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ORIGINAL CONTRIBUTION

STUDY THE PAVEMENT PERFORMANCE FORECASTING MODELS AND THEIR BENEFITS IN ROAD INFRASTRUCTURE MANAGEMENT SYSTEMS

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ABSTRACT

Performance modelling and condition assessment are the systematic approach, that requires for preservation of road infrastructure asset. Use of pavement performance model is very important part of such approach. Pavement performance model can forecast or predict of future condition based on present condition under a defined range of future loading and maintenance scenarios. The perfect implementation of pavement management systems strongly depends upon the successful analysis of future pavement condition, as forecasted by the performance forecasting models of the system, balanced with observed behaviour and engineering knowledge of the road network.

KEYWORDS— Pavement Performance. Pavement Performance Forecasting. Failure of Pavements.

1. INTRODUCTION

Pavement materials will be affected under the influence of wheel loads and environmental issues. The stresses caused by wheel loads generate micro cracking in pavement and also generate deformation in the pavement layers that may be permanent. After traffic movement skid resistance will be reduced due to aggregate polishing. Spring thaw may cause considerable reduction of the permissible stresses in the unbound materials, while frost wave may cause cracking and deformation. Micro cracking can develop by allowing water to penetrate into pavement layers with time. Capturing this pavement deterioration process in a comprehensive manner by considering all factors is known as Pavement Performance Forecasting Model (PPFM).

Already a large number of pavement performance models are available. But the main drawback of those is they are producing different predictions with same input data. Road asset performance models should be based on fundamentally standard engineering principles for reliability and acceptability. In accordance to available historical data and the engineer’s knowledge of local materials, environmental effects, and construction practices, it is also important that these models are easily adjustable.

In spite of an enormous effort that has been made in the pavement engineering field, it still is not possible to make accurate and precise prediction of pavement life (Molenaar 2003). This is because it is very difficult to predict many of the factors that influence the pavement performance. Unusual hot summers, cold winters, wet springs etc. can not be predicted. Traffic forecasts are mostly unreliable and there is a large variation in the characteristics of pavement materials and structures. The available performance prediction models have several limitations in that most of them involve large simplifications (e.g. in material behaviour), some of them contain input factors that are difficult to quantify and most are not comprehensive enough (do not consider all influencing factors). Figure 1 illustrates the complexity of the performance prediction problem.

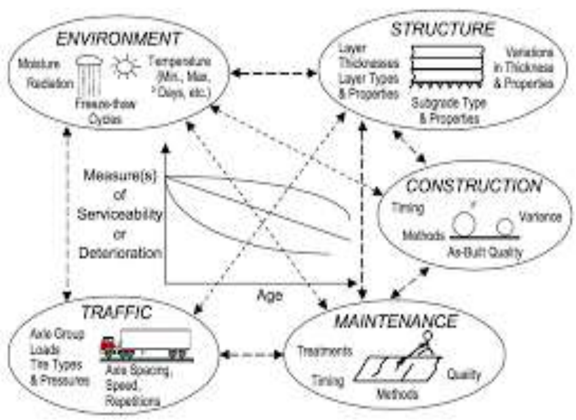

Figure 1: Factors affecting pavement performance (Haas 2003)

2. MEASURES OF PAVEMENT PERFORMANCE

Pavement performance has been expressed in terms of individual pavement distress (such as rutting, cracking etc), pavement condition index, which is often a composite measure involving both the functional and structural condition, and pavement serviceability index, which includes user's evaluation of the condition of the pavement. At the project level it might be appropriate to evaluate the distresses individually, but at the network level definition of some kind of composite measure of performance (performance indicator) is necessary. Currently, a project (COST Action 354) is under way at the European level to define performance measures and performance indicators (Litzka 2006).

3. PAVEMENT PERFORMANCE PREDICTION MODELS

One of the most intense challenges facing pavement managers and engineers has been the development of performance or deterioration prediction models. Several performance prediction models have been proposed over the years, some of which are simple while others more complex. Ralph Haas (2003) grouped the many performance prediction models into classes which indicate their basis as follows:

- Empirical, where certain measured or estimated variables such as deflection, accumulated traffic loads etc. are related to loss of serviceability or some other measure(s) of deterioration and pavement age, usually through regression analysis.
- Mechanistic – empirical, where certain calculated responses, such as subgrade strain, pavement layer stresses and strains etc., together with other variables such as accumulated traffic loads, are related to loss of serviceability or some other measure(s) of deterioration through regression analysis or a model which is calibrated (i.e. the coefficients are determined) by regression analysis.
- Subjective, experience based where serviceability loss or other measure(s) of deterioration vs. age are estimated, for different combination of variables, using Markovian transition process models, Bayesian models etc.

3.1 Empirical Models

Various equations, mostly based on regression analysis, were developed for predicting pavement performance. The usefulness of these empirical equations is limited by the scope of the database that was used in their development.

These kinds of regression equations are valid only under certain conditions and should not be applied when the actual conditions are different. One of the best known examples of the empirical models is the HDM – 4 developed by the World Bank.

The World Bank developed the Highway Design and Maintenance Standards Model (HDM-III) over two decades ago for use in infrastructure investment planning in developing countries. However, in recent years some industrialised countries showed interest in the model and this led to further development of the model. In order to extend the scope of HDM-III and include additional capabilities such as models for traffic congestion, cold climate effects, road safety and environmental effects, the International Study of Highway Development and Management (ISOHDM) was conducted. This project produced the Highway development and Management Tool, HDM-4 (Kerali 2000). The HDM-4 has applications at the strategic, program and project levels and it includes deterioration models for various types of distresses. As an example, the roughness model is described as in Equation 1:

$$\Delta RI = K_{gp} [\Delta RI_s + \Delta RI_c + \Delta RI_r + \Delta RI_t] + \Delta RI_e \quad \text{....(1)}$$

Where:

ΔRI = total incremental change in roughness during the analysis year

K_{gp} = calibration factor for roughness progression

ΔRI_s = incremental change in roughness due to structural deterioration, which is a function of pavement age, number of equivalent standard axles and structural number of the pavement

ΔRI_c = incremental change in roughness due to cracking, which is proportional to the incremental change in area of total cracking during the analysis year (% of total carriageway area)

ΔRI_r = incremental change in roughness due to rutting, which is proportional to the incremental change in standard deviation of rut depth during the analysis year. The rut depth is the sum of four components: initial densification, structural deformation, plastic deformation, and wear from studded tyres.

ΔRI_t = incremental change in roughness due to potholing. The potholing effect depends on the number of vehicles that actually hit the potholes, which in turn depends on the traffic volume and the freedom to manoeuvre.

ΔRI_e = incremental change in roughness due to the environment. This component of roughness is due to factors which include temperature and moisture fluctuations and also foundation movements (e.g. subsidence)

The strength of bituminous pavements is characterized by the adjusted structural number, SNP. To take the effect of seasonal variations into account the average annual SNP is derived from SNP in dry conditions and SNP in wet conditions and the length of the dry and wet seasons. The effect of drainage on SNP is modelled through change in drainage factor, which varies from 1 (excellent) to 5 (very poor). The effect of construction quality is taken into account through a factor termed construction defects indicator.

Crack initiation is modelled using Equation 2:

$$ICA = K_{cia} \{CDS^2 a_0 \exp [a_1 SNP + a_2 (YE4 / SN^2)] + CRT\} \quad \text{..........(2)}$$

Where:

ICA = time to crack initiation in years

CDS = construction defects indicator for bituminous surfacing

SNP = structural number of the pavement

YE4 = annual number of ESALs in millions/lane

K_{cia} = calibration factor for crack initiation

CRT = crack retardation time due to maintenance

a_0 , a_1 , a_2 are calibration parameters

The model for plastic deformation is expressed in Equation 3:

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$$\Delta RDPD = K_{rpd}\, CDS^{3}\, a_0 YE4Sh^{a1}\, HS^{a2} \qquad \text{.........(3)}$$

Where:

ΔRDPD = incremental increase in plastic deformation in analysis year, in mm

CDS = construction defects indicator for bituminous surfacing

Sh = speed of heavy vehicles in km/h

HS = total thickness of bituminous surfacing in mm

K_{rpd} = calibration factor

Another example of empirical performance prediction model is the serviceability equation developed from the AASHO road test and used for many years in the earlier AASHTO design guides. The present serviceability index (PSI) for flexible pavements was expressed as follows in Equation 4 (Huang 1993):

$$PSI = A_0 + A_1 \log(1 + SV) + A_2 (RD)^2 + B_1 \sqrt{C + P} \qquad \text{…..(4)}$$

Where:

SV = mean slope variance

RD = mean rut depth

C = cracking (linear feet per 1000 ft^2)

P = patching (ft^2/1000 ft^2)

A_0, A_1, A_2, and B_1 are coefficients to be determined by linear multiple regression. Equation 4 is not a performance model in itself but it is an expression of the relationship between PSI and distresses. The PSI was used in the performance equation which predicts the allowable number of axle loads to failure i.e., reduction of PSI to terminal serviceability.

Prozzi and Madanat (2004) proposed a more sophisticated statistical performance prediction model based on AASHO road test data and field data from the Minnesota Road Research Project (MnRoad). The proposed model predicts roughness based on layer thicknesses, traffic increment, and frost gradient.

The European project, COST 324 Long Term Performance of Road Pavements, reviewed performance prediction models that were in use in 11 participating countries. These countries were Austria, Belgium, Switzerland, Denmark, Spain, Finland, France, United Kingdom, Greece, Hungary, Ireland, Netherlands, Portugal, Sweden and Slovenia. Most of the countries have developed performance models for the various performance indicators such as longitudinal profile, transverse profile, surface cracking, structural cracking, structural adequacy (deflection), surface defects and skid resistance. Some of the countries use a composite index that combines the various indicators. Detailed list of all the models can be found in the final report of COST 324 (European commission, 1997). The majority of these models are empirical and are mostly based on one independent variable such as the number of repetitions of load or age. The conclusion of the project was that the existing performance prediction models are not suitable for Europe- wide application and development of new performance models was recommended.

3.2 Mechanistic – Empirical Models

In the mechanistic - empirical models, calculated response variables such as tensile strain at the bottom of asphalt layer and vertical strain at the top of subgrade are used in addition to other parameters such as traffic loading to predict performance of the pavement structure. The performance is often expressed in terms of the individual distresses such as fatigue cracking, rut depth etc. The responses, i.e., the strains and the stresses resulting from axle loading are calculated using linear elastic multilayer theory, or, in some cases, finite element method. The material properties, such as the elastic moduli for the various layer materials, are taken into account in the response calculation. The environmental effects, such as the effects of temperature and moisture, can also be taken into account through their effect on the material properties.

Performance prediction models incorporated into the 2002 mechanistic – empirical design guide, developed in USA under the National Cooperative Highway Research Program (NCHRP) 1- 37A, are typical examples of this type of models. The pavement performance measures considered in the guide include permanent deformation (rutting), fatigue cracking (both bottom-up and top-down), thermal cracking and smoothness (International roughness index, IRI). Pavement response is calculated using either the elastic multilayer theory or the finite element method.

The design procedure of the 2002 mechanistic – empirical design guide involves analysis of trial designs to ensure that the designs satisfy user defined performance requirements. The trial design is analyzed for adequacy by dividing the target design life into shorter design analysis periods or increments beginning with traffic opening month. Within each increment all the factors that affect pavement performance/ damage, including traffic levels, asphalt concrete modulus, base and sub base module, and sub grade modulus are held constant. The critical stress and/or strain values are converted to incremental distresses. Rutting is predicted in absolute terms, i.e., the incremental rut depth calculated for each analysis period is accumulated to obtain the total rut depth. Cracking distress is predicted in terms of a damage index, which is mechanistic parameter representing load associated damage within the pavement structure. The incremental damage is accumulated for each analysis period using Miner's law. The cumulative damage is converted to physical cracking using calibrated models that relate the cumulative damage to observable distresses. Calibrated distress prediction models were developed using the LTPP database and other long term pavement performance data. The model equations implemented in the design guide are as follows.

The overall permanent deformation for a given season is the sum of permanent deformation for each individual layer and is mathematically expressed as in Equation 5:

$$RD = \sum_{i=1}^{nsublayers} \varepsilon^{i}_{p} h_i \qquad \text{...... (5)}$$

Where:

RD = pavement permanent deformation

nsublayers = number of sub layers

ε^{i}_{p} = total plastic strain in sub layer i

h_i = thickness of sub layer i

The relationship used to predict rutting in asphalt mixtures is based upon a field calibrated statistical analysis of laboratory repeated load permanent deformation tests. The selected laboratory model is of the form as expressed in Equation 6:

$$\frac{\varepsilon_p}{\varepsilon_r} = a_1 T^{a_2} N^{a_3} \qquad \text{.....(6)}$$

Where:

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

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

T = temperature (Degree F)

N = number of load repetitions

a_1, a_2 , a_3 = non-linear regression coefficients

The final laboratory expression that was initially selected for calibration had coefficients

$a_1 = 10^{-3.15552}$, $a_2 = 1.734$, and $a_3 = 0.39937$. Field calibration factors β_{ri} were added to ascertain field calibration and the final asphalt rutting model has the following form as expressed in Equation 7:

$$\frac{\varepsilon_p}{\varepsilon_r} = \beta_{r1} a_1 T^{\beta_{r2} a2} N^{\beta_{r3} a3} \qquad \text{...(7)}$$

The model for permanent deformation in unbound granular base is expressed as follows in Equation 8:

$$\delta_a(N) = \beta_{GB}\left(\frac{\varepsilon_0}{\varepsilon_r}\right)\exp-\left(\frac{\rho}{N}\right)^{\beta}\quad \varepsilon_v h \qquad ...(8)$$

Where:

δ_a (N) = permanent deformation for a layer/sublayer (in)

N = number of traffic repetitions

ε_0, β, ρ = material parameters

ε_r = resilient strain imposed in a laboratory test to obtain the material properties listed above (in/in)

ε_v = average vertical resilient strain in the layer/sublayer as obtained from primary response model (in/in)

h = thickness of the layer/sublayer (in)

β_{GB} = calibration factor

The model form given in equation 8 is also used for calculation of permanent deformation in all subgrade soils. The parameters ($\varepsilon_0/\varepsilon_r$), ρ, and β are calculated using empirical equations. It has to be noted that according to this procedure permanent deformation for the various layers are calculated separately. This means that calibration of the models requires trenching studies to obtain field data on deformation in the various layers. Available information indicates that the deformation model, particularly that for unbound layers, is being revised.

With regard to fatigue damage, the approach utilized in the design guide models both the bottom-up and top-down cracking. The approach is based on calculating the fatigue damage at the surface for the top-down cracking and at the bottom of each asphalt layer for bottom-up cracking. Estimation of fatigue damage is done according to Miner's law, which can be expressed as follows as in Equation 9:

$$D = \sum_{i=1}^{T} \frac{n_i}{N_i} \qquad ….(9)$$

Where:

D = damage
T = total number of periods
n_i = actual traffic for period i
N_i = traffic allowed under conditions prevailing in i

The relationship used in the design guide for the prediction of the number of repetitions to fatigue cracking is expressed as follows in Equation 10:

$$N_f = C\beta_{f1}k_1(1/\varepsilon_t)^{\beta_{f2}k_2}(1/E)^{\beta_{f3}k_3} \qquad ....(10)$$

Where:

N_f = number of repetitions to fatigue cracking

ε_t = tensile strain at the critical location

E = stiffness of the material

k_1, k_2, k_3 = laboratory regression coefficients

β_{f1}, β_{f2}, β_{f3} = calibration parameters

C = laboratory to field adjustment factors

Transfer functions are used to calculate fatigue cracking as a percent total lane area from the fatigue damage.
The thermal cracking model implemented in the design guide is based on thermal cracking model developed under the Strategic Highway Research Program (SHRP). The model is expressed as follows in Equation 11:

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$$C_f = \beta_1 N\left(\frac{\log C/h_{ac}}{\sigma} \right) \quad \text{….(11)}$$

Where:

C_f = observed amount of thermal cracking (crack frequency)
β_1 = regression coefficient determined through field calibration
N(z) = standard normal distribution evaluated at z
σ = standard deviation of the log of the depth of cracks in the pavement
C = crack depth
h_{ac} = thickness of asphalt layer

The amount of crack propagation induced by a given thermal cooling cycle is predicted using the Paris law of crack propagation expressed as follows in Equation 12:

$$\Delta C = A\Delta K^n \quad \text{….(12)}$$

Where:

ΔC = change in the crack depth due to a cooling cycle

ΔK = change in stress intensity factor due to a cooling cycle

A, n = fracture parameters for the asphalt mixture

The parameters A and n are calculated from creep compliance curve using the principles of visco-elasticity.

The roughness (or smoothness) of flexible pavements is dependent on other distress types such as rutting, variance of rut depth, fatigue cracking, etc. The international roughness index (IRI) is used as a measure of smoothness of flexible pavements in the design guide. The models utilized in the design guide for prediction of IRI are dependent on the base type. For unbound aggregate bases and subbases, the model expressed in Equation 13 below is used.

$$IRI = IRI_0 + 0.0463\,[SF\,(e^{age/20} - 1)] + 0.00119\,(TC_L)_T + 0.1834\,(COV_{RD}) + 0.00384\,(FC)_T + 0.00736\,(BC)_T + 0.00115\,(LC_{SNWP})_{MH} \quad \text{.......... (13)}$$

where:

IRI = IRI at any given time, m/km
IRI_0 = initial IRI, m/km
SF = site factor
$^{age/20}$ - 1 = age term, where age is expressed in years
COV_{RD} = coefficient of variation of the rut depths, % (assumed to be 20%)
$(TC_L)_T$ = total length of transverse cracks (low, medium, and high severity levels), m/km
$(FC)_T$ = fatigue cracking in wheel path, percent total area
$(BC)_T$ = area of block cracking as a percent of total lane area (user input)
$(LC_{SNWP})_{MH}$ = length of moderate and high severity sealed longitudinal cracks outside wheel path, m/km (user input)

The site factor is expressed as in Equation 14:

$$SF = \left[\frac{(R_{SD})(P_{075}+1)(PI)}{2*10^4} \right] \left[\frac{\ln(FI+1)(P_{02}+1)[\ln(R_m+1)]}{10} \right] \quad \text{..........(14)}$$

Where:

R_{SD}=standard deviation of the monthly rainfall, mm

$P_{.075}$= percent passing the 0.075 mm sieve

PI= percent plasticity index of the soil

FI= average annual freezing index, ^{0}C-days

P_{02}= percent passing the 0.02 mm sieve

R_m= average annual rainfall, mm

For asphalt treated bases, the IRI is expressed as follows in Equation 15:

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$IRI = IRI_0 + 0.0099947\,(age) + 0.0005183\,(FI) + 0.00235\,(FC)_T + 18.36\,[1/(TC_S)_H] + 0.9694\,(P)_H$ …..(15)

Where:

$(TC_S)_H$ = average spacing of high severity transverse cracks, m (estimated from thermal cracking model)

$(P)_H$ = area of high severity patches, percent of total lane area (user input)

All other variables are as previously defined.

Another approach to the development of mechanistic-empirical performance prediction models is that pioneered by Danish researchers (Ullidtz 2002, Busch et al 2005, Hildebrand 2006)and it involves computer simulation of pavement performance, based on which, Mathematical Model of Pavement Performance (MMOPP) was developed. MMOPP is capable of predicting longitudinal roughness, rutting and fatigue cracking of a pavement consisting of a bitumen or cement bound layer, a granular base and subbase layer and subgrade. To simulate gradual deterioration over time, MMOPP makes use of an incremental-recursive procedure, where the output from one time increment (one season) is used, recursively, as an input for the next time increment.

The model considers the variation of pavement layer thickness, elastic stiffness, plastic parameters and dynamic load variations along the length of the road. A pavement section is divided into short lengths of 0.3 meters in which the aforementioned parameters are varied. Pavement response is calculated using linear elastic theory. Seasonal changes are considered in MMOPP by using time increments of one season. For each section the effect of the loading is determined in terms of permanent deformation, crack initiation and fatigue induced decrease in asphalt stiffness. Roughness in terms of slope variance is calculated using the permanent deformation. This procedure is repeated with output from one step used as input in the next step for a predefined period or until a certain level of deterioration has been reached.

MMOPP has been calibrated using data from AASHO road test, from full scale accelerated pavement testing and against general experience with pavements in Denmark.

A Model similar to MOPP, referred to as Whole-life Pavement Performance Model (WLPPM) was developed in the UK (Collop and Cebon, 1995). In WLPPM a vehicle simulation is used to generate dynamic tyre forces that are a function of distance along the road. These dynamic tyre forces are then combined with appropriate pavement primary response influence functions (stress, strain and displacement) to give primary response histories at regularly spaced points along the pavement. The primary response histories are then transformed into pavement damage (fatigue and permanent deformation) using an appropriate damage model. The result is an increment of damage at each point along the pavement due to a single vehicle pass. The pavement surface profile is then updated to reflect permanent deformation damage, and the layer material parameters are changed to reflect fatigue damage.

D’Apuzzo et al (2004) developed a model for prediction of roughness progression of asphalt pavements using a modelling approach similar to that used in MMOPP and WLPPM. In this model the road length is discretized in a number of 0.3 – 0.25m long sub-sections. Different layer thicknesses and mix properties are assigned to each section by means of autoregressive time series. Further more dynamic loads are assumed to be applied to the surface at the middle point of each sub-section. Primary response due to dynamic traffic loads and the subsequent permanent deformations of pavement layers and subgrade are evaluated for each section and for each calculation step. The road pavement profile is updated using the total permanent displacement of each sub-section and this process continues until the end of the analysis period.

Information regarding the extent to which these last two models are validated and applied in practice is not readily available.

Mechanistic models for prediction of rutting in granular and bituminous bound materials were also developed under the European SAMARIS project, which was completed in 2006 (Hornych and El Abd 2006, Blab et al 2005). Two models were developed for permanent deformation in unbound granular materials: empirical model and elasto-plastic model. The elasto-plastic model was implemented in a finite-element code developed by LCPC, France. The predictions of these models were compared to measured data from LCPC's testing facility and reasonable agreement has been reported. These models are also being evaluated under the “active design” project in Sweden, the result of which would be interesting for the NordFoU project. The proposed permanent deformation model for bituminous materials was based on linear visco-elasticity.

3.3 Probabilistic Models

The deterioration of pavements is affected by several factors some of which are difficult to observe. Traffic load and environmental conditions change over time and are difficult to predict. This makes the performance or deterioration of pavements to vary greatly showing uncertain or random characteristics. Furthermore uncertainty can arise from the inspection or measurement process and from inability to quantify the factors that affect the deterioration process, and to model the true deterioration process of the materials. Thus pavement deterioration process shows stochastic characteristics.

Probabilistic models attempt to tackle the stochastic characteristics of the pavement deterioration process. Most of the proposed probabilistic models are based on Markov process modelling. A Markov chain is a special type discrete-time stochastic process where the state of the system (for eg. pavement condition) X_{t+1} at time t+1 depends on the state of the system X_t at some previous time t but does not depend on how the state of the system X_t was obtained. In mathematical form this can be expressed as in Equation 16:

$$P\,(X_{t+1}=j \mid X_t=i) \quad \ldots\ldots \quad (16)$$

Where

P is the probability of the state at time t+1 being j given that the state at time t was i, assuming that the probability is independent of time. This assumption is known as the stationary assumption and it represents a major limitation for most of the probabilistic models because it implies that the rate of deterioration of pavements is time independent. Few models use so called non-homogenous (time dependent) Markov chains to overcome this limitation (Li 2005). Some of the probabilistic models are developed based on econometric methods. More detailed review of the probabilistic models is given by Li (Li 2005).

One of the major challenges facing existing probabilistic models is the difficulty in establishing the Transition Probability Matrices (TPMs). A TPM is a square *sxs* matrix where s is the number of possible states in the system. The matrix contains the probabilities of transitioning from state i to state j, i.e, the probability of something being in one state and then changing into another state over a fixed time interval. The TPM can be established using historical data or subjective opinions of experienced engineers through individual interviews and questionnaires, which takes considerable time and expenses.

An example of such models is the Highway Investment Planning System (HIPS) used widely in Finland and Norway. These models usually group the pavements into families (group of pavement sections with similar characteristics) and as such are suited for network-level pavement management systems or strategic investment analysis for the road network. However, they are not suitable for project-level analysis.

4. CONCLUSION

The accurate prediction of pavement performance is very important for efficient management of the road infrastructure. By reducing the prediction error of pavement deterioration agencies can obtain significant budget savings through timely intervention and accurate planning (Prozzi and Madanat 2004).

Pavement performance prediction has been the key component of Pavement Management Systems (PMS). A PMS is considered as a programming tool that collects and monitors information on current pavement, forecasts future conditions, and evaluates and prioritizes alternative reconstruction, rehabilitation and maintenance strategies to achieve steady state of system preservation at a predetermined level of performance. Effective implementation and utilization of pavement management systems in generating and evaluating various alternative strategies based on engineering and economic principles is largely dependent on the ability to predict the future condition of the pavement. The current trend is to integrate pavement management systems, bridge management systems, and other systems related to road infrastructure management into comprehensive road asset management systems. Asset management goes beyond the traditional management practice of examining singular systems within the road network, i.e., pavements, bridge, etc., and looks at the universal system of a network of roads and all of its components to allow comprehensive management of limited resources. Through proper asset management, governments can improve program and infrastructure quality, increase information accessibility and use, enhance and sharpen decision making, make more effective investments and decrease overall costs (OECD quoted in US Department of Transportation 1999). Figure 2 illustrates the components of a generic asset management system. It can be understood that performance modeling and performance monitoring represent key aspects of such a system.

Performance prediction plays a key role not only in pavement management system (or asset management system) but also in pavement structural design. Pavement design involves choice of materials and thickness for the various layers based on sound engineering and economic principles. This requires comparison of alternative materials and thicknesses, which depends heavily on the ability to predict the performance of the alternative material and thickness combinations. In the past, pavement design has relied on empiricism and experience. In recent years, however, mechanistic – empirical design methods, which are based on more fundamental engineering principles, are being applied in various countries. Performance prediction models form the cornerstone of these mechanistic – empirical design procedures.

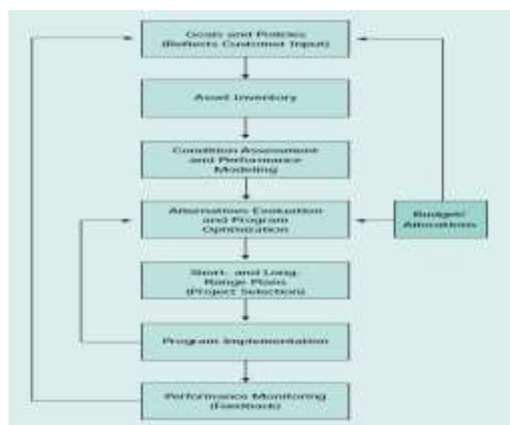

Figure 2: Generic asset management system (US Department of Transportation 1999)

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