 76 MPa (11,000 psi). An equivalent resilient modulus for the subgrade is 39.3 MPa (5700 psi). The drainage index for the base, sub-base, and cushion layers is 1.2, 1, and 0.8, respectively. The coefficients of variation of the base layer, sub-base layer, and cushion layer are 0.22, 0.14, and 0.08, respectively. In this paper, the resilient modulus for the asphalt surface layer was determined from freeze–thaw (F-T) cycle tests.

2.2. AASHTO Design Equation

The AASHTO design method is one of the most widely used methods for designing flexible pavement structures. This design procedure is based on the results from the accelerated pavement testing, known as the AASHO road test [23]. Without considering the term that corrects for the overall variance, the strength of the pavement is defined as:

$$\log W_{18} = 9.36 \log(SN+1) - 0.20 + \frac{\log[\Delta PSI/(4.2-1.5)]}{0.4 + 1.094/(SN+1)^{5.19}} + 2.32 \log M_r - 8.07 \quad (1)$$

where W_{18} is the allowable number of the equivalent 18-kip (80 kN) single-axle loads (ESALs) to cause the reduction of the serviceability level by the amount of ΔPSI; SN is the structural number; M_r is the equivalent resilient modulus of the subgrade soil.

The SN has been described as "an abstract number representing the structural strength of a pavement", which can be calculated as follows [23],

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

where a_1, a_2, and a_3 are the layer coefficients for the surface, base, and sub-base, respectively. D_1, D_2, and D_3 are the thicknesses of the surface, base, and sub-base layers, respectively; m_2 and m_3 are the drainage coefficients for the base and sub-base layers, respectively.

2.3. Calculation of SN under F-T Cycles

The strength and modulus of the AC decline as the number of F-T cycles increases, resulting in a reduction of the pavement's structural capacity. Previous research shows that the SN is able to reflect the structural capacity of the pavement structure. As shown in Equation (2), the layer coefficient, thickness, and drainage coefficient codetermine the SN of a pavement structure. This paper concentrates on the impacts of F-T cycles on the AC layer coefficient.

The coefficients are indicators of the relative ability of a material to function as a structural component in the asphalt pavement. According to the AASHTO 1986 Guide, the resilient modulus of a material is the primary approach to determine its layer coefficient. Layer coefficients for layers made of different materials can be determined using empirical equations derived from field experiments. The following AC coefficient equation is the commonly used relationship.

$$a_1 = 0.4 \times \log\left(\frac{E}{3000\ MPa}\right) + 0.44\ 0.20 < a_1 < 0.44 \quad (3)$$

where a_1 is the AC layer coefficient; E is the resilient modulus of the asphalt mixture, which, in this paper, is not constant but is influenced by F-T cycles. The relationship between the AC coefficients and resilient moduli is generally based on the layered elastic theory.

The AC coefficient $a_{1(F\text{-}T)}$ and the SN after F-T cycles can be obtained by combining Equation (2) with Equation (3). Due to the uncertainty of the resilient modulus under F-T cycles, the $SN_{F\text{-}T}$ maintains the same uncertainty degree as the resilient modulus.

Si et al. investigated and presented the influence of F-T cycles on asphalt materials, and an exponential model was used to fit the variation of the compressive strength and resilient modulus of asphalt mixtures under F-T cycles. The used formula is as below [22,23].

$$y = a + b \cdot e^{c \times x} + \varepsilon \ \varepsilon \sim N(0, \delta^2) \quad (4)$$

where y is the explained variable, which here denotes the compressive strength and resilient modulus (MPa); a is the parameter that represents the initial condition; b is the parameter that represents the rate at which quality deteriorates with F-T cycles; c is the parameter that represents the curvature of the function; x is the number of F-T cycles; ε is the error term. The meaning of the random variable ε is assumed to be zero; the variance is homogeneous and uncorrelated with the dependent variables. Fitting results of the exponential model were shown in Table 1.

Table 1. Fitting results of the exponential model [22,23].

	a		b		c		Statistics	
	Value	**Error**	**Value**	**Error**	**Value**	**Error**	**Reduced Chi-Sqr**	R^2_{adj}
Compressive Strength (MPa)	0.21	0	1.90	0.02	−0.02	0.001	8.78×10^{-4}	0.95
Resilient Modulus (MPa)	78.53	0	675.84	17.85	−0.02	0.003	6.01×10^{-4}	0.81

2.4. Developing the Reliability Function for the AASHTO Design Equation

The supply analysis deals with the material properties and structural design characteristics contributing to the pavement strength, whereas the demand analysis concerns the traffic prediction [24].

The demand equation can be defined as the accumulated number of the equivalent 18-kip (80 kN) single-axle loads during the elapsed time. With a modified yearly traffic in *ESALs* for the initial year ($ESAL_0$) and the total growth factor (TGF), the demand or the accumulated *ESALs* for an arbitrary time period t can be calculated as [25,26],

$$N(t) = ESAL_0 \times TGF = ESAL_0 \times \frac{(1+r)^t - 1}{r} \quad (5)$$

where r is the yearly growth rate and t is the design period.

Given that the demand is defined by Equation (5) and the supply is defined by Equation (1), the limit-state function can be expressed as,

$$G = W_{18} - N(t) \quad (6)$$

Based on the limit-state function, the noncompliance (failure domain) can be defined as $\{G(SN_{F\text{-}T}, \Delta PSI, M_r, ESAL, r, t)\}$, and the probability integral is as follows:

$$F(t) = \int_{G(SN_{F-T}, \Delta PSI, M_r, ESAL_0, r, t) \le 0} f(SN_{F-T}, \Delta PSI, M_r, ESAL_0, r, t) \times dSN_{F-T} d\Delta PSI dM_r dESAL_0 dr \quad (7)$$

The integral Equation (7) can be calculated by the Monte Carlo simulation (MCS). The MCS is a computational algorithm that relies on repeated random sampling to obtain numerical results. With the development of computer technology and programming, the limitations of the MCS with respect to computational intensiveness and mathematical programming that is difficult to implement are not presently critical issues. The basic characteristics of the Monte Carlo method are that the convergence rate of the simulation has nothing to do with the dimension of a basic random vector, and the complexity of the limit-state function has nothing to do with the simulation process. The failure probability of the structure can be obtained after a large number of random variables are sampled and the sampling results are counted. The problem can be solved directly without the linearization of the state function and the equivalent normalization of random variables. The result becomes more accurate with the increase in the sample data, so it is also recognized as a relatively accurate method. Considering this, this research applies the MCS to analyze the limit-state function of the reliability.

2.5. Parameters in Reliability Analysis

By substituting the $SN_{F\text{-}T}$ (after F-T cycles) into the strength equation (i.e., Equation (7)), the reliability index after F-T cycles is obtained. The stress equation is defined by the accumulative equivalent of single-axle loads (80 kN). The traffic volume in this paper refers to China's Specifications for Design of Highway Asphalt Pavement (JTG D50-2006). Take a single lane's equivalent single-axle loads (*ESALs*) as 2.5×10^7 for a typical Qinghai–Tibet highway that has a 15-year design service life. The annual growth factor r is taken as 0.5. Substitute accumulative *ESALs* into Equation (5), and the single-axle $ESAL_0$ is obtained as 1,158,000. In this paper, the reduction of the present serviceability index ΔPSI is taken as 2.

3. Assessment Method for Asphalt Pavement Performance

3.1. Assessment of Asphalt Pavement Performance Evolution Trends

Asphalt pavement performance evolution trends include the deterioration periods and rates. Asphalt pavement deterioration was divided into different periods according to its deterioration rate, which can be regarded as a reference of the highway design, assessment, and decision making. Figure 1 shows asphalt pavement performance evolution trends. The compressive strength and compressive resilience modulus were tested experimentally; then, statistics analysis was performed to build the link with the coefficient of variation (CV). In the figure, the reliability trends vary under the different values of the coefficient of variation. As a result, quantifying the deterioration at a specific time would influence the division of the asphalt pavement performance evolution trends.

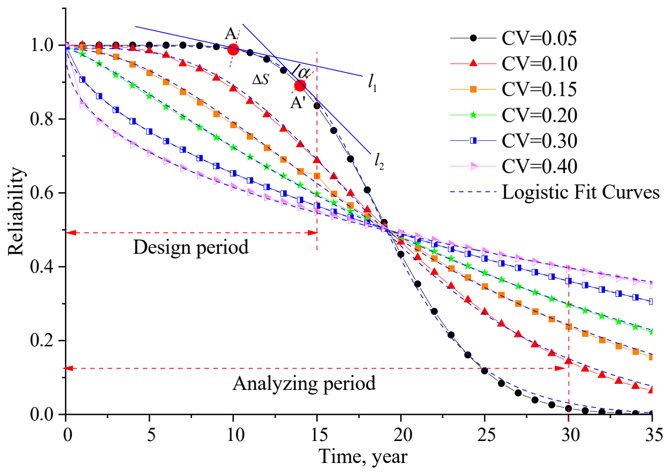

Figure 1. Analysis of evolution trends for pavement performance reliability with time (A and A′ are the selected data points and l_1 and l_2 are the representatives of the slopes).

Rheological Properties

Figure 1 indicates that the pavement performance reliability trends show continuous nonlinear variation. The pavement performance deterioration was classified into three types: convex curve, concave curve, and anti-S curve. The three deterioration types are shown in Figure 1. The concave curve and anti-S curve appear in the analyzing period (30 years). The convex curve and anti-S curve appear in the design period (15 years). Therefore, the pavement performance reliability trends in Figure 1 are representative.

In Figure 1, the convex curve and anti-S curve indicate the trend of 'gentle change-slight change-dramatic change', and they can represent the performance trend of the newly built asphalt pavement structure. For the convex curves, they appear when the CV is relatively small, at which point the reliability deteriorates slowly, which is a result of qualified or excellent construction. It always combines with the anti-S curves when the analyzed range exceeds the design period. In the concave curve, the performance of the pavement decreases from the very beginning, which is uncommon for a newly built asphalt pavement structure. Such concave curves occur only when the highway is unqualified or suffers extreme disasters. The three types of evolution trends can also be applied to the pavement performance after maintenance and rehabilitation.

The pavement performance deterioration trend is closely related to the evolution trend regarding the variation characteristics of the curves. The concept of the 'slope of curve' was used in the paper. The slope of the tangent line of a point on the curve indicates the deterioration trend of the pavement performance. The derivative of a mathematical function at a certain point describes the rate of change near this point. The derivative is the slope of the tangent line of a point on the function curve. Then, the first derivative of the evolution trend curves in Figure 2 can be obtained. A larger slope indicates a larger deterioration rate. It was found that the convex and anti-S curves have smaller slopes at the beginning, significant slopes at the medium point, after which the slope rate finally reduces again. However, for the concave curves, the slopes tend to be sharp at the very beginning and then generally calm down.

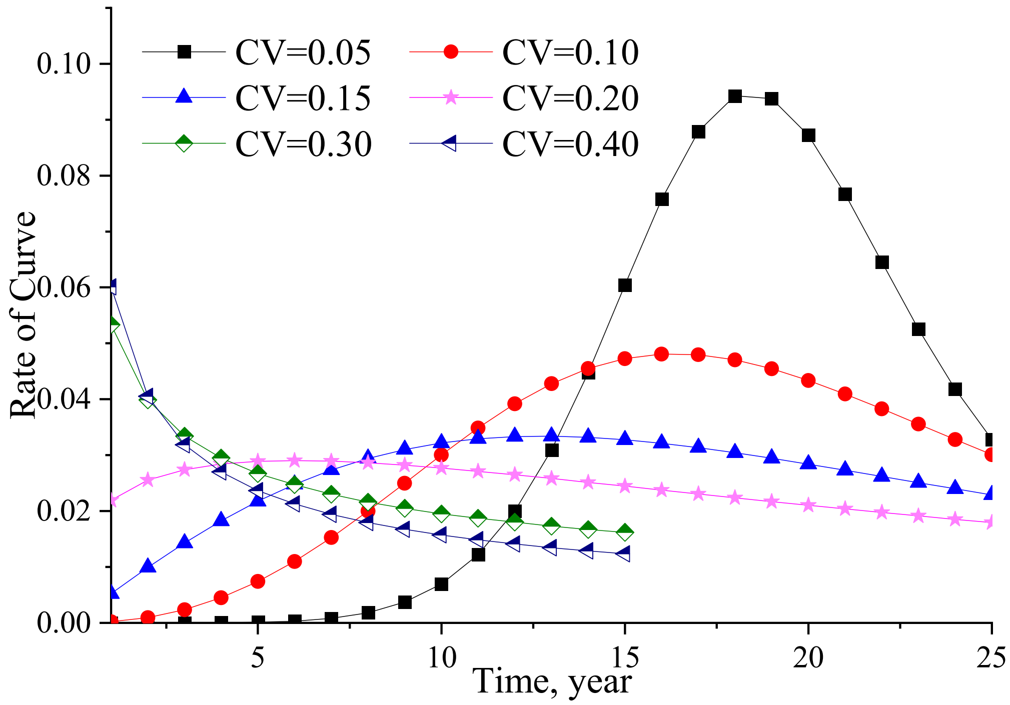

Figure 2. The rate of the curve of evolution trends for structure performance reliability with time.

Logistic models are often used in life prediction and performance deterioration; the logistic model (Equation (8)) is used to show the evolution trends of the pavement strength [27,28].

$$y = \frac{a - b}{1 + (x/x_0)^p} + b \quad (8)$$

where y = pavement strength reliability; x = pavement service time (year); x_0, a, and b are the fitting parameters. A is the initial value; b is the end value; p is the power value, which is larger than one.

Table 2 shows the fitting results of the pavement performance evolution trend curves in Figure 1. It was found that the logistic model can well simulate the evolution trend. In addition, the variance analysis shows that the experimental data have good correlation with the regression equation, at a 5% significance level.

Table 2. Fitting results of the logistic model for pavement performance evolution trends.

	a		b		x_0		P		Statistical Characteristic	
CV	**Value**	**Error**	**Value**	**Error**	**Value**	**Error**	**Value**	**Error**	**Reduced Chi-Sqr**	R^2_{adj}
0.05	0.998	0.002	−0.010	0.002	19.19	0.040	7.083	0.093	6.75×10^{-5}	1
0.10	0.992	0.003	−0.065	0.005	19.83	0.097	3.285	0.051	8.18×10^{-5}	1
0.15	0.990	0.003	−0.196	0.012	22.79	0.259	1.948	0.032	4.70×10^{-5}	1
0.20	0.992	0.003	−0.463	0.023	32.36	0.772	1.268	0.018	1.60×10^{-5}	1
0.30	1.000	0.000	−8.446	0.408	2654	238.6	0.585	0.002	9.22×10^{-8}	1
0.40	0.993	0.004	−21.8	41.1	153908	722650	0.423	0.017	1.35×10^{-5}	1

The first derivative is given to the logistic model function, as shown in Equation (9):

$$y' = \frac{-P(a-b)(x/x_0)^{P-1}}{x_0\left[(x/x_0)^P + 1\right]^2} \tag{9}$$

The fitting parameters are then substituted, as in Table 2, to the first derivative equation. The result is the corresponding curve slope. The evolution trends for the pavement performance reliability are presented in Figure 2.

In Figure 2, for curves with a coefficient of variation (CV) of less than 0.3, the slope of the curve of evolution trends increases first, and then decreases. Additionally, the slope of the curve has less variation for the curves with a larger CV. When the CV is larger than 0.30, the slopes of the curves show a decline as the time elapses. The curve with a larger CV has a greater decline.

When the CV = 0.05, the deterioration rate starts to increase after the slope of the curve reaches 0.005 (the 10th year). The deterioration rate starts to increase dramatically after the slope of the curve reaches 0.02 (the 12th year). Asphalt pavement would fail after the design period (the 15th year), and the slope of the curve in the 15th year is 0.06. For situations in which the CV = 0.10, 0.15, and 0.20, the evolution trends of the slopes of the curves are similar as the curves when the CV = 0.05, but with less variation. Particularly, for the curve with a CV = 0.20, the slope of the curve has very little variation, indicating that the deterioration is more uniform with time. The slope of the curve would exceed approximately 0.005 and 0.02 at the 5th year and the 8th year, respectively, for the evolution trend curve with a CV = 0.10. The points at the 1st year and the 5th year for the evolution trend curve have CVs = 0.15. However, the slope of the curve reaches 0.02 at the 1st year if the CV is more than 0.20.

The analysis above shows that the slope of the curve can represent the rate of change with time for the evolution trend curve, i.e., the deterioration rate. For the evolution trend curve that has obvious curve characteristics, the slope of the curve changes dramatically in a relatively short period. The dramatic change of the slope of the curve indicates the large variation of the pavement performance. Alongside the slope of the curve, it is also necessary to define the curvature. Curvature is used to measure the degree of the bending curve. Curvature is introduced in this paper because it is closely related to the variation trend of the curve. It uses the rate of change of the tangential direction angle of a point on the curve to describe the obvious evolution of the curve characteristics. Curvature is an indicator of the curve's deviation from a straight line. In math, it shows the degree of bending for a point on the curve. A larger curvature means a greater degree of the bending of the curve.

For a section of arc AA′ (length is ΔS) on the curve (CV = 0.05) in Figure 1, l_1 and l_2 denote the tangent lines at points A and A', respectively. When point A goes to point A' along the curve, tangent l_1 becomes tangent l_2 and has a point of rotation of α. Based on the curvature calculation, the slope of the curve (K) is defined as:

$$K = \lim_{\Delta S \to 0}\left|\frac{\Delta\alpha}{\Delta S}\right| = \left|\frac{d\alpha}{dS}\right|$$

However, defining the curvature by its definition is inconvenient. Instead, the curvature can be calculated by the curvature equation (Equation (10)) when curve $y = f(x)$ is known and $y = f(x)$ has a second derivative.

$$K = \frac{|y''|}{\sqrt{(1 + y'^2)^3}} \tag{10}$$

The second derivative of the logistic model is given below:

$$y'' = \frac{2P^2(a-b)(x/x_0)^{2P-2}}{{x_0}^2\left[(x/x_0)^P + 1\right]^3} - \frac{P(a-b)(x/x_0)^{P-2}(P-1)}{{x_0}^2\left[(x/x_0)^P + 1\right]^2} \tag{11}$$

Substituting Equation (9) and Equation (11) into Equation (10), the curvature at each point on the pavement performance evolution trend can be obtained. The three curves with good curve characteristics (CV = 0.05, CV = 0.10, and CV = 0.15) are then plotted in Figure 3. As a result, the curvature changes in Figure 3 can describe the degree of the bending of the evolution trends well. Much evidence indicates the evolution trends tend to be more linear. For example, the evolution trend curve (CV = 0.05) continues to increase in the y direction during the design period (15 years). In addition, the evolution trend curve (CV = 0.10) resembles a parabola that reaches its maximum at the 9th year. For the evolution trend curve (CV = 0.10), however, the curve decreases in the y direction as time elapses.

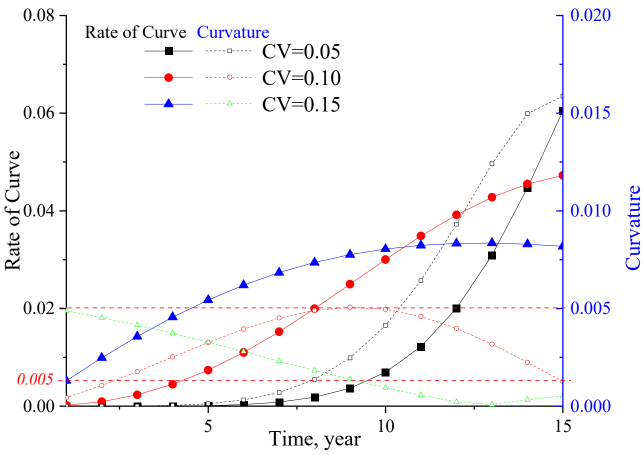

Figure 3. The curvature of evolution trends for structure performance reliability with time.

The conclusion that the change of the curvature is more obvious than the slope of the curve can be drawn from Figure 3. In other words, when the slope of the curve does not have much variation, the curve can be divided into sections based on the curvature to find different stages of the pavement performance evolution trends. For curves that have a CV = 0.05 and 0.10, as in Figure 3, the pavement performance deteriorates slightly when the curvature is from 0 to 0.005. Moreover, this section is called the stable stage. When the curvature is between 0.005 and 0.02, the deterioration increases quickly. Since the deterioration base is small, the deterioration is under control, and the pavement performance is recoverable if effective maintenance and rehabilitation measures are taken. When the curvature exceeds 0.02, the pavement performance still deteriorates at a fast rate, making it more difficult to recover the pavement performance. As a result, much greater costs would be required for maintenance and rehabilitation. As the pavement performance continues to deteriorate, it reaches the minimum value, indicating the failure of the pavement structure.

To sum up, the slope of the curve is an indicator of the deterioration rate in the pavement performance evolution trend analysis. An increase in the slope of the curve indicates more deterioration. The change of the slope of the curve can guide researchers to take measures on maintenance and rehabilitation. Such measures could enhance pavement performance and decrease its deterioration rate (the evolution trend slope of the curve). In the pavement performance evolution trend analysis, attention should be paid to the change of the curvature. Calculation of the curvature can help to analyze the sections that have much more variation on the curve as the curvature exceeds 0.005 and the bending degree of the curve becomes larger. This shows that the deterioration tends to increase. As a result, the evolution trend curvature should be no larger than 0.02 in the early stage to avoid more maintenance and rehabilitation work triggered by a quick deterioration.

This research derived a method of determining the pavement performance evolution trend by comprehensively analyzing the curvature and the slope of the curve. The method discovered that the abrupt change points and stages have much variation. The findings could provide feedback for researchers and help analyze the corresponding reasons. Then, necessary actions could be taken by highway management sectors to prevent the pavement performance from deteriorating too fast.

3.2. Risk Analysis of Asphalt Pavement Performance in Its Life Cycle

Life-cycle analysis usually includes survival analysis and failure analysis. Firstly, introduced in biomedical statistics, survival analysis analyzes the expected duration of time until some event happens, such as disease, treatment response, morbidity, and death. Recently, survival analysis has been widely used in industry, social science, and business [19,27,28]. It is necessary to define the lifetime. In the case of the pavement structure, the lifetime is regarded as the time starting from a critical point and ending by another critical point. The start could be the opening day of the highway, the first maintenance and rehabilitation, or the result of certain highway distress. As for the end, it could be the minimum pavement performance or the failure of the pavement.

Survival analysis is under the impact of random factors, so that the lifetime follows a distribution. Lifetime is described by the survival function, probability density function, and hazard function. The three functions are equivalent, and two functions could be derived if one function is given.

Suppose a random variable T represents a duration of time (survival variable) and it is non-negative and continuous. Define the continuous density function as $f(t)$. Then, its cumulative distribution function $F(t)$ is [27]:

$$F(t) = \int_0^t f(s)ds = prob(T \leq t) \tag{12}$$

Usually, the probability that the research object survives more than the time t is considered, and the survival function is defined as follows:

$$S(t) = \int_t^\infty f(s)ds = 1 - F(t) = prob(T \geq t) \tag{13}$$

When it comes to the product lifetime, $S(t)$ is also called the reliability function. $S(t)$is the nonincreasing function of t, and

$$\begin{cases} S(t) = 1\ (t = 0) \\ S(t) = 0\ (t = \infty) \end{cases} \tag{14}$$

Reliability R_t has already been obtained in the asphalt pavement performance evolution trend and R_t = $S(t)$ is an individual that has survived to time t.

The hazard function describes its instant failure rate at moment t.

$$h(t) = \lim_{\Delta t \to 0} \frac{\text{Prob}(t \leq T < t + \Delta t | T \geq t)}{\Delta t} = \lim_{\Delta t \to 0} \frac{F(t + \Delta t) - F(t)}{\Delta t S(t)} = \frac{f(t)}{S(t)} \tag{15}$$

The hazard function could be defined by the distribution function and the probability density function:

$$h(t) = \frac{f(t)}{1 - F(t)} \tag{16}$$

For the distribution of the random variable $T, f(t) = -S'(t)$. Then, $h(x) = -\frac{d}{dx}\log S(x)$,

$$\log S(x)|_0^t = -\int_0^t h(x)dx \tag{17}$$

$$S(t) = \exp(-\int_0^t h(x)dx) \tag{18}$$

The cumulative function is also widely used, and is defined as:

$$H(t) = \int_0^{\mathrm{t}} h(x)dx \tag{19}$$

The cumulative hazard function has the following relationship with the survival function:

$$S(t) = \exp(-H(t)) \tag{20}$$

When $t = 0$, $S(t) = 1$, $H(t) = 0$; when $t = \infty$, $S(t) = 0$, $H(t) = \infty$, and the cumulative hazard probability range is $[0, \infty)$.

For the continuous lifetime distribution, the hazard function follows: $h(t) \geq 0$; $\int_0^\infty h(t)dt = \infty$.

After the fitting the evolution trends of the pavement performance by the logistic model, the survival function could be expressed as:

$$S(t) = \frac{a - b}{1 + (x/x_0)^p} + b \tag{21}$$

$$f(t) = F(t)' = [1 - S(t)]' = \frac{P(a-b)(x/x_0)^{P-1}}{x_0\left[(x/x_0)^P + 1\right]^2} \tag{22}$$

Then, the hazard function of the asphalt pavement performance evolution $h(t)$ is shown below:

$$h(t) = \frac{f(t)}{1 - F(t)} = \frac{\frac{P(a-b)(x/x_0)^{P-1}}{x_0\left[(x/x_0)^P + 1\right]^2}}{\frac{a-b}{1+(x/x_0)^p} + b} \tag{23}$$

After obtaining the hazard function $h(t)$, the instant hazard rate of the asphalt pavement performance evolution can be calculated. The instant hazard rates during time t add up and compose the cumulative hazard rate $H(t)$. Then, the pavement performance and lifetime can be evaluated by a new method, whereby the change of the hazard rate and the cumulative hazard rate are identified.

3.3. Attenuate Subsection Model of Asphalt Pavement Performance

Although the evolution trend curve and curvature can show the instant change of the pavement performance, the result is not straightforward. To make the result more readable, the evolution trend curve was divided into four stages in Figure 4.

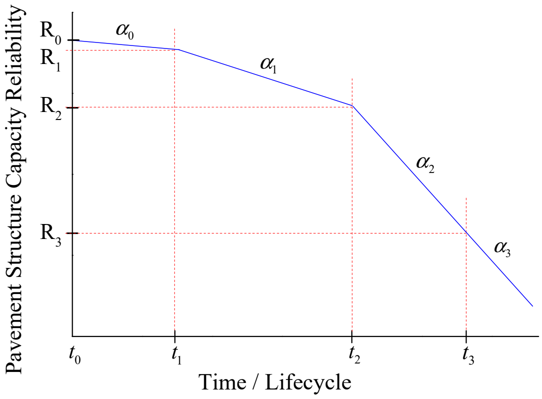

Figure 4. The schematic diagram of pavement reliability piecewise linear degradation model.

In the first stage, $[0, t_1)$, the pavement capacity reliability is relatively stable, indicating that pavement has a good performance. In the second stage, $[t_1, t_2]$, the reliability decreases, but the deterioration would be under control if preventive or corrective maintenance is adopted. When it comes to the third stage, $[t_2, t_3]$, the reliability deteriorates with a faster rate. Under such circumstances, the pavement performance is in a bad condition, so common maintenance would not work. In the last stage, the reliability continues to go down until the pavement fails.

The four-stage degradation model is a typical model that could be applied to any type of pavement performance evolution trend curve. Mathematically, the reliability of the four stages can be expressed in function $R(t)$:

When $t \in [0, t_1]$, $R(t) = R_0 - \alpha_0(t - t_0)$;

When $t \in [t_1, t_2]$, $R(t) = R_0 - \alpha_0(t - t_0) - \alpha_1(t - t_1)$;

When $t \in [t_2, t_3]$, $R(t) = R_0 - \alpha_0(t - t_0) - \alpha_1(t - t_1) - \alpha_2(t - t_2)$;

When $t \in [t_3, \infty]$, $R(t) = R_0 - \alpha_0(t - t_0) - \alpha_1(t - t_1) - \alpha_2(t - t_2) - \alpha_3(t - t_3)$.

Where R_0——initial reliability;

t_0, t_2, t_3——critical evolution time of pavement performance;

$\alpha_1, \alpha_2, \alpha_3$——reliability deterioration rate (also known as the slope of line in each stage).

Combine the subsection functions into one function:

$$F(t_i) = Sign(1 + Sign(t - t_i)) \tag{24}$$

$Sign(x)$ is a sign function and is defined as:

When $x > 0$, $Sign(x) = 1$; when $x < 0$, $Sign(x) = -1$; when $x = 0$, $Sign(x) = 0$.

So, if $t > t_i$, $F(t_i) = 1$; if $t < t_i$, $F(t_i) = 0$.

$$R(t) = R_0 + \begin{bmatrix} (0 - \alpha_0)(t - t_o) \\ (\alpha_0 - \alpha_1)(t - t_1) \\ (\alpha_1 - \alpha_2)(t - t_2) \\ (\alpha_2 - \alpha_3)(t - t_3) \end{bmatrix}^T \begin{bmatrix} F(t_0) \\ F(t_1) \\ F(t_2) \\ F(t_3) \end{bmatrix} \tag{25}$$

$F(t_0)$, $F(t_1)$, $F(t_2)$, and $F(t_3)$ are the validity functions that are used.

By extending the four-stage linear degradation model, an n-stage linear degradation model can be built. Any pavement performance deterioration curve can be regarded as n-segment broken lines. The number n is usually decided by the project accuracy requirement. The n-stage linear degradation model expresses the deterioration process by discrete subsection functions, whose trends are easier to analyze for researchers.

3.4. Threshold Value of Asphalt Pavement Performance

Before applying the life-cycle design evaluation, it is necessary to acquire the pavement performance reliability evaluation index. The evaluation index is the threshold value of the asphalt pavement performance. In its life cycle, if the reliability level of the asphalt pavement performance is lower than the threshold value, detection and evaluation should be taken by the highway agency. The threshold value is regarded as a key index for the research on the pavement performance evolution. A variety of factors, including the highway grade, service level, traffic composition, life-cycle cost, climate, geographic and geological conditions, environmental protection, and low carbon, are taken into consideration when determining the reliability threshold value. The reliability threshold also provides a reference for the detection period optimization.

Some scholars have provided the following asphalt pavement reliability index [23,25,29]: (1) the latent reliability of the pavement built according to design codes; (2) the reliability of a typical built pavement structure; (3) the relationship between the pavement performance and the reliability; (4) the relationship between the pavement maintenance cost and the reliability.

The latent reliability level for China's current asphalt pavement specifications is listed in Table 3 Other research results on pavement reliability are presented in Tables 4 and 5.

Table 3. Minimum reliability levels recommended for pavement structures.

Classification of Highway	Recommendation of the Lowest Reliability Level (%)	
	Urban	Suburb
Interstate/Freeway	95	95
Principal/Arterials	90	85
Collectors	80	75
Local road	75	70

Table 4. The Criterion of Reliability in Asphalt Pavement Design.

Highway Technical Classifications	Express Way	Class I	Class II	Class III, IV
Safety Level	Class I	Class II	Class III	Class IV
Design Life (year)	15	15	12	8
Reliability (%)	95	90	85	80
Reliability Index	1.64	1.28	1.04	0.84
Variation Level	Low	Low–Medium	Medium	Medium–High

Table 5. Latent reliability level for the specifications for current design of asphalt pavement.

Traffic Classification	Express Way	Class I Highway	Class II Highway
Heavy	90–94	85–90	/
Medium	84–90	75–85	70~80
Low	75–82	65–75	60~75

The AASHTO 2020 design guide proposed minimum reliability levels for different grades of roads, which are summarized in Table 6 [26].

According to the Unified Standard for Reliability Design of Highway Engineering Structures, if the ultimate state design of the carrying capacity is applied, the asphalt pavement reliability design should follow Table 4 [30].

Pavement strength is an index that composes the structure performance, functional performance, surface performance, road travel quality, comfortability performance, and safety performance. The functional performance would show more deterioration and a faster deterioration rate than the structure performance once the pavement strength deteriorates. Asphalt pavement functional performance deterioration can cause a decrease in the pavement strength only when the deterioration accumulates to a certain level. As a result, pavement strength is the base for other pavement performance indices. In other words, pavement strength can represent not only the pavement functional performance, but also indicate the pavement performance. This paper used pavement strength reliability as the evaluation index in asphalt pavement performance research.

Table 6. Proposed reliability levels for asphalt pavement strength performance (%).

Evaluation Index		Traffic Classification		
		Express Way	Class I Highway	Class II Highway
Travel Performance	High	95~100	95~100	90~100
	Good	90~95	90~95	80~90
	Medium	85~90	80~90	75~80
	Low	<85	<80	<75
Structure Performance	High	95~100	95~100	90~100
	Good	90~95	90~95	80~90
	Medium	80~90	75~90	70~80
	Low	75~80	65~75	60~70
	Failure	<75	<65	<60

Current research and evaluation on asphalt pavement performance has provided reference for this paper. This research, however, was based on both these reference works and the asphalt pavement's actual evolution trend. This paper carried out the research and evaluation on pavement strength by integrating pavement travel performance with the structure performance. The proposed reliability levels for the pavement strength are summarized in Table 6.

The travel performance in Table 6 represents the travel quality and level of service that users experience, as well as the visual impression brought by road safety performance and pavement appearance. As shown in the table, the travel performance is divided into four grades: high, good, medium, and low. If the express way, class I highway, and class II highway have travel performances lower than 85%, 80%, and 75%, respectively, the pavements are considered to fail to meet the requirement for normal operation. However, the pavement strength does not fail. Maintenance and rehabilitation would help recover travel performance. If there is no maintenance and rehabilitation at this point, the pavement travel performance would finally fail.

The structure performance indicates the pavement structure's capacity for carrying traffic and load. It is a general index that represents the pavement strength and carrying capacity. The index classified the structure performance into five types, as listed in Table 6. Since the evaluation of the travel performance is based on the pavement performance, the travel performance can be regarded as a subset of the structure performance, and has the same evaluation result as the pavement performance in category "High" and "Good". The failure of the structure performance means that the pavement has completely failed and is unrecoverable; therefore, the pavement needs to be reconstructed.

4. Case Study

The case study used the strength performance function (with five parameters) of the Qinghai–Tibet cold regions' asphalt pavement to analyze the risk. Results showed the influence of F-T cycles on the pavement performance. Evaluations of the evolution trend, risk in the lifetime, the subsection model on the strength deterioration, the corresponding maintenance, and the rehabilitation decision making were included in the analysis as well.

Figure 5 presents the strength reliability evolution trend, the change of the slope of curve, the curvature variation, the hazard function, the cumulative hazard rate, and the division of the piecewise linear degradation after the asphalt pavement is subjected to 14 F-T cycles.

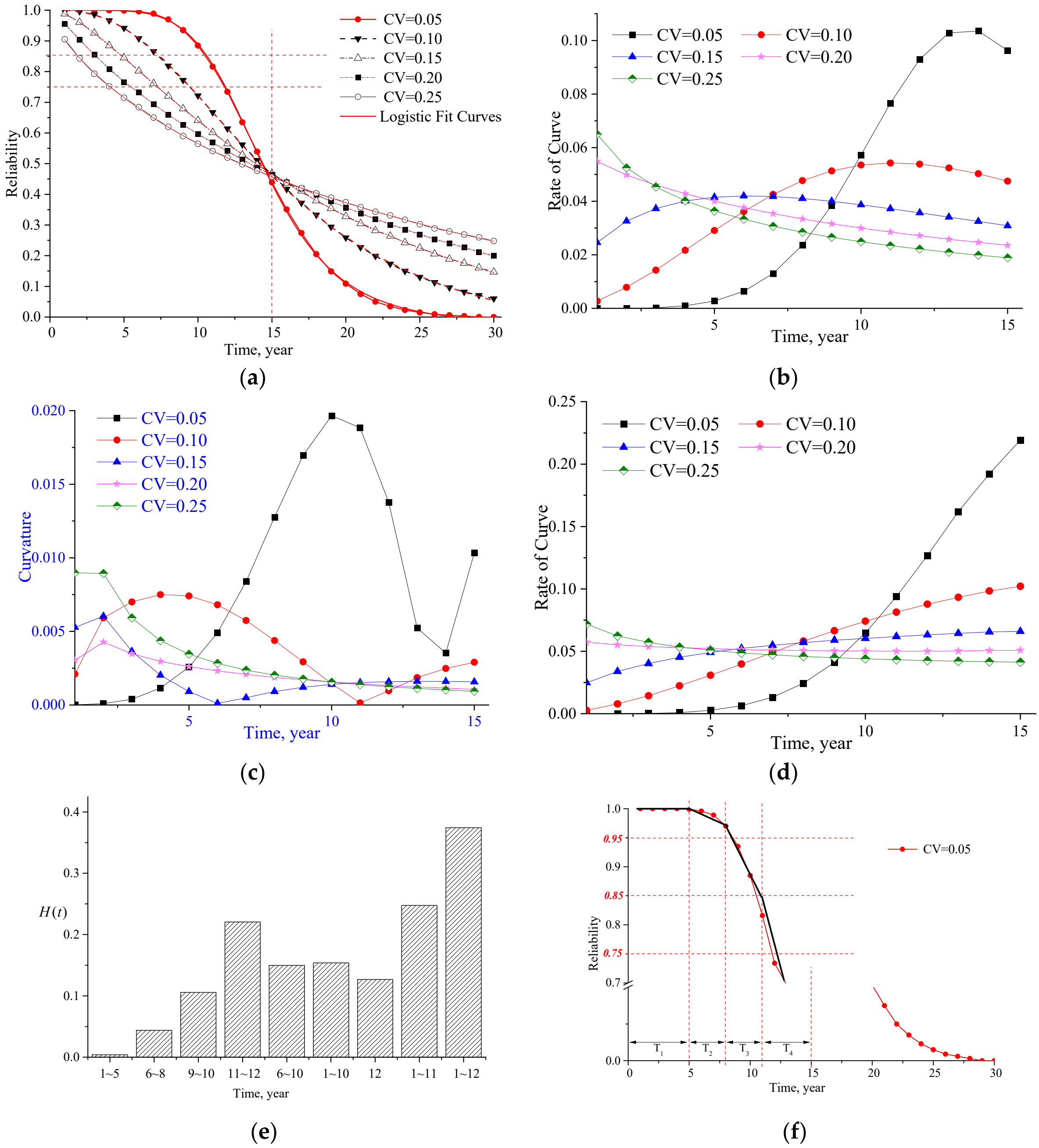

Figure 5. The evolution trends of structure capacity reliability (after 14 F-T cycles) for (**a**) the structure capacity reliability, (**b**) the rate of the curve, (**c**) curvature variation, (**d**) the hazard function $h(t)$, (**e**) the cumulative hazard rate $H(t)$ (CV = 0.05), (**f**) and the division of piecewise linear degradation (CV = 0.05).

After 14 F-T cycles, the evolution trend curve (CV = 0.05) in Figure 5a deteriorates only after the 7th year, while the slope of the curve in Figure 5b has an obvious deterioration after the 5th year and reaches its maximum at the 14th year. This shows that the pavement reliability is in continuous deterioration during its service. The curvature variation follows a convex curve. The curvature reaches the peak point in the 10th year, indicating the maximum deterioration rate around that time. For example, during the 8th year to the 11th year, the pavement strength decreased significantly by 15.4%. Compared to the threshold value of pavement strength reliability, travel performance lower than 85% is considered to be a bad condition. Therefore, necessary maintenance and rehabilitation should be applied before the occurrence of the peak point, ensuring a good pavement condition. To determine when and how to maintain and rehabilitate the pavement structure, life-cycle cost analysis is applied to optimize the work. The evolution trend curves that have different CVs are similar to this one.

The hazard function after 14 F-T cycles is shown in Figure 5d. It can be seen that, when the CV is less than 0.20, the $h(t)$ continues to grow, and the growth rate varies with different CVs. Basically, the pavement with a larger CV has a larger $h(t)$ in the early stage. As time elapses, the pavement with a smaller CV will have a larger $h(t)$. For instance, the hazard rate for the CV = 0.10 exceeds that for the CV = 0.15 at the 8th year, and it exceeds that for the CV = 0.10 at the 11th year. As for the curves having a large CV (CV = 0.20, 0.25), the $h(t)$ starts from a relatively large value and decreases slightly as time elapses. Later, after design period, the $h(t)$ shows a gentle increase. Within the design period, the $h(t)$ changes monotonically. The hazard function $h(t)$ can evaluate the asphalt pavement strength deterioration trend reasonably well.

Taking the pavement (CV = 0.05) that is subjected to 14 F-T cycles as an example, the cumulative hazard rate $H(t)$ in Figure 5e is small during the 1–5 years. Then, the $H(t)$ grows in years 6–8, 9–10, and 11–12. Particularly, the hazard rate at the 12th year is larger than that during 9~10 year, and it is 17.6% less than the $H(t)$during the 1~10 year. It can be concluded that the pavement strength usually has a larger $H(t)$ before and after the structure's failure. Hence, actions (i.e., maintenance and/or rehabilitation) should be taken before the failure.

The division of the piecewise linear degradation after 14 F-T cycles (CV = 0.05) has four stages, and is shown in Figure 5f. In the T_1([0, 5]) stage, the asphalt pavement strength reliability stays stable. The pavement is in good condition and no maintenance or rehabilitation is needed. In the T_2 ([6, 8]) stage, although the reliability deteriorates slightly, the travel performance is still good. Maintenance and rehabilitation are optional in this stage. In the T_3 ([9, 10]) stage, the travel performance changes from good to low, and the pavement strength deteriorates significantly. Maintenance and rehabilitation would be able to recover the pavement strength during this stage. However, when it comes to the last stage, T_4 ([11, 12]), the pavement performance changes from medium to failure. In this stage, reconstruction is required to recover the pavement strength. To conclude, the results from cumulative hazard rate analysis could be used to divide the linear deterioration subsection function. The four-stage linear deterioration model would be able to represent the deterioration trend. As a result, researchers could find the optimized pavement maintenance and rehabilitation by applying life-cycle cost analysis to the different stages.

After the calculation, the reliability deterioration rate of each stage was obtained for this paper: α_0 = 0.000804, α_1 = 0.014353, α_2 = 0.047835, and α_3 = 0.0848 R_0 = 100. Then, by substituting the deterioration rate to the four-stage linear deterioration model, the pavement strength reliability four-stage linear deterioration model was obtained. The pavement strength reliability linear deterioration equation for curves with a different CV could also be obtained by this method. An n-stage linear deterioration equation could be obtained as well.

5. Conclusions

In this paper, a reliability method was applied to investigate the evolution trend of asphalt pavement performance under the effect of F-T cycles with uncertainties. Some indices and methods, such as the rate of the curve, curvature, survival analysis, and attenuation subsection model, were proposed to analyze the change of the pavement performance during its life cycle. Some main conclusions are summarized below:

Since there are few pavement capacity indicators in the present assessment system, pavement reliability well reflects the pavement capacity variation trends during its life cycle. Pavement reliability can be used as one of the indicators to reflect the whole pavement capacity variation.

The rate of the curve and the curvature were proposed to analyze the changes of the evolution trend of the asphalt pavement performance. The rate of the curve indicates the deterioration trend, and larger slope indicates faster a deterioration rate. Curvature is an indicator of the curve's deviation from a straight line. A larger curvature means a greater degree of the bending of the curve. The rate of the curve reflects the degree of the deterioration trend, while the curvature can define the sensitive region of the deterioration process. The combination of these two indicators can identify the abrupt change points and significant variation stages.

Risk analysis was proposed to analyze the asphalt pavement performance during its life cycle. It provides a new method to evaluate the pavement performance evolution by identifying the change of the hazard rate and the cumulative hazard rate. The corresponding calculation equations of the hazard function $h(t)$ and the survival function $S(t)$ are given based on the logistic model, which is a novel way to describe the attenuation trend during its life cycle.

The evolution curve of the asphalt pavement strength reliability can be divided linearly during its life cycle based on the linear piecewise deterioration model, which can be extended to any n-stage linear deterioration model according to the actual situation. The n-stage linear degradation model expresses the deterioration process by discrete subsection functions, which are useful in engineering practice applications.

The reliability levels for the pavement strength were proposed in this research by integrating the pavement travel performance and the structure performance. The travel performance represents the travel quality and level of service that users experience, as well as the visual impression brought by road safety performance and pavement appearance. The travel performance is divided into four grades: high, good, medium, and low. The structure performance indicates the pavement structure's capacity for carrying traffic and load; it is a general index that represents the pavement strength and carrying capacity. The structure performance is divided into five grades: high, good, medium, low, and failure.

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