Department of Civil Engineering

Optimum Use of the Flexible Pavement Condition Indicators in Pavement Management System

Adnan M S H Shiyab

This thesis is presented for the Degree of
Doctor of Philosophy
of
Curtin University of Technology

November 2007

Declaration

To the best of my knowledge and belief this thesis contains no material previously published by any other person except where due acknowledgment has been made.

This thesis contains no material which has been accepted for the award of any other degree or diploma in any university.

Adnan M S H Shiyab

21 November 2007

Abstract

This study aimed at investigating the current practices and methods adopted by roads agencies around the world with regard to collection, analysis and utilization of the data elements pertaining to the main pavement condition indicators in pavement management systems (PMS). It also aimed at identifying the main predictors associated with each condition indicator and the factors that govern pavement structural and functional performance.

Development of a new performance index that incorporates parameters or measures related to the main condition indicators (surface defects, roughness, deflection and skid resistance) and establishing the weight to be assigned to each indicator based on the relative impact on pavement condition was also one of the main objectives of this study.

Thousands of pavement sections were subjected to thorough testing and inspection over the last few years to collect data pertaining to the main condition indicators. The collected data encompass visual distress survey, deflection measurements, roughness and skid resistance measurements. Collection of various condition indicators was accomplished according to well known international standards. The collected data were processed, tabulated and analyzed for the purpose of development of performance models and to prove certain theories or good practices.

Advanced tools and machines were utilized to collect these data with a high degree of accuracy. The Falling Weight Deflectometer (FWD) was used to collect deflection data for structural analysis. Two Non-contact laser roughness measuring devices mounted on vehicles were heavily used for collecting roughness, texture, and rutting data. Distress data were collected using a manual procedure adopted and standardized at the Pavement Management System Unit of Dubai Emirate.

Powerful engineering and statistical softwares were used in the analysis for the purpose of processing the data, back calculating the main parameters pertaining to pavement response, establishing the correlation matrices between various dependent variables and their predictors, and finally, applying linear and non linear regression analysis to develop reliable and predictable deterioration models for the uses of pavement management system.

The analysis procedure was supplemented by a vast literature review for the up to date information along within deep investigations and verifications for some of the current practices, theories and models used in pavement design and pavement evaluation with more emphasis on the inherent drawbacks associated to these models and procedures.

The study confirmed that pavement condition deterioration and performance can be best predicted and evaluated based on four main condition indicators; First, surface distress to assess the physical condition of the pavements and detect the inherent problems and defects caused by various factors affecting pavement performance. Second; roughness measurements to evaluate the riding quality of the pavement. Third; deflection to calculate pavement response (stress and strains) and to assess pavement structural capacity and calculating the remaining life, and finally, skid resistance measurement to assess the level of safety and surface texture properties.

Thorough study and investigation of the physical condition indicators and the associated parameters, confirmed that pavement distress data are vital elements in each pavement management system. Distress data can be used effectively to identify the main problems associated with pavement performance, causes of deterioration, maintenance measures needed to prevent the acceleration of the distress, the rehabilitation schemes needed to improve the pavement condition and finally to prepare maintenance work programs and to estimate the annual maintenance needs under an open or limited budget. Alligator cracking was found to have the heaviest impact on

pavement condition. Distress density, probable causes of deterioration and distress propagation rate are the required parameters in PMS.

Roughness was found to have a basic influence on pavement condition and the type of selected treatment. The use of Roughness data in terms of International Roughness Index (IRI) can be optimized in PMS by using this indicator in the following forms:

- Roughness, as an objective measure, can be used as a good performance predictor of the current riding quality of pavements in service and reflects the inherent imperfections and built-in irregularities embodied in the road pavement surface.
- Roughness measurement can be used as a reference to establish construction specifications and provides through the PMS system an organized feedback approach to correct the persistent design deficiencies detected after road construction.
- Roughness can be used effectively in the planning process for maintenance works and to select the candidate sections through calculating the functional remaining life based on the estimated terminal value using Roughness-Age, Roughness-ESAL, and Roughness-PSI models.
- **Lane–IRI** along with the Difference between the left and right wheel IRI values, termed as ***" Yaw"*** are the most suitable forms to be used in PMS to report about roughness characteristics.
- **Yaw** term can be used effectively to report or feed back about geometric imperfections that exist on the road surface such as improper cross slope, shoving and the probable drainage problems.
- The roughness cumulative distribution curves can be used as a planning tool in PMS to report at the network level. These curves indicate the network health and the required funding at different level of risks, so proactive measures can be taken and the required budgets can be made available.

Deflection data were found to form a basic component of the PMS. It was found that these data can be used at both network and project levels. *Direct deflection* measurements were found **Not** to be the ideal form to report about structural capacity at the network level. It is rather can be used at project level to detect weak spots and critical pavements layers.

At the network level, the back calculated parameters from deflection basin such as **Pavement Modulus (Ep)**, **Asphalt and Pavement Curvature (CUR)**, **Cross Sectional Area** and the other deflection basin characteristics are much more appropriate for reporting about pavement structural conditions and calculating the structural remaining life in PMS.

The design deflection and curvature that characterize the pavement have been found to be calculated based on the mean along with the two times the standard deviation of the measured data.

The **Effective Structural Number (SN_{eff})** was found to have good correlations with the **Total Pavement Thickness (H_t),** the value of the deflection measured at the center of the loading plate **(D_0)** and the difference between D_0 and the deflection measured at 450mm from the center of the loading plate **(D_0 - D_{450}).** The first two variables were found to account for more than ***92%*** of the structural capacity prediction model.

Traffic variable in terms of the accumulated standard repetitions (ESAL) was found to account for more than ***60%*** of the deflection model predictability. Other variables such as E value, asphalt and base layer thicknesses can improve the predictability of the model if included.

The concept of the relative value of effective pavement modulus to the original pavement modulus **(E_{eff}/E_0)** was found to gives a reliable representation about the exhausted and the remaining life of the in-service pavement structure. The study showed that the pavement is reported to be structurally failed, when the effective asphalt or pavement modulus is about

20 - 35 % of its original design value which is equal to the modulus of the unbound material.

It was also found that when the area of the fatigue cracking and the patching distresses exceeds **17%** of the total pavement section area, or the depth of rutting is more than **15mm** , the pavement is reported to be structurally failed and major rehabilitation or reconstruction should be applied.

Skid resistance can be reported in the form of International Friction Index (**IFI**), as a well defined universal index, along with other two numbers ;**F60** Friction (Microtexture) related number measured at 60 km/h velocity and Macrotexture related number and **Vp,** which constitute the IFI index can be used in Pavement management system applications to report about skid resistance characteristics and the network level of safety. These three figures can be used to report about pavement condition, accidents, airports operations, and maintenance management surveys.

In this study, new methods and models were developed and suggested to be used in PMS as an alternative to the current available methods which were found to be impractical in certain cases.

Finally, further research efforts are recommended to explore the uses of other parameters in particular those related to deflection basin analysis, cross sectional area, curvature, and pavement moduli.

Skid resistance testing and reporting method should be subjected to further research works for the purpose of standardizing reporting methods, identifying the relative impact of main predictors i.e. megatexture, macrotexture and microtexture components and to develop performance models.

Acknowledgement

First, praise be to Allah the master and the creator of the universe, Most Gracious Most Merciful, for his blessings, help, generosity and patronage that empowered me to an unlimited horizon to complete this humble work and fulfil the study objectives.

I would like to invest this opportunity to deeply thank my supervisor Dr. Hamid Nikraz for his continuos support, guidance and encouragement that enabled me to complete this work despite the enormous difficulties encountered during my work due to the lack of recourses, tools and capabilities. The technical contributions, personal support and friendly care offered by my supervisor, Professor Hamid Nikraz, are gratefully acknowledged.

Deep appreciations are extended to Professor David Scott, the chairman of the examination committee for his kind assistance, follow up and support.

Thanks are extended to the co-supervisors and the external examiners who had spent invaluable time and exerted much of the efforts for reviewing the thesis chapters and setting up their comments which, of course, enriched the thesis contents and improved the level of credibility and reliability of the offered thesis subjects and outcomes.

Also, I will not forget, never, ever and for ever, the care, support, and assistance offered by my beloved wife, Aya Alwan, and my beloved family members, Solaf, Omar and Lemar for their patience and understanding during the long period of this study.

I will never ever forget the praises and prayers that always extended to me by my beloved mother and father ,who I believed ,after Allah (MCMM), has granted me the power to pursue my studies and to be a creative and a productive man.

My appreciations and thanks are also extended to my family members, brothers and sisters, especially Professor, Said Shiyab and his family, who continuously supported and helped me to pursue my study.

Finally, I would like to express my deep thanks to all individuals who, directly or indirectly, supported and helped me to complete this work.

Table of Contents **Page**

List of Figures

List of Tables

CHAPTER 1. PAVEMENT MANAGEMENT SYSTEM CONCEPT AND CONDITION INDICATORS

Introduction

In the last three decades, pavement maintenance and rehabilitation have become a major problem that faces highways agencies around the world. This problem is compounded by the lack of the proper appropriations to improve the condition of the pavement network. To overcome this problem, many highway agencies started to establish procedures, alternative and practices for the purpose of preserving their pavement network. Such practices are normally constrained by many factors such as staffing, funding and lack of expertise (Wotring *et al*, 1998).

Pavements account for more than 60% of the asset value of any road agency, therefore, it constitutes a major part of the highways infrastructure (Hass *et al*, 1994). Huge investments are required to maintain each country's road network.

Recent statistical figures reported by the Federal Highway Association (FHWA) showed that the annual cost to maintain the pavement of the national highway system in the United States at the existing condition is nearly $50 billion (Zaniewski *et al,* 1996). The United States spends over $25 billion per year. The study also indicated that to bring the entire pavement system up from its current level to a good level would cost around $200 billion.

In 1993, roads agencies spent over $87 billions on US highways, roads and streets, 50% of that amount went directly into pavement construction , maintenance and rehabilitation . This amount of spent money kept the US roads

in an acceptable condition where 50% of these roads were found in good, fair and poor condition(Zaniewski *et al,* 1996).
The US roads network system includes 45,000 miles of interstate highways, and over 111,000 miles of other national highways systems and over 3,700,000 miles of other roads and streets (Zaniewski *et al,* 1996).

In a study conducted by the World Bank in 1987 about road deterioration and financial requirements in 85 developing countries, it was indicated that the backlog of rehabilitation for the main road network is valued at approximately $41 billion. This figure would have been to range only between $10 to $12 billion if these needs were met on a more timely basis. The above figures excludes the bridges and the large tertiary and the lower road networks. A speculative estimate for these components would be on the order of $15 to $25billion additionaly.

The study pointed out that the costs of routine and periodic maintenance needed to prevent the pavement in good condition from deterioration further during the 1986-1999 was estimated to be around $4.6billion /year. Provided that these needs are met on a timely basis, requirements would then tapered off after 1991 as the present surge of roads in fair condition passed. After allowing for this tapering, requirements to meet new deterioration expected over 10 years period(1986-1996) would be total about $43billion.

As an estimate for the cost of the rehabilitation of the roads, if the maintenance needs for 20% of the roads in fair condition are not met on the proper time, the cost will be increased by about $20billion if it is allowed for these roads to deteriorate to the point where they require major rehabilitation.

The above figures provide a clear indication that such a system cannot continue to operate in the traditional approaches in managing and maintaining the road network. There is a vital need to re-evaluate the methods and the strategies

adopted by the highway agencies for these to be more cost effective. This can be achieved through establishing pavement management systems to manage and maintain pavement networks in a much more effective manner (Smith *et al,* 1987).

Pavement management system concept

AASHTO defines a pavement management system (PMS) as "*a set of tools or methods that assist decision makers in finding optimum strategies for providing, evaluating, and maintaining pavements in a serviceable condition over a given period of time* (AASHTO,1993).

The researcher views pavement management systems as *a systematic method for data collection, processing, condition reporting, and decision making for the purpose of optimizing the maintenance and rehabilitation needs*. Therefore, PMS is considered as a philosophy (way of thinking) adopted by each road agency for managing road network using well established procedures that satisfy the agency's needs with the help of information technology tools and programs.

The functions of a PMS is to improve the efficiency of decision making, expand its scope, provide feedback on the consequences of decisions, facilitate the coordination of activities within the agency and ensure the consistency of decisions made at different management levels within the agency.

Figure 1.1 shows different activities incorporated in a pavement management system. As shown in this figure, a pavement management system provides information at the network level at is used to develop a statewide program of new construction, maintenance or rehabilitation that will optimize the use of available funds.

At the project level, more detailed consideration is given to the preferred or optimum design, construction, maintenance or rehabilitation activities for a particular roadway section or project within the overall program.

In this stage, an optimum option that will provide the desired benefits or service levels at the least cost over the analysis period is identified by calculating the benefits and the costs associated with this option.

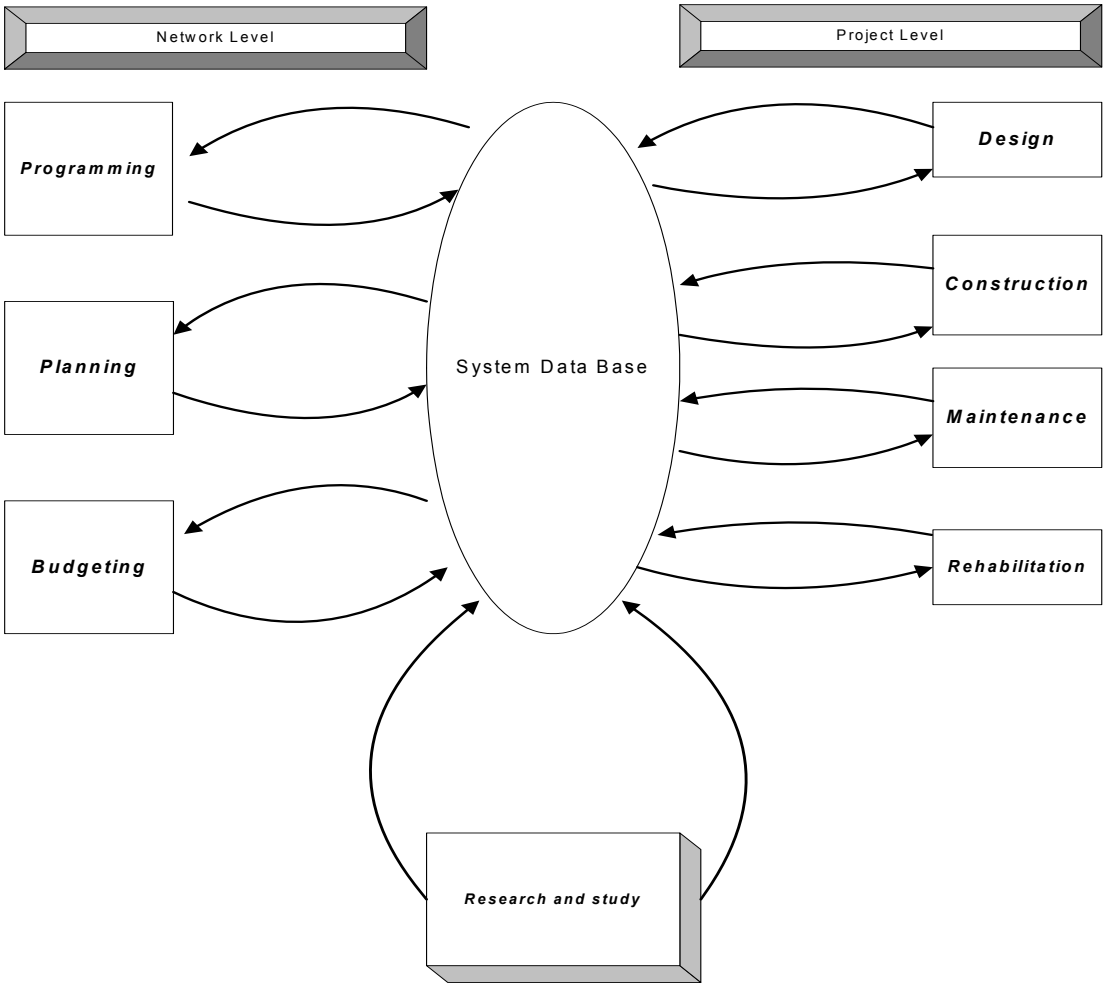


Figure 1.1: Different activities of the pavement management system (AASHTO, 1993).

As can be seen in Figure 1.2, information obtained form pavement evaluation can be used as a feedback in the planning and design phases to modify design standards. Therefore, PMS provides an organized approach to correcting the deficiencies deducted after project construction (AASHTO, 1993).

It should be emphasized that PMS does not make decisions but provides a methodology for processing the data , analyze the feasible options and make the needed comparisons which then permits the decision makers to sort out the results and compare the alternate possibilities based on a set of practical, realistic decision criteria.

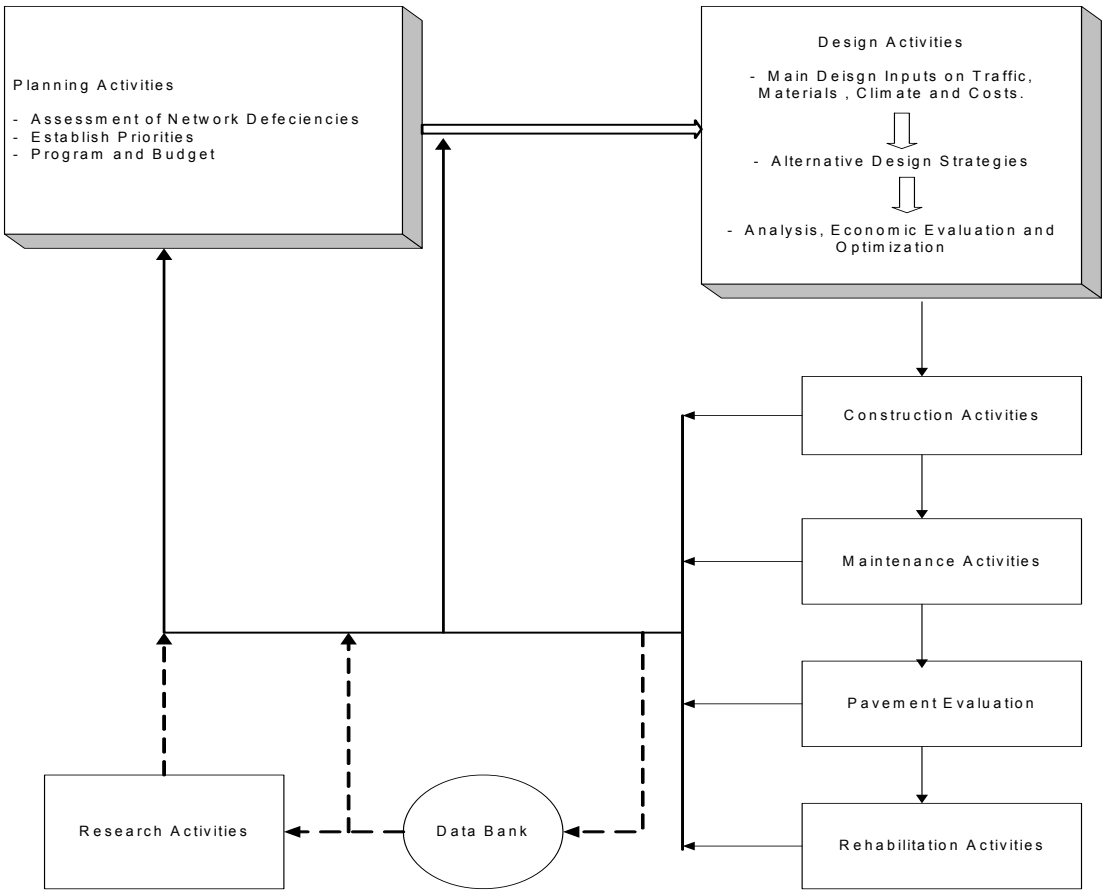


Figure 1.2: Major Classes of Activities in PMS (AASHTO, 1993).

Figure 1.3 depicts this aspect. The solution of a single set of inputs, selected based on a set of practical, realistic criteria, should fulfill the requirements of the design. There are several models for flexible and rigid pavements. Using one of these models will produce an estimate of the design life related to a particular set of inputs. This may or may not meet, with sufficient reliability, the performance period or required design period constraints set forth. If a given design trial satisfies these constraints, then it moves to the economic evaluation block of the process and so on (AASHTO, 1993).

In the last twenty years, many useful pavement management systems have been developed to enhance maintenance planning and field works and to make life easy for maintenance engineers and decisions makers. Despite all the developments in this field, further improvements and enhancements are deemed to be needed to make these systems more comprehensive, reliable and practical.

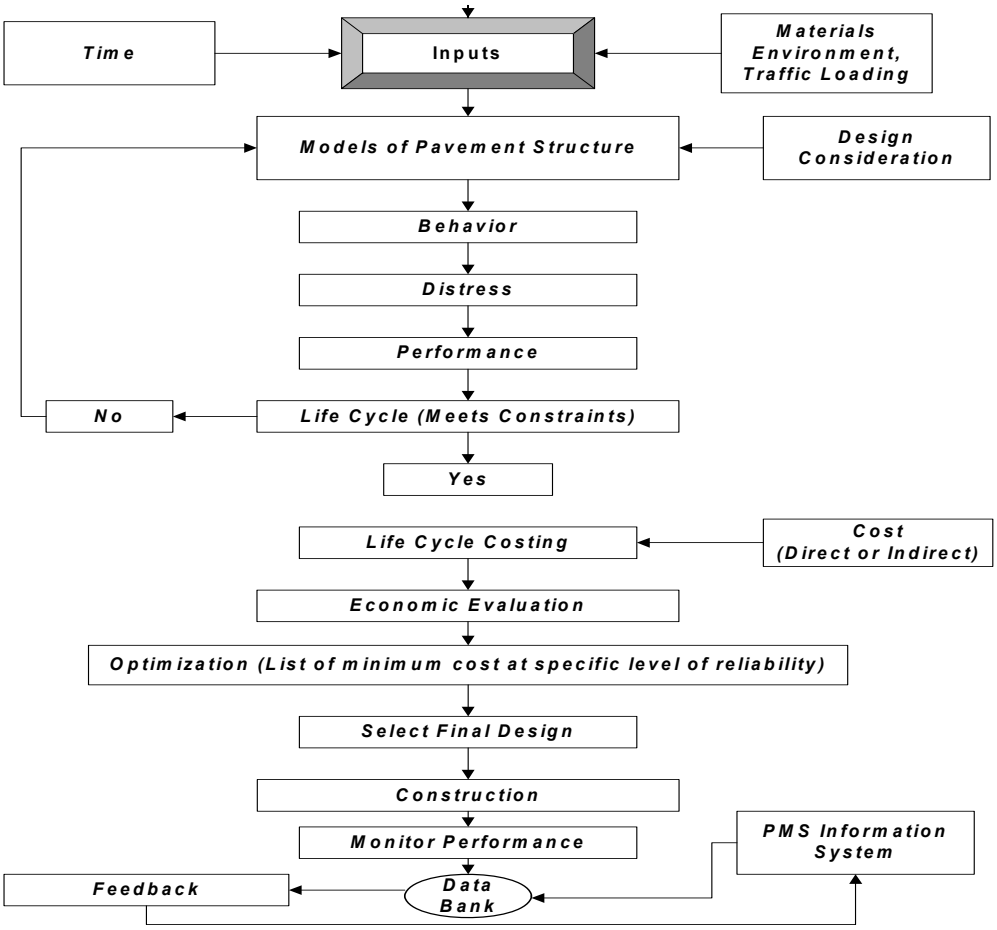


Figure 1.3: Pavement Management System processes flow diagram (AASHTO, 1993).

The need for development a comprehensive, well-established pavement management systems has generated a world wide interest in research to develop such systems (TRB, 1994).

Pavement condition components

The evaluation of Pavement condition or pavement performance is an important factor of pavement design, rehabilitation and management. It includes the evaluation of distress, roughness, friction and structure. It is also, an important part of the pavement management system by which a most cost effective strategy for the maintenance and rehabilitation can be developed (Huang, 1993). In general, pavement condition consists of four main components:

1. Load Bearing Capacity
2. Riding comfort
3. Safety and,
4. Aesthetics

Under the effect of traffic loading and the environment, pavement condition deteriorates with time. Therefore, pavement condition must be considered at the present and in the future. Pavement management evaluation modules include predictive models which are normally developed based on empirical and/or analytical relationships to describe pavement behavior.

There are three generally accepted measures of pavement performance; Structural performance, Functional performance and Safety. Safety performance is normally measured in terms of frictional characteristics of the pavement. Functional performance has been, and is still, one of the most important measures of pavement performance. It is measured in terms of roughness or ride quality of the pavement surface. Structural performance is connected to the loading capacity and the physical defected symptoms.

Pavement condition indicators

Pavement prediction models describe the behavior of the pavement over time which is referred to as pavement performance. Periodic monitoring of the pavement condition variation over time allows adjustment and modification of the prediction models and consequently the improvement of the reliability of the management system by establishing sound predictive models. Sound prediction models allow a more precise life-cycle costing analysis and ensure the selection of the more appropriate and cost effective repair option.

In this sense, the condition of the pavement can be evaluated in terms of factors referred as condition indicators highly associated with distresses manifested on pavement surface (Figure 1.4). These include (Shahin *et al*, 1987):

1- Operational surface indicators. These may include :
   - Roughness (localized and profile roughness).
   - Skid resistance and hydroplaning potential .
2- Structural indicators : These may include structural integrity evaluated based on the observed distress types, quality and severity such as rutting, cracking, distortion and disintegration.
3- Other indicators such as rate of deterioration and the previous amount of maintenance and rehabilitation work applied.

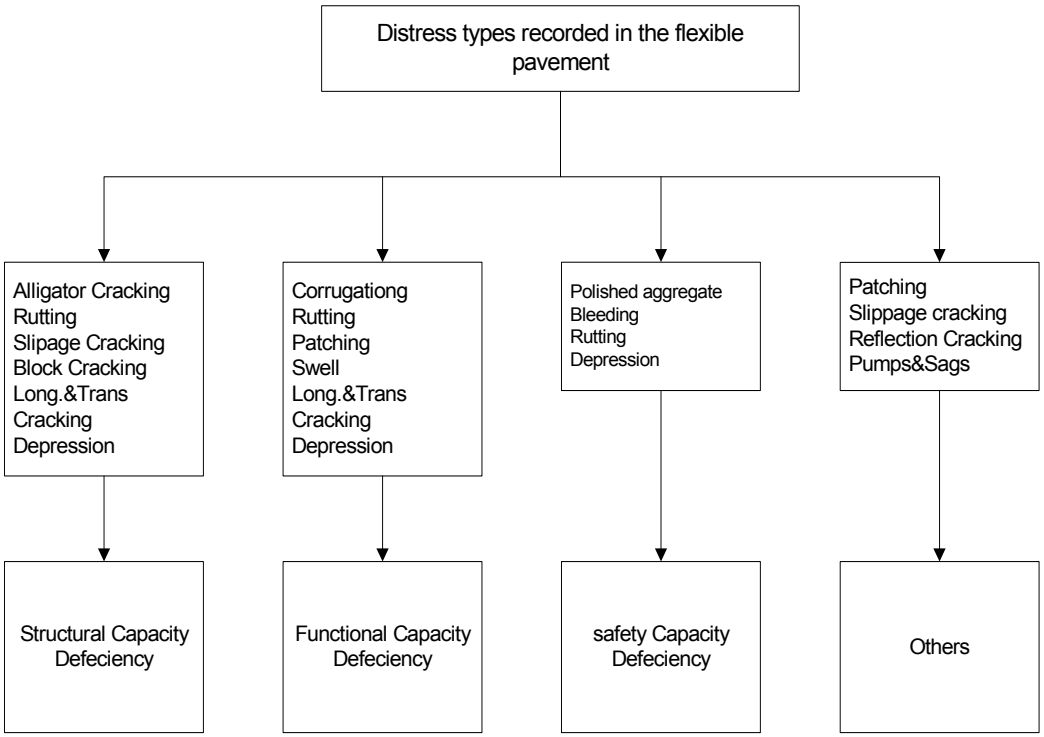


Figure 1.4 :Relationship between condition indicators and various distress types (Shahin *et al*, 1987)

In general, the condition indicators mentioned in the above studies can be re-grouped in the following order:

1. Physical Defects (Type, Severity and Density), with more emphasis on the following distress types:
    - Fatigue (Alligator) Cracking
    - Permanent Deformation or Rutting
2. Structural Capacity parameters
3. Roughness level and
4. Skid resistance

Figure 1.5 below depicts the position of several monitoring methods within the pavement management system as envisaged by the researcher. As shown in this figure, physical strength properties are assessed by analysis of deflection results. The severity level and quantity of each distress present is identified by means of a distress condition survey.

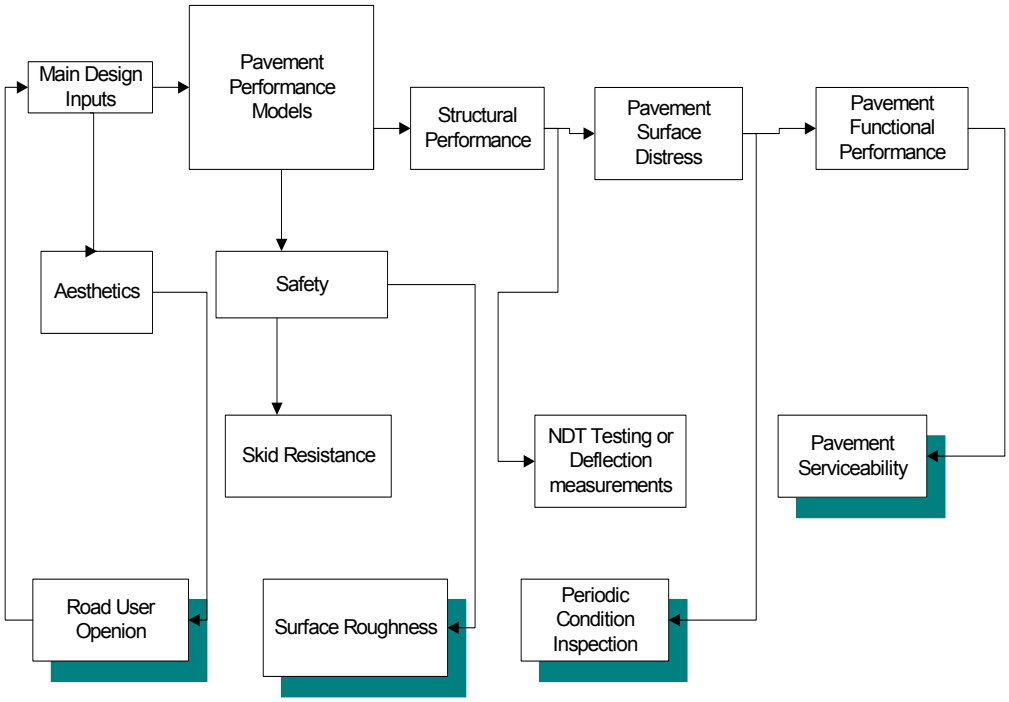


Figure 1.5: Pavement management system monitoring procedures.

The structural behavior of the pavement is monitored with the response of load-deflection measurements. Safety response is handled by measuring the skid resistance and roughness of the surface. Accurate information pertaining to the main design and rehabilitation inputs are important for both road planning, design, and rehabilitation. Whereas the influence of factors affecting pavement design such as traffic loading, environmental factors and material are well defined, the use of such factors in pavement management systems is not. Despite all research efforts that have been made to quantify and optimize the results of pavement condition indicators and performance, further research is still required to verify and further quantify these results in order to optimize both the treatment selection process and the use of the available maintenance funds.

Pavement management system uses and benefits

Performing regular or periodic evaluation for pavement condition usually enables road agencies to plan for the required maintenance with minimum cost.

Carrying out the required preventive maintenance, which is used to stop minor deterioration, retard progressive failure and reduce the need for corrective maintenance, reduces the cost considerably as it prevents the pavement condition from reaching a condition in which the corrective maintenance is no more useful and major rehabilitation or reconstruction is deemed to be necessary (Hicks *et al* , 1999).

Pavement management systems generally include models to predict future pavement condition, prepare work plans and to estimate budgets. These mainly depend on pavement type, condition, traffic, pavement class and age. In order to implement cost effective maintenance policies, the repair action should be applied to the right pavement condition and at the right time.

According to a study conducted by the World Bank 1987, it was indicated that among 85 countries reviewed, some $30billion could have been saved if roads had received periodic maintenance before they deteriorate to the point at which they require major rehabilitation (Smith *et al*, 1987). The appropriate timing to carry out maintenance and rehabilitation is crucial; if delayed, the road structure may fail beyond the scope of restoration (Reddy, *et al* 1997).

Applying pavement preventive maintenance can extend the functional life of the pavement if applied at the right time and at proper deterioration level or threshold (Zaniewski and Mamlouk, 1999). It was proven through analyzing the outcomes extracted from different research programs in the United States that preventive maintenance is cost effective for both low and high trafficked roads if applied to the pavement when in good condition.

The study recommended initiating an educational program for both the highway professionals and the traveling public for better understanding of the effect of preventive maintenance and the different maintenance categories.

Although each type of maintenance is needed in a comprehensive pavement maintenance program, the emphasis should be placed on preventing the pavement from reaching the condition in which corrective maintenance is required as the cost associated with this approach can be substantial (Hicks *et al* , 1999). This situation is depicted in Figure 1.6 below.

The effectiveness of the applied maintenance treatment of flexible pavement has been investigated by the Strategic Highway Research Program (SHRP) through a specific Pavement Study–3 a task force (Eltahan *et a*l 1999). The study examined the effect of the original condition on the performance of the maintained sections. Also life expectancy has been evaluated for each treatment.

The study came up with a finding that after 6 years of the treatment application, pavement sections with a poor original condition had the probability of failure of 83% whereas those with fair or good original condition has an average probability of failure of 38 and 37 percent, respectively.

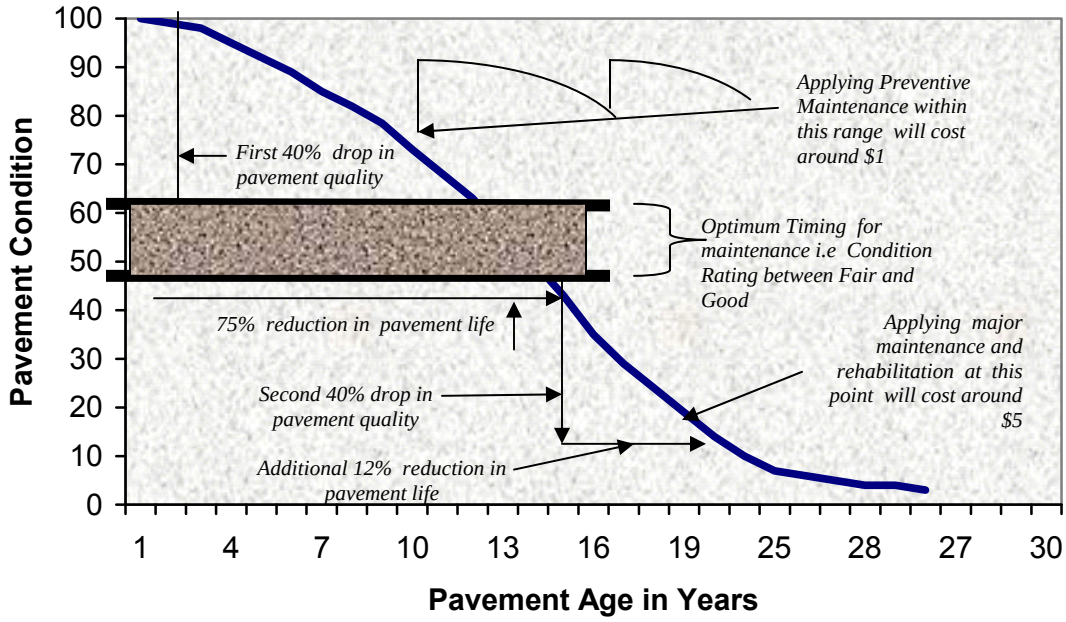


Figure 1.6: Pavement life and the corresponding maintenance cost (Hicks et al, 1999).

Also, it was found that average survival time estimated for three treatments applied in the experiment (i.e. thin overlay, slurry seal and crack seal) were 7, 5.5 and 5.1 years, respectively. The study showed that applying the maintenance to sections with a poor condition increases the risk of failure by 2 to 4 times.

Survival analysis can be used effectively with pavement maintenance data without waiting for all pavement sections to reach failure. This provides efficient use of the data already collected without overspending on data collection activities.

Pavement performance is influenced by many factors such as traffic loading (i.e. wheel loading, tire pressure, loading frequency and duration), environment factors (i.e. temperature, moisture, rainfall and sunlight or ultraviolet exposure), other factors that might affect the pavement performance are material characterization, construction practices, maintenance measures and specifications (Wijk *et al*,1994).

Without the ability to use structural capacity indicators of the pavement as an intervention level for decision making, the basis of the treatment selection is derived only from the visual condition of the pavement (Pain, 1998). The ability to use structural data as well as other pavement condition indicators will enhance considerably the reliability of the decision making process and rationalize the treatment selection procedure.

Earlier, It was indicated that structural monitoring of a pavement network on a regular basis can be accomplished within the scope of network monitoring plan. It was believed that the use of maximum surface deflection under the load as a structural condition indicator is probably all that is required to assess the structural capacity of a pavement (Hass *et al*, 1993 and Hass 1995).

This saying has been argued later on by many researchers whether it will be sufficient to include only this data element to indicate the structural capacity at the network level or not (Paine, 1998). It was concluded that the use of maximum surface deflection for a section as an integral tool in the treatment selection process without relating it to the expected traffic loading (*i.e. determining the effective structural capacity)* will be normally limited. This area of research is still under investigation by many researchers and agencies in the field of pavement management system and it will be handled partly in this study.

The longitudinal profile, translated into roughness, and the transverse profile translated into rutting are the main pavement condition characteristics as they predominantly influence the way the pavement serves the traveling public. Therefore, pavement rutting is an important pavement performance and pavement design characteristic for asphalt concrete pavements. The measurement, analysis and utilization of rutting data should also be integrated with the overall pavement management system (Hajek *et al*, 1998).

Pavement Condition Evaluation and Performance

Many attempts have been made over the last 3 decades to develop models that can predict accurately the structural and functional performance of the highway pavement over time. Unfortunately, most of the developed models were based on theoretical assumptions and few of these were based on actual in-service pavement data. Moreover, the performance models developed based on in-service pavement data were a function of one variable normally the time, and none of the structural, environment and materials data elements were included in such models (Hand A *et al* , 1999).

In Hand's study, pavement management systems data were used by different agencies to develop pavement performance models. These models have been developed through establishing network optimization system which was originally initiated for the purpose of evaluating various alternatives and to recommend the

most cost effective rehabilitation treatment to be used on the various sections of Nevada State highway system.

Estimating the annual maintenance cost figures was the most difficult task in this research. After several attempts, the agency was able to identify a fixed period cumulative cost approach as a best alternative to reducing the variability in the annual maintenance costs. The developed models in conjunction with other supporting database elements were used to conduct a fully automated optimization analysis.

An evaluation system to determine the existing pavement condition is essential component of the pavement management system. Most of the pavement management systems currently in use have this component and they are widely vary in their approach and sophistication. In general , a condition survey is conducted on segments of the existing pavement and various distress data are recorded. The pavement condition of each pavement segment is expressed in terms of a single index calculated based on the collected distress data. Distress data along with other information such as traffic , materials and environment can be used in the analysis of these data for the purpose of determining maintenance and rehabilitation needs.

Understanding why pavements perform as they do will provide the tools for the managers and the engineers to deal with an over-changing world and more effectively manage the risks inherent in highway design, construction, evaluation , maintenance and rehabilitation.

Pavement condition index is an integral component of the pavement management system. It is the only pavement condition indicator that can be used to describe objectively the current network pavement condition and predict the future performance. Currently, performance index is considered the main column in the skeleton of all newly developed pavement management systems. It is

normally a calculated figure on specific scale that combines many different distress types, severities and extents collected for every single pavement section.

Enormous efforts have been paid by many agencies and individuals to develop an easy, straight forward and practical method for pavement evaluation and determining the maintenance needs. Unfortunately, the developed indices were not representative usually lack the accuracy in the practical side.

In a trial to develop a new system that integrates academic and state expertise to solve the state problems regarding the design, construction, rehabilitation, maintenance and management of the pavement network, Michigan Department of Transportation (MDT) established in 1998 the pavement research center of excellence at Michigan State University. The outcome of this research effort was proposing a procedure to be used to select the rehabilitation alternatives for rigid , flexible and composite pavements. The decision tree were designed to be based on few condition indicators which are distress index, ride quality index and average daily traffic-ADT (Wotring , *et al* 1998).

The new system suggested six repair alternatives that could be used to fix the distress type. The shortcoming of this system are numerous among these , it is distress -based treatment which may impart many fix alternatives for the same section leading to confusion in application. Also, all these indicators are open ended scale which may not be easily absorbed or understood by the decision makers (Wotring , *et al* 1998).

Structural data in terms of the back calculated deflection data were found helpful in prediction of pavement performance (Dixon *et al* 1983). The study showed that non destructive test results, pavement age and traffic parameters correlate well with pavement condition. Pavement condition evaluation was performed using pavement condition index (PCI) developed by US Army Corps of Engineers.

Using data extracted from military installation in Virginia, it was found that the outputs of the deflection in different forms can be used to predict pavement condition over time. Deflection measurements were performed at 0 and 12 inches from the points of loading. Two parameters from the NDT deflection measurements were used in the performance prediction model. Analysis indicated that using only deflection measurements taken at these two locations provides the best correlations. The parameters are a normalized deflection factor that gives a measure of the slope of the deflection basin referred to as (DIFF) and a measure of the deflection basin area referred to as (AREA).

The findings of this study indicated that NDT results *do correlate* with pavement condition and can be used to predict its performance. It also showed that using NDT results, in addition to other pavement condition indicators, to accurately predict pavement condition and estimate its performance will allow user benefits to be maximized. Including prediction models based on NDT results in the pavement management system will permit optimum planning , priority ranking and scheduling on a long term basis. The study recommended that further research and development is required. It has been indicated in this important study that improvement of such models could be achieved by introducing and exploring the concept of performance and strength level.

The recommendations of the aforesaid study constitute the basis on which the researcher decided to undertake this study by incorporating more condition indicators in the evaluation and treatment selection processes for the purpose of optimizing the benefits of using such parameters.

The effect of incorporating the results of the structural evaluation extracted from Falling Weight Deflectometer testing has been found considerable in the aspect of reducing the cost of rehabilitation work (Zaghloul *et al* ,1999). The FWD testing was applied to more than 500 km road sections for the purpose of assessing the

structural capacity of the existing pavement, estimate remaining life and predict future rehabilitation. Also, a comparison between the FWD and the PMS analysis regarding the proposed rehabilitation option and cost were investigated.

The FWD rehabilitation analysis is based on the structural capacity of the existing pavement whereas the PMS rehabilitation analysis is based on decision trees in which pavement distress and roughness are considered. The structural improvements were considered only if the proposed strengthening treatment or overlay is more than 50mm overlay thickness

In this study, 27% of the pavement sections tested by FWD were found in agreement with the proposed PMS treatment option. Also, 32% of the proposed treatment by PMS network level analysis were found over designed and costly. The PMS rehabilitation analysis results were found to be under designed for 41% of the total length of pavement sections considered in this study. (i.e. PMS decision process did not select any strengthening option whereas FWD did.). Limited life cycle cost analysis was conducted to identify the differences between the two methods, it was found that the difference in rehabilitation cost for the over designed sections and the under designed sections were relatively considerable.

The concept of the pavement remaining life based on Falling Weight Deflectometer testing and its uses in the management of the pavements was investigated by Daly (Daly, B 2004). In this analytical study, the method adopted by the Austroads for pavement design and rehabilitation was thoroughly reviewed. The study addressed the issue of using FWD test results (back calculated pavement layers moduli and the critical strains) in estimating the remaining life of the pavement. The study tested the ability of current mechanistic pavement design model adopted by Austroads to predict the response of the volcanic subgrade and the pavement remaining life in New Zealand.

The study concluded that determining the remaining life based on FWD is not enough. It was recommended that other factors such as rutting data (using well calibrated rutting transfer function) , roughness progression, subgrade modulus in addition to the measures related to pavement condition that indicate the extent of pavement deterioration of the should be used in conjunction with FWD tests. It was also stressed in this study that the depreciation of pavement moduli must be properly investigated and understood for the purpose of remaining life determination. In summary, the remaining life of the pavement should be determined based on three factors; *traffic loading* , s*tructural strengt*h and *environment loading* (variation of the material strength over time).

Rutting modeling have been tried by many researchers (Archilla *et al* 2001). An interesting findings have been found in this research . Main mix properties such as asphalt content, gradation , air voids and others were studied. In contrary to the traditional understanding that coarser mix resists the rutting more, the researchers found that fine-graded mixes provided the best performance. Asphalt content and voids filled with bitumen affect rutting progression adversely. Air void property by itself was found irrelevant to the rutting or asphalt mixes performance. Rutting progression model based on number of load repetitions applied to pavement section in the past time period has been developed.

Incorporating roughness as a condition indicator measurement at network level has been investigated by a study conducted in Canada (Ningyuan *et al* 2001). The study aimed at calibrating roughness measurements tested by different devices , evaluation the effect of pavement type and the wheel path on the International roughness Index (IRI) values and the effect of the base length on IRI values.

Three types of devices were used to measure the IRI value including Digital incremental Dipstick, High precision (Laser or infrared -based) road

profilometers and response type measuring system. The outcomes of this study showed that the high precision road profilometer are the most accurate and precise. All measurements are independent of vehicle weight , speed Temperature , wind speed and pavement color or texture. Calibration Models that correlate the roughness outputs generated by different devices were developed. It was concluded that roughness measurements at network level provides a valuable and informative data for pavement performance evaluation.
The study recommended that all road agencies should design and implement regular annual program for roughness data collection and verification in case more that one type of measuring devices are used. .

The impact of smoothness on hot mix asphalt pavement performance was investigated through a study carried out using data and information collected from Washington state (Pierce *et al* 2004). In this study, it was found that built smooth roads stay smooth and less deterioration. In this study and other study conducted by Shafizadeh et al, (Shafizadeh *et al,* 2002), it was indicated that 85% of the all acceptable evaluations fell at or below an IRI value of 2.7m/km. Based on this research findings , Washington state department of transportation (WSDOT) has modified the trigger value range for pavement smoothness from 3.5 -5.0 to 2.7-3.5 m/km.

The outcomes of this study indicated that using IRI as the sole trigger for pavement rehabilitation is not cost effective. Alternatively, it is more cost effective to rehabilitate pavement based on other condition indicators such as surface distresses.

A Feasibility Study on data automation for comprehensive pavement condition survey has been conducted by Kilviin et al for the purpose of proving that stereovision techniques for collecting various distress types and roughness are cost effective and more powerful method. The study recommended to use

automated data collection devices to collect and identify various surface cracks, patching and potholes, rutting and shoving (Kilviin et al ,2004).

To have much more better rating procedures and comprehensive management systems, the researchers found that the inclusion of calibrated performance models using functional evaluation indicators such as roughness data and other distress data were effective in pavement management systems decisions (Rohde *et* al 1998). In this study, the HDM-III pavement performance models were calibrated and captured in the PMS. Based on this , the improved models have been proposed for inclusion in HDM-IV. Many pavement sections over wide range of traffic, climate and condition were selected to represent pavement condition.

For structural testing, measured surface deflections were used to find a representative test position per calibration section. The pavement deflection collected at 50 m interval were analyzed to determine the average maximum deflection. A single test position where the measured peak or maximum deflection is close to the calculated average peak deflection was then selected as a representative. Road roughness was measured and rutting was recorded both using a 2m and 1. 2m straight edge.

Structural cracking, reflection cracking, thermal cracking and rutting were then calibrated by observing the increment in the distress quantity over the first three years. The outcomes of this study showed that the inclusion of the calibrated performance models has improved the decision process significantly. The PMS decision process has changed from selecting maintenance and rehabilitation actions based on a ranking process to a decision process based on optimization.

The information about the pavement structural capacity and sub- grade strength were found to be fundamental and it is needed for life cycle costing and pavement network management (Martin and Roberts, 1996). When a new or

rehabilitated pavement is designed , the pavement profile is based on the structural capacity of the pavement to be able to satisfactorily withstand the traffic loading over the design period . Other condition indicators such as pavement roughness are playing more minor role in influencing the pavement design.

Without the ability to continuously monitor and measure the characteristics of the structural capacity of the pavement , it seems to have been a stumbling block to incorporate the structural capacity of the pavement as a network level parameter for analysis. Also, without the ability to monitor and analyze the structural capacity of the pavement, there is obviously some limitations to selecting the most appropriate pavement rehabilitation option for the pavement section on network level. In this regard, It has been observed by many researchers in the field of pavement management that many of these systems do not incorporate any form of structural capacity evaluation at network level management.

In Australia, several condition data element are considered in the process of treatment selection. It is common practice that prior to making the decision to rehabilitation or place an overlay, several steps need to be undertaken such as (AUSTROAD, 1994):

- A thorough evaluation of the existing pavement encompassing both functional and structural aspects.
- Assessment of the adequacy of the existing pavement based on the structural and functional evaluation results together with an estimate of anticipated future traffic loading.
- Adoption of an appropriate strategy for rehabilitation based on the above mentioned aspects and parameters involving consideration of the alternative treatment.

The overlay design procedure requires a deflection survey to be undertaken on the existing pavement. The survey is of prime importance in assessing the structural integrity of the existing pavement and the extent to which it varies.

Pavement evaluation procedure adopted in the Australian pavement design guide is similar to the other well known rating procedure. It includes systematic procedure starting by dividing the road into a set of homogenous sections based on history , structure , subgrade strength and traffic. These information are obtained by detailed site inspection during which all surface defects are recorded to assess in the interpretation of the deflection analysis (AUSTROAD, 1987a).

Two methods of deflection measurement are commonly used in Australia. Data may obtained by means of the Benkelman Beam or Lacroix Deflectograph. A third method of deflection measurement which is the Falling Weight Deflectometer is also used considerably in Australia. Two pavement parameters reactions used for analysis which are (AUSTROAD, 1992):

- Maximum deflection: The maximum value recorded for each site.
- Deflection bowel: which is the shape of the deflected pavement surface caused by an applied load and it is measured directly with FWD device..

Structural data incorporation in a pavement management system has been investigated by David Pain (Paine 1998). Different forms of structural data has been analyzed to determine the most appropriate solution for the incorporation of such data into a pavement management system.

Many analytical approaches to evaluate the structural capacity measurements have been investigated. These include peak deflection readings, deflection and curvature readings, back analysis of deflection bowel and structural capacity indices.

<u>Pain pointed out that further investigation of these analytical approaches may enable a preferred approach to be adopted for use within a pavement</u>

management system that will enable the structural capacity component to be an integral tool in the treatment selection process. This argument has urged the researcher to tackle this aspect as it will be detailed in the subsequent chapters.

Information required to calculate structural number such as deflection measurements, pavement layers thickness have been recommended by many approaches (Table 1.1).

As shown in this table, Falling Weight Deflectometer data has been found the most recommended data element for an easy structural evaluation.

Table 1.1: Information required by various approaches to calculate structural number.

Approach	FWD Data	Total Pavement Thickness	Layer Thickness
AASHTO-1–back calculation	Yes	No	Yes
AASHTO NDT –2	Yes	Yes	No
DCP data	No	No	Yes
Jameson's	Yes	No	No
Howard's	Yes	Yes	no

Pain investigated the optimum way to use the structural data in pavement management system for the purpose of enhancing the decision making processes. Two functions of the structural data have been pointed out; the first is the ability to use this data in determining the pavement deterioration, the second is using this structural data to select the most suitable maintenance and rehabilitation option at network level.

Deterministic deterioration models were adopted in this study to analyze the structural data. These models, which have been used in the World Bank Highway Design and Maintenance Module (HDM3) published in 1992, are the most widely used deterioration models. These models use a structural index in

the form of the modified structural number as the component of structural capacity in pavement deterioration. The same models are used in HDM4 module after a considerable improvement that has been done to these models.

Calculating the structural capacity from Falling Weight Deflectometer-FWD have been found in other studies reasonable and practically acceptable (Pologruto 2001). Deflection data collected over 5 years time span was used to calculate the effective structural capacity and the relevant layer coefficient for sand a, aggregate and asphalt material in Vermont state.

It was found that estimating the Layer coefficients for sub- grade and unbound material based on the FWD testing were reasonable in comparison with AASHTO guide data. The layer coefficient for the asphalt was found to be up to 25% more than that obtained by AASHTO due to many reasons related to environment , location of testing and different material properties. This information can be used in both pavement design and evaluation to give accurate design and manifest the effective pavement structural capacity.

Incorporating of rutting measurements into pavement management system at both network and project level was also found to be helpful and meaningful (Hajek *et al*, 1998). The incorporation of this distress came as a result of the fact that rutting is an important pavement performance and pavement design parameter particularly for asphalt concrete pavements. Rutting depth was measured through a comprehensive research program initiated by the Ontario Ministry of Transport in 1998 using ARAN vehicle which utilizes a smart rut bar with 32 ultrasound sensors and provides an accuracy of about 1mm.

Rutting data or dual rutting was found to occur normally in heavily trafficked roads under each wheel track. The inclusion of this type of distress came as a result of being affecting the safety of the vehicle and the drivers. Severe rutting

can create steering difficulty for the drivers while moving laterally across the rut and it may form hydroplaning.

In this study, rut measurements were accomplished using ARAN vehicle which is based on acoustical technology. Rut depth can be measured by calculating the change in the vertical distance between the measuring vehicle and the pavement surface .This change is related to the change in time it takes the transducer – emitted ultrasound signal to reflect from the pavement surface and return back to the transducer.

Rut depth measurement using ARAN was done by determining the maximum rut depth in the left and right wheel path , calculating the average of these two values and reporting this as ARAN rut Depth. Rut depth measurements were spaced longitudinally at 4-m intervals. Then it is averaged for each 20m or 100 or for any desired section length.

Rut data were found to be useful at *project level*. It can be used for :

- Determination of the exact locations of pavement section requiring remedial treatment.
- Determination of material padding and mill quantities required to restore the pavement cross fall to a correct level as a part of the pre-construction surveys.
- Evaluation of the effectiveness of various pavement maintenance and rehabilitation treatments such as micro surfacing and cold in-place recycling.
- Investigations of claims involving construction work and traffic accidents.

Rut measurement at *network level* can be used to :

- Assess the pavement serviceability.
- Monitor the progression of rutting

- Schedule and plan rehabilitation treatments.

A study to develop simplified and rationalized procedure for interpretation of deflection data has been carried out in Canada (Stolle *et al* ,1995). A simple elasostatic approach to estimate a suitable subgrade modulus from FWD data was developed. Boussinesq-Odemark equations are used to define the effective subgrade modulus. Equivalent thickness of the pavement structure , which reflects the pavement stiffness, is estimated by fitting pre-selected normalized deflection. Given the equivalent thickness, an effective subgrade modulus may be determined. The approach used to determine the subgrade moduli is as follows:

- Estimate the seed moduli via some approximation strategy
- Predict a deflection basin using estimated moduli
- Compare predicted and the measured deflection basins
- Adjust layer moduli through a search techniques (trial and error) to reduce the difference between measured and predicted displacement.
- Repeat steps 2 to 4 until the error between the two deflection basins is within an allowable tolerance.

Temperature adjustment was found the most critical because asphalt concrete temperature can change significantly during the day.

Finally, it was concluded that due to the fact that continuum concepts are no longer applicable to the cracked pavement, it is suggested to use the concept of the effective pavement thickness to obtain better measure of the pavement structural integrity.

Using Roughness measurement for pavement management in South Africa was investigated by Visser and his colleagues.(Visser *et al*, 1998). The study aimed at presenting a number of issues related to the routine measurements of road

roughness in south Africa. The findings of this study showed that calibrated road sections should be used for all types of roughness measuring devices including profiling system. It was found that there can be 20% variation in roughness on roads with an IRI less than 2m/km. Rod and level measurement method was found to be subjected to a number of errors.

It was also found that, control measurements on control section are needed for the purpose of stabilizing the measuring system over time. Also, Care should be taken in relating the new roughness measuring systems to other old systems.

A smoothness model for hot mix asphalt –surfaced pavement has been developed based on the surface distresses and other pavement characteristics (VonQuintis *et al* 2001). The study investigated the relationship between the roughness condition indicator in terms of IRI and many distress types and pavement age. Pavement age, Longitudinal cracking , fatigue cracking and variation in rutting along the pavement were all found to have detrimental effect on the roughness progression value. Also, for the newly constructed or newly overlaid pavement , the initial roughness was found to have a significant influence on the roughness level performance.

The issue raised by the above mentioned study will be discussed thoroughly in the thesis chapters pointing out the results of the field applications carried out on Dubai road network .Paying attention to this problem came as a result of the increased importance to use roughness testing as quality measure for the new roads. Till present time, many road agencies ignore the initial roughness testing and do concentrate only on the traditional lab testing for the asphalt mix as a quality assurance. The effect of the initial roughness value on pavement performance will be studied in this research effort based on data extracted from the field.

Skid resistance information was found to be used as a management tool to help prioritize pavement maintenance and rehabilitation and to select the appropriate maintenance and rehabilitation alternative (Shahin, 1998). Also, such information is needed to prevent or reduce accidents. The data are used to identify the pavement sections with low or rapidly deteriorating level of skid resistance.

Skid resistance is an important pavement evaluation parameter because inadequate skid resistance will lead to higher incidences of skid related accidents. Most agencies have an obligation to provide users with a roadway that is "reasonably" safe. Skid resistance measurements can be used to evaluate various types of materials and construction practices.

Skid resistance of a pavement surface is influenced by both microtexture and macrotexture. Microtexture is generally controlled by the selection of the aggregate type. It refers to the fine-scale texture gritting that is present on the surface of the coarse aggregate and depends on the initial roughness and the ability of the aggregate to retain this surface roughness under traffic. This means that macrotexture is the pavement aggregate component (which controls contact between the tire rubber and the pavement surface). Macrotexture refers to the large-scale texture of the pavement as a whole due to the aggregate particle arrangement. (Jayawickrama *et al*,1996)

Skid resistance varies with time. Typically it increases in the first two years following pavement construction as the bitumen worn away by traffic and rough aggregate surfaces become exposed, then decreases over the remaining pavement life as aggregates become more polished (Anderson *et al*,1986). Seasonal variation was also reported in this study to have detrimental effect on the skid resistance value, i.e. skid resistance rises in Fall and Winter and falls during late Spring and Summer.

Other studies indicated that there is no reduction in skid resistance measure by friction number after the first two years of service. In one of these studies, many sites have been tested several times and no reduction in skid resistance value was noticed (Skerritt *et al* , 1993).

In a study conducted to investigate the effect of friction and surface texture characterization on friction performance(Judith *et al*, 1998), a detailed testing program has been conducted at speed of 64km/hour over 14 pavement test sections in Greenville, north Carolina. It was found that the best friction performance provided by the heavy-duty surface course and the poorest friction performance was provided by the large stone surface course and stone mastic with fibers and stone mastic with polymer. None of the tested 14 sections had an average friction number less than 40 even when tested at 80km/hour.

Common methods of pavement condition rating

In the last century, many methods for pavement evaluation, distress rating and calculation of performance indices have been developed. The development of such methods aimed at establishing an objective and practical pavement condition rating system that includes the actual quantities of the distresses and represents the actual functional and structural condition of the pavement. An important feature of a PMS is the ability both to determine the current condition of a pavement section and to predict the its future condition (Shahin, 1998).

To predict the pavement condition reliably, an objective, repeatable, validated and field-tested rating system for identifying the pavement condition must be used. Among the most common pavement distress rating procedures that have been developed and widely used around the world are the followings:

1- AASHO Present Serviceability Rating
2- Pavement Condition Index -PAVER System.
3- Ministry of Transport of Ontario (MTO) -Canada Methods:

- Method 1
- Method 2

4- Asphalt Institute Method

A brief description of the aforesaid rating procedure is introduced in the following sections:

AASHO Present Serviceability Rating

In the sixtieth's of this century, the concept of Present Serviceability Index (PSI) as an indicative measure of the pavement condition was introduced by Cary and Irick. (Carey and Irick 1975). This quantity has also been incorporated as main design input into the American Association of State Highway Officials –AASHTO design equations for both flexible and rigid pavements. PSI is measured on a scale range from 0-5 with 5 being an excellent. The model that combines these factors is represented in equations 1.1 below:

$$\mathbf{PSI= 5.03-1.91\ \log_{10}(1+SV)-1.38\ RD^2-0.01(C+P)^{0.5}} \quad \text{----------1.1}$$

Where:

SV=Slope Variance %

RD=Rut Depth in inches

C=Density of Cracking (ft^2/1000 ft^2)

P=Density of Patching (ft^2/1000 ft^2)

This concept has been reviewed and argued by other researchers and different approach based on the relevant psychophysical law which describes the human response to the external stimuli proposed by Fechner was introduced (Lui and Herman, 1996, Fechner, 1966).

Using the same source data used to developed the above equation, and varying the principles of the analysis for incorporating the patching, cracking and slope variance, a new equation for the flexible pavement was obtained as follows:

$PSI= 5.0-1.68 \log_{10} (ś/0.71)-1.38 \log ř/(6.1*10^{3)}-0.00871 \log(C/7*10^{-3})$ ---1.2

Where:

ś =Slope Variance %

ř =Mean Rut Depth in inches

C =Density of Cracking ($ft^2/1000\ ft^2$)

The threshold of both ś and ř and c are obtained by forcing the intercept b_o to be 5 such that the maximum R^2 is obtained. Similar equation for rigid pavement using the same approach was also developed.

PSI is one of the most used measures of pavement functional performance This measure was developed through AASHO road test.

As shown in Figure 1.7, this measure is subjective and it is a function of surface characteristics and distresses such as slope variance rutting, patching and cracking areas. Structural performance is a measure of the pavement ability to resist traffic loadings resulted from different types of vehicles.

5	**Very Good**	**Rater:----------**
4	**Good**	**Highway No:----------**
3	**Fair**	**Section No:----------**
2	**Poor**	**Date:----------**
1		**Is pavement of acceptable quality**
		Yes……..

0	Very Poor	No……	
		Undecided…	

Remarks----------

Figure 1.7: Present Serviceability Rating Form

It can be measured visually by distress survey for load- associated defects such as alligator cracking and rutting or by using the non destructive testing such as Falling Weight Deflectometer.

Pavement Condition Index- PAVER System

Pavement Condition Index (PCI) rating system was developed by US Army Corps of Engineers for the benefit of US Air Force at the late eightieth's of the last century. The use of PCI for airfields pavements , roads and parking lots has received wide acceptance and has been formally adopted as a standard procedure by many agencies worldwide. These agencies include Federal Aviation Administration, the U S Department of Defense and the American Public Works Association.

The PCI for airfield has been published as an ASTM test method. The PCI is an objective rating method based on the deduct values-weighing factors from 0 to 100 that indicate the impact of each distress has on pavement condition. It is a numerical index that uses a scale from 0 to 100. Figure 1.8 shows a schematic illustration for the PCI calculation steps and the PCI rating Scale (Shahin *et al* 1990).

Calculation of PCI is based on the results of visual condition survey in which distress types, severity and quantity are identified. It includes a detailed description for 19 distress types at different severities as shown in the asphalt a pavement field inspection sheet (Figure 1.9). The distress severities are of three levels i.e. High, Medium and Low.

PCI method concentrates on the visual inspection and the accurate measurements of the distress quantities. The functional and structural aspects of the pavement are evaluated through accounting for certain distress types.

Functional related distress types such as depressions, pumps and sags, and swelling can be used to estimate the functional characteristics and the riding quality. There is no direct measurement of the pavement riding quality in this method.

Similarly, the structural capacity of the pavement is evaluated by observing certain distress types, quantity and severity. Alligator cracking and rutting are the main structural capacity associated distress types that can be used to evaluate the structural adequacy of the pavement section.

Some limitations and shortcomings in this method as the maintenance decisions are based on a single index (PCI) which in certain most cases does not give an adequate representation of pavement condition and improper selection of the rehabilitation schemes.

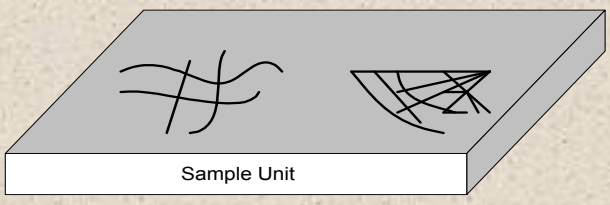
These shortcoming and drawbacks impose limitations on the benefits gained and imparts a duty to carry out other measurements to evaluate the functional and structural capacity levels objectively.

PCI CALCULATION STEPS FOR A SAMPLE UNIT

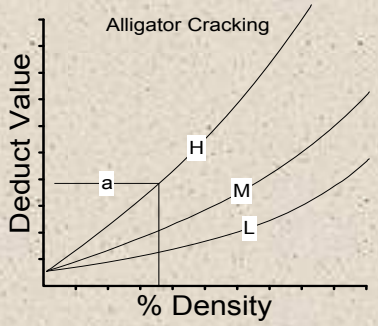
Step 1: Divide pavement network into sections and sample units

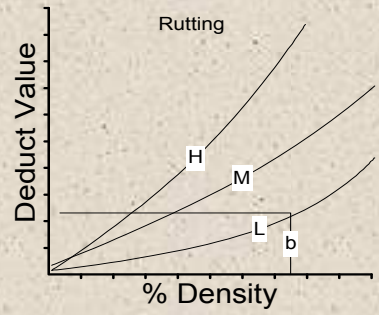
Step 2 : Inspect each sample unit

Step 3 : Determine distress type , Severity and quantity



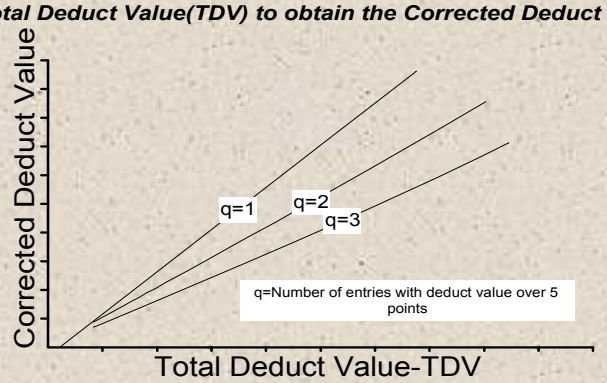
Step 4: Determine the corresponding deduct values for each distress type, severity and density





Step 5 : Calculate Total Deduct Value(TDV) =a+b

Step 6 : Adjust Total Deduct Value(TDV) to obtain the Corrected Deduct Value(CDV)



Step 7: Compute pavement Condition Index (PCI) for each sample unit as PCI=100-CDV

Step 8: Compute pavement Condition Index (PCI) for the entire section =Sum PCI/n

Step 9: Determine pavement condition rating of the section from condition bar below

100 (Excellent)	**85 (V. Good)**	**70 (Good)**	**55 (Fair)**	**40 (V.Poor)**	**25 (Poor)**	**10 (Failed)**	**0**

Figure 1.8: PCI calculation steps proposed by PAVER System (Shahin 1998)

Asphalt Pavement Inspection Sheet	
Branch:	Section:
Date:	Sample:
Surveyed By:	Area of the sample:

Distress Types		**Sketch**
1-Alligator cracking	11-Patcching and utility cut patching	
2- Bleeding	12-Polished Aggregate	
3-Block Cracking	13-Potholes	
4-Bumps and sags	14-Rail Road Crossing	
5-Corrugation	15-Rutting	
6-Depression	16-Shoving	
7- Edge Cracking	17-Slippage Cracking	
8-Jt Reflection Cracking	18-Swell	
9-Lane Shoulder Drop Off	19-Wethering &Raveling	
10-Long. & and Trans. Cracking		

		Existing Distress Types					
Total Severity	L						
	M						
	H						

PCI Calculation						
Distress Type	Density	Severity	Deduct value			
				PCI=100-CDV=		
				Rating=----------		
Deduct Total			q=			
Corrected Deduct Value(CDV)						

0	10	25	40	55	70	85	100	PCI Value
Failed	V. Poor	Poor	Fair	Good	V. Good	Excellent		Rating

Figure 1.9: Asphalt Pavement Inspection Sheet proposed by PAVER System after Shahin 1998.

Ministry of Transport of Ontario (MTO) -Canada Methods

Method No. 1

This method has been developed by the Canadian Ministry of Transport of Ontario. It is a system that includes two parts of evaluation; the first is Pavement Condition Rating on five levels range from “Very Slight” to “Very Severe” evaluated concurrently with the distress density which is also evaluated on a scale range from Few (10% density) to Throughout (100% density) based on the distress type, quantity and quality. The second is the Riding Condition Rating which is divided into five categories range from very poor to excellent corresponding to a scale range from 0 to 10 with 10 being very smooth. Figure 1.10 shows the evaluation form and the information needed to be collected.

In this method, the rating index is calculated based on the distress type and the extent. The distresses are divided into four groups as summarized in the inspection form below. Similar inspection form for rigid pavement has been also developed.

Method No. 2

In 1996, and in order to help Canadian Municipalities to determine the condition of their roads using standardized procedure, the following Pavement Condition Index (PCI) method proposed by the Ontario Good Roads association (OGRA) was selected. The method is similar to the above described method used by MTO to rate the provincial highway system and a method recommended by the Asphalt Institute to evaluate pavement condition. One variation to note is that the provincial method of rating highways uses one (1) objective method of gathering data for *ride comfort* and one (1) subjective method of gathering data for distress manifestations. The new method shown below uses two (2) subjective methods to gather data. With this method, there is also a need to establish a weighting for each type of distress evaluated.

The establishment of an appropriate weighting is somewhat subjective in that the weighting should reflect the effect the distress will have on the structural capacity of the pavement. Figure 1.11 shows the Modified flexible pavement condition evaluation form developed by Ontario Good Roads Association (OGRA) for the benefit of MTO-Canada after 1996.

Riding Comfort Rating (at 80km/h)	Excellent	Fair	Poor	V. Poor

Pavement Distress Manifestation			Severity of Pavement Distress					Density of Pavement Distress (Extent of Occurrence)					Characteristics of Pavement Distress								
			1	2	3	4	5	6	7	8	9	10		Pavement Edge Crack						Alligator Cracking	
			Very Slight	Slight	Moderate	Sever	Very Sever	10% (Few)	10-20% (intermittent)	20-50 % (Frequent)	50-80% (Extensive)	80-100 % (Throughout)	Reflection Cracking	Progressive(12" from edge(0.3m		Non progressive 12 " (0.3m)		Trans. Crack Spacing (mm)		Alligator Block Size mm	Distortion of Alligator mm
Surface Defects	Coarse Aggregate Loss																				
	Raveling																				
	Flushing																				
Surface Deformation	Rippling																				
	Shoving																				
	Wheel track Rutting																				
	Distortion																				
Cracking	Long. Wheel track	Single																			
		Multiple																			
		Alligator																			
	Midland	Single																			
		Multiple																			
	Center line	Single																			
		Multiple																			
		Alligator																			
	Meandering	Single																			
		Multiple																			
	Pavement Edge	Single																			
		Multiple																			
		Alligator																			
	Transverse	Parallel																			
		Hall																			
		Full																			
		Multiple																			
		Alligator																			
	26 Random																				
	27 Slippage																				
	28																				
	29																				

Maintenance Patching	Spray																					
	Skin																					
	Hot Mix																					
Additional Remark																						

Figure 1.10: Flexible pavement condition evaluation form developed by MTO-Canada

FLEXIBLE PAVEMENT CONDITION EVALUATION FORM

Survey Date: _____ Evaluated by: _____

Road (Street) Name: _____ Section Length _____km

Location: from_____to _____

Class of Road (circle both mtce. class & functional class) 1 2 3 4 5 6 freeway arterial collector local

Section # (if applicable) _____

Ride Comfort Rating
(at posted speed)

10 9 8 7 6 5 4
3 2 1

Very Good Good Fair Poor Very Poor

					Severity of Distress (Si)					Density of Distress (Di)				
					Very Slight	Slight	Moderate	Severe	Very Severe	Few <10	Intermittent 10-20	Frequent 20-40	Extensive 40-80	Throughout >80
Pavement				Wi	0.25	.5	1	1.5	2	0.25	.5	1	1.5	2
Surface Defects		Raveling & loss of surface aggregate	1	1.5										
		Flushing	2	0.5										
Surface Deformations		Rippling and Shoving	3	1.0										
		Wheel Track Rutting	4	3.0										
		Distortion	5	3.0										
Cracking	Longitudinal Wheel Track	Single and Multiple	6	1.0										
		Alligator	7	3.0										
	Centerline	Single and Multiple	8	0.5										
		Alligator	9	2.0										
	Pavement Edge	Single and Multiple	10	0.5										
		Alligator	11	1.5										
	Transverse	Half, full and multiple	12	1.0										
		Alligator	13	3.0										

	Longitudinal – meander or mid-lane	14	1.0										
	DMI =∑ Wi x (Si + Di) (for all 14 distresses) = ……												
	PCI = 100 – (DMI + (10 – RCR)) =……												

Figure 1.11: Modified Flexible pavement condition evaluation form developed by Ontario Good Roads Association (OGRA) for the benefit of MTO-Canada after 1996.

This method uses different condition indices defined as follows:

PCI (Pavement Condition Index)

- Is a single performance indicator comprising two components: (1) a roughness component in the form of Ride Comfort Rating (RCR), and (2) a distress component in the form of Distress Manifestation Index (DMI).
- Currently used on pavements with an asphalt concrete surfaces

RCR (Ride Comfort Rating)

- The roughness component can be measured using one of two methods:
    - A subjective ride quality as perceived by the traveling public using the Ride Comfort Rating shown below, or
    - Objectively measured using various mechanical/electronic devices, such as PURD (Portable Universal Roughness Device) or the Mays Meter.

RCR is a rating of ride comfort and for the subjective method of measurement the rating is determined from a drive through of the section at posted speed and assigning a rating based on the scale shown in the chart below.

DMI (Distress Manifestation Index)

- A systematic method for classifying and assessing the visible consequences of various distress mechanisms.
- DMI classifies distress manifestations into 14 categories, which are rated by severity and density.

DMI is calculated by combining the density and severity of all distresses using the following formula:

$$\mathbf{DMI = \sum (of\ all\ distresses)\ Wi \times (Si + Di)} \qquad \text{....1.3}$$

where:

i = one of the following 14 distresses

Wi = Weight of distress i as follows:

Di = Density of Distress

Si = Severity of Distress

The PCI rating is a combination of the RCR and DMI. The RCR can either be gathered through a subjective method (drive through at posted speed) or an objective method (PURD or Mays Meter). The scale used for PCI ranges from zero (worst) to 100 (best). The PCI is calculated by combining the RCR and DMI using the following formula:

$$\mathbf{PCI = 100 - (DMI + (10 - RCR))} \qquad \text{.....1.4}$$

Subsequently the PCI rating must be turned into a decision or action, therefore, according to certain decision matrix designed for this purpose.

Distress surveys are often required as part of the planning and design of pavement rehabilitation projects. These surveys gather information on the various distress types, their location, severity and the extent to which they exist. The accuracy of the survey is dependant upon the consistency of the data gathered.

Trained observers are required to evaluate the pavement condition, their observations should relate to the actions required to repair the distress.

Asphalt Institute Method

The asphalt institute has developed a simple pavement condition rating for low volume asphalt roads (Figure 1.12). Descriptions and photographs help the rater to identify the different types of distress, in addition to giving advice on appropriate corrective actions are provided.

Asphalt Pavement Rating Form (Asphalt Institute)			
Street or Route		City or County	
Length of the project		Width	
Pavement Type		Date	
Note: A rating of 0 indicates defects does not occur			
Defect……...		Weight Mark	Rating
Transverse		0-5	
Carks…….		0-5	
Longitudinal Cracks……		0-10	
Alligator Cracks……..		0-5	
Shrinkage		0-10	
Cracks…….		0-5	
Rutting…….		0-5	
Corrugation…….		0-10	
Raveling…….		0-10	
Shoving or Pushing……		0-5	
Pot holes……		0-5	
Excess Asphalt……..		0-10	
Polished Aggregate……			
Deficient Drainage…….			
Overall Riding Quality (0 is excellent and 10 is very poor)		0-10	
		Sum of Defects	
Condition Rating =100 – sum of defects =100 - ----------			

Condition Rating=		

Figure 1.12: Asphalt pavement rating form for low volume roads developed by the Asphalt Institute.

The system also includes a scale shown in Figure 1.13, which can serve as a guide to when overlays are appropriate or when routine maintenance or complete reconstruction are more applicable (MS-17, 1983).

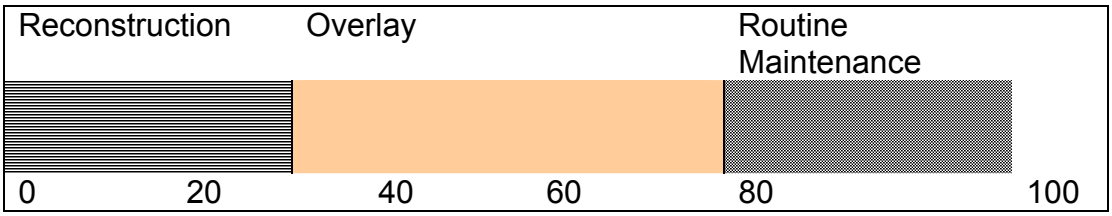


Figure 1.13: Condition rating scale proposed by the Asphalt Institute (MS-17,1983).

The problem and expectations

Despite the significant progress made in the past three decades towards a scientific understanding of the processes and phenomena related to pavement deterioration and performance, it is freely admitted that a considerable gap in engineering knowledge will continue to persist well into the 21st century.

Pavement performance is controlled by many factors. In fact, understanding pavement performance is a complex mission. Many studies have been and are still being conducting to answer a simple twofold question; ***How pavement performs and Why it performs in this manner***? This big question took the road agencies tens of years of research and investigations to answer and still no remedy answers have been found yet. This situation is true due to the fact that pavement is not a homogenous system or material that behaves in the same way at different locations. What is applied in the USA may not be found fully applicable in Australia or Europe. It is a complicated issue that will take the whole next century to find the proper solution if there is any.

To help answering this question, a Long Term Pavement Performance Program-LTPP has been established in the year 1987 in the USA. This program has been designed to last for 20 years period and aimed at studying the in -service pavement by monitoring more than 2400 asphalt and portland cement concrete pavement test sections across the United States and Canada. Several other research programs initiated elsewhere to help in understanding the pavement performance some of these are still going on to the recent days (US DT and FWHA 1998).

In the field of pavement condition rating, many experts and engineers believes that present serviceability PSR or PSI is not sufficient for deciding whether or not the pavement require overlay (MS-17, 1983). Therefore, several agencies have decided to develop systems for combining measures of pavement distress, in some cases including pavement roughness into a single condition distress to be used for this purpose. This distress index is desirable to be used with an automated pavement management systems.

Using only visual distress data collected from the periodic surveys of the roads network is not also sufficient to evaluate the actual pavement condition functionally and structurally. Therefore, other condition indicators must be integrated with the distress data to improve the condition assessment accuracy and enhance treatment selection process. Without incorporating all of the pavement condition indicators into the pavement management systems, there will be obviously many limitations and deficiencies in selecting the optimum maintenance and rehabilitation alternative of pavement section at network level, setting priorities, and selecting the optimal time for carrying out the required repair. The successful utilization of visual pavement condition data , roughness (rideability) data, structural capacity data and other pavement condition indicators will enable the engineers to evaluate the pavement performance (i.e. pavement deterioration over time) and predicting the future condition in much more effective manner. It will provide the pavement management systems with

the capability to be used effectively as a planning tool in the hands of the maintenance engineers and the decision makers.

The problem in this regard stems from the fact that roads agencies uses vast number of management systems that needs different inputs and generate widely variable outputs. These inputs related to various condition indicators. Quantifying or documenting how these inputs impact each condition indictor and pavement performance is a challenging endeavor since many variables must be controlled to isolate the influence of any given factor.

This research effort aims at investigating the optimum way to use parameters related to main condition indicators and how to combine different types of deteriorations into a user defined condition rating index that uses the outputs of different pavement condition indicators with the relevant assigned weight /importunacy levels.
This will include parameters and variables related to structural capacity (in terms of structural number, elastic modulus values , deflection, curvature , or structural remaining life or whatever might be found useful), roughness data (in terms of the international roughness index–IRI or the average roughness index –ARI, and roughness- associated distress types), materials properties (compaction, material strength , effective modulus of elasticity) , skid resistance (in terms of skidding number and skidding associated distress types) and all other pavement properties that may affect the pavement condition and performance.

The development of such an index will reduce the cost of maintenance by assigning the suitable treatment option that will take care of both functional and structural deficiencies, setting their priorities and calculating the life cycle costing. It will also provides a profound advancement towards having theoretically sound and well established, field-tested and validated procedure which can be effectively used in pavement management systems for the purpose of evaluating, managing and maintaining the pavement network .

Road Network in Dubai Emirate (Study Area)

The database available in the PMS of Dubai was used to extract the historical records for the sections under study. These information are important for the development of the prediction models and maintenance and the verification of rehabilitation schemes and to prove some theories and procedures envisaged by the researcher. The road network in Dubai experienced an accelerated rate of growth in the last two decades to provide a high level of service to the road users and to cope with the rapid development in economic and industrial activities.

As shown in Figure 1.14, the total length of the road network according to year 2006 statistics is approximately 2900 km (about 9000 lane-km) of which more than 45% are major roads (i.e. Freeways, Expressways and Arterials) (PMS Unit (2006). The cumulative expenditures on road construction reached to about Dhs8800 millions ($2260 millions) as per year 2003 statistics (Figure 1.15).

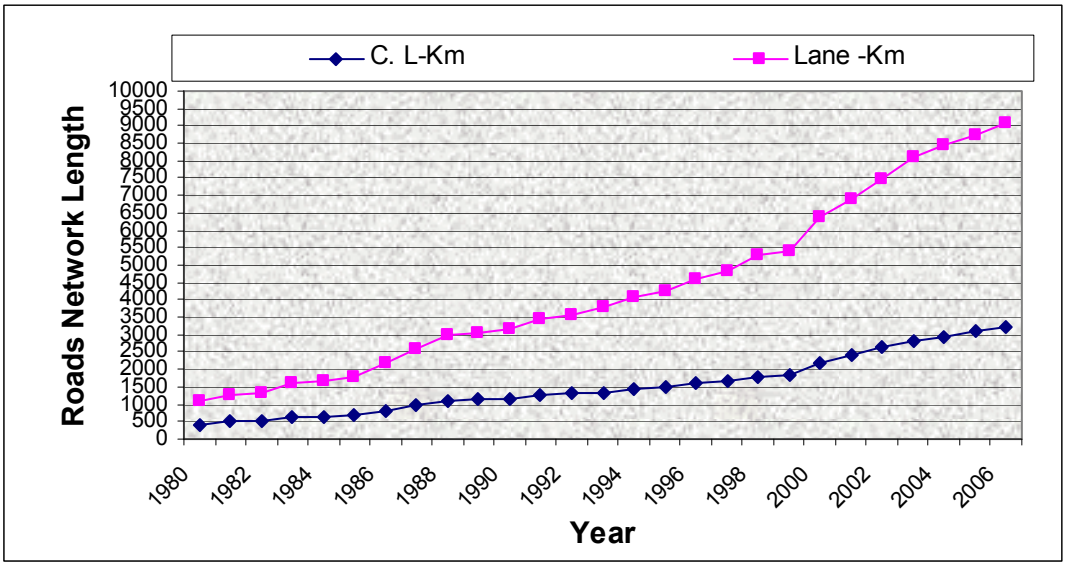


Figure 1.14: Roads network development in Dubai (1980-2006)

This investment necessitated more effort and attention to the development of integrated cost effective and practical M & R schemes, using the results of

different pavement evaluation procedures. Most of the roads are asphalt surfaced pavement and no rigid pavement is used in this area. The distribution of the pavement condition in the study area is shown in Figure 1.16.

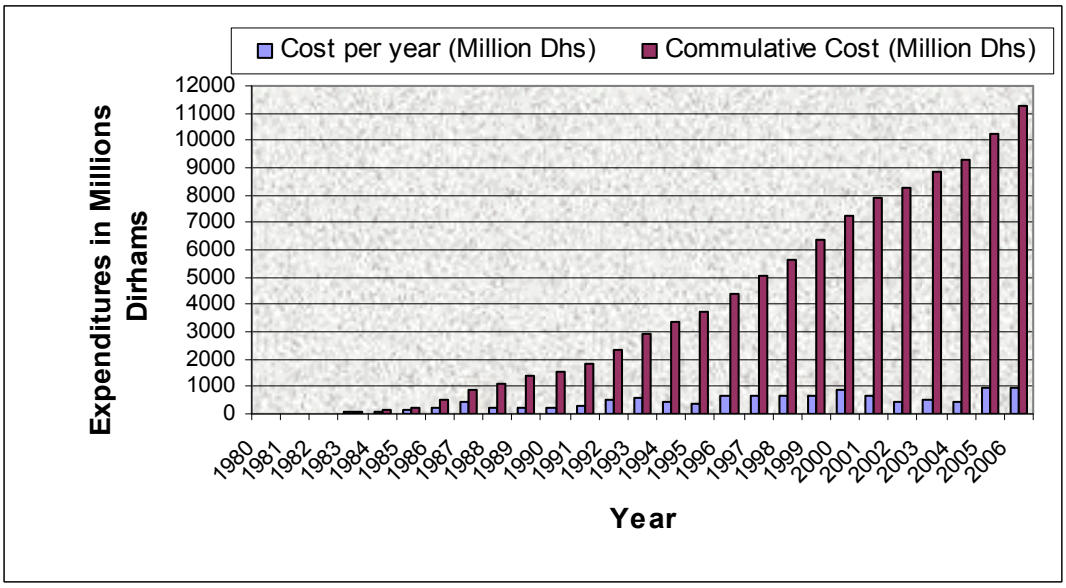


Figure 1.15: Cumulative expenditures on roads construction in Dubai (1983-2006)

It is clear that most of the roads are in excellent condition as more than 50% of the roads are recently built and also, as result of the effective use of PMS in monitoring, evaluating and maintaining the roads network .

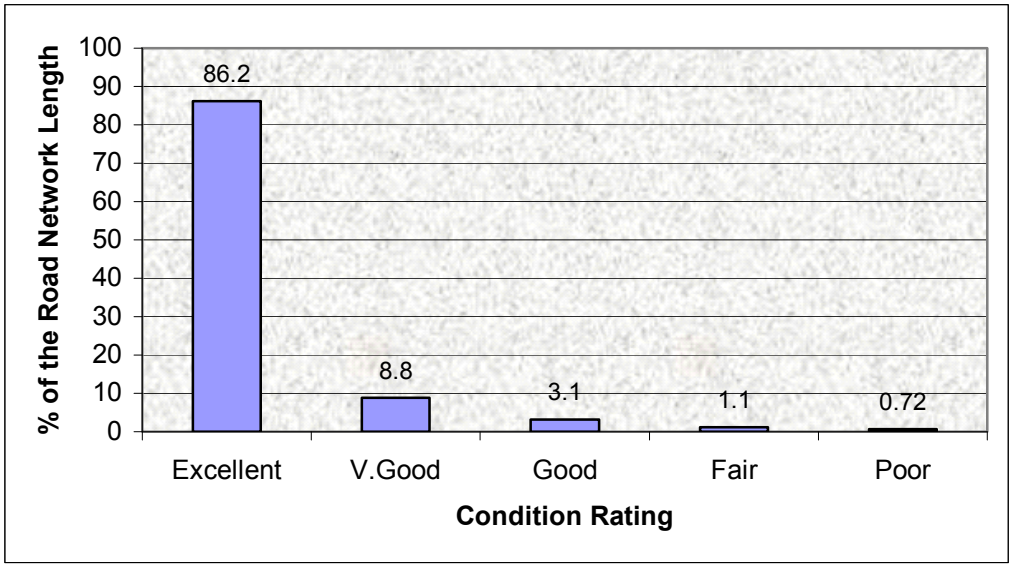


Figure 1.16: Distribution of Dubai Roads Network condition for the year 2006

Research Objectives

Based on the practical experience of the researcher, and the issues raised by other researchers as indicted in the aforesaid literature review, it was noticed that road agencies use condition indicators in different ways and forms to report about pavement condition at network level. Some of the parameters used may be considered misleading if they are used to compare networks performance from region to another. Also, all pavement condition indices used to evaluate the pavement condition suffer from shortcomings and do not satisfy the practical needs of the maintenance agencies. Even at network level, the given index is normally general and sometimes does not represent the actual condition of the pavement.

Since the pavement management system is considered as a planning tool for the maintenance departments, it should provide all necessary information accurately at network level, and to certain extent, at project level. Therefore, this research effort will be directed towards achieving the following main objectives:

- To assess the vital rule of pavement management systems, highlight the concept of pavement deterioration and to study the factors associated with its performance at network level.
- To identify the main pavement condition indicators and the associated parameters that govern pavement structural and functional performance.
- To determine the representative variables pertaining to each condition indicator that can be used effectively in the pavement management system
- To establish conversion rules and weights pertaining to each pavement condition indicator and propose suitable scales for easy use within the pavement management system.

- To investigate the current available models that predict pavement remaining life and to develop sound and predictable statistical deterministic models that simulate the deterioration trends for each pavement condition indicator.
- To determine the main predictors for each condition indicator and investigate the liability of using these predictors to report accurately about the structural and functional capacities of the pavement at network level and to estimate budget.
- To develop a new performance index that incorporates the main condition indicators (surface defects, roughness, deflection and skid resistance) and establish the weight assigned to each indicator based on the relative impact of each indicator on the overall pavement performance.
- To develop the consolidated pavement condition plot that depicts the overall condition of the pavement section which can be used to help managers to make informed decisions regarding maintenance and rehabilitation requirements based on sound and precise engineering analyses outputs.
- To develop rehabilitation schemes and design the decision trees that depend not only on the condition index, as a single figure, but to incorporate other factors such as distress types, distress density, effective structural capacity, roughness level, roads class, location, surface type, failure mode, etc.
- Finally, to investigate the credibility of the new proposed procedures, indices and models to report about pavement condition, optimize the treatment selection process, prepare work programs and estimate the annual maintenance and rehabilitation budgets.

CHAPTER 2 RESEARCH METHODOLOGY, DATA COLLECTION AND ANALYSIS PROCEDURE

Theoretical Concept

In the last three decades, Roads agencies and researchers have dedicated considerable amount of time and efforts to develop a well established, field tested procedures for pavement evaluation and decision making that includes all factors affecting pavement performance.

Despite the fact that the basic engineering techniques are available and well established, they have hardly been used in a form that helps in a pavement management context.

Fortunately, This situation is continuously changing as there is considerable amount of a worldwide research being carried out by individuals and road agencies to investigate the optimum way for using these principles in pavement evaluation and management.

These efforts contributed to better understand the evaluation, performance and rehabilitation procedures of the pavement.

Pavement performance represents the pavement deterioration over time under the prevailing traffic and environmental loading. It is controlled by many parameters or measures called condition indicators. These indicators describe the functional and structural performance of the pavement .

In principal, Pavement condition deterioration can be controlled mainly by six measures, based on which the overall pavement performance curve can be evaluated and the repair option can be optimized. These indicators are shown in Figure 2.1 below:

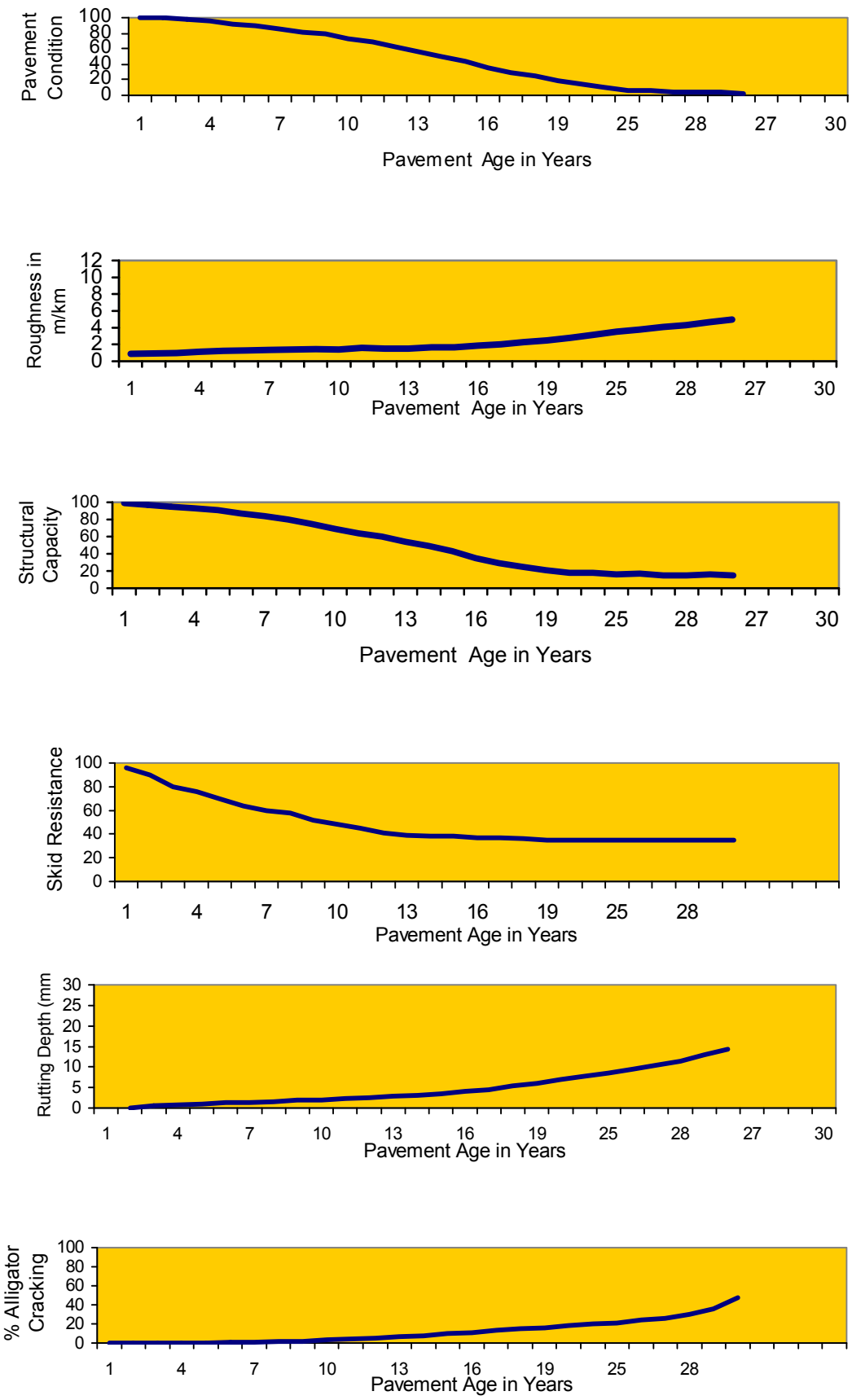


Figure 0.1: Pavement condition deterioration curves (Performance parameters)

In brief, These measures can be defined as follows:

- ***Physical condition (Surface Distress) deterioration:*** The existence of various distress types in a pavement is a clear manifestation of the inherent inadequacies of the pavement system. Therefore, distress surveys are usually carried out in order to describe the physical condition of the pavement and to document the existing distress types and defects. Surface inspection can also be used to describe quantitatively the existing condition in an engineering fashion and indicates the required action to be done.

  Researchers were able to identify around 19 distress types in the flexible pavement caused by various factors such as traffic , environment, mix deficiencies, improper constructions methods and others (Shahin *et al*,1990) .

  Distress evaluation can be performed by many forms. Each agency can design or adopt their own rating procedure based on the prevailing local traffic and environmental loading characteristics. Most of the road agencies in the developed countries are shifting, nowadays, towards the development of an objective rating system, preferably automated. In other countries, manual procedure are still much more preferred due to the availability of the skilled labors of low costing.

  The pavement condition rating method used in this study is an objective method developed solely for Dubai Emirate. This It is a combination of PAVER system and a newly developed pavement condition index developed by Texas Research and Development incorporated (TRDI). This rating system was calibrated by the researcher to suit the prevailing traffic and environmental conditions of Dubai Emirate. Full description of this method will be presented in the next chapters.

- ***Roughness deterioration***: This indicator can be expressed by various means such as Slopes variance(SV), Present Serviceability Index(PSI), Bump Integrator Number(BIN), and lately by International Roughness Index(IRI). Roughness as a factor is a measure of serviceability performance. It can be measured also subjectively by identifying certain distress types and the change in severity level and quantity overtime.

  The IRI was an outcome of the roads roughness experiment conducted in Brazil in 1989 and sponsored by the World Bank (Sayers et al, 1986). The IRI indicator is adopted worldwide as the most representative functional condition deterioration indicator. The International Standards Organization (ISO) has developed a standard for reporting road profile measurements(ISO,1995).

  ***Structural deterioration*:** This indicator can be expressed in various forms. It may be expressed in terms of change in structural number, loss of thickness, change in modulus of elasticity. Environmental effect may cause reduction in the stiffness level of the materials and surface wear caused by the studded tires may reduce the thickness of the pavement surface layer. The structural number or the thickness of the standard materials needed to restore the overall pavement stiffness is used as an indicator of structural deterioration.

- ***Skid Resistance Deterioration***: It is a parameter that measures pavement operational safety. It can be evaluated by skid resistance change or any measure of the tire-pavement friction characteristics. Skid resistance is defined as the force developed between the pavement and the tire when a tire, prevented from rotating, slides along the pavement surface.

  Skid resistance depends on many factors such as pavement surface micro and macro texture (Aggregate properties indicated mainly by the Stone Polished Value (SPV) , water film thickness, tire type and water

pressure etc. Skid resistance can be measured by different techniques such as:

- Mu-Meter-Yaw mode method
- British portable tester
- Swedish roads research skid meter-Slip mode method
- Locked-Wheel method-ASTM specification.

Recently, non contact technology tools, such as laser or infrared based machines, started to be used in measuring the surface friction characteristics by measuring the surface macro texture or the mean texture depth (MTD) of pavement surface.

- ***Alligator or Fatigue Cracking***: It is an indicator of the fatigue failure mode of the asphalt layer. It is manifested by the occurrence of cracking at the pavement surface as a result of repeated traffic loadings. Percentage of the cracked area or the cracking index may be used as a measure of the cracking contribution to the overall pavement condition deterioration.

  Rutting: It is an indicator of the pavement permanent deformation in the pavement layers as a result of heavy axle and traffic loading and the weak resistance of the pavement layers. Percentage of the rutted area, rut depth or the rutting index may be used as a measure of the rutting

The most recent methods for the evaluation of pavement condition indicators and the preferred parameters or forms to be used in pavement management system will be presented in the coming chapters.

The existence of alligator and rutting defects have been found to have a detrimental effect on pavement condition deterioration. The remaining four condition indicators have been studied by different agencies and individuals and different outputs and results were introduced Despite all the accelerated developments in the field of road network managements, still, no full agreement on what inputs are to be incorporated and how to use the outputs

of these indicators to report about pavement condition ,to estimate the budget and to select the optimum treatment.

Flexible Pavements Design and Performance

Conventional flexible pavements are layered systems with better material on the top where the intensity of the stress is high and inferior materials at the bottom where the intensity of the stress is low.

Asphalt-surfaced pavement consists of asphalt mixture layer , granular base layer and subbase layer which rests directly on the subgrade soil. As shown in Figure 2.2, the layers of the pavement distribute the loads throughout a layered system with progressively stronger layer at the top surface.

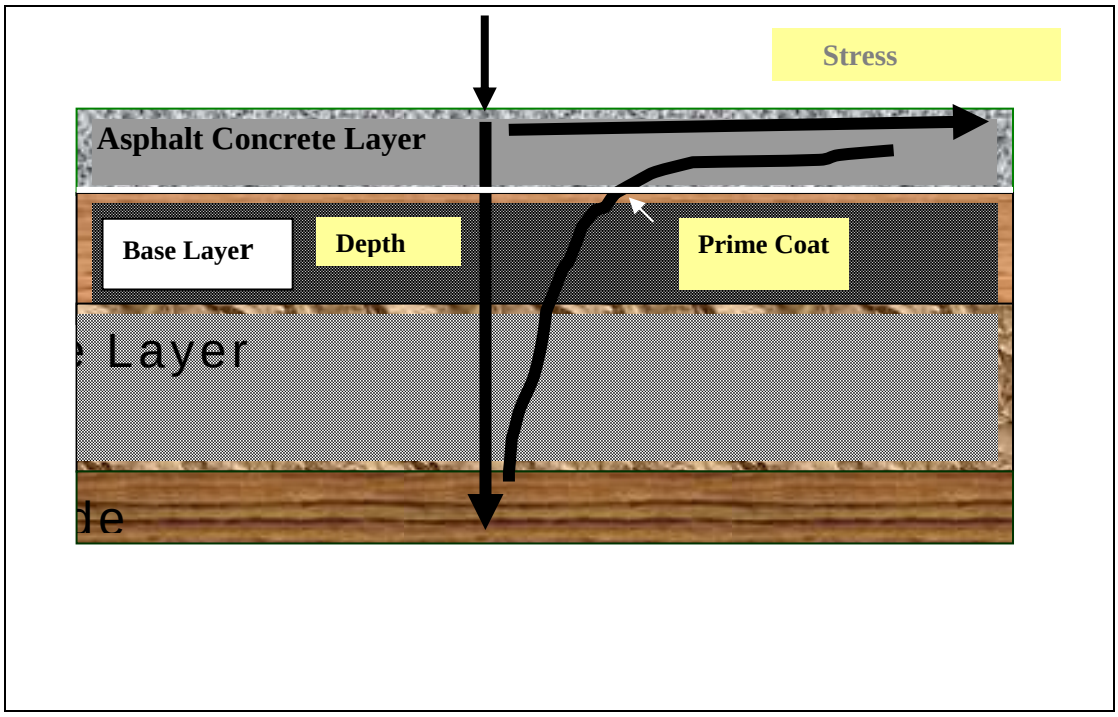


Figure 0.2: Typical cross section of a layered flexible pavement

The definition and the purpose of each layer are summarized as follows (Huang 1993):

- **Asphalt Surface Course:**

It is the top course of the flexible pavement. It represents the structural element designed to resist the effect of traffic loading, tire pressure, weather and abrasion. It also provides smooth surface for vehicle passage. The asphalt surface course can be divided into the following asphalt sub-courses:

- **Asphalt Wearing Course**: It is the top course of an asphalt pavement usually constructed using dense graded hot mix asphalt (HMA). It resists distortion under traffic and provides a smooth and skid resistant riding surface.

- **Asphalt Binder Course**: It is the asphalt layer below the surface wearing course. It is used when the asphalt surface is too thick to be compacted in one layer. Binder course composition and strength is almost the same as wearing course except a slight variation in the gradation sieves.

- **Asphalt Base Course**: It is the asphalt layer below the binder course. It is usually constructed of larger aggregate, less quality and less asphalt in comparison with the asphalt wearing course. It rests directly on the aggregate road base .

- **Road Base Course**:

  It is the layer directly under the AC surface normally composed of granular material (natural Aggregate, Crushed stone, …etc). It is mainly provided to distribute the load and for drainage purposes. Normally, The minimum required strength measured by California Bearing Ratio test (CBR) for this layer is >=80%.

- **Road Subbase Course**:

  It is the layer of material beneath the base course. It is an additional course of granular material but of less quality used to provide further distribution of the imposed load. The minimum required strength measured by California Bearing Ratio test (CBR) for this layer is >=30%.

- **Subgrade**:

  It is the top natural soil on which all pavement layers rest. In roads construction process, the top 150mm of the subgrade is normally scarified and compacted to the desirable density near the optimum moisture content. It may be the in situ soil or a layer of selected material. Additional 150 mm is treated if the subgrade bearing capacity is very weak. The subgrade itself is not part of the pavement layers, but subgrade with good quality of material (i.e. CBR>= 30%) may be used as a subbase layer.

In addition to the above mentioned courses, pavement consists of seal coat, tack coat, binder coat, prime coat. The definition and the use of each coat are as follows:

- **Seal Coat**: It is a thin asphalt material used to waterproof the surface or to provide skid resistance.
- **Tack Coat**: It is a very light application of asphalt material, usually asphalt emulsion diluted with water for the purpose of ensuring a good bond between the asphalt layers (i.e. between asphalt wearing course, asphalt binder course and asphalt base course). It is also used to bond between asphalt layer and PCC pavements or an old asphalt surface.

  Prime Coat: It is a light application of low viscosity cutback asphalt material to an absorbent surface such as untreated granular base for the purpose of binding the asphalt layer to the granular base. The prime coat should penetrate the underlying layer to plug the voids and to form a waterproof surface.

In some cases, the pavement may consists of one or more thick asphalt layer as a structural element for protecting the subgrade, in this case the pavement is referred to as Full Depth Asphalt Pavement (Huang 1993).

Due to the noticeable drawbacks observed in the convectional (empirical) methods of pavement design and performance analysis, considerable efforts have been paid by roads agencies and researchers to shift toward the

Empirical-Mechanistic(EM) approaches. Current and future demands on pavement caused by increased traffic volumes, introduction of new materials and construction techniques, require extrapolation of the performance data well beyond the existing database.

The philosophy of the Empirical-Mechanistic (EM) approach is based on the fact that the structural deterioration of the asphalt-surfaced pavement is associated with two failure modes; Cracking of the asphalt concrete surface as a result of the excessive tensile strains generated by the traffic effect at the bottom of the asphalt layer, and the permanent deformation (Rutting) in the underlying layers in the wheel paths as a results of excessive vertical compressive strains on the top of the subgrade.

The Mechanistic-Empirical approach normally correlates the calculated pavement response in terms of stress / strain or deflection to pavement field performance data. It gives the engineer a rational method to evaluate the effect of traffic loading and configuration, materials and construction methods on pavement design and performance.

Failure Criteria

Accumulated damage to the pavement layers is normally caused by an increased traffic loading or by the effect of environment loading. Two types of strains have frequently been considered most critical for pavement design. These are (Huang 1993):

- Asphalt Fatigue Cracking
  Cracking in the asphalt may be associated with the applied traffic loading and /or the effect of environmental factors. Repeated load associated stresses including those less than the failure stress accumulates progressively resulting in a fatigue cracking. AC Fatigue cracking is controlled by limiting the horizontal tensile strain at the bottom of the asphalt layer.

- Subgrade Permanent deformation

  It is formed as a result of the damage to the soil material such as sub grade and sub base layers caused by repeated traffic loading and other weather factors such as change in water content. Permanent deformation is controlled by the subgrade vertical strains. Subgrade failure limit (rutting) was found to range from 6 to 12 mm.

Study Methodology

The general methodology followed in this study is shown in Figure 2.3. The coming sections and chapters will encompass a detailed description of each study element. As can be seen from the flow chart below, efforts were directed towards standardizing all condition indicators measurements in a simple, meaningful and understandable forms that, when combined together, give the actual representation of the pavement condition structurally and functionally.

For each condition indicator, efforts were dedicated to identify its predictors, investigate the correlation between these predictors and the parameter that represents the condition indicator. Models to predict pavement performance were developed based on various variables pertaining to each condition indicator.

Treatment selection criteria in conjunction with the decision tree variables will be used to determine the optimum treatment alternative which will be able to restore both the riding quality and the structural integrity of the pavement taking into consideration other design aspects such as location , class and drainage condition.

In the methodology, three main procedures will be discussed in details, which are:

- Data collection procedures

- Data standardization
- Data analysis
- Models development

Figure 2.3 below summarizes the general methodology followed in this study.

Pavement Condition Data Collection Procedure

To fulfill the objectives of this study, data pertaining to more than 8000 pavement sections available at Pavement management system of Dubai Emirate and distributed over wide range of age, condition, structure, construction materials and traffic loading were used.

Information about the functional and structural characteristics of these sections were gathered and analyzed. These information include inventory data, condition, location (Urban ,Rural), pavement age, structure, construction materials, traffic loading, surface type, drainage condition, road class, distress types, severity and density, deflection data, roughness data, skid resistance data and any other data that might help in fulfilling the study objectives.

Roughness measurements repeatedly measured on large number of pavement sections subjected to variant traffic level were used to develop roughness propagation models.

Deflection measurements were also conducted on hundreds of pavement sections of various conditions and traffic to develop models that correlate the physical defects to the overall structural capacity of the pavement.

In this study, all condition indicators including physical distress, deflection, roughness and skid resistance, rutting and fatigue cracking were analyzed based on pavement section. This is because the pavement section is

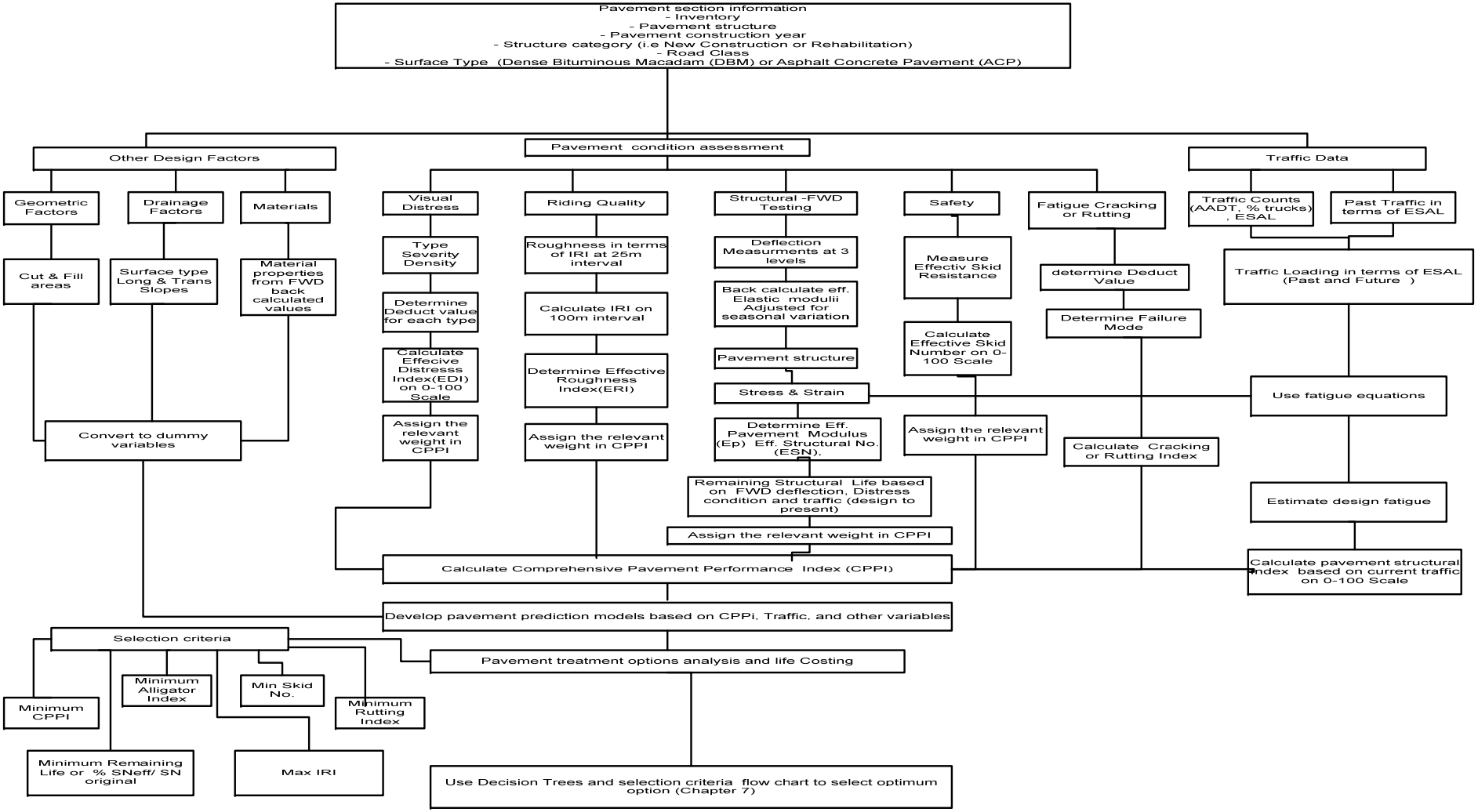


Figure 0.3: General Study Methodology

considered the smallest management unit when it comes to select maintenance and rehabilitation treatment. The procedures adopted for network identification, visual distress data collection, functional and structural evaluation are detailed as follows:

Pavement network identification and definition

This step is manifested by defining and identifying the network elements. Once the network is identified, it is divided into branches , sections and sample units for managerial purposes and easy survey process(Shahin 1989 and Shahin 1998).

Pavement Network

An installation pavement network consists of all surfaced areas which provide access way for ground and air traffic. Roadway, parking areas, aprons and airfield pavements are good examples of a network.

Pavement Branch

Is any identifiable part of the pavement network which is a single entity and has distinct function. Streets, parking area, runways, taxiway and aprons are good example of such element.

Pavement Section:

It is a division of a branch. It has certain consistent characteristics throughout its area or length. Pavement of a section should have the same construction date, traffic level, pavement class drainage facilities, shoulder availability, condition and structural composition in terms of thickness and materials. The pavement section is considered the smallest management unit when it comes to select maintenance and rehabilitation treatment.

Sample Unit:

It is an identifiable segment of a pavement section. It is the smallest component of the pavement network.

Figures 2.4 and 2.5 demonstrate illustrative examples of a network division into sections.

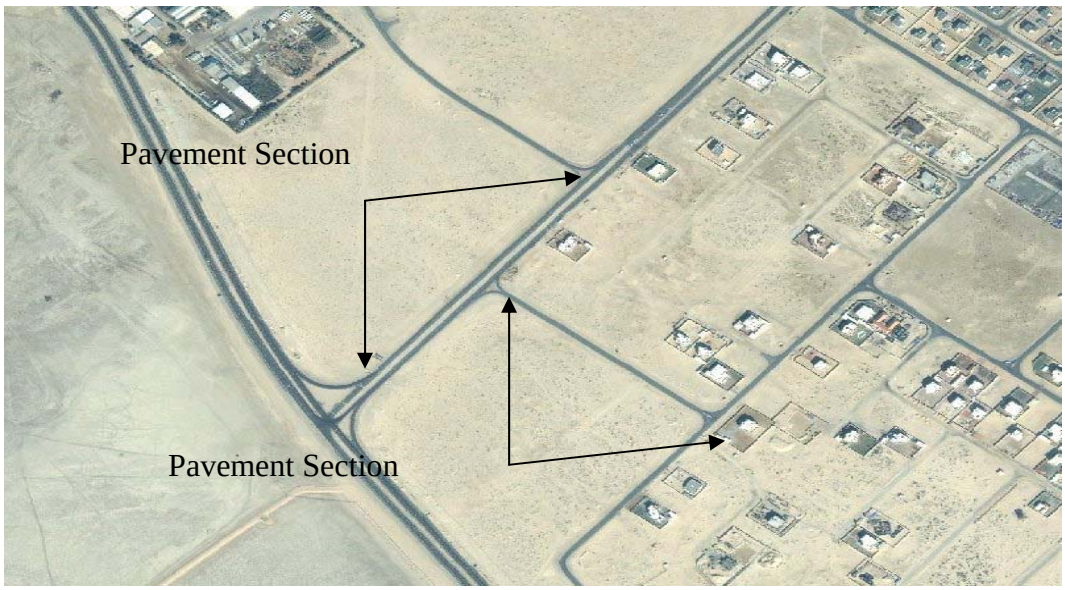


Figure 0.4: An example of road network divisions.

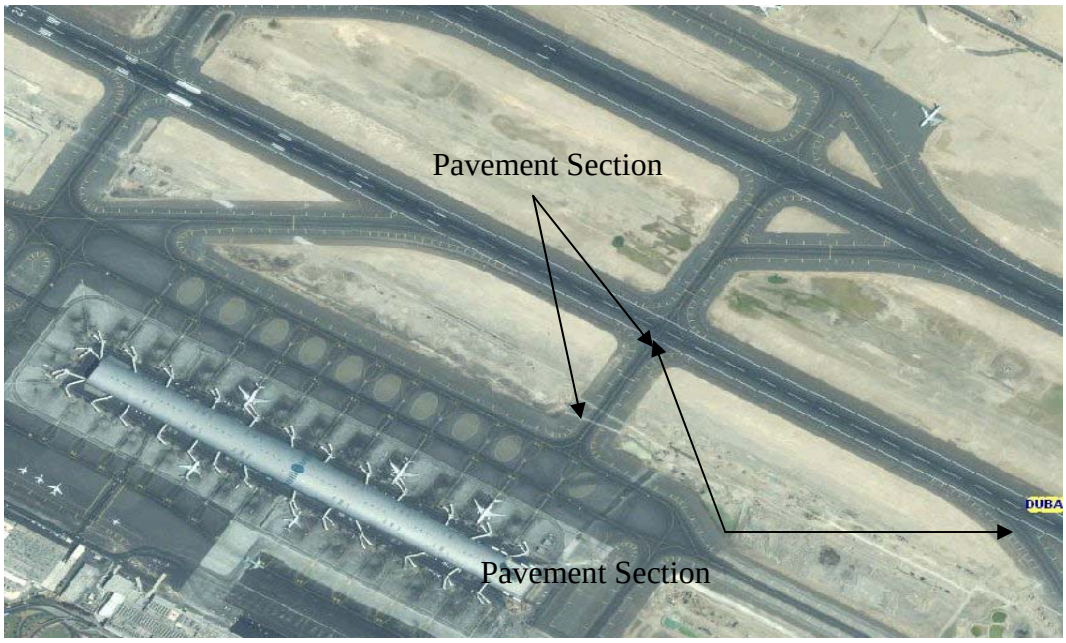


Figure 0.5: An example of an airfield network divisions.

Pavement Defects assessment

Distress survey is normally carried out for the purpose of determining the pavement condition (i.e. pavement condition at the survey time). It is also used to prepare preventive and routine maintenance works programs and estimate the required budget.

Distress surveys are usually carried out based on accurate measurement of the distress area and severity. There are many distress rating procedures around the world most of them are based on estimating the deduct value for the negative effect of each defect on pavement condition.

In this study, physical pavement condition was evaluated in terms of Pavement Quality Index (PQI). This procedure is similar in distress evaluation and physical condition rating to PAVER system developed by Shahin for the benefit of Us Army corps of Engineers (Shahin et al, 1990) . The only main difference between the two methods is the deduct value concept. The details of each method will be outlined in the coming chapters.

This procedure was originally developed by Texas Research and Development Incorporated (TRDI) and modified, enhanced and calibrated by the researcher through a pioneered project for upgrading and devolvement of the old Dubai Pavement Management System (TRDI, 2001). This method of condition survey concentrates on detecting different types of distress and the corresponding density and severity levels.

Most of the condition evaluation methods used around the world consider, more or less ,the following types of defect categories:

1. **Surface Cracking**: All forms of cracking including; Alligator Cracking, Longitudinal and Transverses Cracking, Block Cracking, Slippage Cracking , Joint reflection Cracking and Edge Cracking .
2. **Surface Deformation**: Including Rutting, Shoving
3. **Surface Defects**: Including Weathering and Raveling, Polished Aggregate and potholes.
4. **Surface Mix defects**: including Bleeding ,Corrugation ,Depression
5. **Others**: Including Swelling, Pumps and Sags, Lane shoulder drop off, Patching and utility cut patching and rail road crossing.

More than 19 distress types have been identified by Shahin as the most common types observed and recorded in flexible pavements. In this study, distress evaluation and description outlined in PAVER manual were utilized to record distress information (Shahin 1989 and Shahin 1998). Pavement Quality Index (PQI) was used as a

pavement condition index that represents the physical distress indicator. The reasons behind adopting this compound evaluation method will be discussed in chapter 5.

Normally, Visual pavement condition survey helps in determining the suitable maintenance and rehabilitation alternatives and gives the engineer an idea about the performance of the pavement and the encountered problems to be considered in designing the rehabilitation options.

Pavement Distress Procedure (PQI method)

This method considers distress scores transformed from the raw data is the precursor to the performance index calculations. The distress score is a number ranges from 0 to 100 with 100 being an excellent. (i.e. distress density is zero). All distress types with different distress severity and extents are combined using the multiplicative index approach. The trigger values for distress index calculations are needed to be calibrated each time as more data accumulated. Similar to HDM-3 or HDM-4 model, TRDI model is a general algorithm that should be calibrated for each agency.

Calibrating TRDI model to suit study area prevailing condition (Hot and arid environment) has been carried out in this study. Calibrating distress index calculation trigger values were achieved by assigning a weightage to the different distress types at each severity.

PQI was adopted to be used in this study because it includes general modules to convert condition index regardless the dimension. It uses two methods for calculating distress scores and other condition indicators indices. These methods are as follows:

1. **One dimension distress converter**:

   Distress types of one dimension are directly converted into distress scores using the developed relationships or conversion factors gained from experience. In this study, *efforts were directed to use this method to convert the results of the roughness, skid resistance and structural evaluation into scores on scale 0-100*. Indicators such as International Roughness Index(IRI) and Effective Structural Index (ESI) , Effective Pavement Modulus (EPM-E_p), Asphalt Curvature (CUR) or Remaining Structural Life (RSTL) whichever found convenient will be converted

into scores on scale of 0-100 for the purpose of calculating the comprehensive or overall pavement performance index.

Other distresses such as polished aggregate can also be converted into distress score using this method.

2. **Maximum Allowable Extent**: Distress types of two or three dimensions are directly converted into distress scores using the developed relationship or conversion factors gained from experience. Distresses with both severity and density are good examples of such distress types. In this study, efforts were dedicated to use this method to convert all distress types into distress scores on scale 0-100 to be used later in calculating the overall pavement performance index. This method uses a linear conversion between distress deduct value and extent value with each severity level and a multiplicative combination of deduct values across severity levels.

The maximum allowable extent depends on three linear deduct functions for each severity level. Two coordinates determine each of these lines. Each specified by the user, the low range value and the high range value. More details about this method will be presented in the next chapters.

In this study, sensitivity analysis to calibrate the main parameters was performed. This includes establishing new trigger values for the low and high range values and the corresponding scores based on the actual data extracted from the field. Details of this procedure will be also presented in chapters to come.

Collection procedure for Deflection Data

The required field data pertaining to the structural capacity evaluation were extracted from thousands of in-service pavement sections of various characteristics. Falling Weight Deflectometer (FWD) was used to accomplish this task as it can simulate the field traffic loading in magnitude and the time of application. The surface deflection was measured for hundreds of pavement sections mainly at the slow lane, which is considered the most heavily trafficked lane.

FWD was originally designed by the French engineers in the 1960's. Later on , this machine was modified and upgraded by the Danish and Swedish engineers. It consists basically of a mass that falls under gravity force to hit a spring –loaded segmented plate simulating the effect of a moving wheel load in magnitude and in the loading time over certain pavement point. The generated load is a function of the mass and the drop height. The FWD used in this study for determining the deflection test is a Dynatest 8000 model (Figure 2.6). The FWD is the most advanced tool that has been designed for structural evaluation purposes ever produced. It is a rapid and nondestructive tool for determining the deflection basin response to a measured dynamic load.

The magnitude of loading range from 30-120kN (6500-27000lbf). Several models of this equipment are available. Another model is the Heavy Falling Weight Deflectometer (HFWD) 8081 used for testing the airports pavements.

The range of loading for this tool is between 30-240kN (6500-54000 lbf) which covers half of the axle load imposed by a moderately truck upward through the single wheel of a fully loaded Boeing 747 aircraft.

Figure 0.6: An illustration shows Falling Weight Deflectometer concept

The measurements were carried out according to ASTM D4695-03 (General Pavement Deflection Measurements) and ASTM D4694-87 (Deflection with a Falling Weight Deflectometer-Type Impulse Load Device. All measurements were conducted along the wheel track to measure the accumulated effect of traffic loading.

In general, deflection testing at network level was conducted as per the following steps:

1- Extracting the inventory data of the pavement section such as section designation, date of construction, limits, class, pavement structure material types and traffic volumes.
2- Performing FWD field testing on the slow lane. The test consists of 4 load drops (one seating and 3 at different levels within the range of 35 to 65KN) , at a predetermined staggered spacing within the range of 50m to 250m depending on the section length and construction characteristics.
3- Maximum Pavement temperature measured at depth of 3-10mm was recorded. Temperature for surface and air were also measured at the beginning of each test and continuously updated every two hours during testing.
4- The measured deflections normalized to the standard load and temperature if considerable difference in temperature were recorded. The method of normalization of both load and temperature will be detailed elsewhere in this study.
5- The deflection basin was analyzed and basic engineering parameters of the pavement layers (pavement modulus, modulus of elasticity, for all pavement layers, subgrade modulus Curvature, stress and strains were backcalculated using ELMOD4 program.
6- The back calculated data were then transformed into different format and forms found suitable to represent the effective structural capacity. Various correlations and relationships with the pavement performance were explored and deeply studied. The applicability of using each form as structural capacity indicator in PMS and to determine the remaining live based on various mechanistic approaches were thoroughly investigated and verified.
7- The suitable and representative structural capacity indicators such as structural number, structural capacity index, pavement modulus deterioration index

Deflection Data analysis

Deflection data were analyzed using various structural algorithms developed for such purpose. These algorithms calculate the number of repeated flexural stresses to fatigue failure and the number of vertical compressive strains to permanent deformation failure criteria.

ELMOD4 program can use any mechanistic design procedure by incorporating the transfer function into the parameter file. The allowable stresses σ_{all} or strain ϵ_{all} used in the design takes the following form:

$$\sigma_{all} / \epsilon_{all} = A*N^{B} \; (E/E_{ref})^{c} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 2.1$$

This equation can be written in terms of number of repetitions to failure (N_f) in the following form:

$$N_f = (a/\epsilon_{all})^{B} * (E/E_0)^{C/B} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots 2.2$$

where:

A: Stress or Microstrain at one million application of load

B: Gradient of the distress relationship (stress/strain .vs. Load in MPa or ksi.

The most common algorithms available for deflection analysis are given in the Table 2.1 below.

Table 0.1 Common mechanistic design formulas developed for structural capacity evaluation.

Maximum allowable Strain at the bottom of the asphalt layer used for fatigue mode failure. $\epsilon_{all} = A*N^{B} \; (E/E_{ref})^{c}$				
Algorithm reference	A (mstrain)	B (Mpa)	E_{ref} (Mpa)	C
Shell	538	-0.250	3000	0
TRL (DBM)	201	-0.24	3000	0
TRL (HA)	224	-0.231	3000	0
Asphalt Institute-AI	1162	-0.304	6.9	-0.259
Asphalt Institute-(Ullidtz)	240	-.0304	3000	-0.259
Denmark (Kirk)	195	-0.178	3000	0
NAASRA (Australia)	225	-0.200	3000	0
Maximum allowable Vertical Strain at the top of the Sub-grade used for deformation mode failure. $\sigma_{all} = A*N^{B} \; (E/E_{ref})^{c}$				
Algorithm reference	A (mstrain)	B (Mpa)	E_{ref} (Mpa)	C
TRL & Nottingham	451	-0.280	160	0
Asphalt Institute-(Ullidtz)	484	-0.223	160	0
Shell	885	-0.250	160	0

Maximum allowable Vertical Stress at the top of the Sub-grade used for deformation mode failure. $\sigma_{all} = A*N^{B} (E/_{Eref})^{c}$				
Algorithm reference	A (mstrain)	B (Mpa)	E_{ref} (Mpa)	C
Asphalt Institute	0.1425	-0.307	160	1.16&1
Denmark (Kirk)	0.120	-0.307	160	1.16

C: A constant (MPa or Ksi)

N: No. of load applications in million

E: Back calculated Modulus of elasticity of the material.

E_{ref}: A reference modulus (Mpa or Ksi)

Other models developed to calculate the effective structural capacity for the in-service pavements and TO estimate the remaining life of the pavement sections based on the pavement response (deflection, stress and strain) and at various levels of traffic loading were investigated for comparison purposes and to recommend the best for the use of PMS. The list of these models is shown in Tables 2.2, 2.3 and Table 2.4 below.

Table 0.2: Models of Predicting Rutting.

Model	**Reference**
Model Suggested by Santucci, 1977 $N_R = (\varepsilon c / 0.0105) - 4.84$	David, 1996 Chua et al, 1992
Asphalt Institute permanent deformation model * $N_R = 1.365 * 10\text{-}9 * (\varepsilon c)\text{-}4.47$	Witczak, 1996 Paul, 2000
Model Suggested by Shell ** $N_R = 6.15 * 10\text{-}7 (\varepsilon c)\text{-}4$	Hisham, 1997
Model Suggested by Shook, et al, 1982 $N_R = 1.077 * 1018 (10\text{-}6 / \varepsilon c)^{4.4843}$	Nazarian, 1998
Model Suggested by USACE $N_R = 10000 * \left[\frac{0.000247 + 0.000245 \text{ Log } (E)}{(\varepsilon_c)} \right]^{0.0658 E^{0.559}}$	Manuel, 1998
Model Suggested by Shell Oil $N_R = 5.841 * 10\text{-}9 (1/\varepsilon c)4.775$	Manuel, 1998
Model Suggested by Anderson, 1990 $NR = 30 * (1 / \varepsilon c)1.5$	Anderson, 1990
U. K. Transport and Road Research Laboratory $N_R = 6.18 * 10\text{-}8 (\varepsilon c)\text{-}3.95$	Powell, 1984
Belgium Road Research Center $N_R = 3.05 * 10\text{-}9 (\varepsilon c)\text{-}4.35$	Verstraeten, 1982
Austroads, 2001 $N_R = (0.0093/\varepsilon c)7$	Austroads, 2001

where :

N_R: No. of repetitions to failure due to rutting, **εc** : Maximum compressive strain at the top of subgrade, **E** : Subgrade resilient modulus, psi

* *This model assumes that rutting takes place in the subgrade (i.e., rutting attributed to other pavement layers is negligible) and the failure is defined as the development of 15 to 19 mm (0.5 to 0.75 in) rutting.*

* * The model does meet the one-point criterion of less than 13 mm (0.5 in) rutting if the damage ratio is less than 1.0

Table 0.3: Models of Predicting Roughness

Model	Reference
Model Suggested by Anderson, 1990 $N_{Rg} = 10^{(2.1522 - 597.662 * \varepsilon_c - 1.32967 * \text{Log}\, \varepsilon_c)}$ where NRg = number of load repetitions to failure due to roughness εc = vertical compressive strain at the top of subgrade surface.	Anderson, 1990

Table 2.4 : Models of Predicting Fatigue Cracking

Model	**Reference**
Asphalt Institute MS-I $N_{fc} = 18.4 * C [4.325 * 10^{-3} * (\varepsilon_t)^{-3.291} * E^{-0.854}$ $C = 10^M$ $M = 4.84 [P_{eff} / (P_{eff} + V_V) - 0.69]$	Collop et al, 1995
Model Suggested by Fin et al, 1977 $\text{Log}\, N_{fc} = 15.947 - 3.291 \text{Log} (\varepsilon_t / 10^{-6}) - 0.854 \text{Log} (E / 10^3)$	Hisham, 1997
Asphalt Institute MS-II $N_{fc} = 4.024 * 10^{11} * (\varepsilon_t)^{-4.995} * E^{-4.306}$	Kenis et al, 1982
Model Suggested by Shell Oil $N_{fc} = 0.0685 * (\varepsilon_t)^{-5.671} * E^{-2.363}$	Kenis et al, 1982
Model Suggested by Anderson $N_{fc} = 1.33 * 10^{-1} * (1 / \varepsilon_t)^2$	Anderson, 1990
Illinois Department of Transportation Model $N_{fc} = 5 * 10^{-6} * (1 / \varepsilon_t)^3$	Thompson et al, 1987
U. K. Transport and Road Research Laboratory $N_{fc} = 1.66 * 10^{-10} (\varepsilon_t)^{-4.32}$	Powel et al, 1984
Model Suggested by Shell, 1978 $N_{fc} = \left[\frac{0.006918 \ (0.856 \ P_{eff} + 1.08)}{(E / 145) * \varepsilon_t} \right]^5$	Austroads, 2001

where: N_f: No. of repetitions to failure due to fatigue cracking ε_t: Maximum tensile strain at the bottom of asphalt layer E : Dynamic modulus of asphalt mix., psi P: Effective asphalt content by volume, % V_V: Percent air void	

The effective structural capacity for the in-service pavements were evaluated for the common modes of pavement failure; Fatigue Cracking, Rutting and Roughness. These models were used in the applications to show how the structural capacity can be predicted using the back calculated modulus, stress and strains. The remaining structural life value is envisaged to be a vital condition parameter indicator and can be used to plan the work activities and estimate the budget at short and long range schemes.

The suitable mechanistic design formula will be used according to prevailing conditions and the characteristics of the existing materials to back calculate the performance parameters and the responses of the pavement under loading effect. The suitable algorithms will all be used and verified in this study. Comparison between different models developed to predict pavement failure and remaining life will be presented. The most optimum model envisaged by the researcher to be able to predict pavement failure will be highlighted.

The equipment was configured as shown in Table 2.5 below. For the consistency of the results, the same configuration was kept the same for all pavement sections. 9 geophones were used in the testing. The geophone positions were selected as detailed below:

Table 2.5: FWD geophones position used in the study

Geophone No.	D_0	D_1	D_2	D_3	D_4	D_5	D_6	D_7	D_8
Offset (mm)	0	200	300	450	600	900	1200	1500	1800

The remaining structural life parameter is envisaged to be a vital condition indicator and can be used to plan the work activities and estimate the budget at short and long range schemes.

Load Magnitude

Load magnitude will be designed to range from 40kN to 65 KN . The 40 KN load represents the standard load on each wheel in a single axle dual wheel system. This load is normally adopted as a standard axle load in pavement design and structural capacity back calculations particularly in rehabilitation work.

Roughness data collection procedure

To fulfill the objectives of this study, pavement surface roughness was measured using a KJ Law Model 6500 roughness measuring device-RMD available at Roads and Transport Authority PMS Unit (Figure 2.7). RMD is a vehicle mounted device measuring longitudinal profile and a 3 point transverse profile, with analysis of the profile to produce roughness and rut depth. All roughness measurements and data processing were carried out according to the international standard designated as ASTM E950 (Test Method for Measuring the Longitudinal Profile of Traveled Surfaces with an Accelerometer established inertial profiling reference)) and ASTM E1926 (Computing International Roughness Index of Roads from Longitudinal Profile Measurements).

This road surveyor has the ability to create pavement profile detailing surface roughness and rut depth. It operates at speed between 30 to 110 km/h with no disruption to the traffic movement. The road surveyor employs precision accelerometer (one at each wheel) to measure vehicle motion. Patent , non contact infrared sensors measures displacement between the vehicle body or the front bumper –mounted sensor bar and the road surface. Those two inputs are fed into the system's on board to create pavement profiles for an accurate picture of the road surface roughness and rutting condition.

In order to give the most accurate longitudinal profile ever obtained, independent of vehicle speed, ambient temperature , sunlight or pavement condition, data points are taken every inch (24mm) interval averaged over 12-inch interval and stored as a profile data point.

Figure 2.7: KJ Law 6500 Model Roughness Measuring Device-RMD

The roughness of the pavement surface were used in this study to assess the present serviceability and the remaining service models and forms to be used in PMS for the purpose of reporting about the functional capacity of the road at network level. The following steps were followed to measure roughness on the selected sections:

1. Inventory data of the pavement section such as section designation, date of construction, limits, class, pavement structure material types and traffic volumes were extracted from the current PNS system and by field surveys.
2. Roughness of the pavement in terms of International Roughness Index (IRI) units. I.e. m/km or mm/m at spacing of 25m in both the right and left wheel tracks and on both fast and slow lane were measured for comparison study and models development.
3. Yaw value, which is the difference between IRI values measured in the right and left wheel track. Was calculated for each section.
4. The lane IRI value of 100m and 400m section respectively were calculated.
5. The present serviceability index (PSI) for each homogenous section was determined.

6. The proper relationship between IRI and PSI was investigated. This step is essential for correlating the IRI measurements to international standards used in the design of pavement structures and to establish terminal values that necessitate certain type of maintenance or rehabilitation intervention.
7. The relationship between roughness and surface age was also examined by deterministic approach using historical roughness data.
8. The roughness propagation models and remaining service life for each homogenous section were determined and investigated for the best use in PMS.

Skid Resistance data collection

The required field skid resistance data pertaining to the selected in service pavement sections of various characteristics were extracted . The available tools and methods were used to accomplish this task. The surface skid resistance tests were performed on hundreds of pavement sections mainly at the slow lane, which is considered the most heavily trafficked lane, in order to keep the consistency of data collection process. Also, this is useful in investigating the interrelationships between the main factors affecting pavement performance.

Skid resistance is generally quantified using some form of friction measurement such as a friction factor or skid number:

$$\text{Coefficient of Friction } (f) = F/L$$

where :

f= Friction coefficient

F=Friction resistance force

L=Load applied in perpendicular form to the surface.

Then, Skid Number is calculated as follows:

$$(SN) = f*100$$

Standard friction tests specify standard tires and environmental conditions to overcome the problem of results variability. Variability in results may due to pavement wetness, vehicle speed, temperature, tire wear, tire type, etc.

Skid resistance measurement

The measurements were carried out on the slow lane and along the wheel track to measure the accumulated effect of traffic loading. The tests will be carried out according to the standard procedures outlined in both B.S. 598:Part 105, and ASTM E303.

Detailed description and method of testing on the portable skid resistance tester can be found also, in ROAD NOTE 27 from the Transport and Research Laboratory –UK and ASTM E303 “Standard method for measuring surface friction properties using the British Pendulum Tester”. The skid resistance value were measured in terms of two indicative parameters:

- Mean texture depth using laser machine (ARRB NSV).
- Sand Patch Method to determine Mean Macrotexture Texture Depth and to calibrate the laser measurements.
- Skid number using British Pendulum Tester available at Dubai central laboratory DCL. This test is normally applied to the wet pavement surface . The value of SRV can be measured using British Pendulum Tester. SRV is normally dependent on the micro texture properties (i.e. PSV values). This value is good indicator of the macro texture properties of the pavement surface. Detailed standard procedure for this test can be found in the same reference B.S. 812:Part 114.

Other tests such as Stone Polished Value (SPV) are normally carried out to particular aggregate type using British Pendulum Tester after subjecting the aggregate to polishing action. Full description of the process and the test method can be found in B.S. 812:Part 114. Aggregate with PSV of 60 and more is rated as a High Skid Resistant Aggregate. High Skid Resistant Aggregate has greater resistance to polishing, and it has good ability to retain its own fine texture.

The PSV is good indicator of the micro texture properties of the asphalt-surfaced pavement. In this study, this test was not conducted as most of the roads were constructed using the same source of aggregate (mountains aggregate) with SPV ranges from 55-65. Therefore, macrotexture properties were only measured to account for the skid resistance properties and to develop skid resistance deterioration models.

British Pendulum Tester

This tool is very simple in structure and method of use (Figure 2.8). The pendulum is raised to a locked position, then released from the horizontal position by a quick release button, thus allowing the slider to make contact with the tested road surface .

Figure 2.8: British Pendulum Tester

A drag pointer is used to read the British Pendulum Number (BPN). The greater the friction between the surface and the tester, the larger is the BPN reading.

In general, skid resistance evaluation at network level was carried out according to the following steps:

1. Extracting the inventory data of the pavement section such as section designation, date of construction, limits, class, pavement structure materials types and traffic volumes.
2. Performing Skid resistance field testing, which consists of 3 contacts at each pavement section using Portable British Pendulum described above.
3. Measuring the friction coefficient (Mu) using the available tool (Frictometer) at standard speed of 64km/h.
4. Measuring the Mean Texture Depth (MTD) for the various pavement sections of different surface types (i.e. DBM or ACP) using ARRB NSV and later calibrated by the Sand Patch Method.
5. Calculating the average SN value of the three contacts.
6. Normalizing the average value for each section on a scale from 0-100.
7. Calibrating this value to standard from to used in PMS.

Rutting Measurements

Traditionally, rut is measured by hand using a 1.2m straight edge. Most of the available pavement sections have been measured by this method. Presently, many non contact measurement tools have been developed . for the purpose of fulfilling the objectives of this study, rut data were collected manually and verified by K.J Law roughness measuring device. This machine has the capability of measuring the rut depth over a specified length of pavement section such as 25m or 100m. The average rut depth was reported directly from the machine while measuring at posted speed. The procedure adopted for rut data collection will be as follows:

1. Extracting the inventory data of the pavement section such as section designation, date of construction, limits, class, pavement structure materials types and traffic volumes.
2. Measuring and recording rut depth by hand using a 1.2m straight edge during visual survey.
3. Measuring rut depth in the left and right wheel paths over a specified length of pavement section (adopted length in this study is 25m)
4. Calculating the average rut depth of the two values and estimating the rut area and severity based on rut depth and section length and width.
5. The rut data is reported over the section length in terms of rut index.

Alligator Cracking measurement

Alligator cracking data for most of the available pavement sections is measured manually. This method of measurement is the simplest , economical measuring method for alligator cracking. Data pertinent to alligator cracking will be collected as follows:

1. Extracting the inventory data of the pavement section such as section designation, date of construction, limits, class, pavement structure materials types and traffic volumes.
2. Measuring the area of the alligator cracking for each sample unit and then estimated for pavement section if systematic sampling is used.
3. The density of the alligator cracking along the entire pavement section is calculated.
4. The alligator cracking data is reported over the section length in terms of alligator cracking index.

Development of New Overall Pavement Quality Index(OPQI)

The researcher will use a theoretical concept called *Multiplicative Index Approach* (MIA). In this method all distress types, severity and extents are combined in such a way ,the effect of each distress type can be calculated and then summed up to have one single index at the end of the process. This theoretical concept takes the following form:

$$\mathbf{OPQI_i = 100 \sum_{m=1-n} (1-(1-CI/100)) * Wt_{I,m},}$$

where:

OPQI: Overall Pavement Quality Index (normally assumed on a scale from 0-100).

CI: Condition Indicator Index (also assumed on scale from 0-100).

CI is calculated as follows:

Rutting index, (RI)

Alligator cracking index (ACI)

International Roughness index (IRI)

Effective Structural capacity index (ESCI)

Skid resistance index(SRI)

Bleeding Index(BI)

Patching Index(PCHI)

Potholes Index(PTI)

Weathering and raveling index(WRI)

i= i^{th} Pavement Performance Index.

m=Distress type number

n= Total number of distress included in the performance index.

Wt=The impact weight of each distress type or condition indicator index.

A scale from 0 to 100 was selected in order to convert one –dimensional or multi-dimensional distress types or condition indicators data into a unified scale for the purpose of the overall performance index calculations.

The above formula is considered as the general form that deals with distress indices regardless of the dimension. Therefore, the efforts were paid to develop the conversion rules that convert all pavement condition indicators into a one single index on a scale from(0-100) which can be used to calculate the overall performance index for a pavement section or a sample unit.

As shown in the above formula, Overall Pavement Quality Index includes all condition indicators indices converted into a unified scale range from 0 to 100. This will include in addition to normal distress types indices, rutting index, alligator cracking index, roughness index , deflection or structural capacity index and skid resistance index.

The developed overall index is envisaged to be comprehensive and an indicative measure of pavement condition. The other individual indices are also used in the decision tree of treatment selection. The selected treatment option is expected to rectify the condition of the pavement section and restore both structural integrity and functional serviceability.

Fact No. 1

There is no ideal single pavement management system that is best for all highway agencies.

Fact No. 2

Up to date, most of the developed condition indices incorporate only the distress data. Other condition indicators such as roughness, structural capacity index and friction index are used separately to report about pavement performance

Data Analysis Method

The method of analysis for the main condition indicator parameters as envisaged by the researcher will include the followings:

Structural Capacity Indicator

It became an easy task to evaluate the effective structural capacity of the road pavement using the newly developed Non -Destructive Testing-NDT tools. In this regard, Falling Weight Deflectometer-FWD was used to extract the structural data for the use of pavement condition assessment. In this study , Back calculated elastic modulus of each pavement layer(Ei), overall effective pavement elastic modulus (Ep) using sensors reading at different offsets, Asphalt curvature and the corresponding remaining structural life structural parameters were estimated and used in various forms to reflect the structural pavement characteristics in the performance index.

Deflection data were also used to evaluate the effective subgrade resilient modulus. Deflection data measured by the other sensors can be used to estimate the effective

Despite the fact that most of the agencies prefer to use this data at project level, it is believed that this can be done easily at network level by the devices and tools available now or that to be introduced in the next decade with minimum disturbance to the traffic flow.

Roughness Indicator

This aspect can be manifested through the rideability index in terms of International Roughness Index-IRI. This figure can be treated as a combination of surface defects and surface irregularities. The data were converted into a suitable scale that can be fit within the general form of the performance index.

Skid Resistance indicator

Surface characteristics based on the micro- texture and macro- texture properties can be manifested through this parameter. Skid resistance number can be measured physically with many available tools. This number-SkN can be incorporated into the performance index to reflect surface skidding impact. Alternatively, information extracted from certain distress types (such as polished aggregate, bleeding or weathering and raveling) and the corresponding severity level can be utilized to estimate the safety level prevails on each pavement section.

Estimation of the effective structural capacity pavement remaining life prediction models

Data extracted from the field such as distress data, roughness, skid resistance and deflection were used to develop functional and structural reaming life models based on different pavement characteristics such as pavement age, traffic level in terms of ESAL , Deflection and structural index.

Different methods of analysis outlined in AASHTO manual will be used to calculate the structural and functional remaining life. These methods are:

- Effective structural number SN_{eff} from condition surveys
- Effective structural number SN_{eff} from traffic data based on the past, current and the future volumes.
- Effective structural number SN_{eff} based on deflection analysis.

These methods were compared to other procedures developed in this study based on roughness and deflection data analysis. The IRI was converted to roughness index on a scale 0-100 and the minimum roughness levels were established based on the actual observation from the field.

The same procure was used for deflection and skid resistance analysis. Data for pavement sections of different condition were used to develop the formulas to estimate or predict remaining life for various pavement sections .

The development of these models was based on different analytical approaches including the cumulative distribution function and probability function which are helpful in estimating the pavement mean time to failure and simulating the performance of the pavement systems over time.

Development of maintenance and rehabilitation schemes

Maintenance and rehabilitation schemes developed in this research effort were designed not only based on a single index but also taking other factors into consideration. These schemes include other deterministic factors related to the distress types, distress severity, distress density or distress index, roughness level , structural capacity index , location, inventory data, traffic level (% of trucks),..etc.

The measurement tools and testing devices required to extract the information from the field and the other data components were almost available in the RTA PMS Unit-Dubai. The Portable British Pendulum and Frictometer were made available through the Dubai central laboratory .

The latest version of the Statistical Package for Social Science(SPSS) software was used in generating the correlation matrices between various variable and to carry out linear and non linear regression analysis to develop the required models.

This program is powerful in the linear and multiple regression analysis and the mathematical transformation for the selected functions. Other softwares such as VISIO program for charts and schematic diagrams drawing was also used.

Data Elements and Sources

Pavement Condition Data

The Well-organized and comprehensive condition information database available in RTA Pavement management system for more than 8500 pavement sections of different classes, age, condition, traffic level and structure were used to fulfill the research objectives. In details , the following data sources were approached.

- PMS Unit As Built Drawing Database: Approached to obtain pavement sections history, pavement structure, material characteristics, and sub grade data.

- Distress condition: All pavement sections included in this study were visually evaluated for distress measurements. These include alligator cracking, rutting, weathering and raveling, patching, depression, slippage, bleeding, potholes, and for each pavement section.

- Deflection data : All pavement sections included in this study were tested by Falling weight Deflectometer (Dynatest model 8000) available in PMS Unit.
- Roughness data: All pavement sections included in this study were tested by K J Law 6500 model roughness measuring device available in PMS Unit.

- Skid Resistance data: All pavement sections included in this study were tested to measure skid resistance properties using Frictometer, British Pendulum Tester and Sand Patch method.

- Environmental data from field measurements (daily temperature, seasonal variation, precipitation) from metrological department used to normalize the deflection data for back calculation purposes.
- Literature from the available technical manuals, journals and indices.

Pavement layers thickness and materials description were extracted from the as built drawings available in PMS unit of RTA.

To help in analyzing the deflection data, the seed modulus for various pavement layers (asphalt, aggregate base and aggregate subbase course) were extracted from several samples tested at Dubai laboratory within the context of asphalt research project. These values were referred to as a standard modulus values determined at 25 C°. Typical values for asphalt mix modulus are listed in the Table 2.6 below for easy reference during the analysis.

Table 2.6: Typical values for asphalt mix modulus experimentally measured in the study area

Mix Designation	Mix type	Asphalt Type	Aggregate Type	Poisson Ratio	MR (Mpa) (25 C)
Dubai DBM	Dubai Dense Bituminous Macadam	60-70 pen	Mountain	0.35	6055
Dubai AC	Dubai Asphalt Concrete	40-50 pen	Valley	0.35	6864
Dubai AC	Dubai AC	60-70 pen	Valley	0.35	5303
Dubai AC	Dubai AC	40-50 pen	Mountain	0.35	6984
Dubai AC	Dubai AC	60-70 pen	Mountain	0.35	4746
Base	Aggregate Base	-	Mountain	0.40	427
baseSub	ggregate Subbase	-		0.40	380
Subgrade	Silty sand Subgrdae			0.45	80-180

In practice, temperature at the bottom of the asphalt layer measured in many trial sections in one of the major research project sponsored by Dubai municipality under the supervision of the researcher was found to range from 29 C° to 42 C°. Therefore, it is recommended that asphalt stiffness should be determined at 35 C° for pavement design and evaluation.

Climatic Considerations

These data elements are needed as a complementary information used specifically in the analysis of deflection data. Dubai is a city situated in a dry arid area. All pavement sections were located in the same climatic area, so no considerable differences in the performance of pavement section is expected due to climatic reasons.

Temperature and Humidity

Temperature data in Dubai for the last 12 years (1994-2006) were collected and analyzed. The maximum and the minimum temperature value ever recorded in this region were 47 C° and 6 C° respectively. The temperature as an environmental factor affects the performance of the pavement under traffic action in hot summers. The temperature factor was included in the analysis as a main input in deflection analysis since the asphalt modulus is temperature dependent. Humidity is another climatic aspect of Dubai area. It range from 50 to 100% saturation in the months of July and August.

Precipitation and Water Table

Records related to rainfall in Dubai in the last 10 years shows that the rainy season extends from November to April. The amount of precipitation in this dry arid area range from 9mm to 220mm with average annual precipitation is around 90mm. The water table in this region range from 0 to more than 5 meters with average value in the urban areas , which is very close to the sea shore is around 1.5 m.

Thesis Organization

This study will be comprised of nine chapters, the size of each chapter depends on the subjects and the material included. Each chapter may range in size from 40 to 80 pages. The author will do his best to avoid repetition, generalization and unclear statements. The author will concentrate on fulfilling the study objectives set forth as specific as possible. In this regard, the thesis will be organized as follows:

Chapter 1 includes an introduction to the thesis subject, outlining the pavement management concept, pavement condition components and pavement condition indicators. Common methods of pavement evaluation are briefly described in this chapter . Finally, the chapter includes a summary of the problem and the expectations from conducting this study. Description of the study area in addition to study objectives are also presented in this chapter.

Chapter two encompasses the methodology followed in the study. The theoretical concept of pavement performance and condition indicators measurements are introduced. Procedures for condition indicators data collection are also summarized in this chapter. Devices and measurement tools to be used in data collection processes are also included. This chapter contains a list of data elements and data sources used in this study .

Chapter three includes a detailed description of the factors affecting pavement performance. These mainly include traffic, material, environment and geometric aspects. Other secondary factors such as construction procedures, maintenance measures, work specifications are also discussed and outlined.

Data analysis and discussions are presented in chapters four to six. ***Chapter four*** outlines the analysis procedures to determine the effective structural capacity for the in-service pavements , measuring riding quality and safety. It also describes the attempts carried out in this study to identify the main predictors for the main condition indicators, investigate the correlations and develop models to estimate the current pavement condition and predict the future condition.

Chapter five includes the method of analysis to develop a comprehensive pavement performance index based on pavement condition indicators parameters.

Chapter six contains details about methods of estimations of functional and structural remaining life and the prediction models developed based on different approaches. These include traffic, time roughness and deflection approaches.

Chapter seven include detailed description of the development of maintenance and rehabilitation schemes based on pavement condition indicators using stepwise concept. Treatment selection outcomes will be outlined and the effect of introducing each condition indicator on selection outcomes will be highlighted. Illustrative examples that include in service pavement sections will be presented as a supplementary proof for the effect of each condition aspect on treatment option selection.

Chapter eight, includes an application of the developed pavement condition index and the developed treatment selection processes. It also includes life cycle costing analysis for typical examples of maintenance and rehabilitation schemes.

Chapter nine will summarize the main conclusions drawn based on the outcomes of this study . It also include the recommendations consolidated based on practical applications of the study outcomes to be followed by roads agencies in the field of pavement maintenance and rehabilitation.

CHAPTER 3 FACTORS AFFECTING PAVEMENT PERFORMANCE

3.1 Performance concept

Pavement performance is defined as the ability of a pavement to satisfactorily serve traffic over time (AASHTO, 2003). The serviceability is defined as the ability of a pavement to serve the traffic for which it was designed. Integrating both definitions will yield a new understanding of the performance which can be interpreted as the integration of the serviceability over time (Youder & Witczack, 1975).

Performance is a broad, general term describing how pavement condition changes or how pavement structures serve their intended functions with accumulating use (George, et al 1989). To measure and predict the performance of any pavement, a repeatable , well - established and field calibrated condition rating system must be adopted (Shahin, et al 1984). Several methods and approaches have been developed to measure the pavement performance.

Present Serviceability Index (PSI) on a scale form 0-5 has been developed based on the AASHO road test data. It was the first approach to be used for evaluating subjectively the pavement performance. Later, Pavement Condition Index (PCI) on a scale from 0-100 developed by the US Army Corps of Engineers was introduced as a quantitative measure for estimating pavement condition and performance. Other methods such as pavement Condition Rating (PCR) on a scale 0-100 and Pavement Quality Index on a scale 0-10 were also introduced as a performance measuring approaches .

Pavement is a very sophisticated physical structure that responds in a complex manner to the external traffic and environmental loading. This is mainly due to the non - homogenous composition of the asphalt mixture, aggregate and subgarde soil, and the vast variation in traffic and environmental characteristics from a region to another. In the study area, asphalt pavements demonstrated different types of both structural and functional distresses as a result of the combined effect of traffic and climate. High axle loading and high pavement temperature contribute considerably to the formation of

such distresses. The formation of such distresses in the pavements normally leads to the failure of these roads due to the irreversible strain in the roads pavement.

Therefore, the main task of the pavement engineer is to monitor the performance of the roads in service, scheduling the maintenance and rehabilitation works and maintaining the deteriorating roads according to their needs.

3.2 Factors affecting pavement performance

In general, Pavement performance depends on several factors. These factors can be grouped into the following categories:

- ***Traffic loading associated factors***

  These include traffic volumes, axle load, Number of Equivalent Single Axle Loads (ESAL's), tire pressure, truck type axles, configuration, load application time and mechanism.

- ***Material properties and composition***

  These include the main engineering properties of the materials used in pavement construction such as strength or bearing capacity, gradation, mix properties, elastic and resilience modulus and Poisson ratio in addition to the type of the construction material sued.

- ***Environmental associated factors***

  Such factors include temperature, freeze and thaw , humidity and precipitation, and ground water.

- ***Others***

  Such as Geometric features (longitudinal and cross slopes, provision of drainage facilities), design and construction factors such as pavement structure thickness, maintenance level , surface characteristics (micro and macro texture) and the quality of construction works including initial roughness level , and construction joints.

3.2.1 Traffic characteristics

Accurate traffic estimation is needed for both transportation planning and pavement design. In pavement design , traffic is considered as the most important factor. Traffic configuration, magnitude and the number of repetition are normally considered as the basic traffic parameters which can be used as main design inputs for both pavement design and pavement rehabilitation (Huang, 1993).

3.2.1.1 Traffic volume

In this regard, structural distress propagation which takes place in the in- service pavements is largely associated with the continuous traffic growth. The formation of such distresses leads normally to a failure in one of the pavement components as a result of the accumulated stresses and strains in the pavement layers caused by repeated traffic loading.

AASHTO pavement design procedure requires traffic evaluation for both design and rehabilitation. Since the pavement of the new road or that under rehabilitation is usually designed for periods ranging from 10 to 20 years or more, it is necessary to estimate or predict the design loads for this period of time accurately. The accuracy of estimating the design traffic is affected by many factors such as (AASHTO, 1993):

- The correctness of the equivalency factors used to estimate the relative damage induced by the axle loads of different magnitudes and configurations.
- The accuracy of traffic volumes and weight information used to estimate the actual loading projection.
- The prediction of the traffic loading over the design life.
- The interaction of age and traffic as it affects changes in the serviceability level.

3.2.1.2 Equivalent Single Axle Loads (ESAL's) concept

Roads are normally subjected to various types of vehicles of different configurations and different characteristics. In order to calculate the stiffness of a pavement that subjected to loads applied by a moving truck, many road authorities have developed a concept known as an Equivalent Standard Axle Load (ESAL). An Equivalent Standard Axle is defined as " a Single Axle carrying a load of 8.2 tonnes spread over two sets of dual tyres each dual set separated by 300mm.

To account for traffic variation, the anticipated traffic volumes on certain road is normally calculated or estimated in terms of Equivalent Single Axle Load –ESAL expected to use the road during the design life (Figure 3.1).

The Equivalent Standard Axle

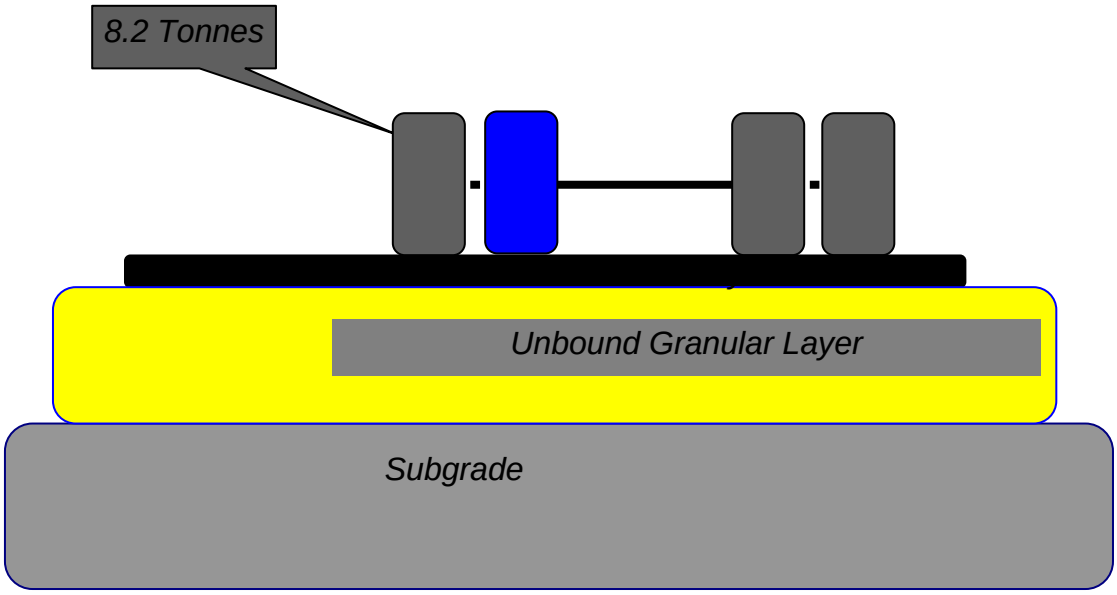


Figure 0.1: Equivalent Standard Axle

The results of AASHO road tests have shown that the damage effect for a passage of an axle of any load can be represented by a number of 18-kip Equivalent Single Axle Loads or ESAL's. Experimentally, it was found that a passage of 12 kip single load is approximately equal to the 0.23 of the damage caused by 18 kip single axle load. The ESAL concept has been applied later to the design equations developed by AASHTO (AASTO,1993).

Three different approaches for calculating traffic volumes for both new roads and roads under rehabilitation are available. These are(Huang, 1993):

- **Fixed traffic**:
  In this approach , the pavement thickness is governed by a single wheel load. It is normally used for airport pavement or for highways pavement with heavy wheel load but light traffic volumes. Presently, This method is rarely used for pavement design.

- **Fixed vehicle**:
  The pavement thickness in this procedure is governed by the number of load repetition of a standard axle load or a standard vehicle. This standard axle load is normally the 18-kips (80kN) single axle load. All axles other than the standard one must be converted to the standard axle to calculate the traffic volumes over the design life. This can be done by multiplying each single or multiple axle by the corresponding equivalent axle load factor (ESALF) for the certain vehicle to obtain the equivalent effect based on 18kips single axle load.

- **Variable traffic and vehicle:**
  This method applies for the mechanistic design procedure where both traffic and vehicle are considered individually. In this procedure, the load of the vehicle can be divided into groups and the response under each load group can be determined separately and used in pavement design.

Roads are designed only for Trucks. Light vehicles and passengers cars are not considered in calculating the design traffic loading. One Truck does as much damage as 8,000 -10000 cars. The increase in damage level with load is not linearly proportional. It takes the shape of the exponential function with a power constant value range from 4 to 6.

Most of the traffic estimation studies consider the 4th power value to calculate equivalent single axle damage for each axle/truck. This is called the “fourth power rule. A moving truck applies a load onto a road which is a function of the total load divided by the

number of wheels in contact with the road. The area of that load (stress) is a function of the air pressure in that tyre.

3.2.1.3 Critical stresses and Strains in pavements

The above details and assumptions related to traffic loading mechanism are needed to compute the critical strains within the pavement developed by the moving truck. Figure 3.2 indicates the position of the critical strains. It is these critical strains within a pavement that cause a pavement to fail structurally. The horizontal tensile at the base of a bound layer (bituminous or cement) causes “fatigue failure” manifested by the readily identifiable “alligator” cracking.

Vertical compressive strain on the subgrade causes “rutting”. Rut Depths are normally measured during profilometry surveys. Instruments such as Falling Weight Deflectometer (FWD) can be used to measure the magnitude of both horizontal tensile strains and vertical compressive strains developed in a particular piece of pavement by a moving truck.

Vertical Compressive strains are inferred by the deflection of the pavement under the load. Horizontal Tensile strains are inferred by the difference of the deflection of the pavement under the load and 200mm away. The stiffer the pavement structure the lower the strains and the greater number of axle loads that the structure will take. Accurate values for these parameters can be computed from the FWD load/deflection data and are used to compute the thickness and stiffness of pavement layers necessary to carry the applied loads.

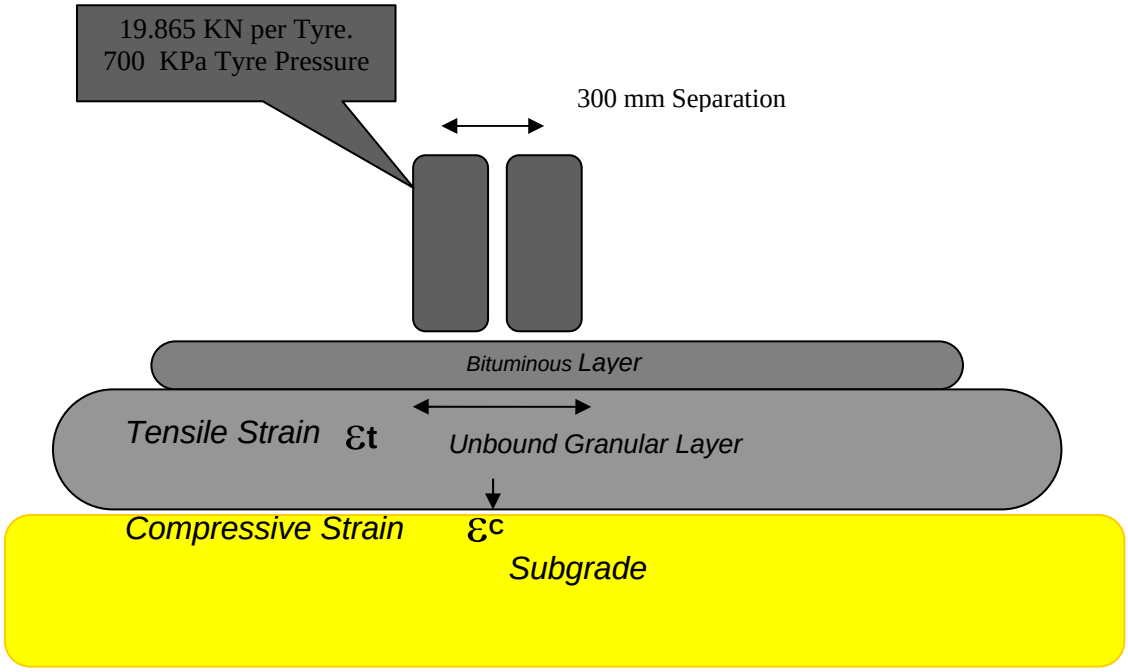


Figure 0.2: Critical Strains within a Pavement

3.2.1.4 Design Loads and Overloading effect

If a truck is carrying a load greater than the 8.2 tonnes per axle, then it is possible to calculate the number of standard axles applied by that truck in a single pass. This can be multiplied by all such trucks that pass over the road in a single year and for 20, 30 or 40 years according to the specified "design life" of the road. Loads heavier than 8.2 tonnes per axle can be converted to ESALs using the "Fourth Power Rule).

Under the "Fourth Power Rule" one truck overloaded by 25% does as much damage to a road as two trucks $(1.25)^4$. Illustrative example is given in Table 3.1 below.

Table 0.1:Example of the damage resulted form overloading

Legal Load	10% Overload	25% overload	Description
55	61	69	Tonnes
13	19	31	Standard Axle Load Repetitions

In Dubai, Traffic is the most important factor that dictates the design of the road structurally and geometrically. Pavement structure, no of lanes, horizontal and vertical alignments and many other roads features are all considerably affected by traffic volumes and vehicle characteristics. Therefore, for the purpose of pavement design and rehabilitation, traffic volumes must be assessed as accurate as possible.

Roads can be classified into different categories based on the cumulative number of ESAL repetition. These categories differ from one country to another. American Association of State Highway and Transportation Officials - AASHTO defines only the low- volume roads as the roads which carry maximum traffic volume between 700,000 to 1000,000 ESAL during a given design period (AASHTO,1993).

Roads classification categories based on ESAL level in Dubai emirate is given in Table 3.2 below. This classification has been established based on data extracted from pavement design inputs for hundreds of road projects carried out in Dubai in the last ten years. As shown in this table, the figure for the low volume roads defined by AASHTO was found to apply for collector and local roads in residential areas where heavy trucks are mainly prohibited from utilizing these types of roads frequently (Al Suleiman et al, 1999).

To standardize the approach to the highway design (geometrically and structurally) and setting maintenance and rehabilitation options, Dubai Emirate has established a well defined road hierarchy that imparts certain characteristics and distinct function to each road class as detailed in Table 3.3 below.

Table 0.2: Roads classification categories based on traffic in terms of ESAL level adopted in Dubai emirate

Traffic Category	ΣESAL in the Design Period	Description Term	Road Class Under this Category
A	≥30, 000,000	High	Freeway, Expressway
B	10,000,000 - 30,000,000	Medium	Arterials
C	1,000,000 - 10,000,000	Low	Collectors, Local Roads

			In Industrial Areas
D	≤1000,000	Very Low	Collector, Local Roads In Residential Areas

Intensive researches efforts have been elaborated throughout the last decades to reach a methodology for traffic calculations and analysis for the purpose of pavement design and rehabilitation. In fact, this field is very sophisticated and needs huge amount of efforts and expertise. Field data is needed almost for every single road project as the traffic volumes and vehicles characteristics vary considerably from on area to another.

3.2.1.5 Truck types and configuration

In the last 10 years, traffic volumes has increased drastically due to the vast industrial, commercial and recreational developments taken place in Dubai emirate. Consequently, the characteristics of the heavy vehicles, types and configurations have changed dramatically to cope with the increase rate in economic and industrial activities , heavy goods and construction materials movements and the individuals traveling demands.

Table 0.3: Dubai functional classification system.

Class	Description and characteristics
Freeway	• Intended for fast and free flowing long distance through traffic. • Full access control by grade separated interchanges
Expressway	• Intended for long distance through traffic. • Full access controlled in general by grade separated interchanges. • Service road normally provided to serve the adjacent land connected to the expressway by free flow ramps
Primary Arterial	• Intended for moderately long distance through traffic. • access generally controlled by at-grade intersections (signalised or roundabout).

Secondary Arterial	• Intended for short to moderate distance through traffic. • Access generally controlled by at-grade intersections (signalised or roundabout).
Collector	• Intended to cater for short trips at relatively low speed. • Minimal access control • Although used for through traffic, access to the adjacent land is very important.
Local	• Intended for short trips only. • No access control. • Access to the adjacent land must be achieved.

The levels of changes in traffic loading characteristics have been monitored and studied in Dubai through conducting a series of manual counts and weigh in motion surveys for the purpose of obtaining realistic figures about traffic loading. These information are usually collected for purpose of utilizing such vital data in designing the pavement of the new roads and proposing the rehabilitation options for the deteriorated in service-roads.

Also, such information are very helpful in studying the behavior of the pavement of different classes under traffic loading and identifying the relative damage contributed by different vehicle types and axles configurations to the pavement structures. Traffic information can also help in the interpretation of the causes of pavement deterioration and shaping the general trend of defects propagation typically caused by traffic action on various roads classes.

In the last few years, many technical field studies have been conducted in the study area to figure out the general frame of traffic loading in volume, configuration, and magnitude in order to obtain more information about the traffic spectrum and wheel load characteristics. Under the keen supervision of the researcher, traffic field surveys have been conducted by the Australian consultant of Snowy Mountains for Engineering Consultancy (SMEC) and ARRB Institute for Transport Research of Australia in December 1998 and May/June 1999 on different road links throughout Dubai Emirate. Weigh in Motion tools and devices were used to record all information on the available vehicles in the study area. Some of the outcomes form this work are included in this study.

3.3 Dubai Standard Axle Groups

Traffic survey studies showed that the traffic composition and loading characteristics in Dubai are slightly different from what is experienced elsewhere in the world. In Dubai there is no axle load limit, therefore, the axle load may reach in some cases to an unaffordable limit. Overloading the axle to a limit that exceeds the allowable axle load limit has resulted in formation of various structural distresses such as alligator cracking and rutting.

For example, it was observed that the load on the axle group of Tandem axle Dual Tire (SADT) reaches sometimes up to 19.5 tons which is considered high in comparison to the standard load on the same axle group which is 8.2 ton. This, of course, has caused severe damage to the pavement structure of many major roads in Dubai specially the expressways and arterials which showed accelerated rate of deterioration in comparison with the pavement of other road classes.

In general, the main axle group configurations recorded in Dubai are the followings:

1. Single Axle Single Tire (SAST)
2. Tandem Axle Single Tire (TAST)
3. Single Axle Dual Tire (SADT)
4. Tandem Axle Dual Tire (TADT)
5. Tri-axle Dual Tire (TRDT)
6. Quad-axle Dual Tire (QADT)

Many types of heavy trucks were recorded in the study area. Most of these trucks are used to haul goods and construction materials from the internal areas to the main industrial zones through the inter-emirate roads. The main truck types configuration observed in Dubai are listed in Table 3.3 below. The

Table 0.4: The truck types and configuration observed in the study area

DM CODE	AXLE CONFIGURATION
T1	SAST X SAST
T2	SAST X SADT
T3	SAST X TADT
T4	SAST X SADT + SADT
T5	SAST X SADT + TADT
T6	SAST X SADT + TRDT
T7	SAST X TADT + TADT
T8	SAST X TADT + TRDT
T9	SAST X SADT + TADT - TADT + TADT
T10	SAST X TADT +TADT - TADT + TADT
T11	SAST X TADT + TRDT - TADT + TADT

Samples for the main truck types observed in the study area are shown in Figure 3.3 below.

DM CODE : T2
2 AXLE RIGID
(SAST X SADT)

DM CODE : T3
3 AXLE RIGID
(SAST X TADT)

DM CODE : T5
4 AXLE ARTICULATED
(SAST X SADT + TADT)

DM CODE : T6
6 AXLE ARTICULATED
(SAST X TADT + TRDT)

Figure 0.3: Illustrations for the truck types observed in the study area.

Also. The main axle group configurations observed in the study area are shown in Figure 3.4.

SINGLE AXLE DUAL TYRE
(SADT)

TANDEM AXLE DUAL TYRE
(TADT)

TRI-AXLE DUAL TYRE (TRDT)

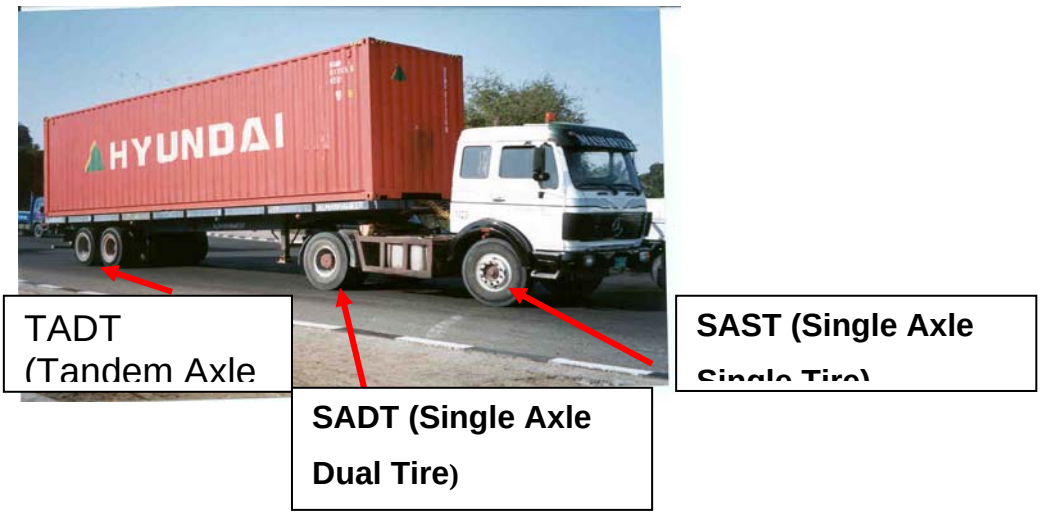


Figure 3.4: Main axle group configurations observed in the study area

3.3.1 Average Axle Factor (Comparison study)

Wheel load surveys have been conducted in the study area to obtain sufficient data about the loading magnitude. Table 3.5 shows the route categories established in Dubai based on route location and truck classification and the weighted average axle factor for each Dubai axle type for each Route Category.

These factors are the average ESAL values determined for the individual axle group in each route category

Table 3.5: Route categories description and Axle factors developed for each axle group in Dubai Emirate.

Axle	AXLE FACTOR FOR ROUTE CATEGORY

	Urban General Freight (1)	Urban-Rural Light Freight (2)	Rural general Freight (3)	Rural Quarry Freight (4)
SAST	1.5	1.3	6.5	1.6
SADT	1.6	2.3	4.3	8.0
TADT	5.9	2.4	4.1	14.6
TRDT	1.6	0.4	3.3	6.3

3.3.2 Average Truck Factor

The average truck factor is calculated from the average axle factors applied to the Dubai Heavy Vehicle Axle Configurations is given in Table 3.6.

These factors are average ESAL values determined for each truck configuration in each route category. Other factors developed AASHTO and FHW are shown in Table 3.7 and Table 3.8 for the purpose of comparison.

Table 3.6: Truck Factors for each truck type classified by Route Category developed for Dubai Emirate.

Truck Type	Axle configuration	Urban General Freight (1)	Urban-Rural Light Freight (2)	Rural general Freight (3)	Rural Quarry Freight (4)
T1	SAST X SAST	3.0	2.6	13.0	3.2
T2	SAST X SADT	3.1	3.6	10.8	9.6
T3	SAST X TADT	7.4	3.7	10.6	16.2
T4	SAST X SADT + SADT	4.7	5.9	15.1	17.6
T5	SAST X SADT + TADT	9.0	6.0	14.9	24.2
T6	SAST X SADT + TRDT	4.7	4.0	14.1	15.9
T7	SAST X TADT + TADT	9.0	6.1	14.7	30.8
T8	SAST X TADT + TRDT	9.0	4.1	13.9	22.5
T9	SAST X SADT + TADT - TADT + TADT	20.8	10.8	23.1	53.4

T10	SAST X TADT +TADT - TADT + TADT	25.1	10.9	22.9	60.0
T11	SAST X TADT + TRDT - TADT + TADT	20.8	8.9	22.1	51.7

Table 3.7: Truck factors suggested by AASHTO, 1993

Vehicle Types	Truck Factor
Passenger Cars	0.0008
Buses	0.6806
Panel and pickup trucks	0.0122
Other 2-Axle/4-Tire Trucks	0.0052
2 axles/6 tires trucks	0.189
3 or More Axle Trucks All Single Unit Trucks	0.1303
3 Axle Tractor Semi-Trailers	0.8646
4 Axle Tractor Semi-Trailers	0.6560
5+ Axle Tractor Semi-Trailers	2.3719
All Tractor Semi-Trailers	
5 Axle Double Trailers	2.3187
6+ Axle Double Tractors	2.3187
All Double Trailer Combos	2.3187
3 axle truck trailers	0.0152
4 axle truck trailers	0.0152
5+ axle truck trailers	0.5317
All truck trailer combos	

Table 3.8: Truck factors on different classes of US Highways developed by FHWA and reproduced by Asphalt Institute (Asphalt Institute, 1981)

Rural Systems				Urban Systems
Vehicle Type	Interstate Rural (Average)	Other Rural (Average)	All Rural Average	All Rural Average
Single-unit trucks				
2-axle, 4-tire	0.02	0.02	0.03	0.03
2-axle, 8-tire	0.19	0.21	0.20	0.26
3-axle or more	0.56	0.73	0.67	1.03
All single- units	0.07	0.07	0.07	0.09
Tractor semi- trailers				
3-axle	0.51	0.47	0.48	0.47
4-axle	0.62	0.83	0.70	0.89
5-axle or more	0.94	0.98	0.95	1.02
All multiple units	0.93	0.97	0.94	1.00

Trucks	0.49	0.31	0.42	0.30

As can be seen from the tables above, Dubai truck factors are much more higher than axle factors experienced in USA and Canada. This is mainly due to the absences of national legislations and traffic regulations that limit the axle load value. Most of the commercial vehicles traveling on the major roads are overloaded much more higher than the permissible limit. Based on the experience of the researcher in the field of pavement design and rehabilitation, traffic volumes in terms of ESAL 's calculated using Dubai truck factors always yield volumes of almost 5 to 7 times of the traffic loading calculated using truck factors developed by Asphalt Institute (AI), AASHTO and FHWA.

The expected damage of such an overloading is enormous on pavement at the long run. To demonstrate this effect, and that the "Fourth Power Rule" state that one truck does as much damage as 8,000 cars it can be seen that one truck 10% overloaded can do the equivalent damage of two trucks. The following table is indicative of the equivalent Standard Axle Repetitions (SAR) that might be expected from this type of traffic. Clearly overloading conditions need to be considered in any assessment of future pavement life and maintenance recommendations.

As a quick glance at the table 3.1 above ,that summarize the effect of the overloading, it shows how considerable effect is generated by an increase in the axle load over the damage level imposed to the pavement.

This means that greater stresses in the pavement layers are introduced and the pavement of the roads subjected to heavy trucks movements deteriorate much more faster than the roads constructed elsewhere for the same design period. Also, this may be used as a justification for what is called a "premature failure" that may occur for certain roads or may occur in future for the pavement structures since the actual accumulating traffic reaches the design traffic level within short period of time causing

the pavement to be defected earlier than expected. The general effect of traffic on pavement performance will be discussed in the next sections.

3.3.3 Tire pressure and loading application time effect

The effect of high tire pressure on the pavement performance is considered major on the primary roads subjected to heavy truck movements. It is well known that as the axle load increases, higher tire pressure becomes more popular for long –haul trucks. (Zhanmin Zhang et al, 2000) . Therefore, High tire pressure increases the value of the primary responses such as stress, strain and deflection and can affect the fatigue life of the pavements.(Hudson et al, 1988)

The loading application time has been found to have more detrimental effect on the pavement deterioration. Pavements of the roads or airports subjected to slow-motion loading mechanism deteriorates faster than those subjected to rapid-motion movement. This means that as the time of load application increases, the deterioration increases (Al Sulieman, et al, 1996, Herrin et al 1986). This is clearly observable in the trafficked areas such as slow lanes used by heavy slow-motion trucks and in the taxiways and aprons in the airports which are also subjected to slow motion airplanes and full load application mechanism. This mechanism always causes more deterioration to the pavement as the total weight of the vehicle or plane load will be fully stressing the pavement layers creating more strains and deflection.

3.3.3.1 Traffic loading effect

traffic loading is considered as the primary factor that affects both Pavement design and performance . Traffic loading characteristics such as axle load magnitude, tire pressure, frequency and duration contribute considerably to pavement deterioration over time.

Despite the fact that traffic loading in pavement design is well formulated and investigated, The method of using axle load in pavement management system as a pavement condition prediction variable is still not well understood. Few studies have discussed the effect of axle loading on pavement performance and how it should be used in pavement management system in an effective manner (Wijk et all, 1998).

A new understanding of the pavement behaviour has been introduced by Shiau et al, 2000). The aim of the study was to interpret the failure of the pavement under repeated traffic loading or what is called the “shake down load limit “. This concept may interprets the behaviour of the pavement systems subjected to both heavy and light traffic. If the pavement is subjected to a repeated load greater than the shake down limit, then the pavement will fail as a result of the excessive plastic deformation. The concept of the shakedown limit can be used as a pavement design parameter for the structures subjected to repeated traffic loadings. I.e. if the design load is made less than the shakedown load limit, incremental failure would not occur and the pavement will perform elastically.

This understanding about the response of the a layered system from repeated moving wheels indicates that if the design load is made more than the shakedown load limit, the pavement may gradually fail by the accumulation of plastic strain, resulting in a form of rutting and surface cracklings. Alternatively , the pavement may purely behave as an elastic structure after the first few cycles after which no further permanent strains will accumulate in the pavement.

Excessive stresses and strains in asphalt surfaced pavement due traffic loading will eventually result in deformation and cracking in different forms and extents. Also, cracking and rutting caused by traffic loading reduce considerably the resistance of the pavement to the eternal traffic and environmental effects by reducing the effective modulus and the thickness. Similarly, stresses induced by traffic effect in the underneath unbound layers (base and subbase) may also result in rutting due to the densification and plastic deformation.

It is very important to realize that the critical tensile and compressive strains under multiple axles are only slight different from those under single axle. I.e. the damage caused by the standard 18-kip (80KN) is almost equal to the damage caused by the 36 –kip (160KN) tandem axles or that of 54-kip (240KN) tridem axles. (Huang, 1993)

Structural distresses (cracking) is normally associated with the tensile stresses or strains in the asphalt material , while rutting , which is also structural distress is

associated with the maximum compressive stresses or strains in the unbound material or at the top of the subgrade.

3.3.4 Materials properties

Pavement performance is highly affected by the properties of the materials used in pavement layers construction. The performance of the asphalt surfaced pavements is affected to certain degree by two material characterization of the following :

- Asphalt mix properties (Asphalt and aggregate)
- Subgrade properties

The above items in addition to the traffic loading are generally control the deterioration trend of the asphalt surfaced pavement. The partial effect of each factor was investigated in this chapter and were handled in much more details in the next chapters which include performance models development.

3.3.5 Engineering properties for Asphalt mix

Asphalt mix is a martial used for construction of roads surface. It is normally consisted of mineral aggregate and a binding material which is asphalt binder. The asphalt binder must have good blending properties such as viscosity, ductility and adhesion to resist cracking so that it maintains stiff when added to the aggregates. The aggregate must be of adequate hardness and angularity in order to resist deformation and abrasion under traffic.

There are many problems associated with evaluating and characterization of the asphalt mix properties. Among these is non homogeneity of the aggregate and the viscoelasticity of the asphalt binder. In addition to that temperature susceptibility and rate of loading. All these parameters must be taken into consideration when selecting the paving material.

Dubai emirates is located in an arid desert area. High summer temperature affects the pavement performance as it soften the asphalt surface leading to permanent

deformation and cracking under the effect of high traffic loading and the absence of axle load limit.

Therefore, special consideration are given to the mixture design so that it can resist rutting and fatigue cracking. This has been achieved through a series of research studies and field investigation which came up with introducing new design criteria manifested by modifying the aggregate gradation, using high viscosity binder such as 40-50 and 60-70 grades

The research efforts paid by the researcher about the pavement performance and the causes of deterioration of the asphalt surfaced pavement in the study area have shown the performance of the roads is affected by the engineering mix properties. These properties are related to mix properties and surface type. These properties are:

a. **Durability**: It is the ability of the material to resist disintegration from traffic, water and temperature. High asphalt content, dense aggregate gradation and well compacted mixture will generally enhance the durability. Distresses such as bleeding, slippage cracking and shoving observed in the study area related to this properties. Improper or high asphalt content will lead to poor performance manifested by appearance of distresses under traffic action.

b. **Stability:** Asphalt mixes with good stability resists the plastic deformation caused by the imposed traffic loading. Asphalt Mixes with low stability will deform under repeated traffic loading causing the pavement to suffer from rutting and corrugation. The level of stability for the asphalt mixes is a function of internal friction and cohesion. Internal friction is attained by good aggregate interlock which is associated to gradation of the aggregate , shape and aggregate surface texture (Micro texture) as explained earlier in chapter 2. Internal friction is reduced by high content of asphalt and always leads to unstable mix as it works as a lubricant between the aggregate particles .

c. **Flexibility**: It is defined as the ability of the asphalt mix to bend elastically under traffic and environmental loading without reaching the cracking point. Proper mix design will fulfill the flexibility requirements . normally, flexibility is

controlled by high asphalt content (within the required limits) , and using open-graded aggregate mix.

d. **Skid resistance:** It is the ability of the aggregate to resist polishing or skidding. Good skid resistant aggregate type should be used in the wearing courses to resist the effect of load and abrasion. Crushed Gabbro material used in Dubai mixes is of good quality and provides good friction quality to pavement surface.

e. **Fatigue resistance:** It is the ability of the material to withstand repeated flexing caused by repeated traffic loading . Normally, Fatigue resistance is improved by using high asphalt content (within the required limits) , and using dense-graded asphalt mix.

f. **Impermeability:** It is the ability of the asphalt mix to prevent the intrusion of the air, water and dust into the voids inside the body of the mix. The effect of the intrusion of air and water is mainly manifested by the appearance of raveling and weathering**.**

3.3.6 Compaction

Most of the desirable properties of the asphalt mixtures are obtained through the compaction process. Stability, air voids, and stiffness are all controlled by the degree of compaction energy exerted on the mixture during construction. Adequate compaction gives the asphalt mixture the needed mechanical interlock and the shear strength and reduce the permeability of the mixture to the water and air.

Compaction can also provide the asphalt mixture with needed resistance to the permanent deformation under the effect of the wheel load and increases the resistance to fatigue cracking resulting from the repeated traffic loading. The effective service life for the asphalt mixture are completely dependent on the acquired proper compaction energy which imparts on the asphalt mixture the engineering properties needed for good performance.

The compaction process in asphalt mixture continues after the completion of the construction work and opening the road to traffic. The viscoelastic properties of the asphalt material keeps the material active and respond to the excess compaction effort or what is called “densification” at the initial few years leading to much more compaction and reducing the percentage of air void. Therefore, care should be taken in dealing with this property in both the lab and during the field construction.

3.3.7 Strength (Elasticity (Resilient) Modulus)

It is well known that under the effect of the wheel loading, most of the asphalt paving mixtures behave elastically and plastically. Usually, Pavement mixtures experience permanent deformation after each load application. As mentioned before, if the applied load is small compared to the strength of the material, and it is repeated for large number of times, the deformation under each load repetition is nearly completely recoverable and can be considered elastic. (Huang,1993) Mainly, the material property that affects pavement performance and the model development is the Modulus of Resilience (MR) . MR of the asphalt mixture is a measure of the material to resist the effect of repeated loading cycles without reaching to the failure limit.

The resilient modulus is also the most important property for characterizing the subgrade in both design and rehabilitation (AASHTO,1993). It is a measure of the elastic property of soil exhibiting non linear characteristics. Modulus of resilient can be used in both mechanistic and empirical approached for designing the pavement structure and predicting rutting, roughness. etc. Methods and techniques for determining modulus of resilient are easy and available at both the lab and in-situ. For asphalt layers, pavements are usually constructed according to the specifications with minor variation created by the quality of the work provided in the initial engineering value allowed by the tolerance limits outlined in the general specifications documents for every single road project.

These values is considerably changed in different level, due to the nature of the asphalt mixtures as being a non- homogenous material of unpredictable behavior after a while as a result of traffic action along the service life of the road. Traffic repetitions will lead to formation of structural and functional distresses such as rutting, alligator cracking and bleeding. The formation of such distresses lead to reduction of the asphalt layers

modulus. More details about the asphalt modulus variation with traffic will be presented in the next chapter.

Pavement performance is also affected by the subgrade strength represented by the value of California Bearing Ratio (CBR%) or by the value of elastic (resilience modulus) . Recently, the subgrade strength started to be reported using the value of Modulus of Elasticity or Resilience(Mr). For many reasons such as (AASHTO,1993):

- It indicates a basic material property which can be used in both mechanistic approaches for analyzing multi layered pavement system.
- It can be used to predict roughness, cracking, rutting etc.
- It is easy to be determined at both lab and field using standard test methods and non destructive testing techniques.

This shift has been also adopted due to the inaccuracies incorporated in the CBR test.

3.3.8 Gradation

Aggregate is characterized by the gradation which imparts to it the main engineering properties. Gradation affects considerably the asphalt mix properties such as stiffness, stability, durability , fatigue resistance and workability.

Aggregate is normally divided into two types, coarse and fine aggregate; fine aggregate is any particle passes sieve no. 4 whilst the coarse aggregate is the one retained by sieve no. 4. The above descriptions is considered as traditional definitions.

In Dubai, the definition is a little bit different, Where the coarse aggregate is defined as the aggregate particles that when placed in a unit volume create void, while the fine aggregate is the aggregate that when placed in a unit volume fill the voids generated by the coarse aggregate. Therefore, the aggregate size is not considered when mix is designed.

In Dubai, there are two asphalt mixes used in pavement construction; Dense Bituminous Macadam (DBM) of which the modulus of resilience was determined in the lab as 310,000 psi ((2100 Mpa) and Asphalt Concrete Pavement (ACP) for which the

modulus of resilience is equal to 245,600 psi (1700 Mpa) tested at temperature 0f 42 C^0 which represents the prevailing asphalt temperature in the Dubai emirate.

DBM is considered a coarse mix while ACP is a fine mix. The aggregate Gradation of the ACP and DBM Mixtures and the specifications for both mixes available in Dubai are listed in Table 3.9 and Table 3.10 below. As can be seen from the tables below, the DBM mixture is coarser mix with maximum size of aggregate of 25mm where as the ACP is a fine mix which has maximum size of aggregate of 19mm.

Table 3.9: Aggregate Gradation of the ACP and DBM Mixtures

Sieve Size	**Dubai AC**	**Dubai DBM**
37.5	100	100
25	100	99.6
19	99	96.3
12.5	76	78
9.5	67	67
4.75	56	43.5
2.36	38	29.6
1.18	25	21
0.6	16	14
0.3	12	9
0.15	6.3	6
0.075	3.4	2.7

Table 3.10: Engineering properties for of the ACP and DBM Mixtures

Mix type / **Criteria**	**ACP**	**DBM**
Stability (KG)	1200	1000
Flow (mm)	2-4	2-4
Stiffness (Stability/Flow) (minimum)	500	320
Voids Filled with Bitumen (VMA%) minimum	15	14-20

Air Void (AV %)	4-8	6-9
Voids Filled with bitumen (VFB)	50-75	48-60
Filler/Bitumen Ratio	0.6-1.4	0.6-1.4

The performance of the both asphalt pavements in Dubai is affected by the characteristics of the materials incorporated in the pavement mix design, engineering properties for the asphalt mix, traffic level and subgrade strength on which the pavement rest.

The above information about the asphalt mixtures adopted in Dubai are given in order to justify the behavior of each mix under traffic loading. The preliminary analysis of the available data showed that these two mixtures performs slightly different. The performance of each mix seems to be highly associated with the gradation and asphalt content.

The course gradation and the low asphalt content of the DBM mix has generated more susceptibility to fatigue cracking and defects caused by water entry; such as depression and raveling, and less susceptibility to permanent deformation or rutting. While the ACP mixture has been found to be more susceptible to rutting due to the relatively high asphalt content. To show the difference in performance for both mixes, Figure 3.5 below was developed based on the data extracted from the field for both mixtures.

As can be seen from this figure, it is clear that the deterioration trend for both mixes is almost the same. This can be referred to the absence of environmental effect on both mixes. DBM could have been experienced more deterioration if the environmental effect is much more sever than existed. It is true that the Gulf region is a hot climate area which suffers from high temperatures, but the effect of temperature will not be so severe if the gradient in temperature variation (day and night, summer and winter) is not that large. In fact temperature cycling mechanism makes the asphalt mixes to crack and deteriorate not the temperature itself.

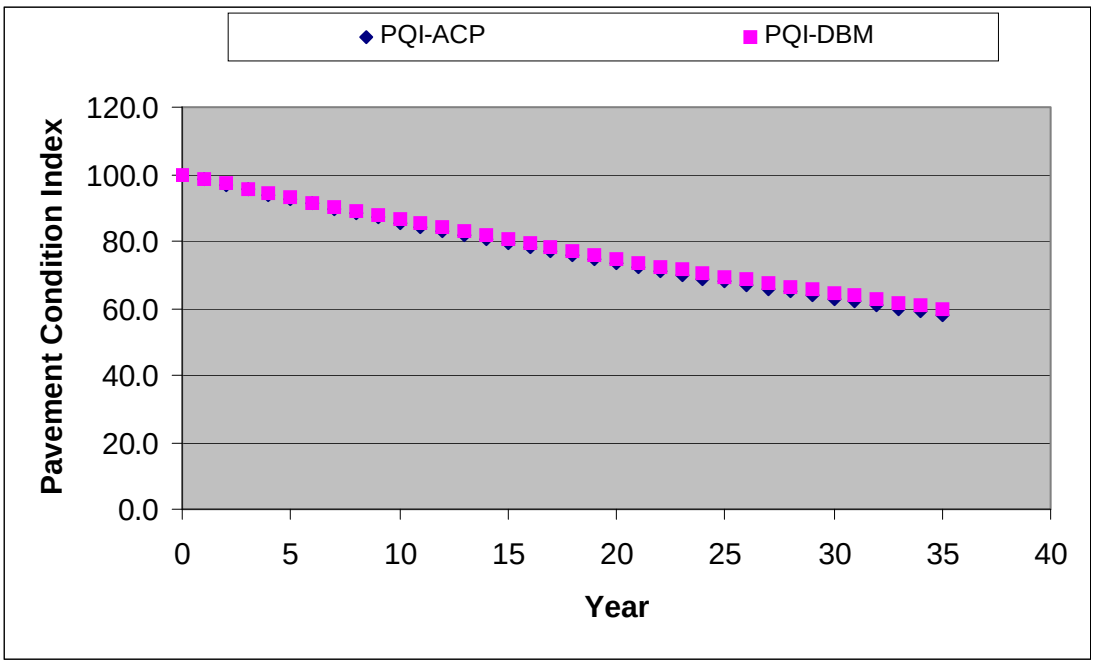


Figure 3.5: DMB and ACP deterioration curves using exponential function.

The main effect of temperature on asphalt pavement performance can be clearly observed in the reduction in the modulus value after years of being in service. The reduction in modulus value affects both mixes. In addition to the above justification, water precipitation is very minimum in this area and the texture is rarely subjected by the water action. Therefore, both mixes behave in almost the same trend and they are still being used in roads construction till present time. More details about this issue will be presented in the following sections.

3.3.9 Environmental effects

3.3.9.1 Temperature and humidity

Temperature is one of the most important environmental factors that affects pavement performance and pavement structural design. It affects creep properties of the asphalt concrete, thermal induced stresses and freezing and thawing of the road bed soil. Temperature data in Dubai for the last 10 years (1994-2004) were collected and

analyzed. The maximum and the minimum temperature value ever recorded in this region was 47 C° and 6 C° respectively.

Maximum Pavement temperature measured at depth of 3-10mm was recorded to be around 74 C° and the average value is around 60 C° . During data collection process, temperature for the asphalt layer, surface and air was measured at the beginning of each test and continuously updated every two hours along the testing period.

As mentioned there before, the effect of the temperature as an climatic factor is manifested by reducing the value of modulus of resilience for the asphalt layers due to the viscoelastic nature of the asphalt material. This material is very susceptible to high temperatures. A lab experiment has been done on the effect of the temperature on Dubai asphalt pavement mixtures showed that asphalt mixes in Dubai are highly affected by the temperature variation. As shown in Figure 3.6, the drop in the MR values is high within the temperature range from 25 to 35. After this point, the drop is relatively small.

The effect of temperature on the unbound material such as granular base and subbase courses is very minimum. No significant indication is found for the temperature variation on modulus of resilience value for the granular materials.

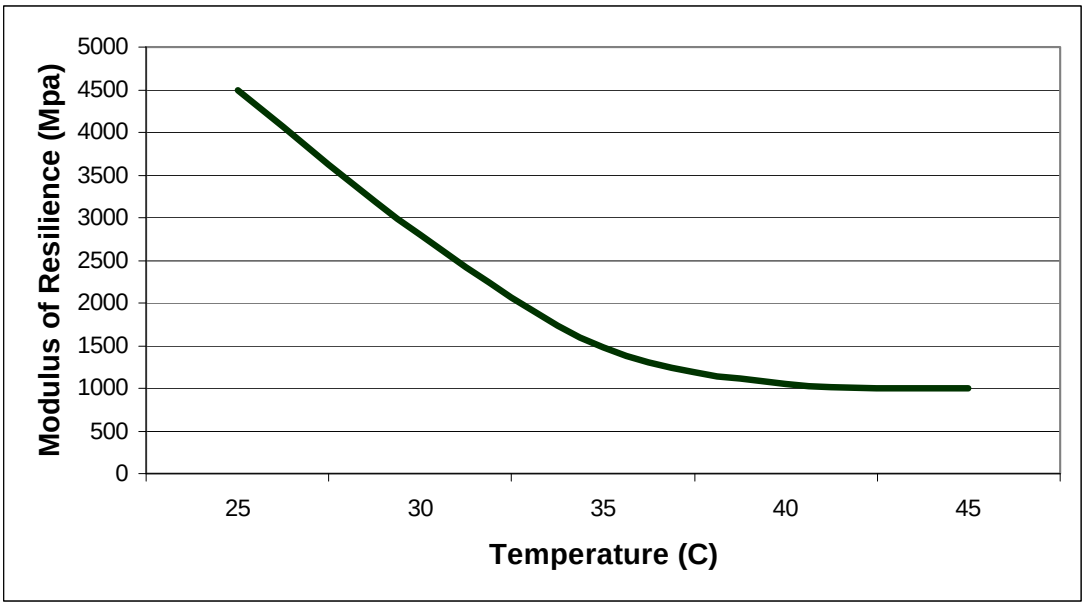


Figure 3.6: Effect of temperature on Resilience Modulus value for ACP and DBM mixtures.

3.3.9.2 Humidity

Humidity is another climatic aspect of Dubai area. It ranges from 50 to 100% saturation in the months of July and August. The effect of this factor is almost negligible as it lasts only for few weeks within the entire year. The main effect of such factor is on the operational characteristics of the pavement surface and traffic movements. Therefore, no more elaboration for the effect of this factor on pavement performance will be considered.

3.3.9.3 Precipitation and Water Table

Rainfall , if allowed to penetrate the pavement structure or the underneath soil will have a detrimental effect on the strength properties of these material.

Water enters the pavement underneath layers through the cracks, joints or as groundwater from the high water table aquifers. Its effect will be manifested in the following:

- Weakening the unbound granular material and reducing its strength
- Reducing the strength of the roadbed soil
- Causing distresses to both asphalt concrete such as swelling, and rigid pavement.
- causing Asphalt stripping

Records related to rainfall in Dubai in the last 10 years shows that the rainy season extends from November to April. The amount of precipitation in this dry arid area range from 9mm to 220mm with average annual precipitation is around 90mm. The effect The water table in this region range from 0 to more than 5 meters with average value in the urban areas , which is very close to the sea shore is around 1.5 m.

Surface Water effect on pavement performance in the study area is very minimum due to low level of precipitation and the provision of good drainage surface system in most

of the urban areas. Sometimes, rain is not seen for many years but short rain showers. Therefore, the effect of such factor on pavement performance is almost negligible.

As for the water table effect, it was found in certain cases that water table created many distress types and defects such as settlement and cracking in the asphalt layers. High water table leads normally to weakening of the strength of the unbound materials and the subgrade soil. Few cases have been found to occur in Dubai as a result of bad construction quality and deviating from abiding to the technical specifications which includes applying the special treatment such as providing a layer of boulders and geotextile material to protect the pavement layers materials. In general, water table problem can not be considered as a factor affecting pavement performance due to the proper cautions taken during pavement construction process.

3.3.9.4 Freeze and thaw

It is one of the major concerns for the pavement design specially in the cold regions. Thawing can reduced the strength of the underneath layers while freezing creates frost heaving causing reduction in serviceability of the pavement. In the study area, there is no such environmental action as Dubai emirate is located in a dry , arid and desert area. Therefore, the study will not concentrate on studying the effect of this factor on pavement performance as it has negligible effect of the performance of the pavement in this area in addition to the unavailability of the data pertains to this element.

3.3.10 Other factors

3.3.10.1 Geometric aspects

Geometric aspects affect pavement performance are manifested by the design of the following elements:

- **Longitudinal and transverse slopes**
  Transverse and longitudinal slopes are provided for the purpose of draining the surface water to the shoulder then by means of drainage facilities the water is drained far away from the pavement body. The effect of longitudinal and

transverse slopes on pavement performance is usually associated with draining the surface water to the edges of the pavement to avoid asphalt stripping and water seepage to the underneath materials. Providing cross and longitudinal slopes is stated and practiced as part of the general geomantic specifications so, all the pavement sections have almost the same characteristics with respect to this features. Few number of pavement sections were found not to have enough slopes in both directions and they were constructed long time ago

- **Drainage system**

  Drainage facility is needed to drain the water away from the pavement structure. Drainage is considered one of the most important factors in pavement design (Huang, 1993). An internal drainage system is not required if the infiltration into the pavement system is less than the drainage capacity of the underneath base and subbase layers in addition to the subgrade .

  Water can inter the pavement structure trough the cracks, joints and shoulder or ground water from high water table etc. mainly the effect of entrapped water in the pavement structure is as follows:

  - It reduces the strength of the unbound base and subbase materials and the subgrade soil.
  - It causes cracking , depression and stripping for the asphalt pavements and pumping for the concrete pavement and may cause shoulder deterioration leading to weakening of the lateral support for the whole pavement body.
  - Cause swelling distress over the swelling soil.

As a simple demonstration to show the effect of providing drainage facilities to the road pavement, the data related to the drainage condition for the expressways and freeways were analyzed by assigning a dummy variable values as 1 if the curb and gutter present and 0 if there is no curb and gutter The general performance trend gave very good observation about the effect of providing drainage facilities on the pavement performance.

As shown in Figure 3.7 below, the pavement sections with curb and gutter were found to be in much more better condition than the condition of the pavement for the road which has no curb and gutter. This is considered practical, as the accumulation of the water on the surface and penetrating the pavement structure through the cracks and affecting the asphalt surface microstructure.

Despite the fact that the precipitation amount in Dubai as mentioned before is very less and does not exceed 90mm per year in average, the provision of drainage facilities still work as a factor to minimize the water accumulation and reduce the risks of high water table effects.

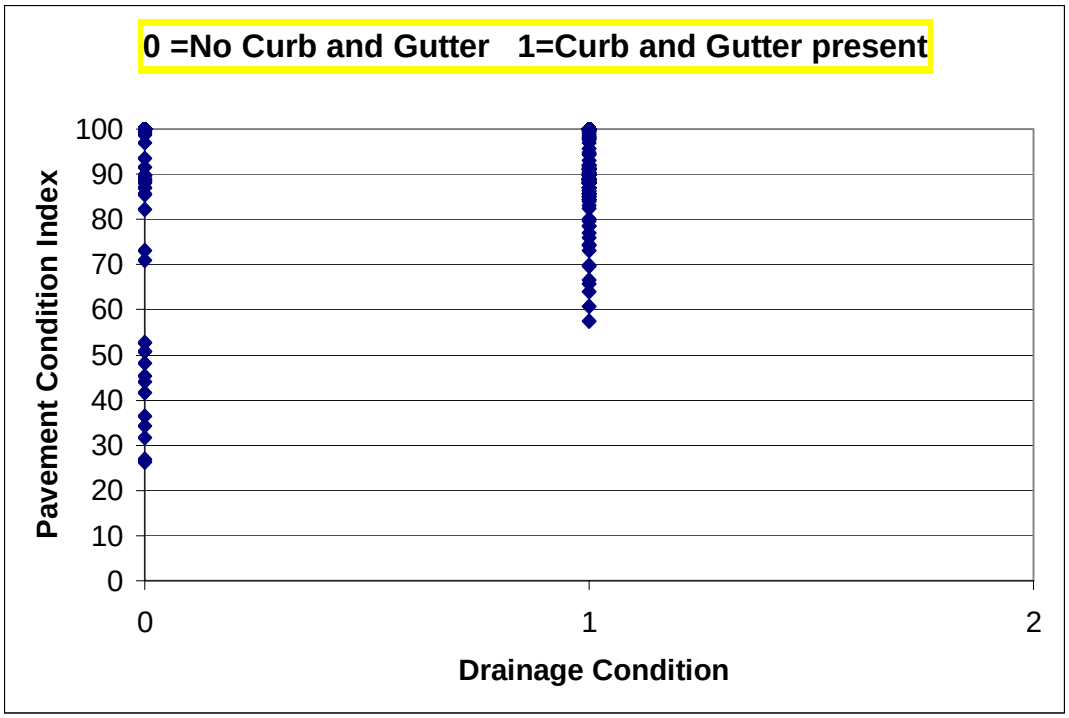


Figure 3.7: Effect of provision of drainage facilities on pavement condition

Despite the importance of discussing in details of various aspects of geometric design and drainage effect, it is viewed that this can be considered outside the scope of the study. But the general effect of providing drainage features can be investigated by considering the difference in pavement performance for roads supported by drainage facilities such as curb and cutter, soak ways, and drainage manholes and those which are not. Fortunately, this type of data is maintained in the PMS system database so it was good chance to investigate the case from this side.

3.3.11 Surface characteristics (Micro texture and Macro texture)

As mentioned in the previous chapters, the performance of pavement is influenced by both microtexture and macrotexture structure of the asphalt surface. Microtexture is generally controlled by the selection of the aggregate type. It refers to the fine-scale texture gritting that is present on the surface of the coarse aggregate and depends on the initial roughness and the ability of the aggregate to retain this surface roughness under traffic (Jayawickrama *et al*,1996). This means that macrotexture is the pavement aggregate component (which controls contact between the tire rubber and the pavement surface). Macrotexture refers to the large-scale texture of the pavement as a whole due to the aggregate particle arrangement.

The effect of these characteristics on pavement performance is controlled by the extent to which the surface material can resist the action of traffic and water. Traffic loading repetitions will result in an abrasion of the aggregate faces and the surface will be slippery and polished. On the other side, entrapping the water particle on the rough surface will results in asphalt stripping and the intrusion of the water and air inside the pavement through the cracks and joints. Part of this aspect has been discussed in the comparison of the DBM and the ACP mixtures. But, in general, as the rainfall amount is very less in the study area, the effect of such properties will not be so clear. More research is needed in this field.

3.3.12 Maintenance level

Pavement maintenance is needed to preserve the pavement from further deterioration . Effective maintenance work will reduce the rate of deterioration and the user cost value. Applying treatment to the pavement structures on time restores the pavement riding quality and structural integrity. In addition to the aforesaid advantages, it increases the life expectancy and prolong the service life for additional few years. Modeling is very helpful if used as a prediction tool in pavement management systems. It can be used to

assign different maintenance and rehabilitation activities to preserve the pavement surface quality and structural integrity.

Preventive maintenance is considered as a strategy designed to arrest light deterioration, retard its failure and reduce the need for routine maintenance activities (Zaniewski, et al ,1999) . The aim of applying such strategy is to extend the functional life of the pavement by applying treatment before the pavement reaches to the condition where the corrective maintenance is needed. The crucial question which is normally asked is ; at what time and at which level the distress the maintenance should be applied?. The answer for this question is still being researched.

Many researchers indicated to the importance of the timing in applying preventive and routine maintenance (Paterson,1985). It was recommended that , the maintenance should be applied before a failure occurs. Good example of such case is “to schedule oil change based on the amount of oil a car burns.”

Models developed for the purpose of predicting pavement performance should always consider the evolution of different distress types and trend of propagation and how they are affected by different types of maintenance applied. As an illustrative example, Figure 3.8 was drawn based on data extracted from the study area to simply shows the effect of applying routine and preventive maintenance; such as surface and full depth patching, crack sealing, on improving the pavement condition . as can be seen from the indicated figure, if the routine maintenance applied at the late age of the section , i.e. the pavement is completely deteriorated, the improvement will be very minimum.

This means that asphalt surfaced pavement suffer from structural distresses in a severe condition do not get benefit form preventive and routine maintenance applications as the pavement condition reaches a state where a major rehabilitation should be applied. Normally, The optimum range of applying the routine maintenance is when the pavement in fair to good condition as indicated in chapter 1. In general, the level of maintenance can affect the pavement performance in a away that it prolong the service life for more few years.

The effectiveness of any maintenance activity can be determined by comparing the condition of the pavement before and the condition after the application of the maintenance activity. Also , evaluation of the rate and the general trend of deterioration as shown in figure 3.8 below can give good indication about this effect.

Maintenance engineer can estimate the benefits gained from applying different types of preventive and routine maintenance activities using an objective evaluation system that takes into consideration all pavement aspects. Also, developing deterioration curves gives the maintenance engineer the ability to predict the remaining life for each pavement section based on age and other variable, plan for the maintenance work and allocate the required budget based on the actual need or based on priority if limited budget is available. This subject will be fully covered in the next chapters .

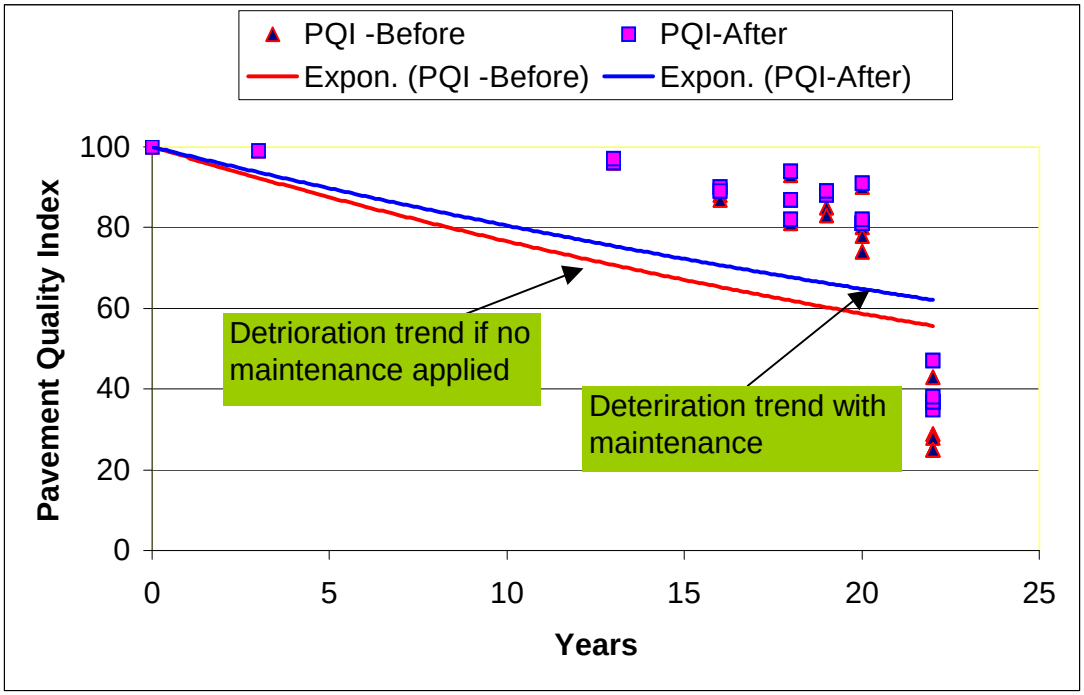


Figure 3.8: The effect of applying routine maintenance (patching and crack sealing) on pavement condition.

3.3.13 Quality of the initial construction work

The quality of the construction works pertains to the pavement plays a crucial rule in determining the deterioration rate and the service life of this element. Engineering properties of the asphalt mixtures such as stability, air voids , gradation and the others are always under control as there are standard procedures and lab tests to assure the

quality of the mixtures provided. Other field aspects such as roughness , segregation, disintegration, delamination .etc are not easy to be managed in the same trend all over the length of the roads under construction. This may partially due to the absence of well established methods to control the quality of such aspects during the field construction.

Initial Roughness is considered as one of the most important condition indicators that dictates the deterioration rate for the roads pavements. Due to the vast variations in the quality of the finished surface and the initial roughness for the newly constructed roads from road to another , it is always expected that these roads behave differently . Many studies investigated the effect of the initial roughness value of the deterioration trend of the pavement sections and found that it plays a crucial rule.

The effect of initial roughness on pavement performance depends mainly on the traffic characteristics and the local standards practiced in each country. Many models to show this relationship have been developed and found to be a good prediction tool for the roughness deterioration over time.

Based on the collected data from the study area, it was found that the roughness deterioration value is proportionally affected by the initial roughness level for the roads subjected to traffic loading. This is mainly due to the dynamic wheel impact of the heavy trucks operating on the roads which increases the pavement deterioration. Dynamic wheel load can elevate the damage experienced by the pavement more than that generated by the static axle load operating on roads with smoother surface (Hassan *et al*, 1999). More details about this subject will be given in the next chapters within the subject concerns the roughness as a condition indicator.

Present Serviceability Index (PSI) and International Roughness Index in conjunction with the initial values of the roughness or PSI can be used effectively to predict the expected service life for the roads under service. Based on data collected from hundreds of pavement sections in the study area, it was possible to establish a kind of relationship between the initial value of the roughness and the expected service life of the pavement.

Usually, pavements constructed with initial serviceability less than the required standard value of PSI say 4.2, which represents the initial serviceability level suggested by AASHTO and adopted in the study area,. It is expected that pavement will serve a period of time less than the time span for which it was originally designed (which is usually selected as 15 years in the study area). This is completely true as the deterioration rate for pavements with initial low serviceability level is expected to be more due to the inherent structural discrepancies and surface irregularities embodied in the pavement structure and the sever effect of the dynamic wheel loading as explained there before.

Applying this rule to the roughness aspect, the researcher was able to establish the deterioration line for the pavement roughness and the expected service life for the roads that do not fulfill the required standards. Based on the analysis, pavements with initial high roughness will deteriorate more and the actual service life will be less than the design life. The details of which will be furnished in the next chapters

The initial standard value for the IRI adopted in the study area is 0.9m/km. All the new roads under construction must satisfy this limit. The terminal value for the roughness which indicates that the road is no more serviceable has been established in Dubai as being equal to IRI=3.0. Monitoring and analyzing the data pertains to the roads with different initial IRI values more and less than the standard limit indicated that , pavements with high initial IRI value experienced much more rate of deterioration than those of lesser values.

The estimated reduction in service life resulted from high initial values can be estimated easily using the following equation formulated by the researcher. It depends on the assumption that roughness is another manifestation of the serviceability. Therefore, as the roughness gets high , the service life will be less. To illustrate this concept. let us assume the followings:

- Initial IRI (IRI_{Std}) = 0.9m/km
- Actual initial IRI = IRI_{Act}
- Terminal IRI (IRI_{Term}) =3.4 (This value corresponds to AASHTO standards of terminal PSI=2.5 and calculated based on a relationship between the PSI and the IRI developed by the researcher for Dubai roads)

Original Design life for the pavement()= 15 years.

Actual Service Life=N_{Act}

The actual service life N_{act} can be estimated using the following formula:

NAct = NDes *{[IRIact —IRITerm]/[IRIStd- IRITerm]} ……1

substituting the terminal IRI value as=3.4 , the formula can be re-written as follows:

NAct = NDes *{[IRIAct —3.4]/[IRIStd- 3.4]}

$$N_{Act} = 15 * \{[IRI_{act} - 3.4]/[0.90 - 3.4]\}$$

The application of this equation is shown in Figure 3.9 below. As shown in this figure, the expected life decreases as the initial roughness value increases. This gives a very good indication about the effect of the quality of the initial works on the performance of the pavement under traffic and environment loading.

Figure 3.9: Estimated reduction in pavement service life (N_{act}) due to high initial roughness level

Finally, one would like to indicate that pavement performance prediction and analysis is a very complex field and it is not an easy task to account for all factors affecting its performance. Different material types of different characteristics are incorporated in the asphalt mixture and this make the task of investigating the partial effect of each element is a difficult mission. Despite of all these complexities, the level of understanding of the asphalt pavement behavior is improving day after day. This may due partially to the efforts being paid by different organizations and establishments to standardize the methods of testing, analysis and evaluation.

CHAPTER 4 PAVEMENT CONDITION INDICATORS AND DETERIORATION MODELS

Introduction

Distress types and failures observed in pavement structure are the physical signs or symptoms of the inherent problems caused by various Inter correlated factors affecting pavement performance outlined in chapter 3. Many organizations and road agencies have established their own distress identification manuals supported by detailed description of each distress, method of formation, severity levels, possible causes and the recommend treatment.

Distress data are essentially needed in PMS to assess the current condition of pavement . it is hard to accept any pavement management system that does not incorporate defects data. Information related to the current condition are used to determine the maintenance and rehabilitation measures which are dedicated to:

- Retard further condition deterioration to extent where major maintenance is needed.
- Prevent acceleration of the distress
- Improve the riding quality and enhance the structural integrity of the pavement.

Distress surveys are required to be carried out periodically. The condition of the pavement can change dramatically over a short period of time especially in severe weather. Therefore, distress information are required to be updated continuously to reflect the current pavement condition to enhance the credibility and the reliability of the maintenance work programs generated by the PMS.

Inventory data related to the pavement section and its surrounding are also required to be collected along with the distress data as these data are used in the selection of suitable maintenance alternative. For example, the presence of sewerage manholes, damaged edges of the kerbs and utility manholes covers are extremely important to the maintenance engineers to decide whether to select an overlay or mill and inlay of the pavement particularly in urban area.

Common defects In flexible pavements

The most worldwide distress manual that has a complete information about each distress in flexible pavement for both roads and airports has been developed by US Army Corps of Engineers (Shahin, 1981). This manual has been adopted later by the Federal Aviation Authority (FAA) and the American Society and Materials Testing (ASTM) under designation ASTM D5340-03 for the airport pavements and ASTM D6433-03 for the roads and parking lots. In general, pavement experts were able to identify around 19 distress types that may occur in flexible pavements.

Table 1 below summarizes the types of different distresses and possible causes of each distress. A detailed description of each distress, severity levels and the possible causes supported by illustrative photos for each distress at different severity level were re-produced by the researcher based on the available references and based data extracted from the field (Shahin, 1981, Shahin 1998, PMS Unit, 2003). Distresses such as alligator cracking and rutting are always associated with traffic loading .Bleeding is always related to improper mix design , weathering and raveling is always associated with the harsh environmental factors.

All pavement management systems contain a rating system to estimate the negative effects of the distresses on the general pavement condition. At the initial life of the pavement, distress occurrence may be delayed for several years till the cumulative effect of both environment and traffic loading become critical . Distress starts at low severity, by time passing, the severity may change to moderate and then to high depending on the traffic level and the severe effect of the environmental factors such as rain or temperature cycling.

Table 4.1: List of common distress types in flexible pavement (Shahin, 1990)

Distress Type	Measurement Unit	Severity levels	Possible Causes
Alligator Cracking	Area (m2)	Low, Medium, High	Load
Rutting	Area (m2)	Low, Medium, High	Load
Potholes	number	Low, Medium, High	Load
Corrugation	Area (m2)	Low, Medium, High	Load

Depression	Area (m2)	Low, Medium, High	Load
Slippage Cracking	Area (m2)	Low, Medium, High	Load ,Others
Edge Cracking	Linear meter	Low, Medium, High	Load
Patching	Area (m2)	Low, Medium, High	Load ,Others
Bleeding	Area (m2)	Low, Medium, High	Climate/Mix
Block Cracking	Area (m2)	Low, Medium, High	Climate/Mix
Joint Reflection Cracking	Linear meter	Low, Medium, High	Climate/Mix
Swell	Area (m2)	Low, Medium, High	Moisture/Drainage
Weathering and Raveling	Area (m2)	Low, Medium, High	Climate/Mix
Bumps and sags	Linear meter	Low, Medium, High	others
Lane Shoulder Drop off	Linear meter	Low, Medium, High	others
Polished aggregate	Area (m2)	One dimension	load
Longitudinal and transverse cracking	Linear meter	Low, Medium, High	Climate/Mix
Rail road crossing		Low, Medium, High	others

The area or the density of each distress can be estimated or predicted for each pavement section by analyzing the historic data about that section. Different model forms can be used to simulate the growth of the distress over time at each severity level which depends on both the climatic factors and the traffic characteristics. The interference time zone between the lower severity and the next one can be estimated based on the general rule that the sum of the distress areas or density should be equal to 100.

Most of the pavement management systems developed by various highway organizations include a general distress performance models. This is mainly due to the shortage of information and unavailability of the data pertain to the historical performance of the distress in each pavement section.

It is recommended that at long term planning that each organization collect sufficient data to model the behavior of every single distress which gives much more accurate estimation for the pavement condition and the level of maintenance needed in the future as it can predict the distress density much more better than the overall pavement condition index.

Therefore, pavement management systems that have the ability to calculate the individual distress index and the pavement performance index will perform better than those depend on the general performance index. The ability to calculate distress index gives insight look about the major problems that each pavement section suffers from.

Common distress types recorded in the study area

The most frequent distress types that have been detected in the study area are shown in Table 4.2 below. The first 9 distress were found to occur frequently in the study area. Other distress types such as Block cracking, Polished aggregate were observed but with very less frequency. Analyzing the distress types experienced in the study area which suffers from high temperature and desert conditions indicate that most of the distress recorded are associated with three main factors:

1. **Traffic loading**: Such as alligator cracking, rutting
2. **Climate**: Such as L&T cracking, Bleeding and weathering and raveling.
3. **Others**: Such as Patching and shoving.

Description and Photos that show common distress types in Dubai at different severity levels extracted from different pavement evaluation cycles conducted in the last ten years are given below:

The description, severity levels, how to measure are extracted mainly from the most well known references in this field (Shahin, 1990, Shahin 1998).

This manual also includes the description of the other distress types that were not found to occur in the study area.

Table 4.2: Frequency of distress types observed in the study area

Distress No.	Distress Types	Frequency	Rank
1	Alligator cracking	9.9	5
2	Bleeding	11.4	3
3	Depression	1.91	9
4	Longitudinal and Transverse Cracking	8.72	6
5	Patch and utility cut patching	44.2	1

6	Potholes	2.6	7
7	Rutting	11.3	4
8	Weathering and Reveling	30.87	2
9	Shoving	2.2	8
10	Block cracking	0	
11	Polished aggregate	0	
12	Bumps and sags	0	
13	Corrugation	0	
14	Railroad crossing	0	
15	Edge cracking	0	
16	Joint reflection cracking	0	
17	Lane shoulder drop off	0	
18	Slippage cracking	0	
19	Swelling	0	

Alligator Cracking (Fatigue)

Alligator or fatigue cracking is a series of interconnecting cracks caused by fatigue failure of the asphalt concrete surface under repeated traffic loading. Cracking begins at the bottom of the asphalt surface (or stabilized base) where tensile stress and strain are highest under a wheel load. The cracks propagate to the surface initially as a series of parallel longitudinal cracks. After repeated traffic loading, the cracks connect, forming many sided, sharp-angled pieces that develop a pattern resembling chicken wire or the skin of an alligator. The pieces are generally less than 0.5m on the longest side. Alligator cracking occurs only in areas subjected to repeated traffic loading such as wheel paths, (Pattern-type cracking that occurs over an entire area not subjected to loading is called “block cracking”, which is not load-associated distress).

Severity Levels

- L - Fine, longitudinal hairline cracks running parallel to each other with no, or only a few interconnecting cracks. The cracks are not spalled (Figure 1a).
- M - Further development of light alligator cracks into a pattern or network of cracks that may be lightly spalled (Figure 1b).

- H - Network or pattern cracking has progressed so that the pieces are well defined and spalled at the edges. Some of the pieces may rock under traffic (Figure 1c).

How to Measure

Alligator cracking is measured in square meters of surface area. The major difficulty in measuring this type of distress is that two or three levels of severity often exist within one distressed area. If these portions can be easily distinguished from each other, they should be measured and recorded separately. However, if the different levels of severity cannot be divided easily, the entire area should be rated at the highest severity present. If alligator cracking and rutting occur in the same area, each is recorded separately as its respective severity level.

Low severity alligator cracking (Figure 1a)

Medium severity alligator cracking (Figure 1b)

High severity

alligator cracking(Figure 1c)

Bleeding

Bleeding is a film of bituminous material on the pavement surface that creates a shiny, glass like, reflecting surface that usually becomes quite sticky. Bleeding is caused by excessive amounts of asphalted cement or tars in the mix. Excess application of a bituminous sealant and, or low air void content. It occurs when asphalt fills the voids of the mix during hot weather and then expands onto the pavement surface. Since the bleeding progress is not reversible during cold weather, Asphalt or tar will accumulate on the surface.

Severity Levels

- L - Bleeding has only occurred to a very slight degree, and is Notice able only during a few days of the year. Asphalt does not stick under shoes or vehicles (Figure 2a).
- M - Bleeding has occurred to the extent that asphalt sticks to shoes and vehicles during only a few weeks of the year (Figure 2b).
- H - Bleeding has occurred extensively and considerable asphalt sticks to shoes and vehicles during at least several weeks of the year (Figure 2c).

How to Measure

Bleeding is measured in square meters of surface area.

Low Severity Bleeding (Figure 2a)

Medium Severity Bleeding (Figure 2b)

High Severity Bleeding (Figure 2c)

Depression

Depressions are localized pavement surface areas with elevations slightly lower that those of the surrounding pavement. In many instances light depressions are not noticeable until after a rain. When pounding water creates a “birdbath” area on dry pavement, depressions can spot by looking for stains caused by pounding water. Depressions are created by settlement of the foundation soil or are a result of improper construction. Depressions cause some roughness and when deep enough or filled with water, can cause hydro pounding.

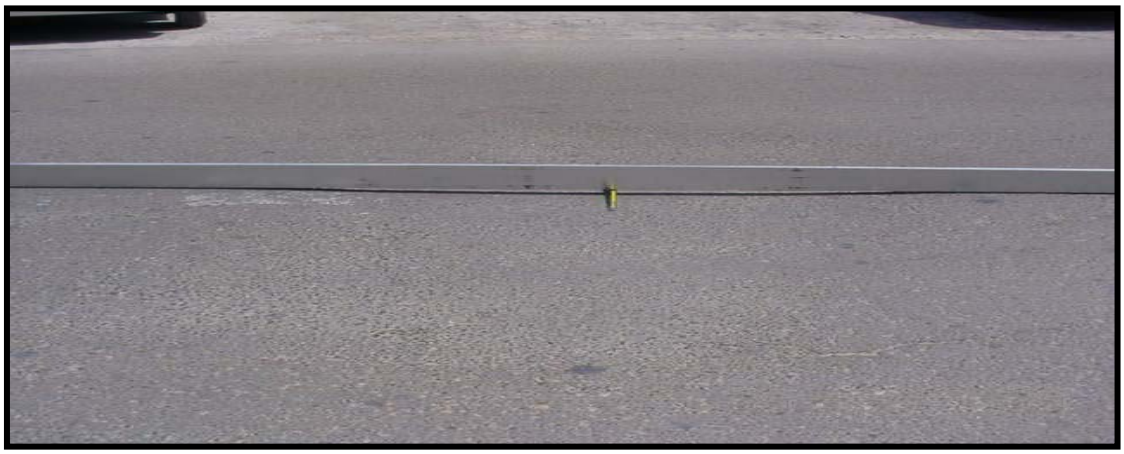
Severity Levels

Maximum Depth of Depression

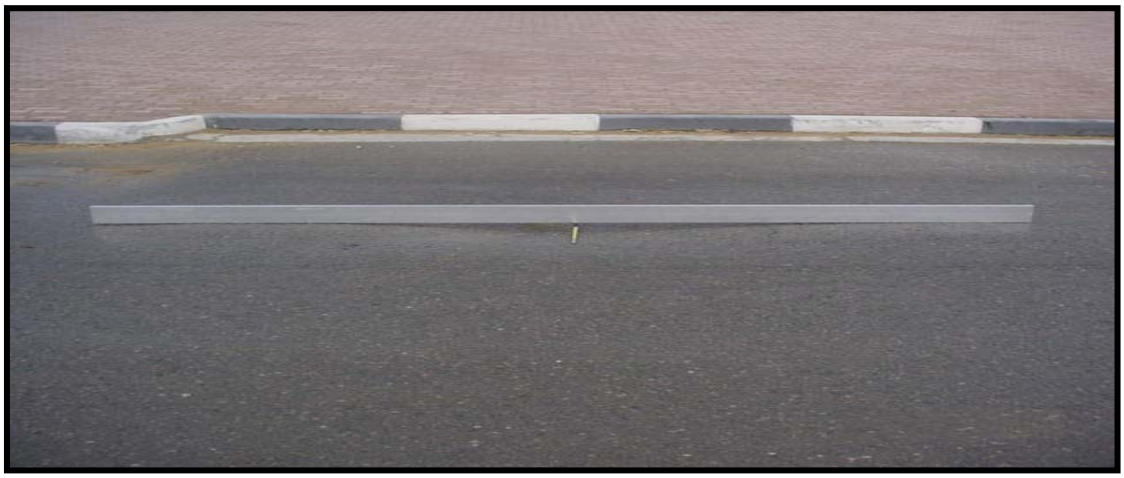
- L 13 to 25 mm (Figure 3a).
- M 25 to 50 mm (Figure 3b).
- H more than 50 mm (Figure 3c).

How to Measure

Depressions are measured in square meters of surfaces area.



Low Severity Depression (Figure 3a)



Medium Severity Depression (Figure 3b)

High Severity Depression (Figure 3c)

Longitudinal and Transverse Cracking

Longitudinal cracks are parallel to the pavement's centerline or lay down direction. They may be caused by:

- A poorly constructed paving lane joint.
- Shrinkage of the AC surface due to low temperature or hardening of the asphalt and/or daily temperature cycling.
- A reflective crack caused by cracking beneath the surface course, including cracks in PCC slabs (but not PCC joints).

Transverse cracks extend across the pavement at approximately right angle to the pavement centerline or direction of lay down. These types of cracks are not usually load-associated.

Severity Levels

- L One of the following conditions exists (Figure 4a): (1) non-filled crack width is less than 10 mm, or (2) Filled crack of any width (filler in satisfactory condition).
- M One of the following conditions exists (Figure 4b): (1) non-filled crack width is greater than or equal to 10 mm and less than 75 mm; (2) non-filled crack is less than or equal to 75 mm (3 in.) surrounded by light and random cracking. Or (3) filled crack is of any width surrounded by light random cracking.
- H One of the following conditions exists (Figure 4c): (1) any crack filled of non-filled surrounded by medium- or high- severity random cracking, (2) non-filled crack greater than 75 mm, or (3) a crack of any width where approximately 100 mm of pavement around the crack is severely broken.

How to Measure

Longitudinal and transverse cracks are measured in linear meters. The length and severity of each crack should be recorded. If the crack does not have the same severity level along its entire length, each portion of the crack having a different severity level should be recorded separately.

Low Severity Longitudinal and Transverse Cracking (Figure 4a)

Medium

Severity Longitudinal and Transverse Cracking (Figure 4b)

High Severity Longitudinal and Transverse Cracking (Figure 4c)

Patching and Utility Cut Patching

A patch is an area of pavement that has been replaced with new material to repair the existing pavement. A patch is considered a defect no matter how well it is performing (a patched area or adjacent area usually does not perform as will as an original pavement section). Generally, some roughness is associated with this distress.

Severity Levels

- L - Patch is in good condition and satisfactory. Ride quality is rated as low severity or better (Figure 5a).
- M - Patch is moderately deteriorated and/or ride quality is rated as medium severity (Figure 5b).
- H - Patch is badly deteriorated and/or ride quality is rated as high severity. Needs replacement soon (Figure 5c).

How to Measure

Patching is rated in square meters of surface area. However. If a single patch has areas of differing severity, these areas should be measured and recorded separately. For example a 2.5 square meter patch may have 1 square meter of medium severity and 1.5 square meter of low severity, then these areas would be recorded separately. Any distress found in a patched area will not be recorded; however. Its effect on the patch will be considered when determining the patch's severity level. No other distresses (e.g.. shoving and cracking) are recorded within a patch; even if the patch material is shoving or cracking, the area is rated only as a patch. If a large amount of pavement has been replaced, it should not be recorded as a patch, but considered as new pavement (e.g.. replacement of a complete section).

Low Severity Patching and Utility Cut Patching (Figure 5a)

Medium Severity Patching and Utility Cut Patching (Figure 5b)

High Severity Patching and Utility Cut Patching (Figure 5c)

Potholes

Potholes are small—usually less than 750 mm in diameter—bowl-shaped depressions in the pavement surface. They generally have sharp edges and vertical sides near the top of the hole. When holes are created high severity alligator cracking, they should be identified as potholes. Not as weathering.

Severity Levels

The levels of severity for potholes less than 750 mm in diameter are based on both the diameter and the depth of the pothole according to Table 1 below.

	Average Diameter (mm)		
Maximum Depth of Pothole	100 to 200 mm	200 to 450 mm	450 to 750 mm
13 to ≤ 25 mm	L	L	M
>25 and ≤ 50 mm	L	M	H
>50 mm	M	M	H

If the pothole is less than 750 mm in diameter, and the depth is less than 25 mm, then hole is considered medium severity. If the depth is more than 25 mm, then they are considered high severity (Figures 6a through 6c).

How to Measure

Potholes are measured and determined in square meters that are low, Medium, and high severity and recording them separately.

Low Severity Pothole (Figure 6a)

Medium Severity Pothole (Figure 6b)

High Severity Pothole (Figure 6c)

Rutting

A rutting is a surface depression in the wheel paths. Pavement uplift may occur along the sides of the rut, but. In many instances, ruts are noticeable only after a rainfall when the paths are filled with water. Rutting stems from a permanent deformation in any of the pavement layers or sub grades usually caused by consolidated or lateral movement of the materials due to traffic load.

Severity Levels

Mean Rut Depth

- L 6 to 13 mm (Figure 7a)
- M >13 to 25 mm (Figure 7b)
- H >25 mm (Figure 7c)

How to Measure

Rutting is measured in square meters of surface area and it severity is determined by the mean depth of the rut (see above). The mean rut depth is calculated by laying a straight edge across the rut, measuring its depth, then using measurements taken along the length of the rut to compute its mean depth in millimeters.

Low Severity Rutting (figure 7a)

Medium

Severity Rutting (Figure 7b)

High Severity Rutting (Figure 7c)

Weathering and Raveling

Weathering and raveling are the wearing away of the pavement surface due to a loss of asphalt or tar binder and dislodged aggregate particles. These distresses indicate that either the asphalt binder has hardened appreciable or that a poor-quality mixture is present. In addition, raveling may caused by certain types of traffic, example tracked vehicles, softening of the surface, and dislodging of the aggregates due to oil spillage and also included under raveling.

Severity Levels

- L Aggregate or binder has started to wear away in some areas, the surface is starting to pit (Figure 8a). In the case of oil spillage the oil stain can be seen, but the surface is hard and cannot be penetrated with a coin.

- M Aggregate or binder has worn away. The surface texture is moderately rough and pilled (Figure 8b). In the case of oil spillage, the surface is soft and can be penetrated with a coin.

- H Aggravate or binder has been worn away considerably, the surface texture is very rough and severity pilled the pitted areas are less than 10 mm in diameter and less than 13 mm deep (Figure 8c); pilled areas larger than this are counted as pothole. In this case oil spillage, the asphalt binder has lost its binding effect and aggregate has become loose.

How to Measure

Weathering and raveling are measured in square meters of surface area.

Low Severity Weathering and Raveling (Figure 8a)

Medium Severity

Weathering and Raveling (Figure 8b)

High Severity

Weathering and Raveling (Figure 8c)

Shoving

Shoving is a permanent, longitudinal displacement of a localized area of the pavement surface caused by traffic loading. When traffic pushes against the pavement, it produces a short, abrupt wave in the pavement surface. This distress normally occurs only in unstable liquid asphalt mix (cutback or emulsion) pavements. Shoves also occur where asphalt pavements above PCC pavement; the PCC pavements increase in length and push the asphalt pavement, causing the shoving.

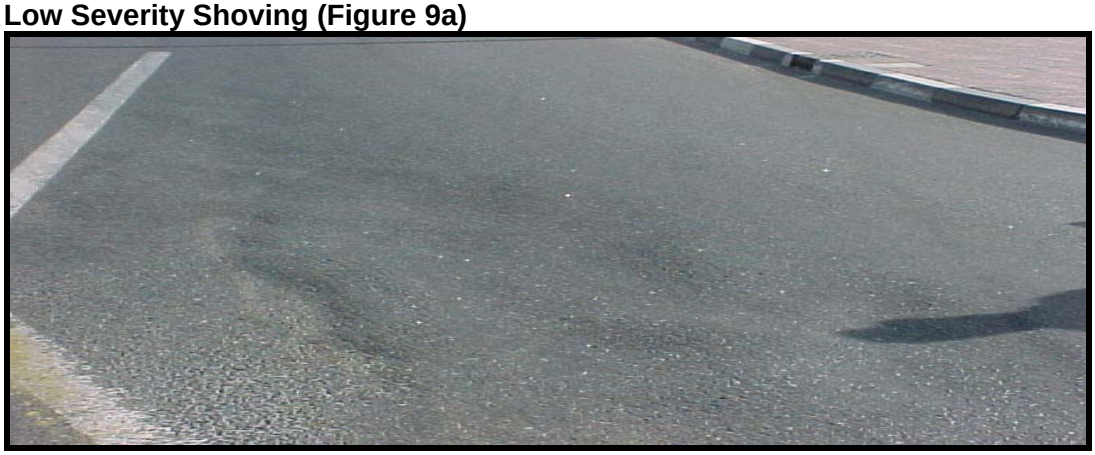
Severity Levels

- L Shove causes low-severity ride quality (Figure 9a)
- M Shove causes medium-severity ride quality (Figure 9b)
- H Shove causes high-severity ride quality (Figure 9c)

How to Measure

Shoves are measured in square meters of surface area. Shoves occurring in patches are considered in rating the patch, not as a separate distress

Low Severity Shoving (Figure 9a)



Medium Severity Shoving (Figure 9b)

High Severity Shoving (Figure 9c)

Pavement – Age performance models

Pavement performance models are considered crucial component of the pavement management system as it enable the maintenance engineers and decision makers to plan for the next years and to establish maintenance works program based on the predicted pavement condition. In this regard, age is considered the crucial factor in predicting the pavement deterioration as it implicitly include the effect of both cumulative traffic loads and environmental factors . Each road family behaves differently, therefore, separate models should be developed for each.

To investigate this effect in the study area, data pertain to more than 9000 pavement sections were analyzed based on differences in traffic level implied by the class and the use of the road. The general trend of the pavement deterioration over time under different traffic levels shows that almost each class behaves differently.

Similar trend of deterioration was found to exist between freeways and expressways classes, which are subjected to the same level of traffic, so it was decided to develop one deterioration model for both of them. As most of the data pertaining to the same climatic region, it was found that climate does not vary that much in the study area and the need to divide the study area into different climatic regions was deemed to un warranted.

Based on the results of the correlation between pavement performance and traffic loading shown in Table A -1, Appendix A, different transformations functions were tried to simulate the deterioration trend of the data as shown in the Figures 4.1 to 4.4. Also, transformation is warranted to facilitate calculations of the models coefficients through the use of non- linear regression. The general trend of the data indicated that several equation forms were found to fit data trend and can be used as a for prediction purposes.

Linear ,Exponential, Logarithmic, Power and polynomial functions were all tried as many research works have indicated the suitability of such functions to represent the pavement deterioration trend. As can be seen from Figures 4.1 to 4.4 below,

exponential function and polynomial function were found to have good fitness with general data trend with sufficient accuracy and satisfy the general boundary conditions applied to the deterioration of the pavement system. The statistical characteristics of these models are listed in Tables A-2 to A-5, in Appendix A. Finally, for the use of PMS , the following models were developed.

Freeways and Expressways

$PQI=100e^{-0.0259\ Age}$ $R^2=0.676$ 4.1

$PQI=100-0.0503\ Age-0.065Age^2$.....$R^2=0.783$ 4.2

Arterials

$PQI=100e^{-0.0229\ Age}$ .....$R^2=0.540$ 4.3

$PQI=100-0.221\ Age-0.07Age^2$..... $R^2=0.813$ 4.4

Collectors

$PQI=100e^{-0.01\ Age}$ .....$R^2=0.64$ 4.5

$PQI=100-0.0837\ Age-0.035Age^2$.....$R^2=0.803$ 4.6

Local-Industrial/Commercial

$PQI=100e^{-0.015\ Age}$ .....$R^2=0.515$ 4.7

$PQI=100-0.408\ Age-0.035Age^2$..... $R^2=0.76$ 4.8

Local Residential

$PQI=100e^{-0.011\ Age}$ .....$R^2=0.676$ 4.9

$PQI=100-0.276\ Age-0.030Age^2$.....$R^2=0.743$ 4.10

In this regards, selection of certain function to represent the general deterioration trend for any pavement system should satisfy the following boundary conditions:

1. The Pavement Quality Index at zero age should be equal to the maximum limit of the rating scale i.e. 10 or 100 , whatever adopted.
2. The minimum value for the condition index should not be negative at any value of the pavement age.
3. It should have high value of Coefficient of Multiple Determination (R^2) which means good predictability.

4. The number of the Independent variables should be kept minimum and easy to measure.

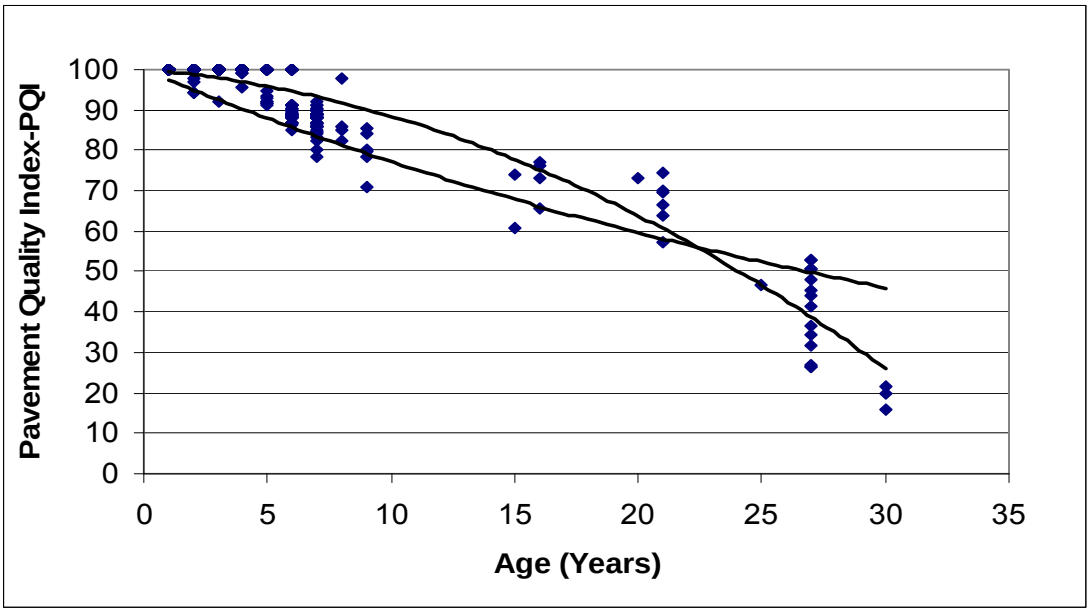


Figure 0.1: Scatter gram shows the general deterioration trend for Freeways and Expressways

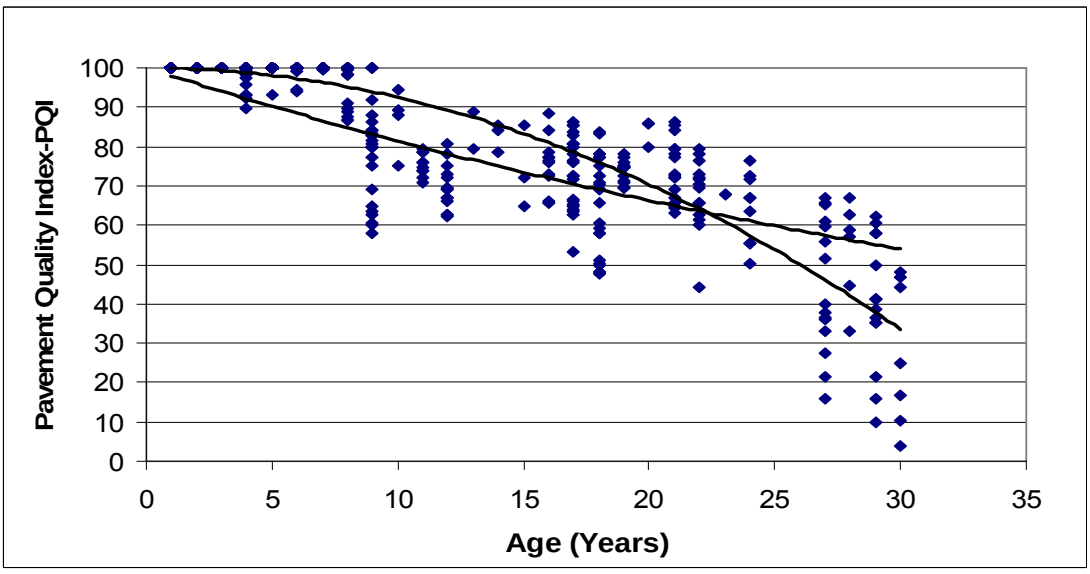


Figure 0.2: Scatter gram shows the general deterioration trend for Arterials.

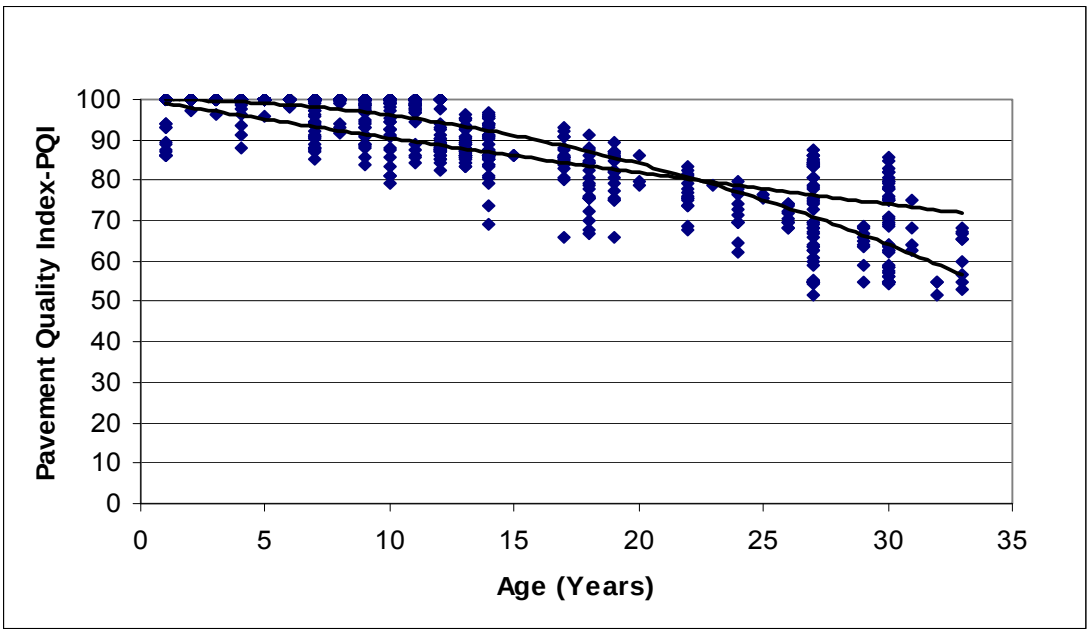


Figure 0.3: Scatter gram shows the general deterioration trend for Collectors

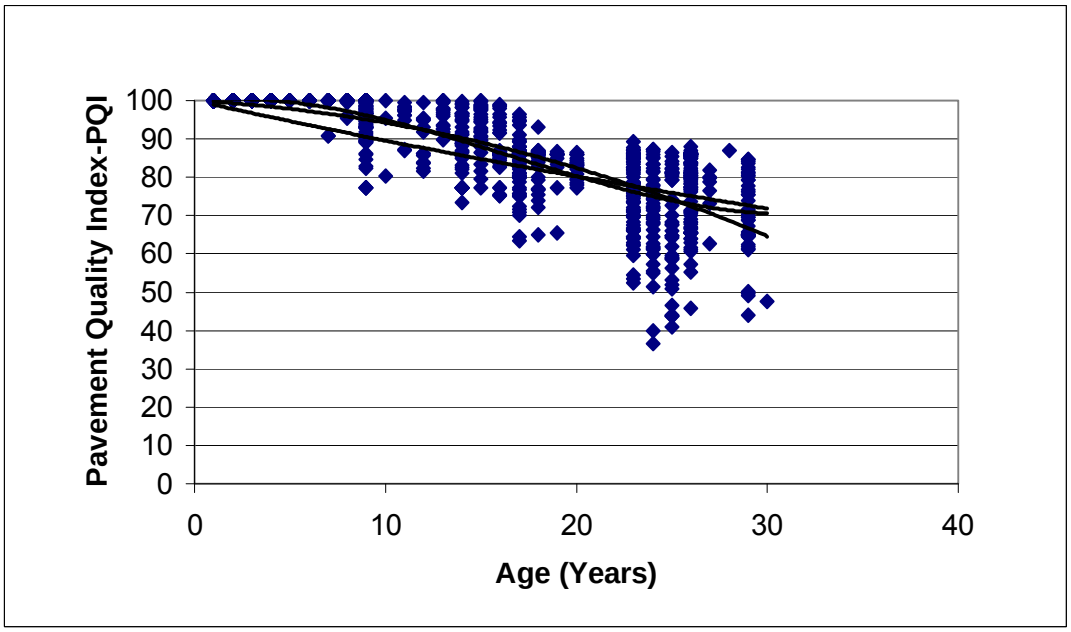


Figure 0.4: Scatter gram shows the general deterioration trend for Local residential

The analysis conducted using part of the data collected for this research from hundreds of pavement sections subjected to various levels of traffic loading indicated clearly that, traffic effect on pavement deterioration is considered major if compared to the other factors. This effect is manifested by the high rate of deterioration for the major roads in comparison with local roads. The preliminary correlation matrix established between the pavement condition indicators and the major factors that are expected to affect the

pavement deterioration showed good correlation with traffic loading in terms of the accumulated ESAL 's calculated for the past years since pavement construction. The details of models and prediction models will be discussed in the next chapter.

The correlation coefficients shown in Table A.6 Appendix A, were developed using part of the data pertaining to 73 pavement sections on which the traffic figures were found available at the date of writing this chapter. This correlation matrix was developed just to show the general effect of traffic and the other factors on pavement performance. Many other pavement sections are now under investigation and all the data about condition, Age, traffic levels, subgrade modulus , and the subgrade strength in terms of California Bearing Ratio (CBR) will be gathered and analyzed for final models developments. This correlation matrix and the deterioration curves will be revised once the data collection program completed in the next few months. In general, the same partial contribution of each factor to the deterioration trend is expected to prevail.

Several studies indicated that the contribution of traffic or axle loading to pavement performance may range from 60 to 85% . The remainder is contributed by the environment and other factors (AASHTO, 1993, and Watanatada, 1987). As shown in the correlation Table No 3.8 below, ESAL partial correlation coefficient reaches to R=-0.875 which means that the pavement deterioration is highly associated with traffic loading level and the contribution of traffic in the model that predicts the pavement performance is within the values range mentioned above.

This can be seen from the developed general deterioration curves that show the pavement deterioration for roads of different classes. The developed curves indicate that the pavement deteriorate at faster rate when approaching the end of the service life if both routine and major maintenance are not applied on time. These deterministic curves developed using the collected field data for hundreds of in-service pavement sections in the study area. These pavement sections are subjected to various levels of traffic loading. The general trend of the pavement deterioration for different classes is shown in Figure 4.5 and Figure 4.6 below. The aim of developing such curves here is just to show how the performance of the pavement is highly affected by traffic levels.

Based on the above conditions, exponential function was found to have the capability to represent the deterioration trend with a reasonable accuracy as represented by the value of $R^{2.}$

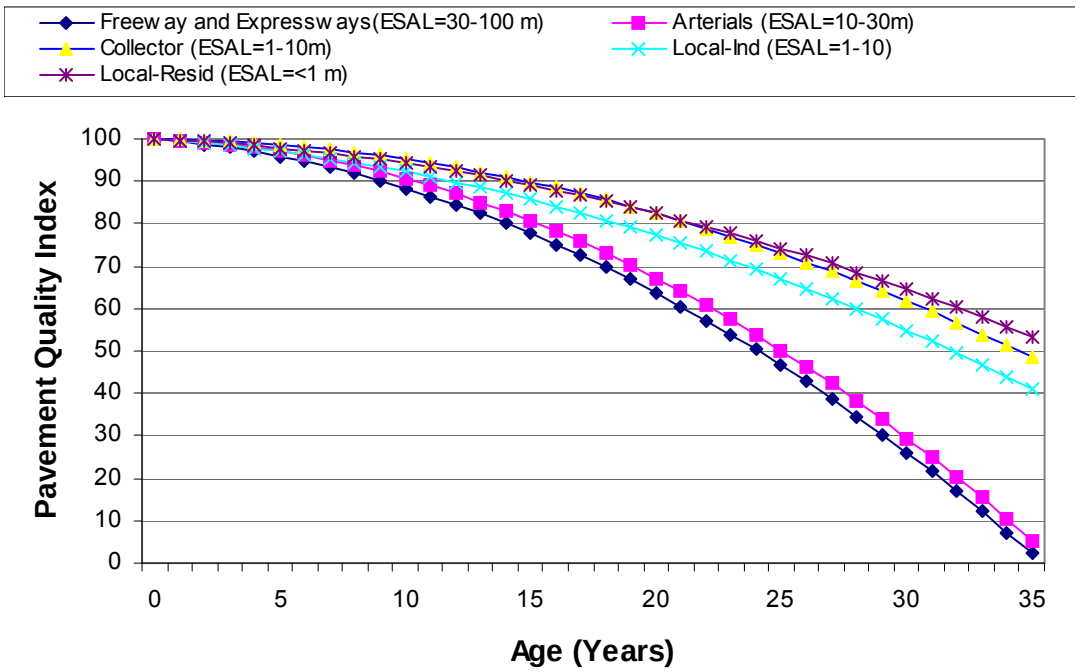


Figure 0.5: Deterioration curves developed using polynomial function for different road classes under various traffic levels.

Figures 4.5 and 4.6 were developed for all road classed subjected to different traffic loading levels. One model is developed for the major roads (i.e. Freeway and Expressway) as the preliminary investigation for the data collected up to date showed that the trend of the deterioration curves for both classes is almost the same.

This is considered practical as both classes are almost subjected to same traffic level and operate under similar loading pattern and environmental condition.

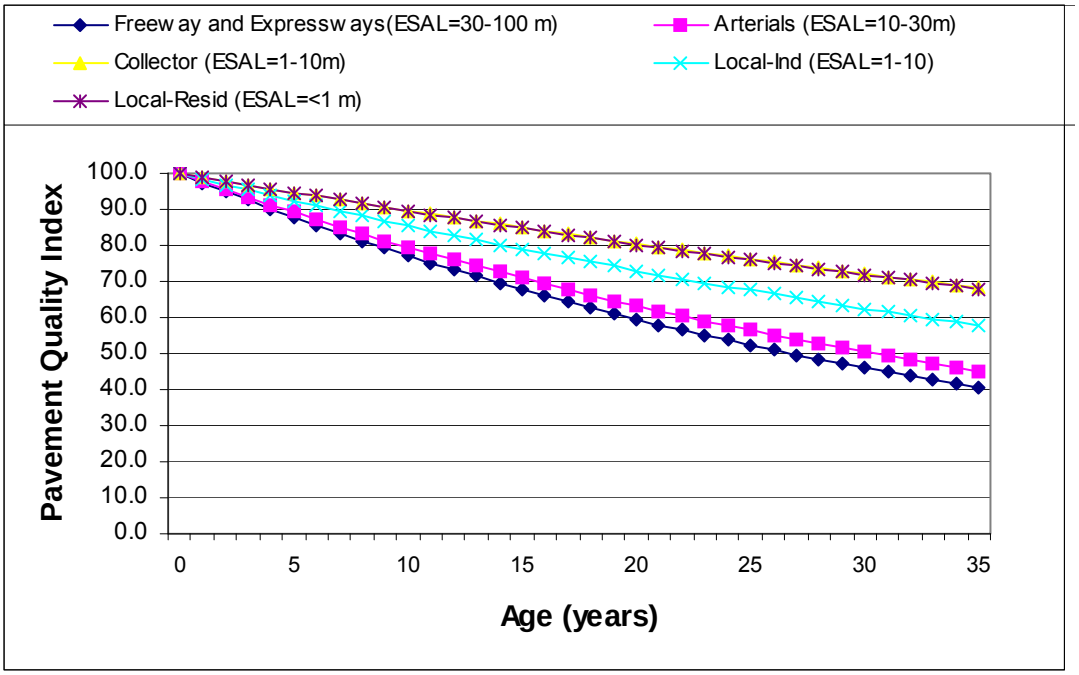


Figure 0.6: Deterioration curves developed using exponential function for the different road classes under various traffic levels.

Optimum Use of distress data as health indicator at network level

At network level, distress data are required to report about the physical pavement condition and the extent of defects that necessitate an immediate maintenance intervention. The optimum use of the distress data at network level envisaged by the researcher is summarized as below:

Distress quantities and cause

1- Determine the type , extent and severity of the surface defects depending on the cause of the defects , in this regard, defects to be reported at network level based on the cause of the distress and should include:
   a. Traffic loading associated defects which include:
      i. Alligator Cracking
      ii. Rutting
   b. Climatic (Environmental) defects which include:
      i. Longitudinal and Transverse Cracking

ii. Block Cracking
iii. Weathering and Raveling

c. Mix associated defects which include:
    i. Slippage Cracking
    ii. Bleeding

d. Traffic/Environment associated defects which include:
    i. Pothole

e. Others which include:
    i. Patching
    ii. Utility cut patching

2- Calculate/ estimate the density of the defects relative to the section area.
3- If the density of the structural defects (Traffic/ Traffic-Environment) exceeds 15%, the section is reported to be structurally failed. Traffic loading and material properties should be considered during project level study. Using Modifiers to enhance asphalt mix resistance and durability may look essential in this case.
4- If the density of the mix associated defects exceeds 50% , the section is reported to be functionally failed. Mix design should be considered during project level investigation.
5- If the density of the environmental associated defects exceeds 50% , the section is reported to be functionally failed. Modifier inclusion in Mix design should be considered during project level investigation.

Network cumulative distribution curves

At network level, the cumulative distribution curves (Survival Curves) that indicate the “health level “ of the roads network based on visual defects data can be used to report about the roads network condition and the required maintenance works and costing. The use of the “ Survival Curves” in PMS can give a clear representation of the current status of the network and help road agencies to plan ahead all maintenance works based on predetermined performance indicators or limits.

As shown in one of the application below (Figure 4.7), road agency can determine the current status of the network represented by the percentage of the pavement sections of PQI that exceeds a predetermined failure trigger value such as PQI=30. All

pavement sections located below failure limits must be planned for major maintenance or rehabilitation. Also, accurate deterioration curves can predict the percentage of the failed section after x years and the estimated budget that should be allocated for M&R works.

For example, in the figure below, the graph indicates that more than 83% of the road network is in very good and above condition. Less than 3% are in failed condition. These figures can be used as an indication of the required maintenance work to maintain the network condition at the same level. Based on the above, the survival curves established based on statistical characteristics of the each condition indicator represent the optimal use for these information at network level.

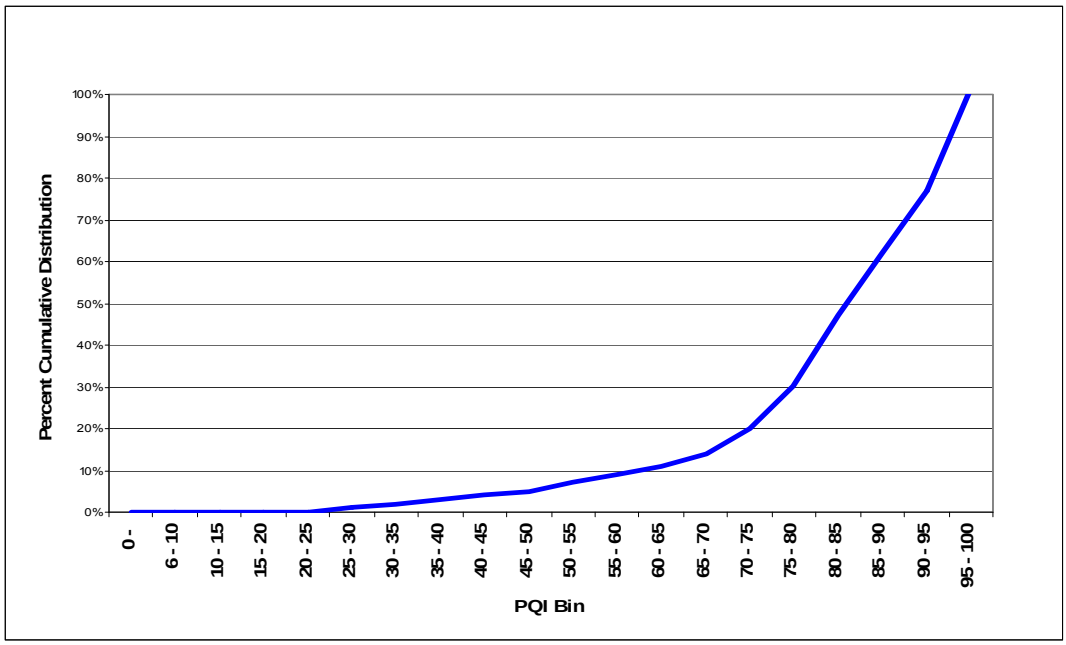


Figure 0.7: Cumulative Distribution of PQI values used as network health indicator (Dubai example)

Rideability (Serviceability) Concept

As stated previously in chapter 1, serviceability is manifested by the ability of the pavement section to serve traffic in its existing condition. Historically, there are two methods that can be used to determine serviceability or riding quality; one methods is the Present Serviceability Index (PSI) developed by AASHO Road test taking into consideration the pavement roughness and the distress types such as rutting, cracking

and patching. The second method is using roughness index based on roughness measurements only. Each method will be discussed thoroughly in the next sections.

The Serviceability (riding comfort) is determined by the pavement profile. A pavement Profile is measured in terms of two parameters Longitudinal Profile (or change in height along the pavement in the direction of travel – this measurement is termed “Roughness” and Transverse Profile (or change in height across the pavement and transverse to the direction of travel – this measurement is termed “Rutting”. There is however a third measurement of profile Texture Depth which is measured longitudinally in and between the wheel paths and is used in the analysis of Skid Resistance as a function of “Slip Speed”.

1. Longitudinally, the two parameters are:
    - Roughness with wavelength of between 0.5 and 50m and frequency between 20 and 2000 Hz and
    - Macrotexture with wavelengths less than 0.5mm and frequencies greater than 2000 Hz

2. Transversally , the parameter is:
   Rut depth, defined as “the maximum vertical pavement displacement, in the transverse profile, either across the wheel path (wheel path rut) or across the lane width (lane rut) measured from a reference plane.”

Each of these parameters can be measured by mechanical or non-contact distance measuring transducers such as acoustic or laser methods. The three above parameters have undergone considerable evolution over the past three decades.

Present Serviceability Index-PSI

The concept of Present Serviceability Index- PSI as an indicative measure of the pavement condition was introduced by Cary and Irick in the sixtieth’s of the last century (Carey and Irick 1975). This parameter has also been included as a main factor into the American Association of State Highway Officials –AASHTO design equations for both flexible and rigid pavements. PSI is measured on a scale ranges from 0-5 with 5 being

an excellent. This measure is subjective and it is a function of longitudinal surface characteristics and distresses such as slope variance rutting, patching and cracking areas. The model that combines these factors is represented in equation below:

$$PSI = 5.03 - 1.91 \log_{10}(1+SV) - 1.38\, RD^2 - 0.01(C+P)^{0.5} \quad \text{----------} \quad 4.11$$

Where:

SV=Slope Variance %

RD=Rut Depth in inches

C=Density of Cracking ($ft^2/1000\ ft^2$)

P=Density of Patching ($ft^2/1000\ ft^2$)

There are four terms of serviceability, each one has a definitive meaning and implications, these terms were established to correlate the subjective measurements on pavement serviceability and the objective measurements of the pavement performance, there terms are (Huang,1993):

- Present serviceability: It is the ability of a pavement section to serve high speed, high volume, mixed traffic in its existing condition.
- Individual Present serviceability rating: It is the individual rating for the present serviceability of a specific pavement section given by an individual member of the rating panel.
- Present serviceability rating: It is the mean of the individual ratings made by the members of the rating panel.
- Present serviceability Index: It is the mathematical combination of values obtained from the field measurements formulated to predict the PSR for those pavement within the prescribed limits.
- Performance Index: A summary of PSI over time which can be represented by the area under the performance curve of PSI over time.

The above terminologies are very important as there is always a confusion in understanding the meaning and the implication of each term. More information on methods of calculating PSR and PSI is detailed in the same reference.

Despite the fact that PSI depends on both roughness (longitudinal profile characteristics) and distress data such as rutting, patching and cracking, it is the roughness that contribute to the major part of the PSI value. The contribution of the distress data was found to be around 5% only. (Zaniewski, et al ,1985).

Many researches have indicated to the shortcoming inherent in the PSI method for pavement evaluation, among these are the followings:

1. The models developed were based on the AASHO road test panel. The perception of the road user nowadays may be completely different due to the vast change taken place in trucks configurations, speed and material used.
2. PSI incorporates both surface distresses and rideability at the same time. For better pavement management system, it is better to always include separate measure for surface distress and separate measure for riding quality. Therefore, the need for development a separate measure for riding quality was very crucial. In this study, one of the main objectives is to develop separate model for each condition indicator.
3. The profilometer used in AASHO road test to measure PSI is no longer in use. Errors are expected to be high when measuring roughness with other devices.

Based on the above , it was a necessity for various highway agencies to develop an objective measurement method to evaluate the serviceability which was finally accomplished with the introduction of the International Roughness Index-IRI method for measuring roughness or serviceability.

Pavement Roughness

Roughness has gained considerable attention and it has been subjected to a huge research effort when it was identified by Cary and Irick as the most important component in pavement present serviceability(Carey and Irick 1975). Since then, Roughness has become one of the principal pavement condition indicators that dictates the pavement performance and determines the limits and type of the required maintenance and rehabilitation needs. It is also considered the most important factor affecting operational characteristics and the road user cost. Recently, most of the

highway organizations started to use roughness parameters as a trigger line for pavement rehabilitation and reconstruction.

Longitudinal roughness can be defined as " *The longitudinal deviation of the pavement surface from a true planar surface with characteristics dimensions that affect vehicle dynamics, ride quality and dynamic pavement load"* (Shahin, 1998). In general, pavement roughness consists of random multi-frequency waves of many wave length and amplitudes. Factors affecting roughness include construction quality (I.e. .built in construction defects and irregularities) and construction materials, traffic loading and climate.

Methods of measuring roughness

There are several methods for calculating roughness index for pavements. In general, they are based on two principals:

- Pavement Surface Profile Measuring System (PSPMS)
- Response Type Road Roughness Measurement System(RTRRMS)

The first method is considered as a repeatable and accurate measuring method as it basically depends on a mathematical modeling of the measured profile. The second method is not repeatable because it depends on the vehicle characteristics which may change over time. IRI method can be classified under the pavement surface profile measuring systems as it is pavement surface profile based index.

International Roughness Index-IRI concept

Due to the shortcomings incorporated in the PSI method in evaluating pavement roughness or ride quality , most of the roads agencies started to search for an objective method that can be used to evaluate roughness accurately. The subsequent research efforts in this field has led to the development of the International Roughness Index (IRI). The development of the IRI was one of the most important outputs of the international road roughness experiment that has been conducted in 1982 by the World Bank in Brazil.

This experiment aimed at developing a measurement procedure that gives the same measurement at different regions with different machines or measuring tools. The IRI has been found to fulfill these requirements as it is based on a mathematical simulation of the road profile variation. Since the World Bank published a technical circular about the guidelines for conducting and calibrating roughness measurements (Sayers et al 1986), the IRI started to be used as a standard methods by which the longitudinal road profile can be analyzed and has become a standard scale on which road roughness level is reported in most of the world countries for the purpose of evaluating the pavement riding quality. It has been also adopted by many road agencies such as Federal Highway Agency (FHWA)(FHWA, 1987b).

The World Bank experiment has indicated that IRI is a unique roughness index in the ease of measurements and the extent to which its measurability has been demonstrated. The experiment has shown that IRI can be measured by many equipments and devices including Rod and level, Mays meter car(USA) , NAASRA car (Australia) French APL, BPR roughmeter, TRRL Bump Integrator, TRRL beam, Swedish road surface Tester, ARAN, Face Dipstick, GMR –Types profilometer, South Dakota Type profiling system and many others.

IRI is defined as "*The average rectified slope which is the ratio of the accumulated suspension motion to the distance traveled obtained from a mathematical model of a standard quarter car traversing a measured profile at standard speed of 80km/h"* (Huang, 1993). IRI is a scale for roughness based on the response of a generic vehicle to roughness of the roads surface. It is determined by measuring the profile of the road, then, by using an algorithm that simulates the quarter car response, the profile data is processed and accumulates the suspension travel. The cumulative deviations per mile (km) are a summary of the road slope deviation.

IRI is considered as a profile based index calculated using mathematical model applied to a measured pavement profile. It is based on the theory of Quarter–Car mathematical Model of the suspension (Figure 4.8). This principle has been used as a basic for developing the roughmeters which depends on a single wheel trailer with masses and suspension characteristics similar to the front wheel of a passenger car.

This tools has been later developed to different device named Bump integrator. The quarter car response usually takes place in a very definitive manner irrespective the way in which the roughness is viewed whether as slope (velocity input) or deviation in elevation (displacement input) or change in slope (acceleration input). Mathematically , The response can be described using dynamic equations used in quarter car simulation. I.e. at low frequency (pavement surface with long wavelength), the suspension response is zero as the wheel and vehicle body move together up and down. If the frequency is increased a little bit, say at one Hertz, the sprung mass will start to resonate on the suspension producing stroke that is slightly grater than the road input.

The same response is kept the same up to frequency near 10 Hertz where the vehicle axle resonance takes place. Above the axle resonance , the response again drops to zero as the road surface bumps deflect the tire without producing suspension stroke. The measurement of the suspension stroke as the roughness response was chosen for the convenience back in the early development period of the Via-Log and roadmeter.

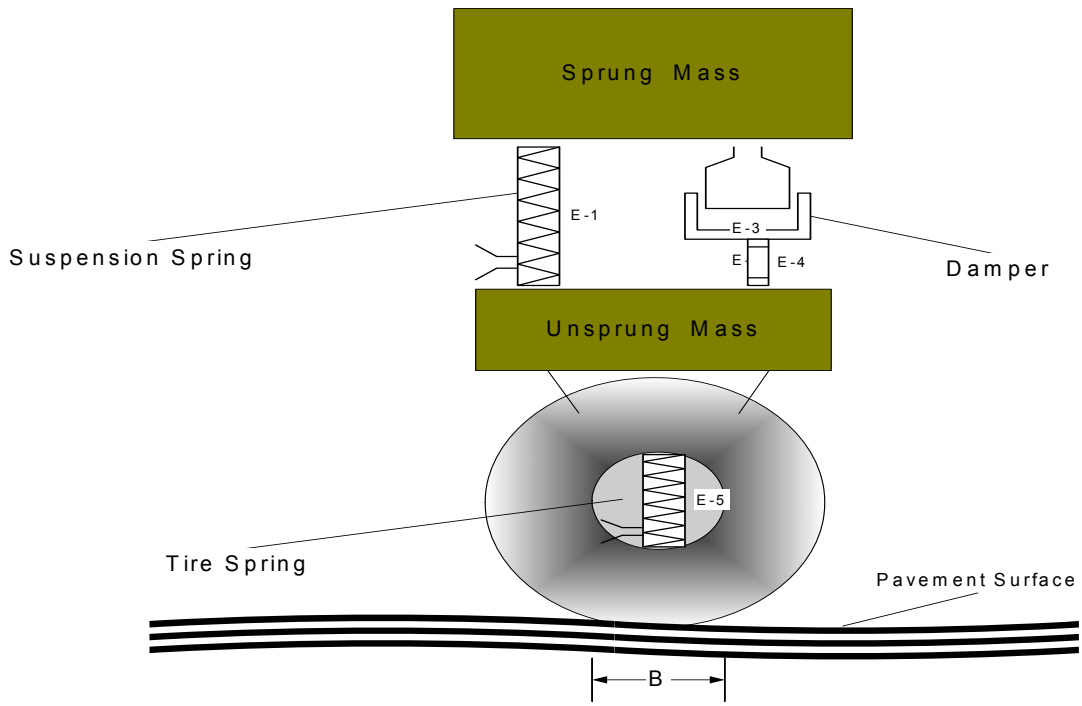


Figure 0.8: Quarter Car Mathematical Model used for the development of the International Roughness Index (IRI).

The standardized simulated speed of the quarter car model is normally selected as 80km/h. Similar to the real wheel suspension action, the model acts as a wavelength filter amplifying the response to certain wavelength range while damping the response of the wavelength outside this range.

In analyzing the road profile, it is usually desirable to remove the long wave length and include the short wave length in the filtering process to calculate roughness (Shahin,1998).

The relationship between the wavelength and the frequency takes the following form:

$$L=v/f \qquad 4.12$$

Where:

L=Wavelength (m)

V=Speed (m/s)

f=Frequency in Hz (1/s)

Many researches have indicated that the wavelength that affects the IRI computation ranges from 1.7m to 28m (6 ft to 90 ft) which corresponds to the frequency in the range from 0.80 to 13 Hz. Waves of lengths that lie outside this range of the wave band deducted by IRI model (i.e. more than the 28m (0.80Hz) and less than 1.7m (13Hz)) are both damped drastically.

The above statement indicates that waves of frequency more than 13Hz will increase more slowly with increasing frequency. Also, the amplitude of the short wave outside this range will be very small and will have negligible effect on the IRI value.

Sayers and karamihas ,1996 indicated that the frequency response function of the IRI quarter car shows a significant response to the pavement profile wave length within the range from 1.1m to 44m . These wavelength correspond to frequency between 0.5 and 20Hz. Despite the fact that researches based on field testing indicated that IRI measurements has an evident response to the waves of lengths range from 1.1m to 44m , still there are some evidence that IRI has some response to waves of length more

than this range. The exact wave lengths range included in the IRI calculation is still argued by different authors and researchers in this field. From the above theoretical explanations, the IRI model can be defined as follows:

IRI= vertical movements (displacements) of the sprung mass with reference to the unsprung mass / horizontal traveled distance

$$IRI = \sum (D_v/D_h)$$

Where:

IRI=International Roughness Index (mm/m or or m/km)

D_v =Vertical Displacement of the sprung mass with reference to the unsprung mass (mm or m)

D_h =Horizontal traveled distance (m or km).

Level of condition measurements based on Pavement roughness

Three Levels of condition measurement are proposed as a function of need, or as a path to providing road condition data that is “Fit for Purpose”. These levels should be considered when reporting about network condition:

Condition Level 1

It is considered as the basic level. It provides data and outcomes suitable for ranking sections or sub-networks. The outcomes are not generally suitable for either demonstration of “duty of care” or preparation of maintenance works Program (An Operational Works Program because they provide insufficient engineering properties.

Condition Level 2

The Intermediate Level. Measurements at this level are sufficient to demonstrate “duty of care” and to allocate sections of a network as candidate sections for further analysis. These measurements can be used to prepare a pavement management system, but would be unlikely to support the modeling desirable for a Performance Specified Maintenance Contract or a long-term maintenance program involving Key Performance Measures.

Condition Level 3

The most detailed level of measurement. This level permits the allocation of levels of risk through the sample size. Level 3 requires highly sophisticated equipment operation, and evaluation by engineering professionals. It provides defensible evidence of road management plans and for litigation. Level 3 is the preferred option for "Project Level Investigation" (PLI).

The IRI index at recent time is the most acceptable measure that quantifies pavement surface profile because it encompasses almost all wave lengths that affect significantly the motor vehicles. Most reliable deterioration models are now based on roughness in terms of The International Roughness Index or IRI.

IRI as a condition indicator that measures pavement riding quality is expected to remain the basic indicator or index for reporting about pavement serviceability level for the next decades. Figure 4.9 shows the IRI scale used for establishing the roughness specification and defining the limits for different types of pavements for roads and airports (Sayers *et al, 1986).*

Uses of roughness characteristic value (RCV) in PMS

The concept of "the characteristic value" is considered as one of the methods available to decision makers and maintenance engineers that allows to apply statistics to current data and analyze the risk inherent in such data (Austroad, 2000). It can be applied to all data pertaining to all condition indicators collected for condition assessment in order to establish the condition for each road and the network as a whole.

This concept is particularly useful where the data is to be used for selection and allocation of candidate sections at network level. The characteristic value is defined as :

$$\text{Characteristic value} = \mu \pm f\,\sigma \qquad 4.13$$

where;

μ = The population mean

σ = The standard deviation of the population and f is given by Table :

Road Class	f	% of all values that will be covered by the

		characteristic value
Freeways, Expressways ,Arterials	2.00	97.5
Collectors & Distributors	1.65	95
Regional Roads, Local Access and Residential	1.30	90

Provided the mean and standard deviation for any set of data can be easily determined, the chance of the characteristic value being representative of the true population is increased depending on the road usage.

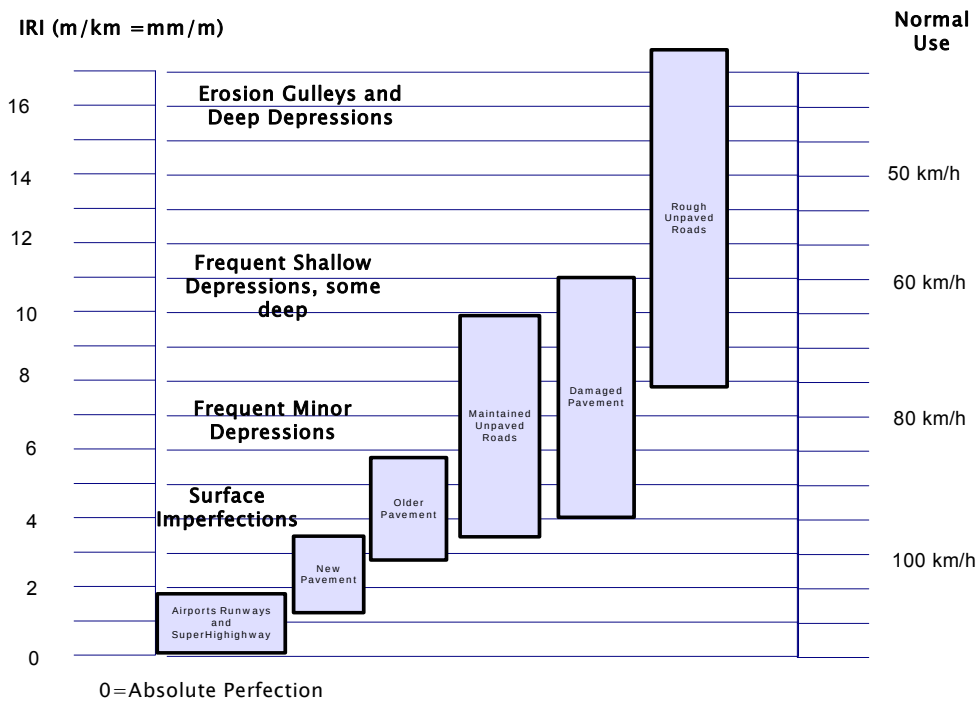


Figure 0.9: The International Roughness Index–IRI Scale (Source: World Bank, reproduced by the author)

For example, for major roads one needs to be very sure that the intervention levels chosen for maintenance alternatives should have a low risk of error, whereas for minor roads a 10% risk of error is considered acceptable.

Currently, there are several problems associated with the use of the IRI measurements at network level. Many of the road agencies and municipalities collect roughness data but the benefits gained from these data are limited particularly in the issues related to treatment selection and budget estimation. These problems are mainly associated with the limits of intervention for maintenance work and the incorrect understanding of the roughness concept as a condition indicator.

Referring to the original hypotheses included in the world bank report No. 46, "Guidelines for conducting and calibrating road roughness measurements" proposed by Sayers et al,1986, it is clear that the interpretation of IRI is a reasonably coarse measurement. This saying is based on the relatively wide range allowed to describe roughness in terms of IRI. As shown in Figure 4.9 (above), the IRI ranges from IRI =2m /km, which represents a fairly smooth pavement on which the vehicle can travel on reasonable comfortably at 110kph, to IRI =16 m/ km (i.e. a pavement that it would be difficult to control a vehicle at 40kph (for example, speed humps in suburban roads are of the order of IRI=16).

Therefore, IRI acceptance level for the new roads varies from one region to another, for example, in Sweden, it is stipulated that for the highest road class, the IRI should not exceed 2.5m/km over any 400m section. In Italy and France, the IRI acceptance level for the new roads is set to 1.5m/km over any 400m section. In USA, Federal Highway Administration adopted the classification shown in Table 4.3 for roads roughness rating (Waheed Uddin *et al* 1998). It is very clear that roughness as a road response perceived by the road user, considerably vary from area to another.

The implications of such the wide range of the IRI scale imposed on road agencies to adopt different views and policies regarding the target and the terminal values of the roughness levels. As shown in Table 4.3 , the road of a value of IRI > 3 is considered a smooth road whereas it is considered a value that warrant heavy maintenance or rehabilitation in other regions.

Table 4.3: IRI ranges and the corresponding roughness rating proposed by Federal Highway Administration (FHWA)

Range of IRI	Roughness Rating
IRI<3	Smooth
3<IRI<5	Moderate
IRI>5	Rough

The above argument constitutes one of the sub-cores of this thesis problem. Based on the vast experience of the searcher in this field, the roughness measurements should

be used differently to in pavement management systems to report about network functional level.

It is envisaged that the intervention level for maintenance and rehabilitation should be function of the posted speed on each roads. This suggestion is based on the original hypothesis of Sayers et al, illustrated in Figure 4.10 below. *Therefore, establishing maintenance intervention levels based on road class and posted speed, is envisaged to be one of the optimum uses of roughness measurements at network level.* This issue will receive more elaboration in the next sections.

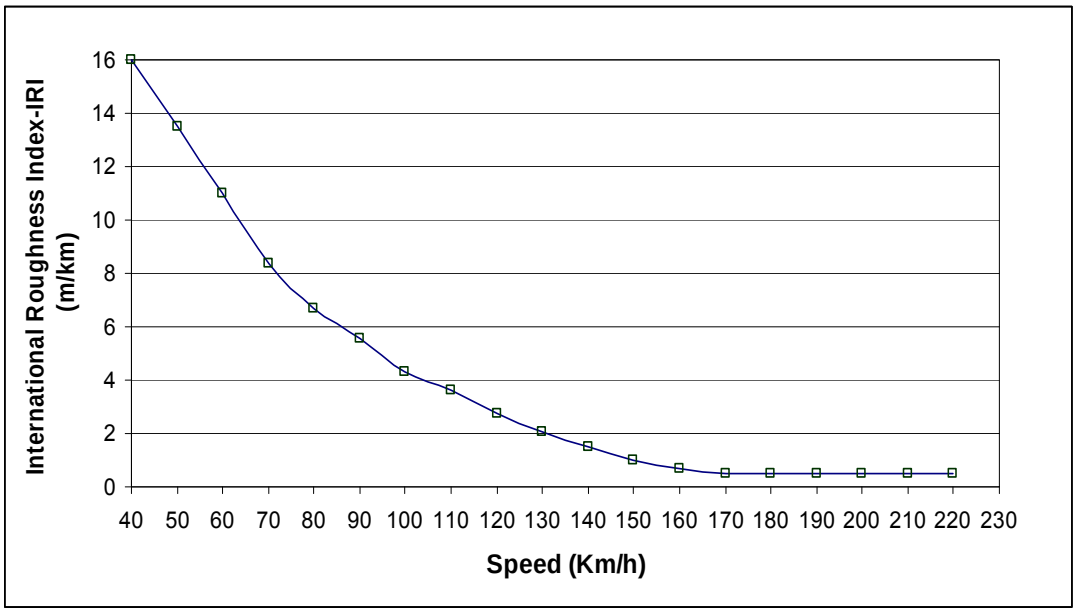


Figure 0.10: Roughness as a function of Speed (after Sayers & Gillespie-1986)

Roughness measurements are usually collected by roads agencies using both *Single Track IRI* in which the roughness measurement is based on a quarter car model run over a single track of the longitudinal profile. These measurements are represented by the values of IRI in the right and left wheel track. The other form of the roughness is the *Lane IRI,* which is the composite IRI values estimated based on the average value of the Left and right wheel paths of the lane. The roughness measurements are usually averaged over 100m or more segments for the purpose of facilitating treatments selection process.

To evaluate the network current status from roughness point of view, terminal and target values must be established. The comparison of the current roughness measurements

against the target value (smooth surface of newly constructed roads) imparts applying certain type of major repair or surface rehabilitation for the pavement sections that experience high level of roughness or those segments that, at least, exceeds the terminal value.

Use of IRI cumulative distribution to report at network level

The cumulative distribution of roughness measurements can be used at both project and network levels. Typical cumulative distribution of roughness example for roughness measured for one of the roads in the study area is shown in Figure 4.11 below.

From this graph it can be seen that there is a distinct difference between the Lane IRI (half car index) and the individual wheel path IRI (quarter car indices).

As can be seen from the same graph, the 90th percentile range, which is normally used to report about network functional health, can be determined and the required budget can be allocated.

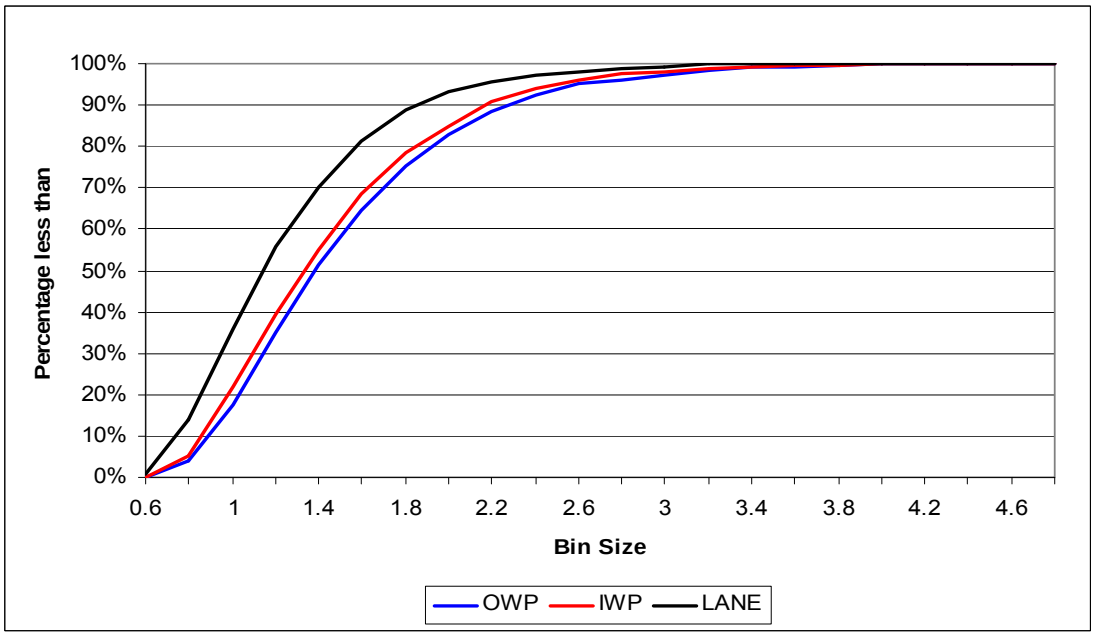


Figure Error! No text of specified style in document.**.1: Use of cumulative distribution curves of roughness results to report roughness at network level**

Uses of "Yaw" measurements as intervention for maintenance action.

Most of roads agencies use lane IRI to report about the roughness level at both network and project levels. Roughness are usually measured on both wheel track. The

new term “Yaw” is used to describe the motion set up within a vehicle by different longitudinal values in the adjacent wheel paths.

Clearly, if a vehicle is traveling at high speed on a road of un-even surface, then the vehicle may become unstable as a result of the lateral “wobble” induced by the road surface. It is therefore advisable to monitor the difference in IRI between the outer and inner wheel paths and to place an intervention levels based on this difference as a “trigger” for maintenance action. Based on the experience gained from the field, it is recommended that the cumulative distribution of the difference in roughness between wheel paths is to be considered as a main PMS reporting parameter on roughness. *In this regards, the 90th percentile of the IRI or Yaw values can be suggested in this study as an intervention level for major maintenance work.*
Figure 4-12, which is given as an example, shows the cumulative distribution of the difference in roughness between right and left wheel paths.

If road agency desires to establish an intervention levels for major maintenance recommendation at 10% risk level, It can be seen that 90% of all values fall below 0.35 IRI. For this reason an intervention level of 0.35 IRI can be proposed as an intervention value for major correction action.

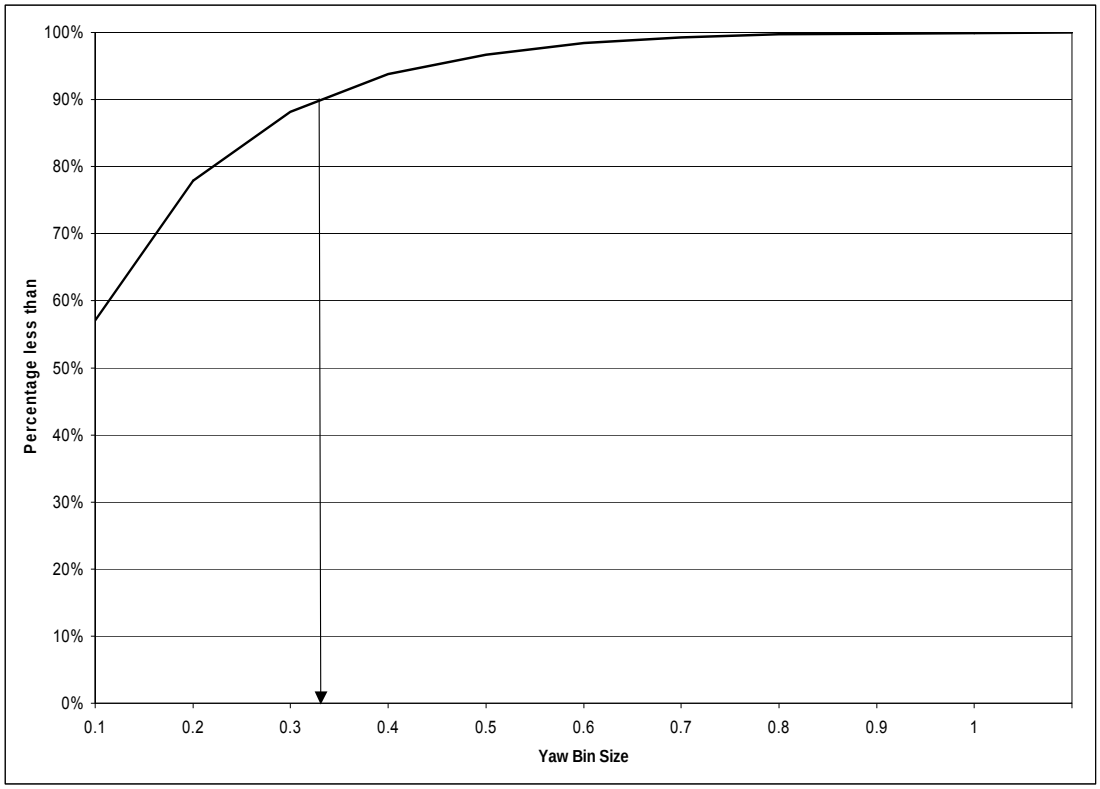


Figure Error! No text of specified style in document.**.2: Cumulative distribution of "Yaw" values on one of the roads in the study area used to establish the failure limit.**

Establishing permissible roughness level based on road class

Based on the above argument, the researcher in this study effort proposes and recommends that *roughness intervention level are to be set as function of road class or highway posted speed.* Combining Figures 4.9, 4.10, and 4.11; Table 4.4 following, is suggested as interim intervention levels for the uses of PMS to be used in treatment selection decision trees to compute the likely maintenance program and associated budget , and /or to be used as a reference for particular design specification.

Table 4.4: Proposed IRI values as a intervention levels for roads rehabilitation classified by posted speed.

Posted Speed	Maximum permissible roughness (IRI)
80	4.0
100	3.5
120	3
130	2.5
140	2
>140	1

In Australia , roughness is reported using NAASRA measurements in the units of counts/km or by the International Roughness Index (IRI) measured using “quarter car model” (Austroad 2000). Each count is equal to 15.2mm. NAASRA roughness meter is a mechanical device used extensively in Australia and New Zealand since the seventieth of the last century for measuring roads roughness. It records the upward vertical movement of the rear axle of a standard station sedan relative to the relative body as the vehicle travels at a standard speed along the road section being tested. A cumulative upward vertical movement of 15.2 mm corresponds to one NAASRA roughness counts (i.e. 1 NRM/km).

Similar to the IRI concept, any road segment of NAASRA number exceeds certain predetermined terminal value is reported to be failed and warrants sort of repair or rehabilitation action. Roughness measurements can be converted to NAASRA units. Linear equation conversion is used to convert NRM to IRI as shown in Table 4.5 below. The following equation is used to convert from NAASRA counts to IRI values:

$$NAASRA = 26.49 * IRI - 1.27 \qquad 4.14$$

where:

NAASRA = Roughness level measured by NAASRA model (counts/km)

IRI= International Roughness Index in m/km..

In the study area, a stricter limit of IRI acceptance level is adopted as 0.90m/m over any 400m section. This limit has been adopted to control the rate of deterioration for the pavement with high initial roughness.

The dynamic load caused by high initial roughness level reduce the structural ability of the pavement to resists the imposed load and contributes considerably to pavement damage resulting from heavy truck traffic.

Table 4.5: IRI ranges and the corresponding roughness rating adopted in Australia.

Range of IRI	NRM (Counts/ km)	Ride Quality	Roughness Rating
IRI<1.46	40	Very Smooth	Excellent
1.46<IRI<2.97	40-80	Few minor pubs encountered	Good
2.97<IRI<4.1	80-110	Small Up and down movement, reasonably comfortable driving	Fair
4.1<IRI<5.23	110-140	Small Up and down movement, feel rough in trucks, low comfortable driving with	Poor
RI>5.23	>140	Uncomfortable driving, severe up and down sideway movement. Good control of steering must be maintained. Reduction in speed is practiced sometimes	Very Poor
1 count=15.2mm			

Roughness prediction models

Sound predictable models that predict the change in the roughness over time became a crucial tool in all modern pavement management systems to determine and program the future rehabilitations needs and allocate the required budget. Local deterministic modeling is much more preferable in this case as the roughness change is a function of the prevailing operational and loading conditions in each area.

The importance of the roughness is not limited to the riding quality of the road user, but it is also the principal measure of pavement condition that affect considerably the vehicle operational cost (Reddy et al ,1998).

Based on the various research works done in this field and the practical experience of the researcher, roughness change is envisaged to be a function of the following three main factors:

- Initial roughness level in terms of IRI
- Traffic loading
- Age (Traffic , Climate and the interaction effect between traffic and climate).

Other minor factors including pavement thickness, climatic factors such as temperatures and rainfall amount, can also affect roughness deterioration but in a much lesser extent.

Age is considered as the main performance predictor in many pavement condition indicators deterioration models as it represents the accumulated effect of both traffic loading and climate and the interaction effect between both traffic and climate. I.e. the effect of both factors are implicitly incorporated in the age variable. As can be seen from the correlation matrix, Table A-2, appendix A, age variable always have high partial correlation coefficient (R) with both pavement quality index and roughness index in comparison with the other variables.

Roughness propagation modeling was achieved by analyzing the data collected for hundreds of pavement sections over wide range of traffic and pavement age. Despite

the fact that a particular model for each pavement section is the preferred approach, a family deterioration curves based on road class, surface type or any other criteria can be successfully utilized to predict roughness deterioration, as well as pavement condition deterioration, curves with relatively sufficient accuracy. The roughness deterioration model can be developed based on the data pertains to the initial roughness data at zero age and the data for the roughness in the subsequent few years after being exposed to traffic and climate.

In the World Bank pavement deterioration models used in the Highway Design and Management (HDM 3) and used later in HDM4, roughness variable was modeled based on many factors including the initial roughness, surface distresses, subgrade and pavement strength, environment and traffic loading. It was basically designed for life cycle costing analysis of the proposed rehabilitation options.

The use of such model is somehow cumbersome and needs a lot of efforts to collect the relevant data pertaining to each factor incorporated in the models which is often unreachable. Therefore, many researchers tried to simplify the equation so that roughness change can be predicted easily with less effort and reasonable accuracy.

Based on the above criticism for the world bank roughness prediction models, a new simplified model has been introduced to be use in pavement management systems for determining the pavement maintenance and rehabilitation of the asphalt- surfaced pavements with relatively good predictability (Paterson et al ,1992). It is designed to predict roughness values at any time after construction. The proposed model takes the following form:

$$IRI_t = 1.04\, e^{mt}\{IRI_0\} + 263(1+SNC)^{-5}\,(CESAL)_t \qquad 4.15$$

Where:

IRI_t= Roughness values at time t.

IRI_0=Initial roughness value at 0 age. (directly after completing construction and before opening to the traffic.

t= Time since last construction or overlay in years

m=Environmental coefficient varies from 0.01-0.7 (from dry, non freeze, wet , freeze conditions)

SNC=Structural Number modified by subgrade strength

$CESAL_t$= Traffic in terms of Cumulative equivalent single axle load at time t in millions.

As stated earlier in this chapter, the initial IRI varies from region to another. It may range from 0.90 to 2.0 depending on the specifications adopted in each country.

The application of such model in PMS will be also cumbersome and inconsistent for simple reason, structural numbers can not always be obtained at network level, and the environment factor selection procedure will be very subjective.

Models for roughness propagation trend for Australia were developed for both arterials and national highways (ARRB,1994). These models can predict the roughness level at any time based on inputs related to the initial roughness, structural number, traffic and maintenance expenditures (routine and periodic maintenance). The model takes the following form:

For national Highways

$$R(t)= R0+R0*A1\ (I+100/SNC)A2*t^{A3}*(1+A4*L^{A5})/ME+4000)^{A6} \qquad 4.16$$

For Rural Arterial

$$R(t)= R0+R0*A1\ (I+100/SNC)A2*t^{A3}*(1+A4*L^{A5})/ME+200)^{A6} \qquad 4.17$$

Where:

R(t)= Roughness level at any time t

R0= Roughness level at zero age

I= Thornthwaite Index

SNC= Modified structural Number

L=Annual Traffic load (CESAL/Lane/year*10^6)

ME=Average maintenance expenditures ($/Lane.km in 1992/1993 $s)

T=Time in years since original construction or last overlay

The coefficient A used in the above equations are shown in Table 4.6 below.

Table 4.6: Coefficient A used in the equations developed for both Australian arterials and national highways

Class / coefficient	**National Highways**	**Rural Arterials**
A1	0.0016	0.014
A2	0.497	0.308
A3	1.07	1.00
A4	55.5	7.56
A5	0.02	0.295
A6	0.265	0.177

Other models to predict roughness (unevenness) as a function of the initial roughness ,traffic and deflection were proposed elsewhere (Reddy et al ,1998). The proposed model was developed using very limited data, so it is expected to be valid only for the range of data based on which it was developed with very limited applications.

In general, sound models can help in obtaining much more accurate condition prediction. But , the models should depend on variables that can be easily measured at site. For example, structural number can be used as a variable to predict roughness change over time for each pavement section. Unfortunately, data pertains to structural number information can not be easily obtained from the field. If the as built drawings or the original design details are not available, destructive and nondestructive testing to calculate the design and the effective structural number values should be carried out.

Therefore, for this simple reason, it was decided in this study to develop the roughness propagation models based on one or two of the three important factors that were found highly correlated with each other and can be easily measured or estimated. These include; Initial roughness, Age and Traffic loading in terms of ESAL.

To fulfill the objective of this study in this regard, the IRI values were calculated over a 400m pavement section. This value was selected because the IRI model sometimes deduct the amplitude of the longitudinal profile of the pavement surface for the waves more than 44m in length and includes it in the IRI calculations. In addition to this

reason, it is much more practical to calculate IRI over lengthy sections for the purpose of assigning practical maintenance and rehabilitation options.

Roughness prediction models using Age variable

There is almost a complete agreement that after a given initial time, IRI starts to increase and this is assumed to be related to traffic level ,particularly heavy vehicles. The issue of predicting the IRI based on distress types and other variables that are more difficult to measure in the filed is impractical. Therefore, a simplified approach to develop roughness propagation models based on few independent factors that can be easily measured at network level has been adopted in this study . Prediction models should be able to represent the actual general behavior of the road pavement.

The preliminary analysis showed that roughness data for heavily trafficked lane (slow lane) and the fast traveling lane vary a little bit. Therefore, it was decided to develop different model for each lane. Figure 4.13 shows the scatter gram of the IRI –Age data pertains to the fast lane in addition to the suitable functions that were found to fit the general trend of the data. Various mathematical transformations to fit the general nonlinear trend of the data were tried. Nonlinear relationship can be analyzed as a linear model by transforming the independent variable (Shahin 1998).

As expected, exponential and 2nd degree polynomial functions were found to have good fitness with the general data trend. The roughness prediction models were found to take the following forms:

- ***For the fast lane***

$$IRI\text{-}f = 0.0028\ Age^2 + 0.0121\ Age + 0.744 \qquad 4.18$$
$$R^2 = 0.7701$$

$$IRI\text{-}f = 0.7442e^{(0.0507Age)} \qquad 4.19$$
$$R^2 = 0.8563$$

- ***For the slow lane***

$$IRI\text{-}s = 0.0035\ Age^2 + 0.0215Age + 0.769 \qquad 4.20$$
$$R^2 = 0.8257$$

$$IRI\text{-}s = 0.7691e^{(0.059Age)} \qquad 4.21$$
$$R^2 = 0.939$$

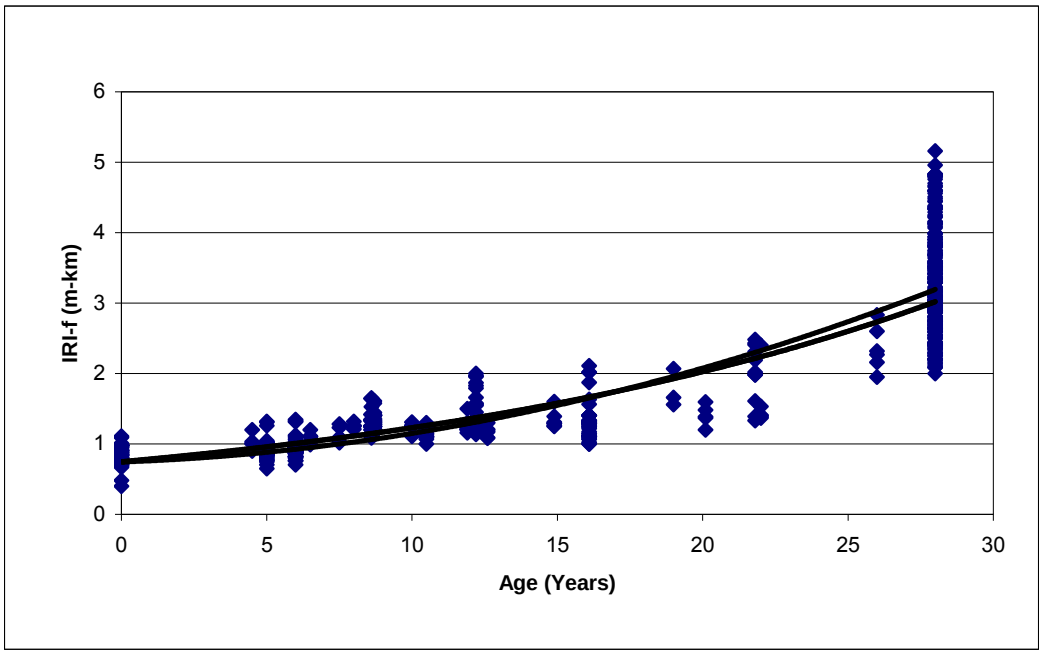


Figure Error! No text of specified style in document.**.3: Scatter gram for the IRI against Age for the fast lane and suitable functions fit the general trend.**

As can be seen from Figure 4.14 and Figure 4.15, Age variable accounts for around 90% of the deterioration models which indicates the relative importance of such factor on pavement performance.

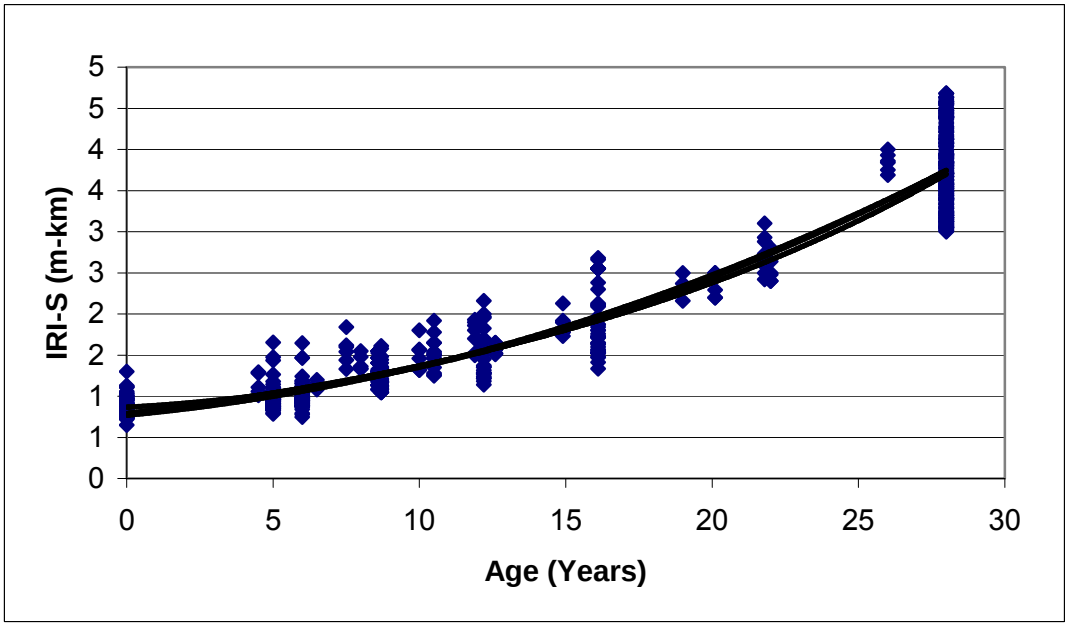


Figure Error! No text of specified style in document.**.4: Scatter gram for the IRI against Age for the slow lane and suitable functions fit the general trend.**

The model shows that, as the age increases, the roughness increases accordingly. At the early ages of the pavement service life, the rate of IRI changes is relatively small, after the year say around 15 years, the change rate of roughness increases dramatically . This mainly due to the formation of high severity cracking and the formation of potholes.

The rate of roughness change is very important factor in determining the length of the service life of the pavement section. In this regard, high initial roughness will lead to an accelerated rate of deterioration and thus control the life cycle of the pavement. The service life of the pavement initially constructed with IRI of 0.90 (PSI=4.2) will have significantly longer service life that the pavement section with initial roughness of IRI of 1.2 (PSI =4.0)._The application of the model developed for the slow lane is shown in Figure 4.15 below:

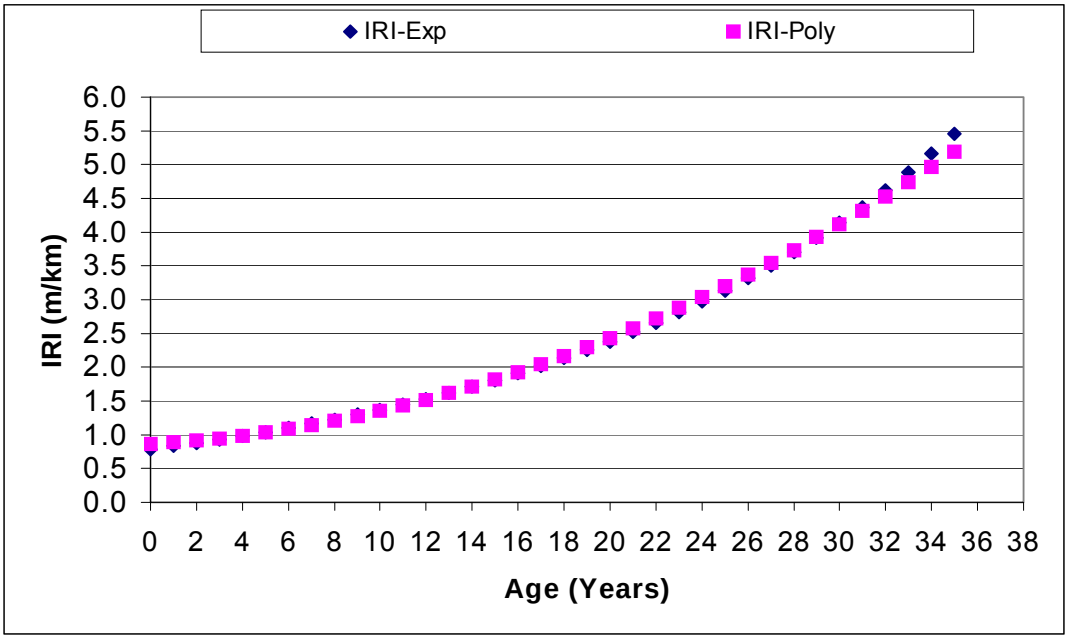


Figure Error! No text of specified style in document.**.5: The application of the IRI-Age model (Slow lane) developed using exponential and polynomial functions.**

The value of models goodness of fit represented by R^2 is high. This conclusion is expected as the Age variable accounts for the effect of both traffic and climatic

variables. In general, the selected models should satisfy the boundary conditions of the pavement system behavior which are:

- The IRI value should be equal to the initial value (a constant) at pavement age equal to zero.
- The model should not give a negative value at any value of pavement age..
- The curvature of the line, represented by the value of b constant, should take the suitable general trend of the collected data (concave down or concave up).
- The model should have high predictability as indicated by R^2 value.

The predicted IRI against the measured values for the slow lane was drawn for showing the accuracy of models prediction. It is clear from Figure 4.16 below that all the predicted and the measured IRI are on 45^0 line which indicates the goodness of fit for the developed model.

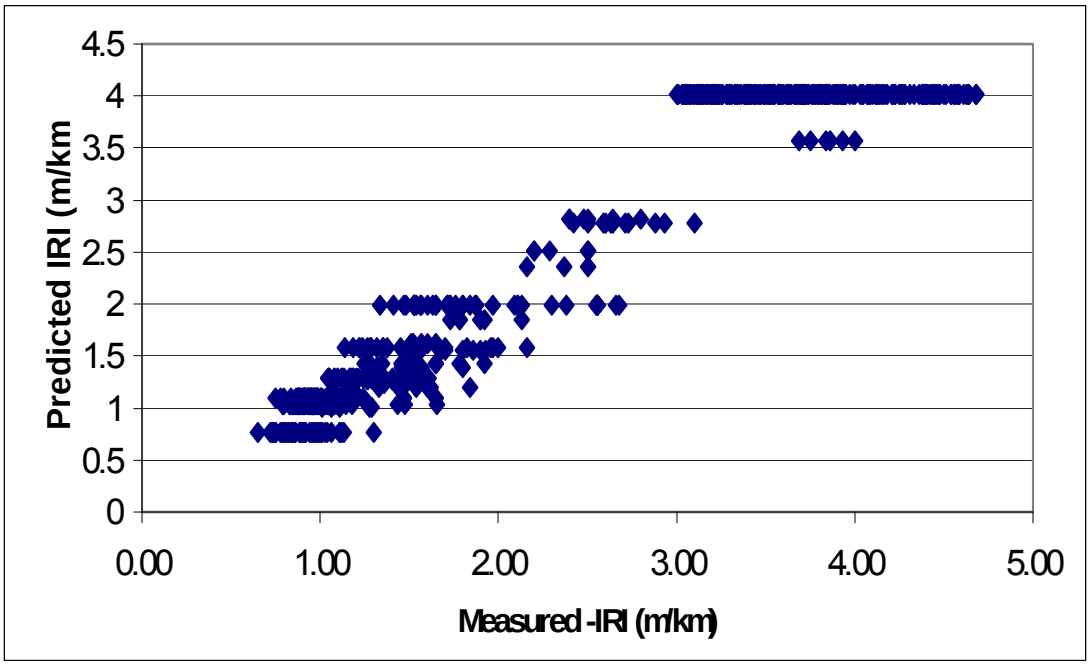


Figure Error! No text of specified style in document.**.6: Scatter gram for the predicted - measured IRI values for the model developed using exponential function**

Roughness change prediction models using ESAL variable

Roughness prediction models based on traffic loading data were also developed. This gives the engineers and the planners other options to predict roughness deterioration if

the traffic data is available. The models that describe the change in the IRI values Is shown in Figure 4.24 below and found to take the following forms:

$$\Delta IRI = 0.0003\ ESAL^2 + 0.0077*ESAL \qquad 4.22$$

$$R^2 = 0.6776$$

$$\Delta IRI = -2E\text{-}05\ ESAL^3 + 0.0011*ESAL^2 - 0.0024*ESAL + 0.0348 \qquad 4.23$$

$$R^2 = 0.6786$$

$$\Delta IRI = 0.0343e^{(0.0965\ ESAL)} \qquad 4.24 \qquad R^2 = 0.6089$$

$$\Delta IRI = 0.0078\ *ESAL^{(1.1164)} \qquad 4.25$$

$$R^2 = 0.6408$$

As the initial roughness contributes considerably to the value of the final roughness value, the final form of the model for calculating the effective IRI value at any time based on traffic data using the function with highest R^2 can be calculated as follows:

$$R(t) = R_0 + \Delta IRI \qquad 4.26$$

Where:

$R(t)$= Roughness level at any time t

R_0= Roughness level at zero age and it is equal to :

R_0= 0.744 for the fast lane

R0= 0.866 for the slow lane

The predictability of the above models can be enhanced by incorporating more data about pavement sections to cover wider range of the existing conditions. As can be seen from Figure 4.17 below, traffic loading in terms of ESAL accounts for about 67% of the deterioration models which indicates the partial importance of such factor on pavement performance.

Other studies have confirmed similar conclusions that IRI changes under traffic effect but the variation in the IRI values was very small(Charles E *et al* ,2003).

In comparison with models developed using Age factor, the value of models good of fitness represented by R^2 is relatively lower as the age variable accounts for climatic

variables which also affects pavement performance with percentage of at least 15% of the pavement deterioration. the remaining of the models predictability can be obtained by introducing other minor factors such as structural number and the level of maintenance applied to the pavement section. The model shows that as the number of axle loads ESAL increases, the roughness increases accordingly. The application of the Traffic–IRI models is shown in Figure 4.18. As can be seen from the figure, polynomial function gives much more accurate and realistic predictions than the power function. In deterministic models developed based on local data , polynomial functions has been found to suitably represent the deterioration trend of the collected data.

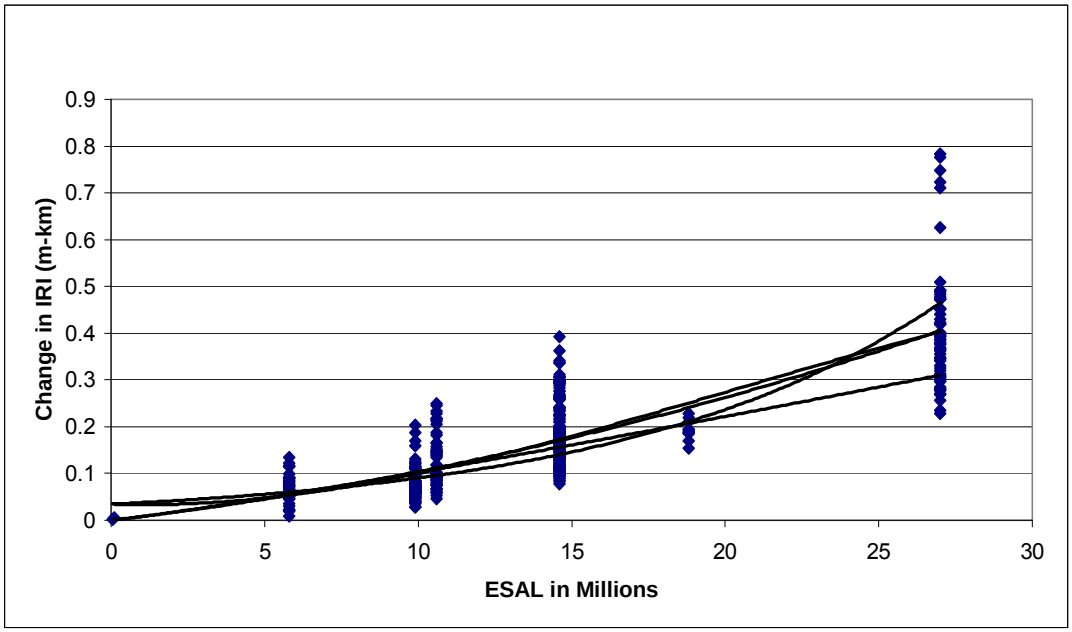


Figure Error! No text of specified style in document.**.7: Scatter gram for the change in IRI values against ESAL for the slow lane (design lane) and suitable functions fit the general trend.**

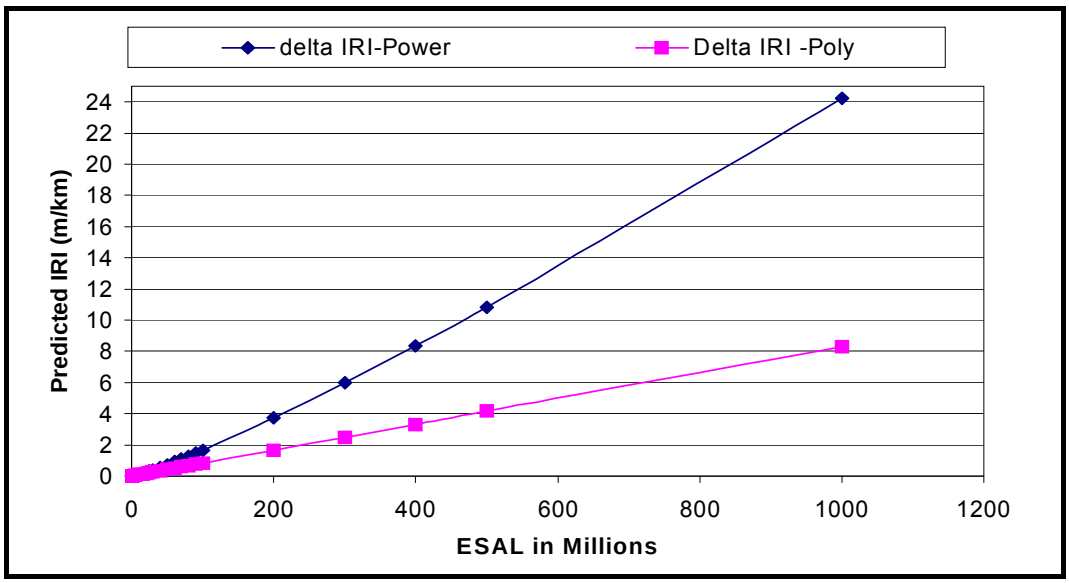


Figure Error! No text of specified style in document.**.8: Predicted IRI values against ESAL using power and polynomial functions**

Roughness -Present Serviceability Relationship

Many research works showed that IRI scale is closely correlated to the PSR scale as both measures are based on vehicle riding perception and the response of various modes of the vehicle performance. World Bank studies showed A relationship between IRI and Present Serviceability Index (PSI) can be described as shown in the equation below (Patterson 1986).

$$PSI = 5\ e^{-(IRI\ /\ 5.5)} \qquad 4.27$$

Where:

PSI = Current Present Serviceability Index for each homogenous roughness section.

IRI = The mean International Roughness Index of the homogenous roughness section.

Data collected from various pavement sections in the study areas showed a highly correlated relationship between PSR and the IRI measurements (Figure 4.19). The models that correlate the IRI with PSR were found to take the following forms:

$$PSR = 4.6836e^{-0.1859IRI} \qquad 4.28$$

$R^2 = 0.9143$

$PSR = 0.0524\ IRI^2 - 0.8373IRI + 4.7136$ 4.29

$R^2 = 0.8886$

$PSR = -0.0066IRI^3 + 0.1267IRI^2 - 1.0723IRI + 4.8962$ 4.30

$R^2 = 0.8901$

$PSR = 4.0444\ IRI^{-0.4459}$ 4.31

$R^2 = 0.8832$

The relationships in the above equations can be used to determine the current PSR of pavement sections under consideration if international standards such as AASHTO for the new roads to be applied. Models developed in this study indicate that the PSI values can be predicted using direct roughness measurements.

The goodness of fit of the regression line in terms of Coefficient of Simple Determination (R^2), which measures the proportion of the total variation about the mean based on one independent variable , is relatively high for all models developed using different function forms. These models are practically acceptable in the view of using one independent variable easily measured in the field i.e. roughness index-IRI. All the models indicate that , as the roughness increases, the PSI decreases accordingly.

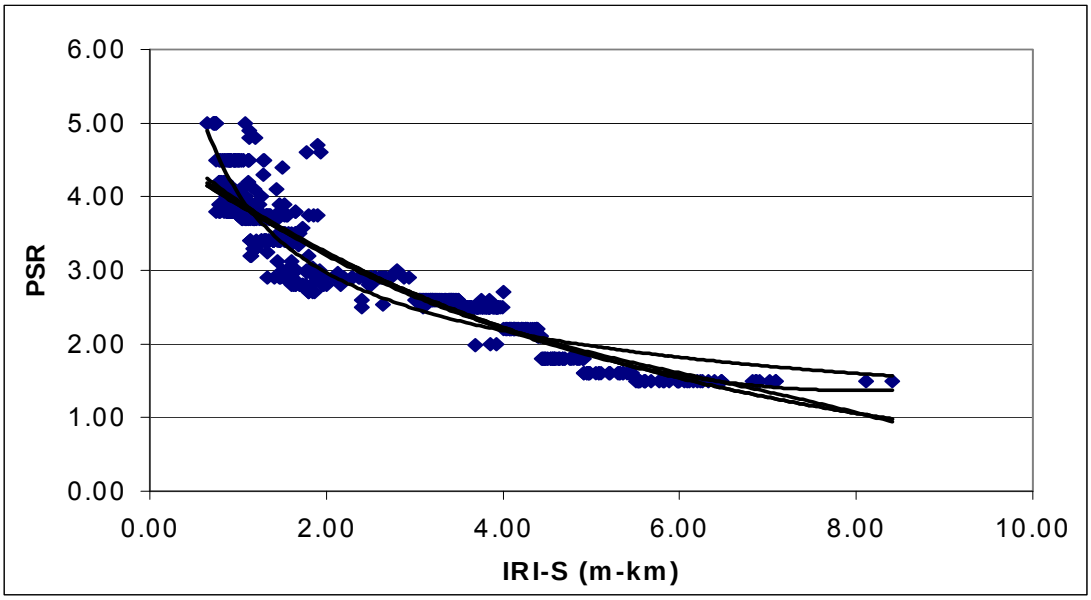


Figure Error! No text of specified style in document.**.9: Scatter gram for the control IRI-s for the slow lane against PSR and suitable functions fit the general trend.**

The application of all models developed to predict PSR value is shown in Figure 4.20. below. Among the functions selected to model the data trend, Exponential function showed the best fit for the relationship between the Present Serviceability Rating PSR and the IRI as shown in the application of this function in Figure 4.21.

The polynomial (both 2nd and 3rd degree), Logarithmic and Power functions were all excluded from the applications as they give unrealistic values at certain IRI values. The predicted PSR against actually measured PSR values calculated using the exponential function models are shown in Figure 4.22 below.

As can be seen in this figure, the values of both measured and estimated or predicted PSR lie almost on 45^0 line which indicate good predictability.

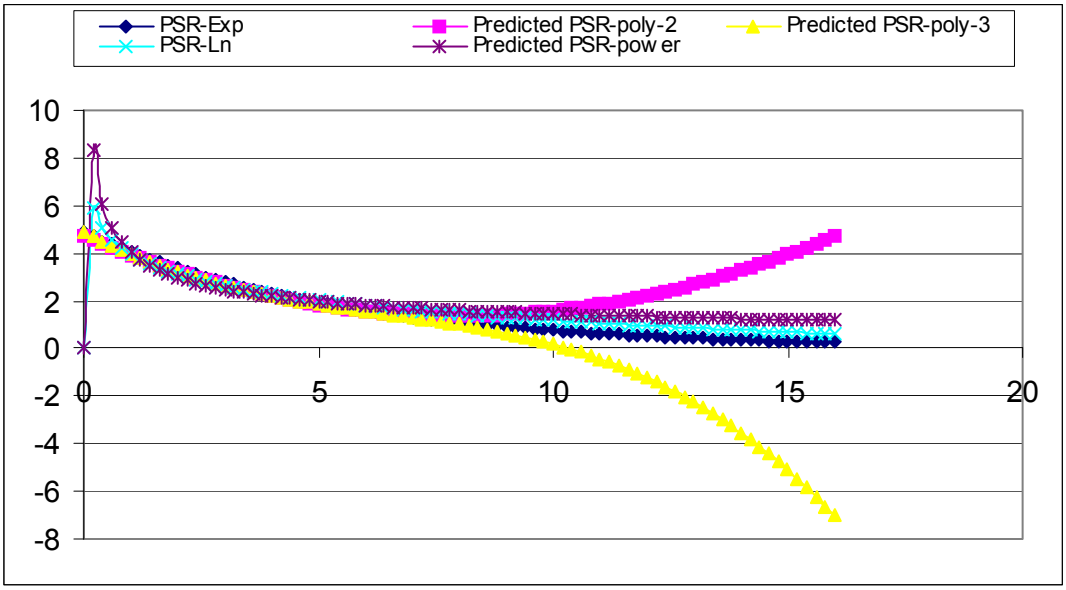


Figure Error! No text of specified style in document.**.10: The application of the IRI-PSR models developed using different functions.**

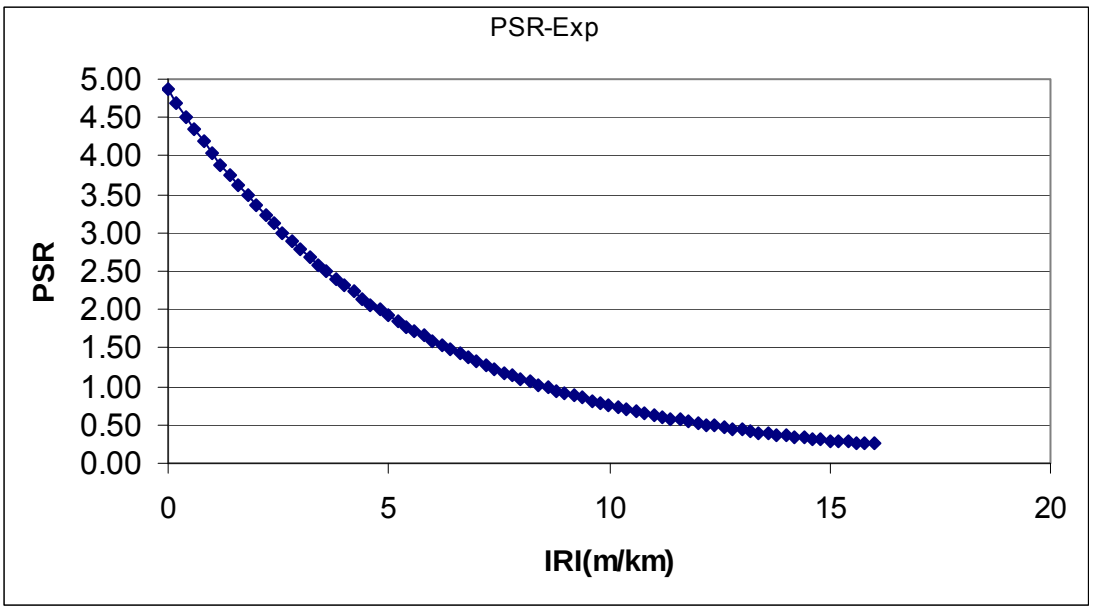


Figure Error! No text of specified style in document.**.11: The application of the IRI-PSR models developed using exponential function.**

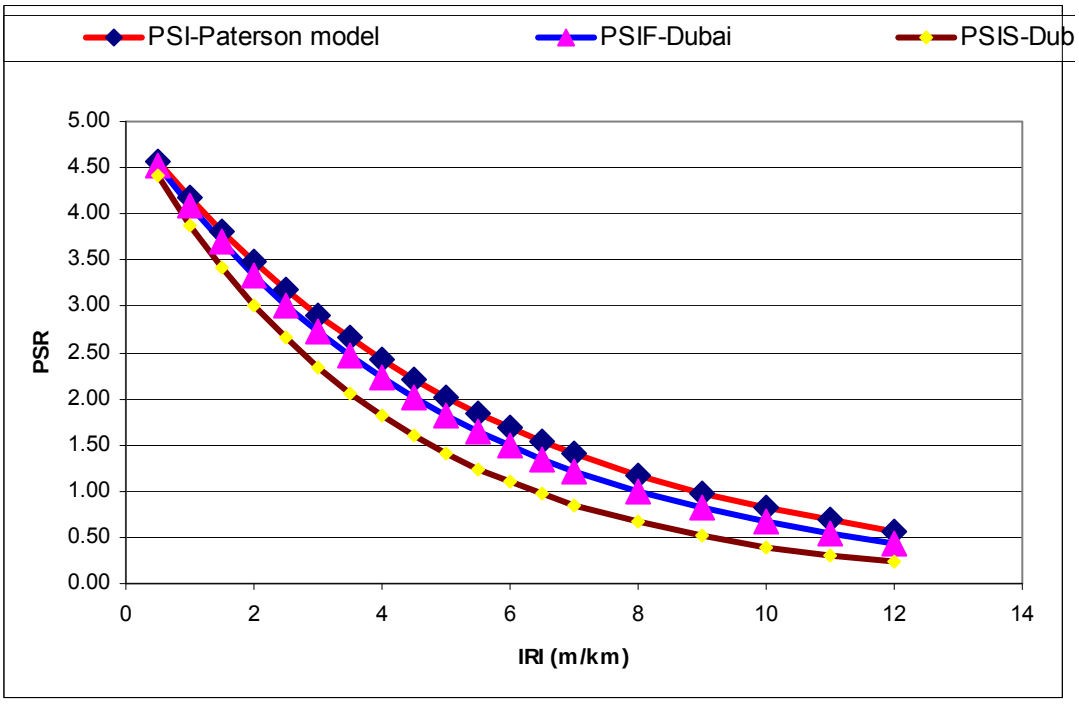


Figure Error! No text of specified style in document.**.12: IRI-PSR developed model in comparison with the World Bank model (Paterson 1986)**

Functional failure approach based on roughness

It is well known that , as the number of axle loads increases, the PSI decreases accordingly (Chiu Liu 2000). Based on the developed models, the roughness in terms of IRI corresponding to the terminal PSI value of 2.5 is approximately equal to 2.9-3.4 using all models developed.

As shown in the application of all models developed for this purpose, the exponential function showed the highest goodness of fit with R^2 =0.91. Other functions such as polynomial and power functions were excluded as they give unrealistic values at certain IRI values. Therefore, it was decided to calculate the failure limit for the pavement using this function. The limits were found to be as follows:

Table 4.7: Terminal PSI and the corresponding Terminal IRI established using the developed models.

Road Class	PSI-t	IRI-t (Power)	IRI-t (Exp)	IRI-t ($Poly^2$)	IRI-t ($Poly^3$)
Freeways, Expressways, Arterial, collector	2.5	2.9	3.4	3.0	3.2
Residential collector and Local roads	2.0	4.9	4.4	3.6	4.4

These IRI values are referred to as the Terminal IRI values. Therefore, pavement sections with an existing roughness greater than 3.4 which corresponds to a PSI less than 2.5 are considered to be at the end of their service lives. *The Remaining Service life(RSL) is defined as number of years until a pavement section provides no more service to the traveling public based on roughness level*. At this time, it will be in need for a treatment to reduce the roughness, such as milling the surface and/or applying a thin or structural overlay and it may in some cases go far beyond these options to require reconstruction.

Structural Condition Indicator Performance

As pointed out previously in chapter 3, pavement performance can be considered from two aspects (AASHTO1993). First; *Functional performance* which describes how well the pavement serves the road user. It is mainly characterized by the riding comfort or ride quality measures. The second aspect is the *Structural performance* which is related to the pavement physical condition such as the occurrence of cracking, rutting and other distresses that adversely affect the load carrying capacity of the pavement structure.

The use of non destructive deflection testing has been an integral part of the structural evaluation and rehabilitation process for many decades. Deflection data has been used in different format to report about the effective structural capacity and the remaining life of the existing pavement.

To be an effective reporting system, Pavement Management System should encompass a complete set of information about the main condition indicators are needed to optimize the selection procedure for the maintenance and rehabilitation alternatives. The optimization of the treatment selection method and the relevant life cycle costing for the selected options will be achieved by combining all pavement condition indices in such a way , all encountered problems will be over come.

In PMS applications at network level, roads agencies always prefer to use simple procedures based on a few indicators such as distress survey, and age to report about pavement condition and the needed treatment. *This simplification makes the treatment selection not fully optimum and resulted in an increase in life cycle costing.* Therefore, it was found that using multi –indices condition indictors is much more helpful in selecting the appropriate treatment to fully restore the riding quality and the surface structural integrity (kameyama et al , 1998). The sophisticated issue in using such method is the determining the suitable weight that should be given to each index.

Structural Number Concept

Structural Number (SN) is considered the basic parameter for designing flexible pavement proposed by AASHTO. Despite the fact that most of the roads agencies, including AASHTO, are directing their efforts towards the development of a mechanistic-

empirical procedures for pavement design, AASHTO pavement design procedure is still considered the most widely used methods for flexible and rigid pavement design. The AASHTO pavement design procedure has promoted the notion of assigning a Structural Number (SN) to represent the strength exhibited by the pavement structure. In principle, Structural Number is an index that quantifies the strength of the total pavement structure by giving simple and abstract quality. (Michael Pologruto, 2001). To be able to use this number, the layer coefficient for the materials used in construction should be determined. Unfortunately, the determination of such coefficients is not an easy task to be used directly in calculating the effective structural capacity which is usually used in PMS. The general equation to calculate the structural number for the new roads is given below (AASHTO1993):

$$SN_0=a1*D1+a2*D2*m2+a3*D3*m3 \qquad 4.32$$

Where:

SN_0 =Structural number of the new pavement structure .

a1,a2,a3=Layer coefficient for asphalt, aggregate base, and Aggregate subbase layers.

m2,m3=Drainage coefficient for the unbound material determined based on certain criteria.

One of the main shortcomings of the structural number method is the determination of the layer coefficient for different pavement layers. The layer coefficient proposed by AASHTO were developed for particular materials used in the construction of the AASHO experimental road sections. AASHTO design model does not recommend any procedure to determine these layer coefficients. Therefore, a well established method to evaluate the effective structural capacity is deemed to be badly needed.

Use of deflection data to Calculating Effective structure Number

For the determination of the structural capacity of the existing pavements, various methods are available among which the Non-destructive (NDT) testing (Deflection) is the most credited and inexpensive(Shahin,1998). This method does not disturb the traffic movement and the underneath pavement layers. In general, NDT can be used in

evaluation process of the asphalt pavement for obtaining deflections, stress and strains, elastic modulus of the pavement layers , deficiency in the pavement layers (critical layers), remaining structural life and the overlay thickness needed to enhance the structural capacity to resist the imposed future traffic.

For the purpose of development a simple, reliable and easy to use procedure to estimate the effective structural capacity and the remaining life of pavements, hundreds of asphalt-surfaced pavement sections were tested using FWD and analyzed. The selected pavements represented a wide range of traffic levels, ages, structural compositions. The total thickness of the pavement (combined asphalt, base and subbase layers) for the pavement sections tested ranged from 300mm for local roads to 900mm for major roads.

Deflection testing of the pavements was conducted using a Dynatest 8000 series FWD, with a load level ranging from 40-80 kN, simulating the range of traffic level that might be traveling on these pavements. All deflection values were normalized to a standard temperature of 21°C and of a load of 40 kN.
Structural Number is a commonly used index that attempts to quantify the strength of the total pavement structure in a simple and abstract quality (Pologruto 2001).

Many researches have indicated that the effective SN_{eff} can be calculated by various mechanistic methods based on deflection and other data elements measured in the field. One of the approaches to evaluate the effective SN_{eff} is by using the direct deflection measurements. The following model was suggested in this regard (Jameson, 1992):

$$\mathbf{SN_{eff} = 13.47 - 6.47 * \log(D_0) + 3.697 * \log(D_{900})} \qquad 4.33$$

where:
SN_{eff}=Effective Pavement Structural Number
D0=Central deflection corrected to the standard temperature in microstrains
D900=Normalized deflection at a distance of 900mm from the center of the loading plate in microstrains

Rohde (1994) proposed the following model to evaluate the effective structural number based on deflection measurements. It takes the following form:

$$\mathbf{SN_{eff} = k1*(D_0 - D_{1.5}*Hp)k2*Hpk3} \qquad 4.34$$

where:

D0= Central Deflection in microstrains

Hp= Total Pavement Thickness (mm)

k1, k2, k3 =Regression Coefficients.

AASHTO (1993) recommends the following simple relationship to estimate the effective structural number for use in PMSs:

$$\mathbf{SN_{eff} = 0.0045*D*Ep1/3} \qquad 4.35$$

where:

SN_{eff} = Effective Structural Number in inches

D=Total thickness of the pavement structure above the subgrade in inches

E_p = Effective pavement modulus in psi effective elastic modulus for the total pavement structure using the deflection reading measured at the center of the loading plate (D_0)

The World Bank's deterioration models adopted in HDM3 used a modified structural number as an indicator of the structural capacity in pavement deterioration modeling. The model used in HDM3, and later adopted in HDM4, correlates the deflection measured by Benkelman Beam and the structural number, and takes the following forms (N.D Lea 1994):

$$\mathbf{DEF = 6.5*SNC^{-1.6} \quad or \quad SNC = 3.2\ DEF^{-0.63}} \qquad 4.36$$

where:

DEF=Deflection measured by Benkelman Beam

SNC= The Corrected Pavement Structural Number.

Benkelman Beam is no more in use as it is not accurate enough to determine the mechanistic behavior of the pavement structure particularly with the evolution in the

powerful software packages that can quickly and accurately calculate the stresses and strains in the critical points in the pavement structure and estimate the remaining life.

It can be seen from the above formulas that different methodologies give varying structural number results. The HDM3/HDM4 and Rohde formulas calculate the structure number for peak deflections, whilst the Jameson formulas and AASHTO are using central deflection, deflection at 900mm and 1500mm from the load and the pavement modulus Ep For local use, these models should be studied thoroughly and calibrated to suit the local conditions if adopted for use in PMS.

To develop deterministic models for PMS uses, the Statistical Software Package SPSS was used to investigate the correlation between SN_{eff} and the list of variables listed in the correlation table shown Appendix A, (SPSS 2003). The actual SN_{eff} was determined using pavement surface condition data along with the pavement layers thickness obtained from the as built documents which are well maintained in Dubai RTA PMS Unit. Pavement sections with missing layer thickness information were cored and cut out to obtain the required information.

The layer coefficient values were assigned to the materials of the in service pavement depending on the types and amounts of defects present. Table 5.2 of the AASHTO pavement design guidelines was used to estimate the layer coefficient values (AASHTO 1993). The statistical characteristics tables show that the Pearson correlation coefficient between the dependent variable SN_{eff} and many independent variables are relatively high.

As shown in this table, SN_{eff} correlates well with the maximum deflection (D_0, r =-0.615), pavement thickness (H_P, r =0.963) and the pavement asphalt curvature represented by the value of (D_0-D_{200}, r=-0.784), or (D_0-D_{300}, r =-0.832) and the curvature in the whole pavement represented by the value of (D_0-D_{450} r =-0.834),) and the Log(D_0-D_{450}, r =-0.839),) and the deflection in subgrade represented by the value of (D_0-D_{1200}, r =-0.716). In this regard, r was used to preliminary check the strength and the trend of the relationship between various variables. After considering many different transformations, it was found that the best model to predict the SN_{eff} using only two variables took the following form:

$$\mathbf{SN_{eff}=-33.01+0.251*H_p+0.185*D_0}$$

r^2=0.934, adj- r^2 =0.933 4.37

Adding the variable (D_0-D_{450}) improved the predictability of the model slightly.

Other models were developed based on deflection (D_0) found suitable with less predictability. These models were found to take the following forms:

$$\mathbf{SN_{eff} = 2807.9\,(D_0)^{-0.65}}\ ,\quad r^2=0.31\ ,\ \text{adj-}\ r^2=0.30 \qquad 4.38$$

$$\mathbf{SN_{eff} = 304.8\ e^{(-0.0074D0)}}\ ,\ r^2=0.35\ ,\ \text{adj-}\ r^2=0.34 \qquad 4.39$$

where:

SN_{eff}=Effective Structural Number in mm

D_0 =Maximum normalized deflection measured at the plate center in microstrains

Table A-10, Appendix A contains Analysis of Variance (ANOVA) tables for the analyses. These tables show that the goodness of fit, represented by the Coefficient of Multiple Determination (r^2), for the model in equation 4.37 was high, indicating that all the variables and the regression coefficients included in the model are significant and can be considered as good predictors of the effective structural capacity of the pavement.

The included variables in both models were found statistically significant at significance level ά=0.001. The slight difference between the value of r^2, and the adjusted-r^2 indicates that all the variables included in the model are significant at the selected level of significance (ά=0.001). The predictability of the model in equation 4.37 is also verified by the high F value and the corresponding significance level in addition to the low value for the standard error of estimate as shown in the ANOVA table.

The introduction of the variable D_0-D_{450} has slightly improved the predictability of the model. Therefore, for PMS practical use, the model in equation 4.37 is recommended. Figure 4.23 contains a plot of predicted SN_{eff}, determined using equation 4.38, and observed values of SN.

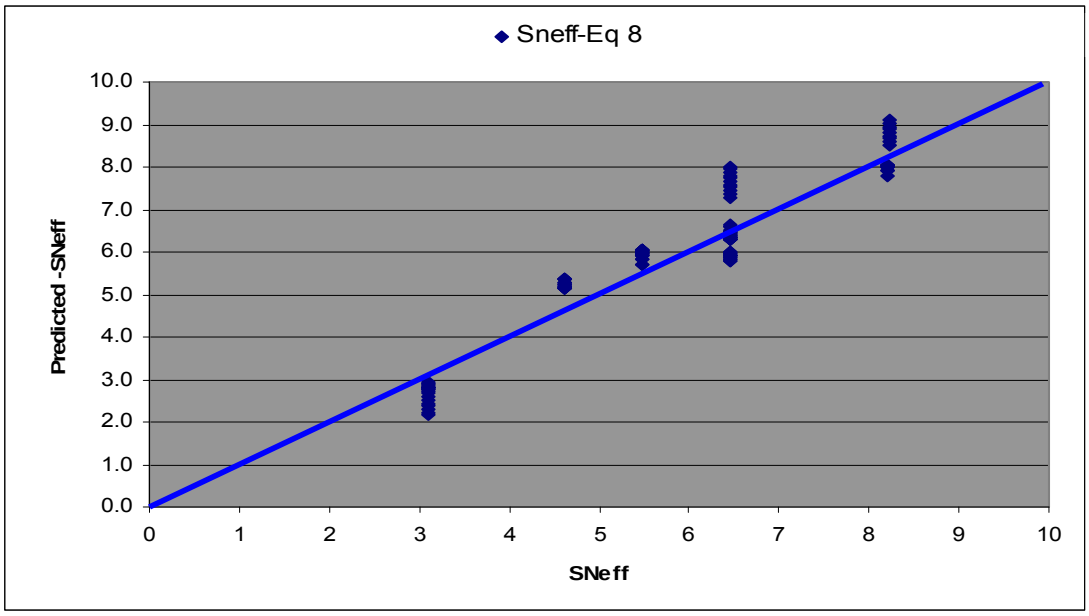


Figure Error! No text of specified style in document.**.13: Predicted Vs measured SN_{eff} (inches) values using the developed models**

The statistical characteristics of the developed model indicates high predictability. These models can be used in PMS to estimate the SN_{eff} for pavement sections based on the direct deflection measurements and pavement thickness information. Other models shown in equations 4.38, and 4.39, were developed based only on deflection variable (D_0), and introduced for comparison against the HDM and Jameson models (see Figure 4.24).

Comparing the developed model with other models, it can be seen that Jameson and Dubai models give close values of SN_{eff}, while the HDM model give higher values which suits very thick pavement. According to the experience gained form the field and practice, it is rarely to find a pavement of SN more that 7.0 . This difference may due to the fact that HDM method for calculating the SN_{eff} depends on the peak deflection using Benkelman beam which should be corrected for temperature and load, while Dubai and Jameson models are dependent on the deflection under the load and at 900mm and 1500mm from the load.

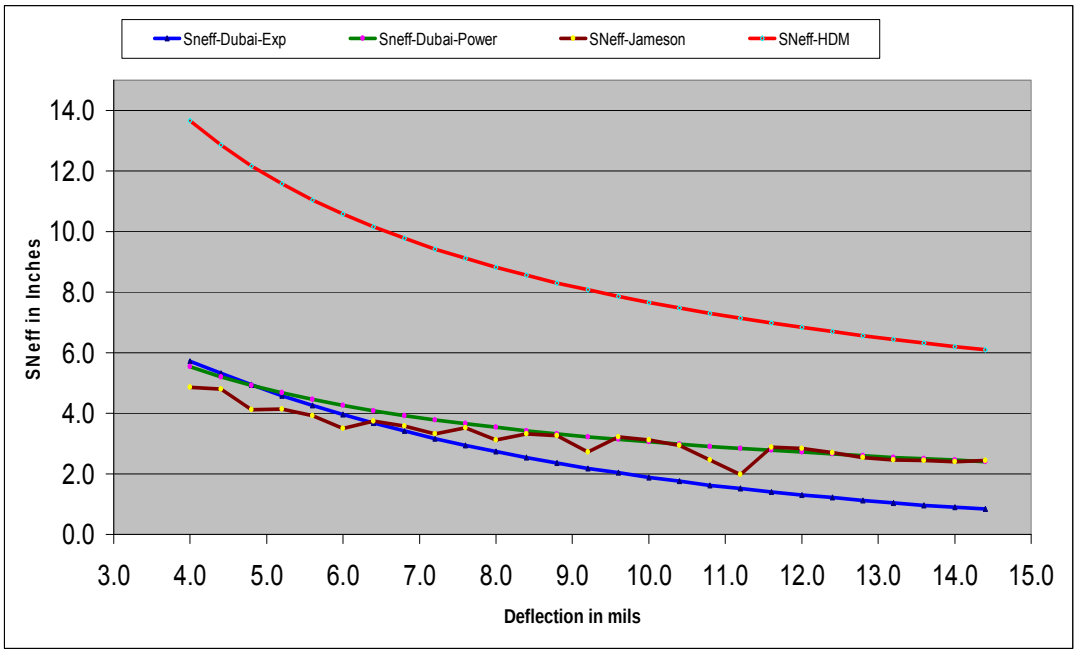


Figure Error! No text of specified style in document.**.14: Comparison of the Predicted SN_{eff} (inches) using HDM, Jameson and Dubai models using Power and exponential function.**

Effective structural Number based on Distress Data

.The concept behind this correlation stems from the fact that the structural capacity decreases as cracks and rutting start to appear in the pavement body. This deterioration is proportional i.e. as the percentage of cracks and rutting increases, the pavement capacity to absorb more traffic repetitions decreases until it reaches a point where it needs rehabilitation. At this point, the pavement will be reported as structurally failed.

This index is easy to calculate as a stand alone condition indicator or it can be easily merged into the OPQI index.

$$\mathbf{SN_{eff} = 0.992\ (PQI/PQI_{max})^{0.383}}$$

$$\mathbf{SN_{eff} = 0.992\ (PQI/100)^{0.383}} \qquad 4.40$$

where:

Effective Structural Capacity Index (0-100)

SN_{eff} = Effective Pavement Structural Number.

PQI = Pavement Quality Index of the pavement section (0-100) at time x

PQI_0= pavement Quality Index at zero age and it is by default =100.

The SN_{eff} value was calculated according to the guidelines Table 5.2 in the AASHTO method . This model was developed in this study based on Dubai data and also found to have good predictability.

ESCI can be calculated relative to the original structural number measured or estimated before opening the road to traffic. Using one of the above models developed to calculate the effective structural number, The ESCI can then be determined using the following expression:

$$\mathbf{ESCI=100-((1-(SN_{eff}/SN_o))*100)} \qquad 4.41$$

where:

ESCI= Effective Structural Capacity Index (0-100)

SN_{eff} = Effective Pavement Structural Number.

SN_o =Original (Design) Pavement Structural Number

As an attempt to explore the possibility of using central deflection D_0 as an indicator to evaluate structural capacity, the relationship between ESAL and the deflection D_0 was investigated. It showed that a reasonable correlation exists between these two factors. The following models were developed:

$$\mathbf{D_0 = 2E\text{-}05\ (ESAL)^{0.5343}} \qquad ,R^2 = 0.6009 \qquad 4.42$$

$$\mathbf{D_0 = 0.0765e^{3E\_-08\ (ESAL)}} \qquad ,R^2 = 0.536 \qquad 4.43$$

It was noticed that deflection as a single value can not be considered a good indicator of the structural capacity of the pavement. As shown in the Figure 4.25 below, two different pavement sections exposed to different traffic loading of 15million and 35million are having the same deflection value of deflection of around 0.19mm.

There are many reasons for this criterion among these; deflection value is usually decreases proportionally with the pavement depth. On the other hand, stresses and

strains decrease proportionally with the square of pavement depth. *This means that surface deflection parameter is not the ideal parameter to be selected to predict the effective structural capacity of the existing pavement at network level.* It could be used successfully to locate the weak spots within the pavement of a single section of the same characteristics by comparing the deflection data along the entire length of the section itself. Therefore, the search for another parameter to evaluate the actual effective structural capacity is deemed to be vital.

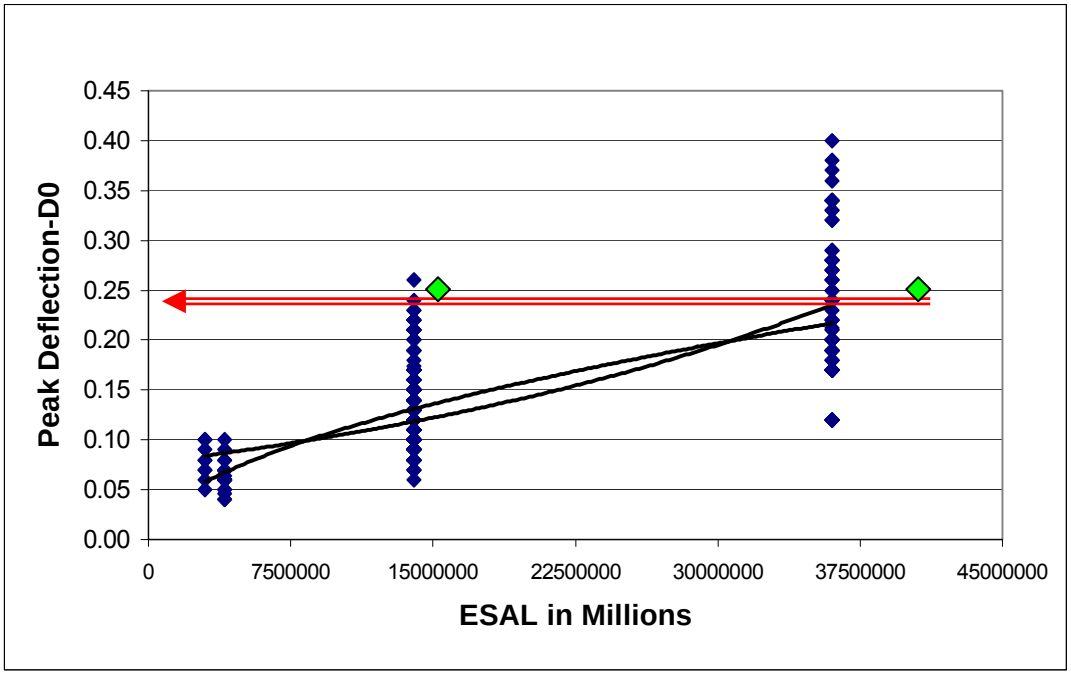


Figure Error! No text of specified style in document.**.15: Peak Deflection (D_0) against traffic loading in terms of ESAL.**

Bearing capacity of the pavement is defined as "*The number of the standard loads that pavement can carry before reaching to certain amount of structural or functional distress*". All new pavements deteriorate slowly in the first 10 to 15 years of service , and then it deteriorate much more rapidly in the subsequent years unless timely maintenance is applied. In pavement management system, the Effective Bearing Capacity or the Remaining Bearing Capacity that the pavement can serve under the prevailing conditions is what is needed in PMS (i.e. how much more load in terms of ESAL can the pavement handle in the remaining years of service life)

After many researches and investigations, recent technical advances in this field indicated that, if deflection is to be considered as performance predictor, the most

accurate assessment of the pavement performance can be achieved through the use of ***maximum deflection*** in combination with an indicator of the ***radius of curvature*** of the pavement under load (which means high stiffness value). Therefore, the NDT deflection pavement structural capacity assessment method requires the use of deflection basin measurement rather than the maximum deflection alone (AASHTO1993).

Effective pavement structural capacity based on Pavement Modulus (Ep) value

Pavement Modulus (E_p) is the most important parameter that indicates the effective structural status for the pavement in service. It is highly correlated to the physical condition of the pavement. It also reflects the cracking and rutting existence so that accurate predictions can be expected if this parameter is used in PMS systems for estimating the remaining structural life and calculate the future maintenance and rehabilitation needs.

Excessive stresses resulted from both traffic loading and environmental loading will eventually result in the formation of micro-cracking of the material. These cracks will in turn reduce the effective cross sectional area of the layer. The reduction of the cross sectional area of the pavement layer will increase the intensity of the stresses in the remaining intact material.

The results of the aforesaid actions will lead to considerable reduction in the value of the modulus of the layers under stresses leading to one of the main failure modes in pavement i.e. fatigue cracking or permanent deformation.

Therefore, the author of this thesis used data collected from hundreds of deflection basins to develop a simple equation for estimating the effective Ep and predicting the effective structural capacity for each pavement section using only one independent factor which is the traffic level in terms of ESAL.

The developed equations were found to take the following forms (Figure 4.32):

$$E_p = 2.5841ESAL^2 - 150.8\ ESAL + 2799.2 \qquad 4.44$$

$R^2 = 0.9515$

$$E_p = 2759.6e^{-0.0451ESAL} \qquad 4.45$$

$R^2 = 0.8562$

Where

E_{peff}= Effective pavement modulus (Mpa)

ESAL= Accumulated equivalent axle loading traffic to date (Millions)

As can be seen from the Figure 4.32 below, the exponential functions proved again that it represents the actual deterioration trend much more better than the polynomial function.

The polynomial function changes its sign at certain traffic level yielding higher Ep value as the ESAL increases. On the other hand exponential function showed a logical decreasing trend as the ESAL increases.

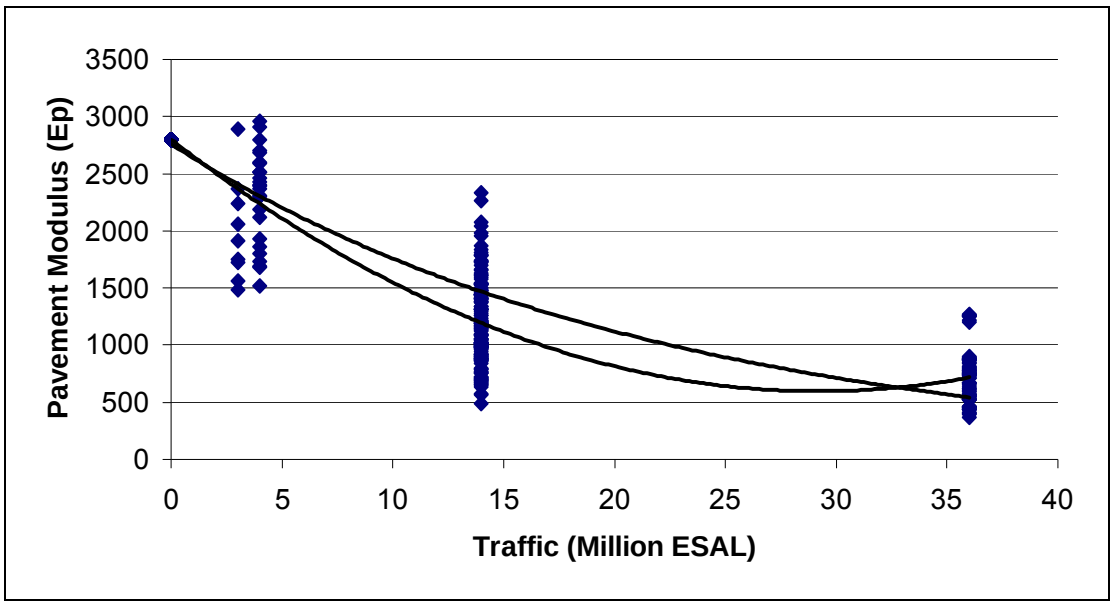


Figure Error! No text of specified style in document.**.16: Pavement Surface Modulus (Ep) against traffic loading in terms of ESAL.**

The application of the above developed model is shown in Figure 4.27 below. As can be seen from this figure, the predicted value of the Epeff/Ep0 decreases as the ESAL increases. At traffic level of 35millions standard repetition, the value of the E_p is approximately equal to 433 Mpa which is almost equal to the value of aggregate base course. This indicates that the pavement is heavily cracked and the asphalt is no more working as a bound material. This condition was frequently observed in the heavily traffic roads in which the pavement was completely damaged and the surface is badly cracked and raveled.

For an easy use in PMS , linear approximation was adopted to calculate the effective structural capacity -ESCI of the pavement based on decrease in modulus value using the following proposed equation:

$$ESCI=100-((1-(Ep_{eff}/Ep_{o})))*100) \qquad 4.46$$

Where:

ESCI= Effective Structural Capacity Index

Ep_{eff}= Effective pavement modulus.

Ep_o=Original (Design) pavement modulus assumed 2800 Mpa

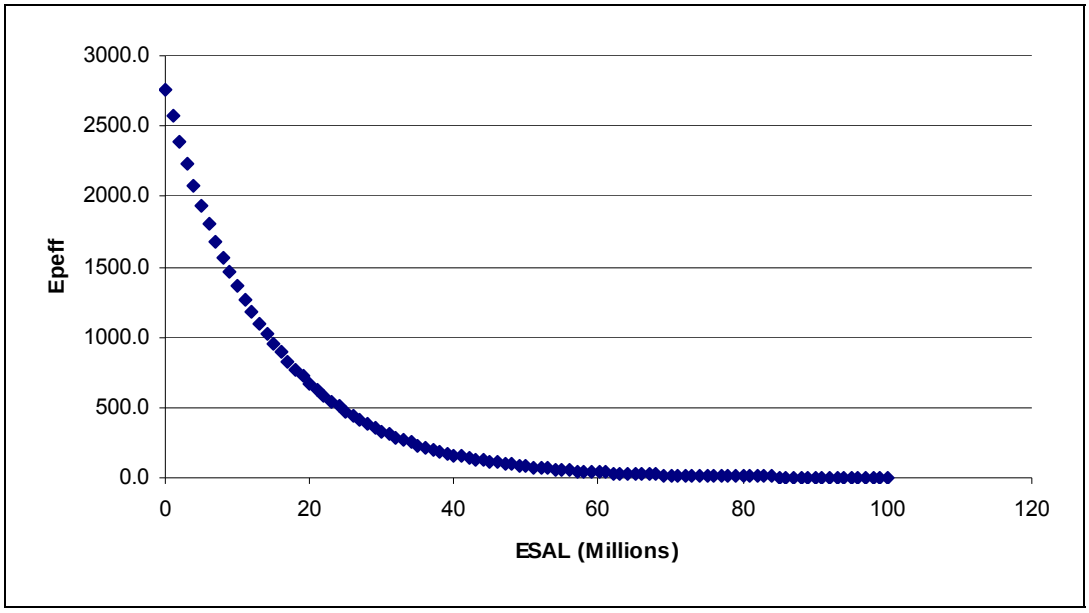


Figure Error! No text of specified style in document.**.17: The application of the developed exponential model to predict the Pavement Surface Modulus (Ep) value.**

The changes in the value of the asphalt modulus can be used effectively as a structural failure criteria. As shown in the aforesaid sections, the reduction in asphalt modulus is a function of the applied traffic loading represented by the value of past ESAL. As shown in Figure 4.27, the value of the Ep_{eff} or the relative value of Ep_{eff} decreases linearly with the number of ESAL up to the value of 15-20 millions ESAL. Then it decreases exponentially at the remaining potion of the figure until it reaches the complete failure at the end of its service life. In general, exponential function is the dominant form of the deterioration trend as indicted by all developed models.

The analysis of the collected data for the cracked portions for many pavement sections tested by FWD has indicated that When the cracking becomes observable or visible on the pavement surface (i.e. the severity ranges from low to high), the effective asphalt or pavement modulus was found to be about 20 - 35 %of its original design value. This means that the pavement structure has already lost around 65 to 80% of its effective cross sectional area. *This value was chosen as a structural failure criteria to be used in treatment selection procedure as will be discussed in the chapters to come.*

The asphalt modulus can be considered as a representation for the pavement modulus since the value of the elastic modulus for the unbound material do not vary considerably with time passing unless it is affected by water. Most of the fatigue or permanent deformation was observed to occur in the upper courses of the asphalt layer.

In conclusion, It is worth indicating here that the concept of the relative value of effective pavement modulus to the original pavement modulus at the pavement age equal to zero (i.e zero ESAL) or the relative value of effective SN_{eff} to the original SN_0 has been adopted in this study as it gives much more better representation about the *exhausted* and the *remaining* lives of the existing pavement structure.

Rutting and Fatigue Cracking prediction models

Rutting and Alligator cracking are considered as the most important distress types that lead to the structural and functional pavement failure. Therefore, in all newly developed mechanistic -empirical approaches used in pavement design and analysis. The occurrence of those types is considered as the main failure modes in the flexible pavement.

Rutting can be used to infer the structural capacity of a pavement and to identify locations where water (from rainfall or vegetation watering operations) may pond and induce “aquaplaning” in vehicular traffic.

In any pavement management system, the condition and distress types must be always known at any time. Unfortunately, it might happen that certain circumstances could prevent carrying out the pavement condition survey on time each year. Therefore, models to predict the future condition and distress propagation by time under traffic and environmental loading, in particular for the main distress types such as alligator cracking and rutting, are of great interest.

If the condition surveys are not undertaken on time for a reason or another, PMS can use various deterministic prediction models developed based on data collected from the field at network level. The following models developed by the researchers can be used as an alternative to predict both alligator cracking and rutting using different parameters (Fwa, et al, 1998) :

$$C = 21{,}600(N)\,(SN)^{-SN} \qquad 4.47$$

$$R = 4.98(Y)^{0.166}\,(SN)^{-0.5}\,(N)^{0.13} \qquad 4.48$$

where:

C=Total area of the cracking in m/km/lane

N=Past Traffic Loading in ESAL (Millions)

R=Rut Depth in mm

Y=Age of the pavement since construction or last overlay

S=total Area disintegrated in m/km/lane

SN=pavement structural Number.

Rutting can also be predicted using the following model(Neil et al, 1993):

$$RD = 0.026 + 0.000191\,(ESAL) \qquad 4.49$$

Where:

R=Rut Depth in mm

ESAL=Past Traffic Loading in ESAL (Millions).

And (Hans Ertman et al, 1998)

$$R=1.44*10^{-6}*N^{0.23}*\mu\epsilon^{1.536} \qquad 4.50$$

Where:

N=Past Traffic Loading in ESAL (Millions)

$\mu\epsilon$=Measured Plastic strain (microstrain)

R=Rut Depth in mm

Skid resistance prediction models

There are several methods and tools to evaluate the pavement skid resistance and the safety characteristics. The simplest and the cheapest method to evaluate the skid resistance is to use British Pendulum fiction tester. It is basically composed of a hard rubber pad attached to a free-swinging metal arm that is designed to touch a fixed length of pavement at each testing point. Upon testing, the result is directly marked on a dial scale by a recording arm which represents the BPN. This tool is portable and it has the capability to measure the skid resistance in terms skid resistance value (SRV) that is equivalent to the value of patterned tire traveling at speed of about 50 km/h. In addition to using this tools as a skid resistance tester for the asphalt-surfaced pavement, it is being used also, to test road marking friction characteristics and skid resistance

Skid resistance of a pavement surface is influenced by both micro-texture and macro-texture. Micro-texture is generally controlled by the selection of the aggregate type. Macro-texture refers to the large-scale texture of the pavement as a whole due to the aggregate particle arrangement. (Jayawickrama *et al*,1996)

As a safety measure, inadequate skid resistance will lead to higher incidences of skid related accidents. Most agencies have an obligation to provide users with a roadway that is "reasonably" safe. Skid resistance measurements can be used to evaluate various types of materials and construction practices.

Many researches have indicated that skid resistance varies with time. Typically it increases in the first two years following pavement construction as the bitumen worn away by traffic and rough aggregate surfaces become exposed, then decreases over the remaining pavement life as aggregates become more polished (Anderson *et al*,1986, Skerritt *et al* , 1993). Seasonal variation was also reported in these studies to have detrimental effect on the skid resistance value, i.e. skid resistance rises in Fall and Winter and falls during late Spring and Summer.

The critical case of skidding is envisaged to be when the surface is wet. Therefore, all tests are recommended to be carried out using watering system to simulate the actual critical case. As previously indicated, pavement will perform effectively and resist skidding if both the macrotexture and microtexture are well designed. The macrotexture and microtexture properties of the pavement surface control the frictional characteristics in two aspects:

1. Microtexture controls the frictional capability of the surface in dry condition through the effect of the aggregate surface grittiness, roughness, and angular surface.
2. On the other hand, the macrotexture controls the drainage ability of the pavement surface by preventing the formation of the water film .therefore, both textures determine the effectiveness of the pavement surface from frictional and safety point of view.

The International Friction Index (IFI) is an international scale of friction value similar to the IRI measuring index developed to report about the friction characteristics of the pavement in wet condition. This index was developed to account for the variation in methods and systems from one country to another. Till present time , general correlation between the output from these systems had not been achieved as it is necessary always to include the effect of the surface (microtexture) texture in the calculations.

In PIARC experiment, pavement texture and friction data were analyzed using different models. Based on this experiment, it is recognized that Friction scale should consist of two numbers(Odoki, *et al*, 2000):

1- Friction (Microtexture) related number

2- Macrotexture related number

The model developed by PIARC takes the following form:

$$F(s)=F60.\ e^{(S-60/\ Vp)} \qquad 4.51$$

Where:

F(S)= True Friction –velocity relationship for a pavement section.

S= Slip –Speed in km/h

F60= friction value (Factor) at 60km slip speed (i.e microtexture friction number).

Vp=Macro-texture related parameter of the model called velocity number.

The velocity number is related to the macro-texture (aggregate arrangements on the pavement surface). It can be predicted by analyzing the texture measurements obtained by texture measuring devices. The form of the model is as follows:

$$Vp = a+bTx \qquad 4.52$$

Where:

Tx= texture measurement ,such as Mean Texture Depth in mm) measured by the device.

a, b= Assigned coefficients for each measuring equipment

IFI, as a well defined universal index, along with the two numbers ;F60 and Vp, which constitute the IFI index can be used in Pavement management system applications. These three figures can be used to report about pavement condition , accidents, airports operations, and maintenance management surveys.

Modeling skid resistance performance with time under the effect of environment and traffic is crucial for predicting the future condition for the pavement sections. This issue has been investigated by many researchers. In PMS , the following models was

proposed to be used as a planning tool to predict skid resistance and its effect of pavement condition(Anderson *et al*,1986)

$$SN_{64}=SN_0\, e^{-0.64PNG} \qquad 4.53$$

Where:

SN_{64}= Friction Number at velocity of 64 km/hr.

SN_0=Friction Number at zero speed (intercept of the curve)

PNG=Normalized gradient in % =-100($dSN_v/dv)/SN_v$

The above model is given just as a guide about factors that may be incorporated in skid resistance prediction.

The analysis of the collected data showed that there is a correlation between the Friction Coefficient (Mu) and the macrotexture properties measured by the Mean Texture Depth (MTD). The aggregate type which controls the microtexture properties was found to have negligible effect as all the asphalt surfaced pavement are made of one or two aggregate types of similar physical characteristics. Regression analysis for the data collected showed a sort of trend that takes the shape of exponential function between Mu and both the MTD and the Age. The form of the model was as follows:

$$Mu=0.73 +0.183\ \text{Lin (MTD)} \qquad R^2=0.622 \qquad 4.54$$

$$Mu= 0.371+0.557\ (MTD) -0.027\ (Age) \qquad R^2=0.522 \qquad 4.55$$

The application of the second model is shown in Figure below. The above developed models indicate that the macrotexture and Age have considerable effect of the friction level. The first model indicate that as the Age increases, the Mu value decreases but at different rates depending of the value of MTD. At high values of MTD,

the decrease in Mu values is less than that at low MTD values. This due to the fact that the potential for the formation of the hydroplaning film on the surface decreases as MTD increases. It works as a draining tracks for the water so preventing accumulation of the water on the surface. Also, friction factor decrease with age progression due to the traffic abrasion and polishing effect on the surface aggregate. Such models can be used to estimate the friction level at network level with an acceptable accuracy.

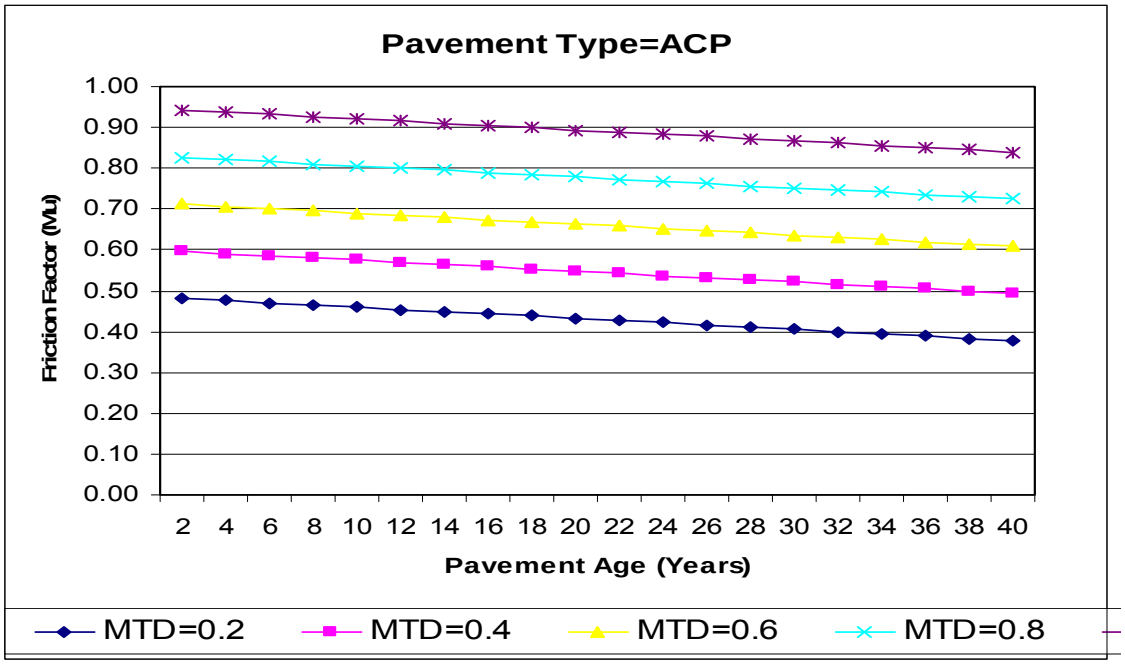


Figure Error! No text of specified style in document.**.18: Friction level deterioration with time under various levels of mean texture depth.**

CHAPTER 5 DEVELOPMENT OF THE OVERALL PAVEMENT QUALITY INDEX (OPQI) BASED ON VARIOUS CONDITION INDICATORS

General Concept

At network level, Pavement Management System (PMS) aims at optimizing the multi-year planning of the pavement maintenance and rehabilitation work programs by proposing different implementation scenarios and allocating the required funds to carry out various maintenance works. At project level, PMS is helpful in optimizing the maintenance and rehabilitation option from among the most feasible alternatives so that the proposed scheme is the most effective and economical alternative which has the ability to restore both the pavement riding quality and the structural integrity.

To fulfill these requirements, an objective quality index to report about pavement condition and sound performance models to evaluate the life cycle costing for various rehabilitation options are needed. Many researchers have indicated the keen need for a reliable serviceability index and accurate prediction models of the pavement condition deterioration in order to establish an effective pavement management system that manages the roads network in the proper manner (kameyama, et al, 1998).

It was envisaged that such an index should include factors related to all pavement condition indicators such as surface distresses, riding quality, skid resistance and deflection (structural capacity). Therefore, for successful implementation of the Maintenance and Rehabilitation (M&R) schemes, an acceptable level must be set for each of these four condition indicators which represent pavement serviceability main factors.

To report about pavement condition and predicting the future condition, the pavement performance prediction indices may range in sophistication from simple individual distress-based index, to the most comprehensive complex indices that are based on combined condition indicators . The optimum method to achieve this objective is to develop a comprehensive pavement performance index that takes into account the contribution of all condition indicators related indices in such a way a reliable pavement

condition evaluation can be achieved. However, few years ago, the issue of combining more than one index was viewed to yield some difficulty in the calculations of the network requirements and makes the M&R selection process and life cycle costing analysis more sophisticated. Therefore, most of the roads agencies adopted an index that is basically based on surface distress index and some times they include riding quality index. In the view of the recent developments in the field of data collection and analysis, roads agencies started to develop management systems that are based on more than one condition indicator to report about pavement condition.

In this study, the effort was firstly, dedicated to develop an individual pavement performance indices based on data pertains to each condition indicator I.e. Pavement Quality Distress-based Index ($PQDI_{-Distress}$), Pavement Quality Roughness-based Index ($PQRI_{-Roughness}$), Pavement Quality Structural Capacity-based Index ($PQSCI_{-Structure}$) and Pavement Quality Skid Resistance-based Index ($PQSKI_{-Skid}$). Then all these indices was combined together by adding or multiplying together the individual condition indicators indices to form the Overall Pavement Quality Index (OPQI) taking into account the relative importance of every single indicator. This combined index is expected to give well representation of the pavement condition and will be able to predict the performance accurately. Furthermore, it will enhance the process of optimizing the life cycle costs in the selected M&R option through utilizing the acceptance levels of the aforesaid four condition indicators indices.

Utilizing such an index in pavement management systems will create good systems that lead to more savings of the roads maintenance funds and enhance the ability of the roads network to provide better service at network level. Therefore, it is widely appreciated today that if the PMS is not able to program the maintenance and rehabilitation works over multi year planning period, the full benefits of the PMS cannot be fully attained(Fwa, et al 1988 and Feighan et al, 1987).

As stated in the methodology in chapter 2, the theoretical concept behind combining the indices of various condition indicators is called the Multiplicative Index Approach (MIAP). MIAP combines together all distress types (severity and extents) roughness, effective structural capacity and skid resistance accounting for the effect of each indicator. Individual indices are first calculated and at the end of the process , they are

summed up or multiplied to yield one single index. This theoretical concept takes the following form:

$$OPQI_k = 100\sum_{i=1}^{i=n}\left[1-\left(1-\frac{CI_i}{100}\right)*W_{i,k}\right] \ldots\ldots\ldots \qquad 5.1$$

Where:

OPQI$_k$: Overall Pavement Quality Index (selected to be on a scale from 0-100).

CI: Condition Indicator or distress Index (also selected to be on scale from 0-100). **CI** includes the followings:

Rutting Index, (RI)
Alligator Cracking Index (ACI)
Bleeding Index(BI)
Patching Index(PCHI)
Potholes Index(PTI)
Depression Index(DEPI)
Longitudinal and Transverse Cracking Index (LTI)
Weathering and raveling index(WRI)
Shoving Index(SHI)
Etc…

In addition to the above distress types, other condition indicators calculated on the same scale from 0-100 , were included in this study in order to determine the Overall Pavement Quality Index. These indicators include the followings:

Average International Roughness Index (AIRI)
Effective Structural Capacity Index (ESCI)
Skid Resistance Index(SKRI)

K= Kth Pavement Performance Index.

i= ith Distress or condition indicator out of the total number of the “n distresses or condition indicators

n= Total number of distress types or condition indicators included in the performance index.

Wt i,k=The impact or the relative weight of each distress type or condition indicator.

The above form of the performance index is created based on the deduct scale which means that a perfect pavement would have a score of 100. As distress accumulate, the deduct values for each distress increases. This in turn , lowers the value of the performance index of the pavement.

Based on the statistics from the field in the study area, the above distress types were only selected to be incorporated in the distress- based pavement quality index as these distresses were found to be the most frequent types observed to cause most of the pavement network deterioration. Therefore, it was decided to calculate the distress-based pavement performance index based only on these 9 types. For other areas, other or additional different distress types can be included after assigning the corresponding distress weight for each distress.

The above formula is considered as the general form that deals with condition indicators regardless of the dimension. Therefore, the conversion rules that convert all pavement condition indicators into a single and unified scale that ranges from 0-100 were developed in this chapter to describe the overall quality of a pavement section or a sample unit. The OPQI index proposed in this study is designed to include all the indices of pavement condition indicators after being converted into a unified scale that ranges from 0 to 100. This includes roughness index, deflection or structural capacity index and skid resistance index, in addition to basic distress types indices (i.e. rutting index, alligator cracking index…etc).

The proposed index is expected to be a good indicative of pavement condition and performance. This index along with the individual condition indicator indices will be used separately or combined to report about the condition of the pavement and will be used also in the structure of the decision trees for treatment type selection. Based on this index, the treatment options selected to rectify pavement section will be able to restore both structural integrity and other required functional and safety characteristics.

A scale from 0 to 100 is selected in order to convert one –dimensional or multi-dimensional distress types or condition indicators data into a unified scale for the purpose of the quality Index calculations and easy understanding and comparison. The method of analysis for the main condition indicator parameters as envisaged by the

researcher is based on a step by step development procedure described in details in the following sections.

Pavement Quality Index (PQI_{Dist}**Error! Bookmark not defined.**) based on distress

Distress data collection and analysis is considered as the basic element of all pavement management systems developed around the world. Many pavement distress evaluation methods were developed based on this element and are being used to report about pavement condition and maintenance budgeting. Examples of such management systems include PAVER rating system represented by PCI condition index, the Ministry of Transport of Ontario MTO in Canada, Asphalt Institute (AI) rating methods, Texas Research and Development Incorporated (TRDI) rating system etc. One of these methods will be adopted here in this study after calibration just to show *how* these individual indices can be used separately or combined to finally come up with a Overall Pavement Quality Index and the *extent or the effect* of incorporating such indices on optimizing the treatment selection procedure.

In this study, distress evaluation method developed by Texas Research and Development Incorporated (TRDI) will be used as it can be used as one dimensional and multi-dimension distress converters (TRDI, 2002). Similar to the other well known methods such as PAVER system and MTO method, TRDI method calculates the pavement condition index based on distress data (types, severity and density). This method is much more flexible in the way it absorbs the incorporation of one or multiple-dimensions condition indicators. In this study this method will be modified and calibrated to suit the study area characteristics.

<u>It is worth to indicate here that, the aim of this study is not to develop a new rating system for various condition indicators rather than to show how these indicators can be used individually or combined together to calculate a comprehensive performance index that can be used effectively to report about pavement condition and in life cycle costing analysis.</u>

As stated earlier in chapter 4, most of the pavement deterioration in the study area was found to be caused by 9 main distress types. These distresses were calibrated to suit

the local conditions by assigning a new deduct values for each distress severity and density. The Maximum Allowable Extent (MAE) developed by TRDI explained in chapter 2, is adopted in this study and used to calculate the distress index for each type.

The resultant Individual Pavement Quality Index based on distress data can be calculated as follows:

$$PQI_{Dist\ k} = 100 \sum_{i=1}^{i=n} \left[1 - \left(1 - \frac{D_i}{100} \right) * W_{i,k} \right] \ldots\ldots \qquad .5.2$$

Where:

PQI_{Dist}: Pavement Quality Index based on distress types only (selected to be on a scale from 0-100).

DI: Individual Distress Index (also selected to be on scale from 0-100). Here DI replaces the Condition Indicator (CI) term mentioned in the general performance form in equation 1.

DI terminology for each distress is as follows:

Rutting index, (RI)
Alligator Cracking Index (ACI)
Bleeding Index(BI)
Patching Index(PCHI)
Potholes Index (PTI)
Depression Index (DEPI)
Longitudinal and Transverse Cracking Index (LTI)
Weathering and raveling Index (WRI)
Shoving Index (SHI)

K= K^{th} Pavement Performance Index.

i= i^{th} Distress type out of the total number of the “n distresses.

n= Total number of distress types included in the performance index.

Wt i, k=the impact or the relative weight of each distress type.

Distress weights are normally range from 0 to 1. It mainly depends on the relative impact that each distress has on the pavement condition. The weights for distresses are not required to sum to a value of 1. This impact can not be calculated numerically but rather it is estimated based on field experience. In this study, the distress and other

condition indicators weights have been established based on both engineering judgment and filed cross checking and calibration examples undertaken by a panel of experienced pavement specialists and experts in the field of pavement evaluation. The proposed weights for each distress are estimated to be as shown in Table 5.1 below. This table can be used for all distress types that may observed in the flexible pavements. It also includes the main cause of each distress which is very helpful in evaluating the impact of each distress on pavement condition deterioration.

Table 1: Common distress types in flexible pavement and the corresponding estimated impact weights.

Distress Type	Severity levels	Possible Causes	Weight (W_t)
Alligator Cracking	Low, Medium, High	Load	1
Rutting	Low, Medium, High	Load	1
Potholes	Low, Medium, High	Load	0.1
Corrugation	Low, Medium, High	Load	0.5
Depression	Low, Medium, High	Load	0.4
Slippage Cracking	Low, Medium, High	Load	0.6
Shoving	Low, Medium, High	Load	0.1
Edge Cracking	Low, Medium, High	Load	0.1
Patching	Low, Medium, High	others	0.65
Bleeding	Low, Medium, High	Climate/Mix	0.4
Block Cracking	Low, Medium, High	Climate/Mix	0.1
Joint Reflection Cracking	Low, Medium, High	Climate/Mix	0.1
Swell	Low, Medium, High	Moisture/Drainage	0.5
Weathering and Raveling	Low, Medium, High	Climate/Mix	0.4
Bumps and sags	Low, Medium, High	others	0.4
Lane Shoulder Drop off	Low, Medium, High	others	0.1
Polished aggregate	One dimension	load	0.1
Longitudinal and Transverse cracking	Low, Medium, High	Climate/Mix	0.3
Rail road crossing	Low, Medium, High	others	0.4

In this study, the Pavement Quality Index based only on these main 9 distress types observed in the study area can be calculated as follows:

$$PQI_{Dist_k} = 100\left[1-\left(1-\frac{\mathrm{RI}}{100}\right)*1\right]*\left[1-\left(1-\frac{\mathrm{ACI}}{100}\right)*1\right]*\left[1-\left(1-\frac{\mathrm{BI}_i}{100}\right)*0.40\right]*$$

$$\left[1-\left(1-\frac{\mathrm{PCHI}}{100}\right)*0.65\right]*\left[1-\left(1-\frac{\mathrm{PTI}}{100}\right)*0.10\right]*\left[1-\left(1-\frac{\mathrm{DEPI}}{100}\right)*0.40\right]*\left[1-\left(1-\frac{\mathrm{LTI}}{100}\right)*0.30\right]*$$

$$\left[1-\left(1-\frac{\mathrm{WRI}}{100}\right)*0.40\right]*\left[1-\left(1-\frac{\mathrm{SHI}_i}{100}\right)*0.10\right] \qquad 5.3$$

Where:

RI=Rutting index

ACI =Alligator Cracking index

BI=Bleeding Index

PCHI=Patching Index

POTI=Potholes Index

DEPI=Depression Index

LTI=Longitudinal and Transverse Cracking Index

WRI=Weathering and raveling Index

SHI=Shoving Index

K= K^{th} Pavement Performance Index.

i= i^{th} Distress type out of the total number of the “n distresses.

n= Total number of distress types included in the performance index.

Wt i, k=The impact or the relative weight of each distress type.

Distress Index (DI) is calculated using the principle of the Maximum Allowable Extent (MAE) which converts the two dimensional distress into a distress index. The two dimensions of each distress are usually the Severity and the Density(Extent). The following steps summarize the procedure adopted to calculate the distress index:

Calculate each severity index (low severity index, medium severity index and high severity index). This severity index is equal to the individual severity score when there is only one severity of the distress surveyed. If more than one severity is recorded, then the severity score will be a *dampened additive combination* of the individual severity scores and the result will be limited to the highest single severity minimum score.

1- Calculate distress score at each severity level score (SS) using the following formula:

SS=Max (HRS, 100-(100-HRS / (HRE –LRE)*(Extent-LRV)…….5.4

Where:

HRS=High Range Score

HRE=High Range Extent

LRE=Low Range Extent

LRV=Low Range value

The definitions of the terms outlined in the above table are as follows:

- **High Range Value (HRV)**: It is the highest extent (density) of the distress after which the score will no longer decrease.
- **Low Range Value (LRV)**: It is the extent (density) at which the deduct points start to accumulate for each distress.
- **Minimum Score (MS)**: It is the lowest score that will be calculated for a given severity level when the distress extent (density) recorded on a specific pavement section is *greater than* or equal to the high range value.

The researcher has calibrated the module for calculating the distress index by assigning the values of the Low Range, High range and the Minimum Score for each distress type at different severities as shown in Table 2 below.

It is worth mentioning here that distress indices can also be calculated or estimated using one of the common methods mentioned in chapter 1 such as US PAVER system, or the Canadian MTO or the Asphalt institute (AI) methods. Unfortunately, these methods do not account for any other condition indicators such as roughness, deflection or skid resistance. It evaluates these indicators subjectively which is considered not accurate enough in pavement evaluation and life cycle cost analysis.

To facilitate the issue of understanding the method of calculation of the distress index, the following example is given. Assume during the field survey, it was found that alligator cracking has the following quantities:

Alligator cracking density (extent) at High severity =10%

Alligator cracking density (extent) at medium severity =0%

Alligator cracking density (extent) at Low severity=40%

Table 2: Low and High Range values and the corresponding Minimum Score selected for each distress type at different severities.

Distress	Severity	Low Range value	High Range value	Minimum Score
Alligator Cracking	Low	0	50	20
	Medium	0	25	15
	High	0	15	10
Bleeding	Low	10	100	70
	Medium	6	70	40
	High	0	70	15
Depressions	Low	1	40	50
	Medium	0	20	40
	High	0	10	25
Longitudinal and Transverse Cracking	Low	5	25	70
	Medium	2	20	50
	High	0	10	25
Patching / Utility Cut Patching	Low	5	50	65
	Medium	0	15	20
	High	0	10	10
Potholes	Low	0	1	50
	Medium	0	1	20
	High	0	0.5	0
Rutting	Low	0	30	60
	Medium	0	15	50
	High	0	10	25
Weathering / Raveling	Low	30	100	80
	Medium	5	50	50
	High	0	20	30
Shoving	Low	0	20	80
	Medium	0	15	35
	High	0	5	30

To calculate the distress index, you need first to calculate the severity score at each level as outlined in Equation 2 and Table 2, so:

SL1= High Severity Score=max (0,100 –100-0/50-0 (10%-0)) =max (0,100-2(10%)) =80

SL_2= Medium Severity Score= max (20,100 –100-20/80-0 (0%-0)) =max (20,100-0)) =100

SL_3= Low Severity Score= max (40,100 –100-40/80-20 (40%-20)) =max (40,100-1(20%)) =80

Then, calculate the deduct value at each severity level, $Dl=100-SL_l$ as follows:

D1=100-80=20

D2=100-100=0

D3=100-80=20

Now, to calculate the distress index, the following steps are performed:

1- Calculate the Low severity deduct value as , D_1=100-80=20
2- Calculate the weighted medium severity deduct value (composite deduct value) as:

   $D_{2w}=D_2$+ (MaxD2-D2)/MaxD2))*D1

   Where:

   Max Di: is the Maximum deduct value of D_i, so:

   D_{2w}=0+ {100-0)/100}*20=20
3- Calculate the weighted High severity deduct value (composite deduct value) as:

   $D_{3w}=D_3$+(MaxD3-D3)/MaxD3))*D2,=20+ ((100-20)/100)*20=20+16=36
4- Calculate the composite distress index=DI=100-Weighted High severity deduct value, therefore,

The Distress Index (DI) =100-36=64

Interested individuals may refer to the reference list for detailed description of the rating system and individual indices calculation.

Combined Pavement Quality Index ($OPQI_{Dist}$**Error! Bookmark not defined.**$_{+Roughness}$) based on Distress and Roughness

Roughness aspect can be manifested through the rideability index in terms of International Roughness Index (IRI). This figure can be treated as a combination of surface defects and surface irregularities. As stated earlier. The conversion of the IRI,

which is set according to the World Bank scaling system on a scale that ranges from 0-12, to a rating system on 0-100 scale, has been accomplished by a panel of raters who are aware of the IRI concept and purpose.

Similar to PSR method of evaluation. The raters pass over the entire length of hundreds of selected pavement sections for which the IRI values have been measured to give a rating on a scale from 0-100. The individual values of the roughness index then averaged to yield a value termed as **Average Roughness Index (ARI)**. In addition to that the PSR value has been also recorded for the purpose of the development of the model that describes the relationship between the IRI and the PSR as detailed in chapter 4. The relationship between the IRI and the Average roughness index was found to take the following form (Figure 5.1). The best regression model obtained to estimate ARI from IRI measurements takes the following form:

$$\mathbf{ARI=100-15.2\ (IRI)+0.6\ (IRI)^2} \qquad \mathbf{R^2 = 0.985} \qquad 5.5$$

$$\mathbf{ARI=100\ e^{-0.518*IRI}} \qquad \mathbf{R^2=0.70} \qquad 5.6$$

Where:

ARI= Average Roughness Index on scale from 0-100

IRI= International Roughness Index in mm/m or m/km

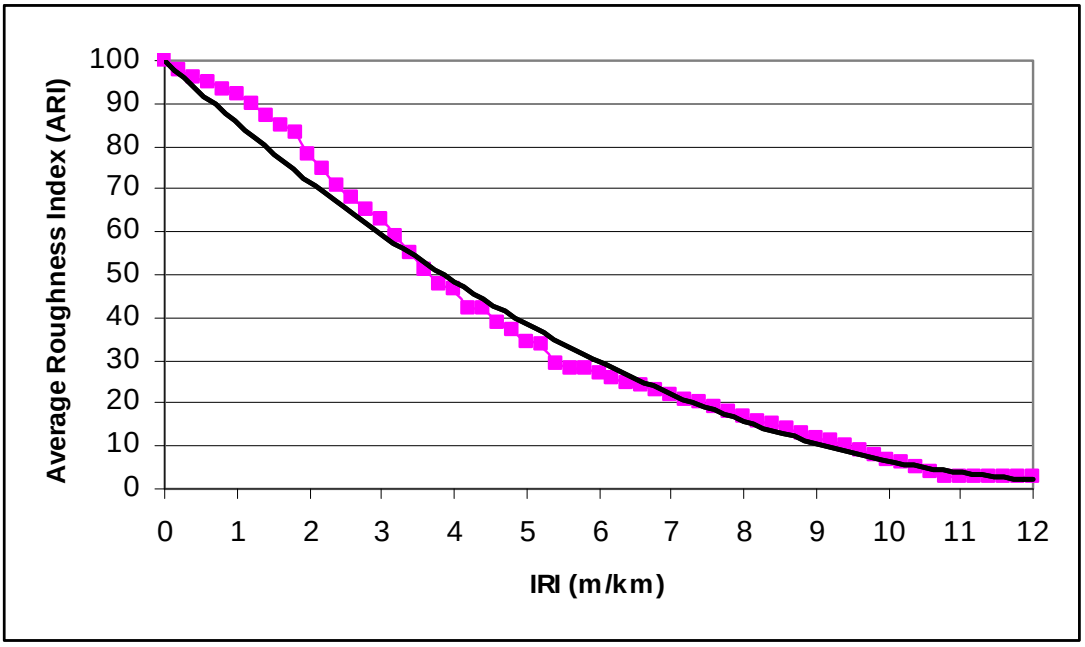


Figure 5.1: The relationship between the IRI and the ARI

Using the above equation, the data in this case will be converted into a suitable scale that fits the general formula of the performance index. The contribution of the Average Roughness Index to the overall pavement quality index can be expressed as follows:

$$CPQI_{Dist+Roughness_k} = 100\left[1-\left(1-\frac{\mathrm{RI}}{100}\right)*1\right]*\left[1-\left(1-\frac{\mathrm{ACI}}{100}\right)*1\right]*\left[1-\left(1-\frac{\mathrm{BI}_i}{100}\right)*0.40\right]*$$

$$\left[1-\left(1-\frac{\mathrm{PCHI}}{100}\right)*0.65\right]*\left[1-\left(1-\frac{\mathrm{PTI}}{100}\right)*0.10\right]*\left[1-\left(1-\frac{\mathrm{DEPI}}{100}\right)*0.40\right]*\left[1-\left(1-\frac{\mathrm{LTI}}{100}\right)*0.30\right]*$$

$$\left[1-\left(1-\frac{\mathrm{WRI}}{100}\right)*0.40\right]*\left[1-\left(1-\frac{\mathrm{SHI}_i}{100}\right)*0.10\right]*\left[1-\left(1-\frac{ARI_i}{100}\right)*Wt_{Roughness}\right]$$

5.7

The ARI can be used in the PMS condition reporting system individually or combined with other condition indicators that form the OPQI depending on the quantity and the type of data collected at network level.

If tools and labor are not available to carry out the filed inspection, IRI values can also be estimated using various models developed using various factors.

Based on the calculated IRI, the ARI can be also estimated easily based on one of the models developed for this purpose. In using prediction models for the purpose of predicting the IRI deterioration over time, it is deemed worthwhile to always consider and attempt to adopt the most simple and straight forward models. Statistically, as more variable are added to the model structure, the accuracy of prediction will increase proportionally.

At network level, high accuracy in prediction is not always required since all the analysis depends on the establishing trigger zones for different maintenance categories.

Combined Pavement Quality Index (OPQI $_{\text{Dist}}$**Error! Bookmark not defined.**+ARI+ESC) based on Distress, Roughness and Effective Structural Capacity

In order to manage the pavement properly by assigning the required budget and recourses of maintenance and rehabilitation work, quick and reliable testing tool and

analysis procedures must be used. In this regard, , structural evaluation for the in-service pavements at network level can be accomplished by two common methods:

- Destructive testing which includes taking asphalt cores, cut outs of the existing unbound pavement layers, material condition evaluation and measuring the thickness of every single pavement layer. This type of evaluation is very costly for both road users and highway agencies and traffic disturbance is expected to be high.
- Non-destructive Testing which includes carrying out deflection testing on the existing pavement using the available devices that might include Benkelman beam, Road Rater and recently, the developed Falling Weight Deflectometer and the Rolling Weight Deflectometer shown in Figure 5.2.

  The RWD is a new tool developed to evaluate the structural capacity of the existing pavement at the highway posted speed. It utilizes a spatially coincident methodology for measuring pavement response such as deflection, stress and strains (ARA, 2004). RWD uses three laser sensors to measure the unloaded pavement surface (which is composed of the forward and outside the deflection basin), and a fourth laser sensor, located between the dual tires and just behind the rear axle, measures the deflected pavement surface. In this case, the profile of the undeflected pavement surface is subtracted from the profile of the deflected pavement surface measured at the same locations. The deflection is therefore calculated by comparing spatially coincident scans as the RWD moves forward.

The extracted data can be directly or indirectly used in pavement condition assessment and pavement management at network level.

Figure 5.2: Rolling Weight Deflectometer developed by Applied Research Associates Inc, USA.

Under traffic and environmental loadings, flexible pavements deteriorate with time passing until it reaches a complete failure; firstly, when the stresses accumulated at the bottom of the asphalt layer are more than the allowable limit, and secondly, when the accumulated vertical strain caused by traffic loading reaches a value of more than 10-12 mm at the end of its service life.

The measured parameters that are normally obtained directly from such advanced machines include Deflection value at different levels, Load, Stress and Strains values. Other parameters that can be calculated based on these measurements include Modulus of Elasticity, Remaining Structural life in years, Critical Layer and the Overlay Thickness that can prolong the service life for another selected design period which may range from 10 to 15 years.

Deflection data measured by the FWD or RWD sensors can also be used to estimate the effective pavement modulus and the subgrade modulus. There are many forms to use the deflection data for the purpose of reporting about the structural capacity of the existing pavement.

The E value can be used effectively in PMS to compare pavement sections at network level as it is layer property governed by the effective physical condition of the pavement layers. It well known that E value is a function of layer thickness, compaction and gradation. Based on this, the E value will be inversely affected if the characteristics of the pavement layers changed as a result of cracking or any surface distortion. Therefore, The E value is envisaged to be the needed parameter in PMS more than any other parameter to predict the future deterioration of the pavement. In this regards, replacing the E value with the deflection parameter will not give accurate prediction about the future pavement performance and the remaining structural capacity.

Other reasons behind adopting the E value as the prediction parameter for the pavement performance is that E provides an early warning of the impending structural deterioration long before it shows up as a visual surface distress or as an increase in roughness(Ullidtz, *et al*, 1990).

The justifications behind this adoption is that excessive stresses and strains in flexible pavement layers resulted from the traffic and environmental impacts are usually lead to a formation of micro cracking in the layer material. The formation of such micro cracking will eventually reduce the cross sectional area of the layers that constitute the pavement structure. As a result of this reduction in cross sectional area, the impact of the stresses and strains are considerably increased over the remaining intact area.

The increased level of the stresses and strains will in turn reduce the value of the modulus of elasticity for the pavement layers. Therefore, the reduction of the E value has occurred as a result of the direct impact of the reduction in the cross sectional area resulted from the formation of the micro cracking.

As the pavement gets aged, these micro cracking will develop into macro or visible cracking manifested in different forms on the pavement surface. These forms may take a shape of alligator cracking or rutting as a result of excessive densifications or shear forces, or raveling due to the fatigue action and the loss of the binder due to the hardening and cracking. The reduction in pavement modulus of elasticity is directly affected by the formation of such distress.

Effective Structural Capacity index (ESCI) based on the calculated pavement modulus can be used easily to report about structural condition of the existing pavement. To be easily used in the PMS system calculations and life cycle costing analysis, this number should be converted into an index on a scale similar to the other condition indicators (such as distress , roughness. etc).In general, the value of ESCI can then be estimated using one of the following methods:

In evaluating the effectiveness of using such parameter in PMS, it is envisaged that there will be many difficulties in obtaining such an index in the field by assigning a value to the structural layer coefficient (a_i) based on visual pavement condition. Even more, it is almost impossible to estimate the effective structural number accurately using this coefficient to the extent that allows the user to adopt it in PMS reporting.

This conclusion has been reached by many researchers who investigated the possibility of using structural number at network level management for predicting the performance of the pavement. It was concluded that layer coefficient for the asphalt and other pavement layers can not be assigned on the basis of the moduli values from back calculation analysis. This coefficient can only be determined in laboratory using the ASTM Standard Test D4123-82; “Standard Test Method for Indirect Tension Test for Resilient Modulus of Bituminous Mixtures”. Also, it was found that the back calculated moduli differ from the laboratory moduli by an order of magnitude (Romanoschi , *et al*, 1999).

Alternatively, using the empirical models listed in the previous chapter as a simple way to estimate the structural number of the pavement, the structural number can be determined based on distress data and deflection data and used in PMS for structural condition reporting and analysis. The disadvantage of using such models is that these models were developed in different areas based on local data extracted from the field. So, inaccurate estimation could be obtained if any of the above models adopted in areas of different characteristics.

As indicated in chapter 4, $\boldsymbol{Ep_{eff}}$ can be estimated based on various parameters or measurements; among these is the past traffic in terms of ESAL. Based on data collected from hundreds of deflection basins, a simple equation for estimating the effective $\boldsymbol{Ep_{eff}}$ was developed.

As times passes, the damage caused by traffic loading estimated by the number of the ESAL repetitions leads to a progressive decrease in pavement modulus, I.e. E value decreases as the ESAL increases. Based on certain measurements obtained form the field, Pavement usually starts with high modulus which may range from 1700-5500 Mpa similar to the condition shown in Figure 5.5 below.

Figure 5.3: Pavement with sound E value for a newly constructed road.

These initial or the design modulus values were determined based on extensive laboratory testing and field investigations carried out in the study area through a pioneered asphalt research project designed to develop both pavement design and a validated, field tested mix design procedures.

Upon the initiation of the cracking, the modulus decreases drastically. By time passing and the propagation of the cracking in density and severity, similar to the condition shown in Figure 5.6, the E may reach a value around 500 Mpa approaching the aggregate base value. In this case, the asphalt layers will no more working as a fully bonded material. Alternatively, it will behave as unbound material similar to the aggregate road base and subbase layer.

Based on the above explanations, it is envisaged that the modulus value can be used to determine the Structural Remaining Life (SRL) estimated by the number of the ESAL repetitions or the number of years needed to pavement failure. SRL value can then be used in the decision making process to establish the point at which a remedial

intervention is needed. It can be also used to select the required treatment type needed to restore pavement capacity structurally.

Figure 5.4: Cracked pavement with low E value that ranges from 500 to 700 Mpa for a deteriorated road.

Changing rate in modulus value was estimated by other researchers based on data collected elsewhere in the following form ((Ullidtz, 1999)

$$E(t) = E_0 * (a - b * Age)^c \qquad 5.8$$

The above formula estimates the change in modulus as a result of aging. In real life, at the early ages of the pavement i.e. first and the second year after opening to traffic, the modulus value increases a little bit due to the densification effect then it will start to decrease continuously and progressively by time under the traffic and the climate effect. In cases where cracking takes place in the upper pavement layers, the intrusion of water and air will weaken the underneath unbound material, reducing the moduli of these materials considerably. This effect is calculated by multiplying the modulus of the unbound material by a constant less than 1, when the moduli of the bound material drop below certain value.

After the determination of the parameter Ep_{eff}, the value of ESCI needed in PMS applications can be calculated using the following expression:

$$ESCI=100-((1-(Ep_{eff}/Ep_o))*100) \quad 5.9$$

Where:

ESCI= Effective Structural Capacity Index (0-100)

Ep_{eff}= Effective pavement modulus.

Ep_o= Original (Design) pavement modulus

The term $(1-(Ep_{eff}/Ep_o)*100)$ represent the consumed portion of the pavement strength or capacity in terms of modulus value. Therefore, the value of $100-(1-(Ep_{eff}/Ep_o))*100)$ represent the remaining or the effective capacity of the pavement. This value decreases continuously with time passing under traffic loading until a major structural failure mode takes place in the pavement. Such failure may take the shape of permanent deformation (rutting) or fatigue cracking. As discussed previously in chapter 4, cracked asphalt may have a value that ranges to 500-700Mpa which is almost equal to the value on unbound material such as aggregate road base.

Despite the fact that most of the agencies prefer to use this data at project level, it is believed that this can be also done easily on network level with devices and tools available now or that to be introduced in the next few years with minimum disturbance to the traffic flow.

Measuring the effective pavement modulus should be practiced based on certain rules and restrictions, i.e. for each pavement section, the value of E should be normalized to standard load and temperature. It is always preferable that the deflection testing to be performed annually or periodically (i.e. say every 2 - 3 years.) in the same month or season.

As previously explained, the E value may represent the structural capacity much better than the peak deflection at network level. This is because, different pavement sections with the same deflection value may have different structural capacity in terms of the traffic loading that can be absorbed by these two sections.

Many programs has been developed to back calculate the effective pavement modulus among them ELMOD4 is the most appropriate. ELMOD 4 has the ability to undertake back calculation of pavement layers moduli using either Odmark-Boussinesque

transformed section approach or using the “Curve fit” procedure. This method of analysis is fast and considered appropriate for determining the characteristic moduli of each pavement section.

Using the E value in pavement management system is highly recommended due to the simplicity of the back calculations and the easy processing of the deflection data. Estimating the effective pavement modulus can be obtained directly from the deflection testing. Most of the back calculation programs give the surface modulus E_0 and the E value for each pavement layer as an output.

The effective structural capacity index can be used in PMS separately or combined with other condition indicators to report about pavement condition. This index will be incorporated in the treatment selection procedure as it is expected to be found of significant effect on the selected pavement treatment type. The details of this effect and method of using will be thoroughly discussed in chapter 8.

The contribution of the effective (remaining) structural capacity to the Overall Pavement Quality Index can be expressed as follows:

$$CPQI_{Dist+Roughness+ESCI} = 100\left[1-\left(1-\frac{\mathrm{RI}}{100}\right)*1\right]*\left[1-\left(1-\frac{\mathrm{ACI}}{100}\right)*1\right]+\left[1-\left(1-\frac{\mathrm{BI}_i}{100}\right)*0.40\right]*$$
$$\left[1-\left(1-\frac{\mathrm{PCHI}}{100}\right)*0.65\right]*\left[1-\left(1-\frac{\mathrm{PTI}}{100}\right)*0.10\right]*\left[1-\left(1-\frac{\mathrm{DEPI}}{100}\right)*0.40\right]*\left[1-\left(1-\frac{\mathrm{LTI}}{100}\right)*0.30\right]*$$
$$\left[1-\left(1-\frac{\mathrm{WRI}}{100}\right)*0.40\right]*\left[1-\left(1-\frac{\mathrm{SHI}_i}{100}\right)*0.10\right]*\left[1-\left(1-\frac{ESCI}{100}\right)*Wt_{Structure}\right]$$

5.10

Skid resistance indicator

Surface characteristics based on the micro- texture and macro- texture properties can be manifested through this parameter. Skid resistance number can be measured physically with many available tools. This number-SN can be incorporated into the

performance index to reflect surface skidding impact. Alternatively, information extracted from certain distress types (such as polished aggregate or weathering and raveling) and the corresponding severity level can be utilized to subjectively estimate the safety level prevails on each pavement section. Skid resistance index as a condition indicator is very crucial in the areas exposed to frequent rain and wet weather conditions.

In the study area, where the rain is very scarce, this portion may be dropped due to the relative infrequency of rain and wet condition. But, since the development of such index can be used in all pavement management system around the world, and not limited to the study area, skid resistance portion may play a crucial role in determining the type and the extent of the pavement treatment along with the remaining condition indicators. In this regard, the contribution of the skid resistance to the overall pavement quality index will be as follows:

$$CPQI_{Dist+Roughness+ESCI+SKRN} = 100\left[1-\left(1-\frac{\mathrm{RI}}{100}\right)*1\right]*\left[1-\left(1-\frac{\mathrm{ACI}}{100}\right)*1\right]*\left[1-\left(1-\frac{\mathrm{BI}_i}{100}\right)*0.40\right]*$$
$$\left[1-\left(1-\frac{\mathrm{PCHI}}{100}\right)*0.65\right]*\left[1-\left(1-\frac{\mathrm{PTI}}{100}\right)*0.10\right]*\left[1-\left(1-\frac{\mathrm{DEPI}}{100}\right)*0.40\right]*\left[1-\left(1-\frac{\mathrm{LTI}}{100}\right)*0.30\right]*$$
$$\left[1-\left(1-\frac{\mathrm{WRI}}{100}\right)*0.40\right]*\left[1-\left(1-\frac{\mathrm{SHI}_i}{100}\right)*0.10\right]*\left[1-\left(1-\frac{SKRN}{100}\right)*WT_{Skid-\mathrm{Re}sistance}\right]$$

5.11

Skid resistance can be measured directly from the field on a scale from 0 to 100. Skid resistance can be measured easily using many available tools in the market such as pendulum skid tester, grip tester, friction tester .etc. All these tools give skid resistance number directly and on the same scale.

As mentioned in details in chapter 4, Skid resistance varies over time in certain manner. It increases in the first 2 years after construction as a result of the wear a way of the bitumen under traffic effect. After that, it starts to decrease over time as a result of traffic action, mixture design and aggregate properties and the environment.

.

The skid resistance index can be used in PMS separately or combined with other condition indicators if needed to report about pavement condition. This index can be incorporated in the treatment selection procedure if found of significant effect on the pavement condition. The details of this effect and method of using will be thoroughly discussed in chapter 8.

Development of Overall Pavement Quality Index based on different condition indicators parameters.

The above condition indicators are formulated so that it can be used to calculate the individual performance indices separately. For more accurate estimation of pavement condition , All the above developed indices can be combined together to form a Overall Pavement Performance Index (OPQI) which describes the pavement structural and functional capacities of the road section taking into consideration all data collected for the surface condition. Therefore, it is envisaged that this index combines all distress types (severity and extents) roughness, effective structural capacity and skid resistance value according to the relative importance of each condition indicator. The overall theoretical concept takes the following form:

$$OPQI_k = 100\sum_{i=1}^{i=n}\left[1-\left(1-\frac{CI_i}{100}\right)*W_{i,k}\right] \quad \text{.....5.12}$$

Substituting each pat developed for each condition indicator will yield the general equation that encompasses the following terms:

$$OPQI = 100\left[1-\left(1-\frac{\text{RI}}{100}\right)*1\right]*\left[1-\left(1-\frac{\text{ACI}}{100}\right)*1\right]*\left[1-\left(1-\frac{\text{BI}_i}{100}\right)*0.40\right]*$$

$$\left[1-\left(1-\frac{\text{PCHI}}{100}\right)*0.65\right]*\left[1-\left(1-\frac{\text{PTI}}{100}\right)*0.10\right]*\left[1-\left(1-\frac{\text{DEPI}}{100}\right)*0.40\right]*\left[1-\left(1-\frac{\text{LTI}}{100}\right)*0.30\right]*$$

$$\left[1-\left(1-\frac{\text{WRI}}{100}\right)*0.40\right]*\left[1-\left(1-\frac{\text{SHI}_i}{100}\right)*0.10\right]*\left[1-\left(1-\frac{ARI_i}{100}\right)*WT_{Roughness}\right]*$$

$$\left[1-\left(1-\frac{ESCI}{100}\right)*WT_{Structure}\right]*\left[1-\left(1-\frac{SKRI}{100}\right)*WT_{Skid-\text{Resistance}}\right] \quad 5.13$$

Where:

OPQI: Overall Pavement Quality Index (selected to be on a scale from 0-100).

CI: Condition Indicator Index (also selected to be on scale from 0-100). **CI** includes the followings:

Rutting Index, (RI)
Alligator Cracking Index (ACI)
Bleeding Index (BI)
Patching Index (PCHI)
Potholes Index (PTI)
Depression Index (DEPI)
Longitudinal and Transverse Cracking Index (LTI)
Weathering and raveling index (WRI)
Shoving Index (SHI)
Average International Roughness Index (AIRI)
Effective Structural Capacity Index (ESCI)
Skid Resistance Index (SKRI)

K= K^{th} Pavement Performance Index.

i= i^{th} Distress or condition indicator out of the total number of the "n distresses or condition indicators

n= Total number of distress types or condition indicators included in the performance index.

Wt i, k=The impact or the relative weight of each distress type or condition indicator.

How to use the Overall Pavement Quality Index

It is envisaged that incorporation of the Average Roughness Index, Effective Structural Capacity Index and Skid Resistance Index condition indicator in the Overall Pavement Quality Index will necessitate a light modifications of the impact weights of many distress types as the effect of certain distresses may be doubled if used along with roughness, deflection and skid resistance parameters.

Various reporting mechanisms using various condition indicators indices individually or combined can be design to be used in pavement management systems. Using the

suitable performance index is subjected to the availability and the accuracy of the data for that condition indicator needed to be incorporated. However, all new PMS can absorb the possibility of using more than one performance indicator in reporting about pavement condition and in the calculation of the network master file. Also, decision trees can be designed easily to take each condition indicator into account in the treatment selection process. The recommended combinations can be described as follows:

$OPQI_4$= {($PQI\text{-}_{Distress}$) (Wt_{14})} + {($PQI\text{-}_{Roughness}$)*(Wt_{24})} + {($PQI\text{-}_{Structure}$) (Wt_{34})} + {($PQI\text{-}_{Skid}$)*(Wt_{44})}

$OPQI_3$= {($PQI\text{-}_{Distress}$) (Wt_{13})} + {($PQI\text{-}_{Roughness}$)*(Wt_{23})} + {($PQI\text{-}_{Structure}$) (Wt_{33})}

$OPQI_2$= {($PQI\text{-}_{Distress}$) (Wt_{12})} + {($PQI\text{-}_{Roughness}$)*(Wt_{22})}

$OPQI_1$= {($PQI\text{-}_{Distress}$) (Wt_{11})}

In this regards, the following rules may apply to the OPQI calculations:

1. OPQI will be equal to **$OPQI_1$** which is mainly a function of distress types i.e. $PQI_{Distress}$ If the data pertains to the remaining condition indicators are not measured or not available. The value of **Wt_{11}** in this case will be equal to 1.0. In this case, OPQI will be in the following form:

$$CPQI = 100\left[1-\left(1-\frac{RI}{100}\right)*1\right]*\left[1-\left(1-\frac{ACI}{100}\right)*1\right]*\left[1-\left(1-\frac{BI_i}{100}\right)*0.40\right]*$$
$$\left[1-\left(1-\frac{PCHI}{100}\right)*0.65\right]*\left[1-\left(1-\frac{PTI}{100}\right)*0.10\right]*\left[1-\left(1-\frac{DEPI}{100}\right)*0.40\right]*$$
$$\left[1-\left(1-\frac{LTI}{100}\right)*0.30\right]*\left[1-\left(1-\frac{WRI}{100}\right)*0.40\right]*\left[1-\left(1-\frac{SHI_i}{100}\right)*0.10\right]* \quad [Wt_{11}=1.0]$$

2. OPQI will be equal to **$OPQI_2$** which is mainly a function of distress types i.e. $PQI_{Distress}$ and $PQI\text{-}_{Roughness}$ If the data pertains to both structural capacity and skid resistance are not measured or not available. The value of **Wt_{11}** and **Wt_{22}** in this case will be equal to 1.0 and 0.50 respectively. This is because roughness is

affected by the following distress types which creates short waves of noticeable amplitudes which in turn is considered by the IRI model:

- L&T cracking
- Depression
- Pumps and sags
- Corrugation
- Potholes

So, the weight of roughness will be damped a little bit to avoid doubling the effect of such distresses.

$$CPQI = 100\left[1-\left(1-\frac{\mathrm{RI}}{100}\right)*1\right]*\left[1-\left(1-\frac{\mathrm{ACI}}{100}\right)*1\right]*\left[1-\left(1-\frac{\mathrm{BI}_i}{100}\right)*0.40\right]*$$

$$\left[1-\left(1-\frac{\mathrm{PCHI}}{100}\right)*0.65\right]*\left[1-\left(1-\frac{\mathrm{PTI}}{100}\right)*0.10\right]*\left[1-\left(1-\frac{\mathrm{DEPI}}{100}\right)*0.40\right]*$$

$$\left[1-\left(1-\frac{\mathrm{LTI}}{100}\right)*0.30\right]*\left[1-\left(1-\frac{\mathrm{WRI}}{100}\right)*0.40\right]*\left[1-\left(1-\frac{\mathrm{SHI}_i}{100}\right)*0.10\right]*$$

$$\left[1-\left(1-\frac{ARI_i}{100}\right)*0.50\right]$$

3. OPQI will be equal to **$OPQI_3$** which is mainly a function of distress types i.e. $PQI_{Distress}$ and $PQI\text{-}_{Roughness}$ and $PQI\text{-}_{Structure}$ If the data pertains to skid resistance is not measured or not available. The value of **Wt_{11}** ,**Wt_{22}** and **Wt_{33}** this case will be equal to 1.0 and 0.5 and 0.75 respectively. This is because structural capacity is affected by the following distress types:

- Alligator Cracking
- Rutting

So, the weight of structural capacity will be damped a little bit to avoid doubling the effect of such distresses. the weight given for the structural term is more than that assigned for both roughness and skid resistance as the structural characteristics have a detrimental effect on the overall pavement condition. In this case, OPQI will be in the following form:

$$CPQI = 100\left[1-\left(1-\frac{\mathrm{RI}}{100}\right)*1\right]*\left[1-\left(1-\frac{\mathrm{ACI}}{100}\right)*1\right]*\left[1-\left(1-\frac{\mathrm{BI}_i}{100}\right)*0.40\right]*$$

$$\left[1-\left(1-\frac{\mathrm{PCHI}}{100}\right)*0.65\right]*\left[1-\left(1-\frac{\mathrm{PTI}}{100}\right)*0.10\right]*\left[1-\left(1-\frac{\mathrm{DEPI}}{100}\right)*0.40\right]*$$

$$\left[1-\left(1-\frac{\mathrm{LTI}}{100}\right)*0.30\right]*\left[1-\left(1-\frac{\mathrm{WRI}}{100}\right)*0.40\right]*\left[1-\left(1-\frac{\mathrm{SHI}_i}{100}\right)*0.10\right]*$$

$$\left[1-\left(1-\frac{ARI_i}{100}\right)*0.50\right]*\left[1-\left(1-\frac{ESCI}{100}\right)*0.75\right]$$

4. OPQI will equal to **$OPQI_4$** which is mainly a function of distress types i.e. $PQI_{Distress}$ and $PQI\text{-}_{Roughness}$ and $PQI\text{-}_{Structure}$ and $PQI\text{-}_{Skid}$ If the data pertains to all condition indicators are measured or available. The value of the value of **Wt_{11}** ,**Wt_{22}**, **Wt_{33}** and **Wt_{44}** in this case will be equal to 1.0 and 0.50 and 0.75 and 0.25 respectively. This is because skid resistance is affected by the following non structural distress types:

   - Bleeding
   - Weathering and Raveling

So, the weight of skid resistance **Wt_{44}** will be damped relatively more than the value assigned for **Wt_{22}**, **Wt_{33}** due to the aforementioned reasons and to avoid doubling the effect of such distresses.

$$CPQI = 100\left[1-\left(1-\frac{\mathrm{RI}}{100}\right)*1\right]*\left[1-\left(1-\frac{\mathrm{ACI}}{100}\right)*1\right]*\left[1-\left(1-\frac{\mathrm{BI}_i}{100}\right)*0.40\right]*$$

$$\left[1-\left(1-\frac{\mathrm{PCHI}}{100}\right)*0.65\right]*\left[1-\left(1-\frac{\mathrm{PTI}}{100}\right)*0.10\right]*\left[1-\left(1-\frac{\mathrm{DEPI}}{100}\right)*0.40\right]*$$

$$\left[1-\left(1-\frac{\mathrm{LTI}}{100}\right)*0.30\right]*\left[1-\left(1-\frac{\mathrm{WRI}}{100}\right)*0.40\right]*\left[1-\left(1-\frac{\mathrm{SHI}_i}{100}\right)*0.10\right]*$$

$$\left[1-\left(1-\frac{ARI_i}{100}\right)*0.50\right]*\left[1-\left(1-\frac{ESCI}{100}\right)*0.75\right]*\left[1-\left(1-\frac{SKRI}{100}\right)*0.25\right]$$

The reduction in the weights of each condition indicator made upon a stepwise introduction of each one of the condition indicators into the Overall Pavement Quality

Index is a subjective estimation made by the researcher based on filed experience and the relative effects of each indicator on the performance of the pavement. Low value for the weight of the skid resistance in the general equation indicates the low importance of such indicator on the pavement performance particularly in the study area which is characterized as being an arid, desert area with scarce rainy days. These weights can be changed if these models are to be applied in other area of high precipitation amount.

Different prospects are seen by different roads users and highways agencies regarding pavement evaluation process and the relative impact of various pavement condition indicators. In this regards, roughness has been rated as the most important aspect of pavement condition which has a noticeable effect on pavement deterioration models as well as in establishing trigger levels for maintenance and rehabilitation works.

From road users' point of view, pavement that provides good riding quality and adequate skid resistance for safety reasons is a good pavement. From road agencies point of view, a good pavement is that pavement which should require less maintenance and provide adequate structural support to traffic and environmental loading (Hass, et al, 1994).

Pavement maintenance decisions in most of the cases are made based on personal judgment and experience of the individuals. Different individuals use different principals and different set of information to rely on. For more effective pavement management and proper maintenance programming, effective decision making tools should be adopted. These tools should not help only in determining the pavement sections which are in need for rehabilitation or maintenance, but also, in providing information about, how, and when to perform these maintenance programs under the condition of a limited maintenance budget.

CHAPTER 6 ESTABLISHING STRUCTURAL AND FUNCTIONAL REMAINING LIVES

Theoretical concept of pavement life

Pavement service life can be divided into two main concepts; first, is the ***structural pavement life*** which depends on the pavement structural performance and mainly related to the occurrence of cracking, rutting and ravelling and other conditions that would adversely affect the load carrying capacity of the pavement structure and necessitate carrying out maintenance works(AASHTO 1993). The second concept is the ***functional service life*** which depends mainly on the pavement functional performance that is concerns with the rideability and safety levels (i.e. how well and how safe the road is serving its users). In this regards, both structural and functional performances are evaluated under routine maintenance work. Ignoring the routine maintenance work for pavements will lead to a considerable reduction in pavement life and consequently massive increase in maintenance costs. Usually, the structural deterioration leads to a reduction in the functional life of the pavement at the same time. The occurrence of high severity structural distresses is considered as the major evidence or symptom of the end of pavement service life.

Despite the huge advances in the field of pavement evaluation , the analysis of the resulting data is still elusive, especially in predicting how long before a major rehabilitation would be needed which is designated as remaining life (Vepa et al 1996). This fact was among the main reasons that forced the writer of this thesis to conduct this research effort in order to partially fill the knowledge gap persists in this respect

The determination of remaining structural life level at which rehabilitation intervention is needed is very detrimental in the planning of maintenance and rehabilitation works that should be designed for restoring both the structural capacity and the riding quality of the in-service pavement. As explained in Chapter 1, Figure 1.1 , delaying the application of routine or

preventive maintenance work far beyond the optimum timing of the treatment application will lead to more deterioration and more cost. In this case, the pavement will be in a condition where the major rehabilitation work is not economically nor technically valid.

Establishing remaining service life criteria is very crucial in pavement management system as it is used ***for defining when (time in years) and how (mode of failure) each pavement section will fail*** . This task can be accomplished by using sound prediction models developed for each road class and condition indicator along with the pavement failure thresholds values set for each condition indicator using individual or combined performance indices developed by each road agency.

Based on these elements, the individual performance index that meets the failure criteria first will be considered as the decisive factor that controls the failure mode of pavement section. In this case , the pavement will be rated as failed based on one or a single condition indicator. If combined index such as the OPQI developed in this research effort is used , then this index will represent the overall condition of the pavement section and estimates the number of years needed for a pavement section to reach a complete failure. This value is termed as the **R*esidual or Remaining Service Life***.

For calculation of the remaining service life for each condition indicator or for the combined overall condition index, there are certain elements that should be available;

- Accurate field measurements for the pavement performance variables such as age, traffic level, distress quantities and severities, roughness, structural parameters etc.
- A well formulated field-tested performance index that combines all pavement condition indicators with the relative weights for each indicator.
- The *condition failure thresholds* or *serviceability terminal limits* established for each condition indicator at which the pavement is no

more serviceable. whether by considering the overall condition performance or by considering individual condition indicators.

- And finally, sound prediction models that predict the condition of the pavement accurately over the selected analysis period based on the selected performance variables such as age, traffic , roughness etc.

Prediction of the pavement remaining life can be fulfilled by using sound prediction models, preferably deterministic type, developed based on the prevailing local conditions for each road class, under certain traffic and environmental loading effects, and for each region (Local Modelling) or what is called family performance curves so that high prediction accuracy can be attained. The importance of developing such local models can be referred to the followings:

1. Local prediction models provide a reliable tool for estimating the existing pavement condition and predicting the future condition during the entire life of the pavement. It can lead to money saving by reducing the time and the need for repetitive field surveying.
2. It can be used for predicting the remaining service life and helping in determining the critical timing for maintenance intervention by predicting the residual life or specifying the end of service year (Terminal condition limit).
3. It can be used for life cycle costing analysis and accurately establishing different maintenance and rehabilitation scenarios under limited budget by calculating both cost and benefits resulted from each scenario.

Adopting accurate pavement prediction models enhances the usefulness of the pavement management system considerably at both network and project levels. At network level, prediction models can help in the selection of the optimum maintenance and rehabilitation strategies, setting priorities for the rehabilitation projects and estimating the budget. It also assists in scheduling the field inspection programs so that the user benefits are maximized. At project level, the prediction models can be used to select the optimum and

most advantageous maintenance and rehabilitation alternative(Dixon et al 1983). Modelling is considered as the most vital activity in PMS as it represents the change in pavement condition under the accumulated effect of both traffic and environment. It is absolutely essential for pavement management systems at both project and national networks(Xin, et al 1995). In pavement performance modelling, the most preferred method for building deterministic models is the regression analysis based on pavement historical data.

As stated in the previous chapters, most of the PMS's available around the world use the current physical condition as a trigger line to prepare work programs and estimate maintenance budget. Depending on one condition indicator will not yield the optimum repair option. This task can only be achieved through development a sound prediction models that represent the actual deterioration of the pavement.

In general, performance of flexible pavement is mainly associated to their resistance to fatigue cracking, permanent deformation (rutting) and durability (Anderson et al 1995) . Performance Prediction models along with the lab testing can give good estimation of the pavement performance life. For the ease of taking the decision about the seriousness of the pavement condition, the treatment extent is to be applied and the application priority.

Pavement deterioration can be grouped into two types (AASHTO1993):

1. ***Structural deterioration***: It is defined as any condition that reduces the load carrying capacity of the existing pavement. It is manifested through the formation of various types of distresses such as rutting and alligator cracking or depression in the underneath pavement layers of the pavement. Visual condition assessment and deflection measurements and analysis of the fatigue damage resulted from traffic loading can provide good evaluation methods of the effective structural capacity of the pavement in service for the purpose of condition investigation and the recommendation of the proper maintenance and rehabilitation strategies.

2. ***Functional deterioration***: it is defined as any condition that adversely affect the road user and the riding quality . It is manifested through two condition indicators which are; ***pavement surface roughness*** *and* ***surface friction and hydroplaning***. Surface roughness is associated with various waves of different lengths, potholes, cracking and ravelling. On the other hand, surface friction is associated with the poor, wet weather friction due to polishing of the surface as a result of inadequate resistance offered by the macrotexture and microtexture properties of the pavement. Poor skid resistance is also associated with the existence of certain distress types such as bleeding.

It is always believed that structural failure occurs before functional failure and normally the structural deterioration eventually lead to functional deterioration. In some exceptional cases, functional failure may takes place while the pavement is structurally sound. This case may due to high roughness and built -in irregularities during construction. As shown in Figure 6.1 below, the capacity of the pavement to carry more traffic is considerably reduced as the pavement gets aged. The procedures for estimating the remaining life using both traffic loadings and age are detailed in the sections below.

Pavement Remaining Life in Pavement Manaagement system

The evaluation of pavement remaining life is necessary to make optimal use of the structural capacity of the in service pavement (Vepa et al 1996). Estimation pavement remaining life is very crucial step in all PMS's and for all roads agencies as such information is badly needed for preparing the master work plan, setting priorities, budgeting and carrying out various work scenarios. Therefore, various methods were proposed for estimation or calculating this parameter as precise as possible. The proposed methods vary in complication from simple parameter such as pavement age and

distress data to more sophisticated approaches that utilize computer software to back calculated the effective pavement capacity based on material characteristics such as modulus of elasticity, curvature, deflection and thickness .

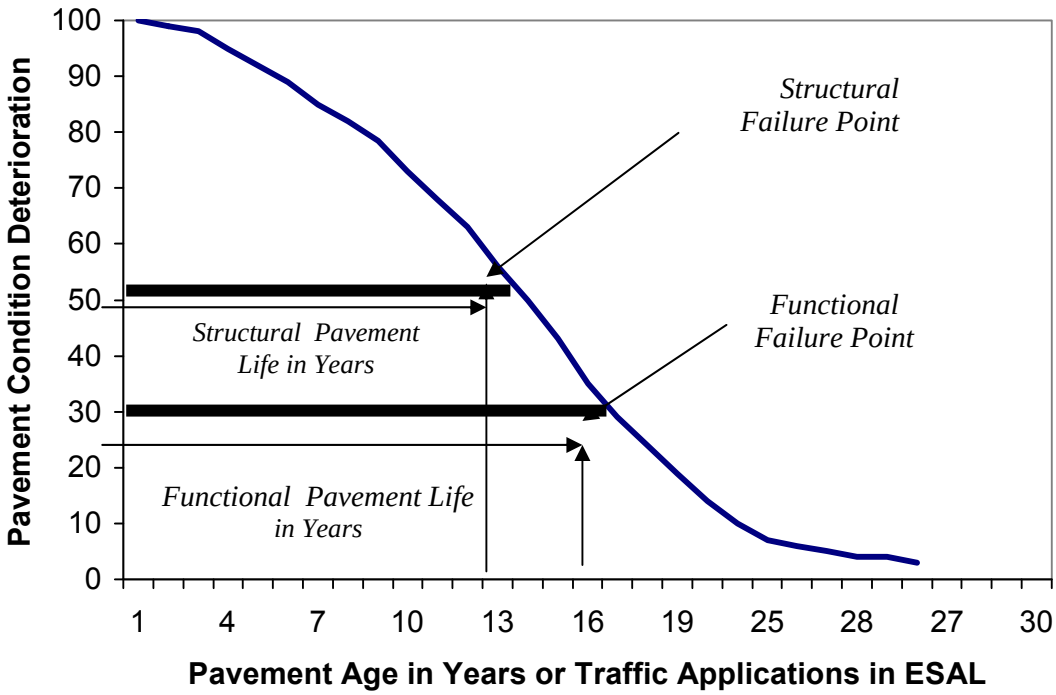


Figure 0.1: Functional and structural deterioration of flexible pavements

In summary, for estimating the pavement remaining life using functional failure approaches, the decrease in the performance index with time (age) and traffic loading repetitions(ESAL) can be used in conjunctions with a pre-selected functional failure criteria.

On the other hand, for estimating the pavement remaining life using structural failure approaches, the failure modes criterion (fatigue and rutting) along non destructive testing back analysis can be used in conjunction with an established failure criteria and pavement characteristics. These approaches will be thoroughly discussed in the following sections.

Estimating Pavemenet Remaining Life Based Visual Condition.

As indicated in chapter 4, the selection of the model to represent the general deterioration trend and predicting the remaining life for any pavement system should satisfy the following boundary conditions:

- The Condition Index at zero age should be equal to the maximum limit of the rating scale i.e. 10 or 100 , whatever adopted.
- The minimum value for the condition index should not be negative at any value of the pavement age.
- It should have high value of Coefficient of simple or Multiple Determination (R^2) which means good predictability.
- The number of the Independent variables should be kept minimum and it should be easily obtained from the field.

Using physical condition evaluation results (Distress data and OPQI index) as a trigger to identify the type and the level of maintenance and rehabilitation action to be taken was and still is the only widely available method for all PMS around the world. This procedure has some drawbacks in the treatment selection process as it may not yield the optimum rehabilitation strategy for pavement sections on medium to long term maintenance scenarios (3 to 5 years plans) if the correct deterioration trend is not well selected.

Therefore, it is obvious that, to predict the remaining life in years for any pavement system based on the calculated physical condition index, it is very important first to develop the representative deterioration curves that simulate the future deterioration so that the predicted value will be accurate enough. This will be reflected through the treatment selection decision process or in the life cycle costing analysis. *It is freely admitted that there is low probability of selecting the optimum rehabilitation option using only condition index method, if the deterioration trend is not representing the actual case.*

Predicting the remaining life of the pavement for roads subjected to different levels of traffic and environmental loading depends to a large extent on the deterioration trend. As indicated in the previous chapters, the exponential function was found to have the capability to represent the deterioration trend with a reasonable accuracy as represented by the value of $R^{2.}$

The remaining life from time point of view can be defined as "*The additional number of years that the pavement can serve before reaching the failure point expressed as terminal PQI value.* For the purpose of generating the maintenance and rehabilitation work programs on short, medium or long range, trigger limits based on PQI values should be established to be used by the PMS system in life cycle costing analysis and in budgeting scenarios.

In this study, the following values are established as the terminal limits recommended for preparing or taking an action. It is suggested in this regard that two limit are to be considered; these are:

Interference limit:
It represents the condition state at which the pavement section provides unacceptable level of service (functionally or structurally). In this case, it should be nominated for further detailed study at project level.

Failure limit:
It represents the condition state at which the pavement section provides no service to the public and an action for major rehabilitation or reconstruction should be taken immediately.

The terminal value for PQI as well as other distress types were proposed based on the extensive experience of the researcher and depending on an international references in the field of pavement rehabilitation. These values are established for each road class as the importance and the deterioration trend for each class are different. Table 6.1 below lists the values proposed for different roads classes.

Table 0.1: Interference and failure limits proposed for PQI and various types of distress.

Road Class / Condition Index	Major Roads (Freeways, Expressways, and Arterials)		Minor Roads (Collectors and Local Roads)	
	Interference (warning) Level	Failure Level	Interference (warning) Level	Failure Level
PQI	55	40	55	30
Alligator Cracking	55	40	55	30
Rutting	55	40	55	30
Potholes	55	40	55	30
Corrugation	55	40	55	30
Depression	55	40	55	30
Slippage Cracking	55	40	55	30
Edge Cracking	55	40	55	30
Patching	55	40	55	30
Bleeding	55	40	55	30
Block Cracking	55	40	55	30
Joint Reflection Cracking	55	40	55	30
Swell	55	40	55	30
Weathering and Raveling	55	40	55	30
Bumps and sags	55	40	55	30
Lane Shoulder Drop off	55	40	55	30
Polished aggregate	55	40	55	30
Longitudinal and Transverse Cracking	55	40	55	30
Rail road crossing	55	40	55	30

These interference trigger values can give a warning about the expected pavement condition so that preventive maintenance or any other remedial measures can be planned. Condition trigger limits can also help in

determining the expected number of years for the road to reach the failure point and predict the optimum timing for maintenance interference.

The above two limits are illustrated in Figure 6.2 below. Most of the highway agencies establish these levels to help in predicting the remaining life to failure based on the pavement deterioration prediction models developed for various road classes.

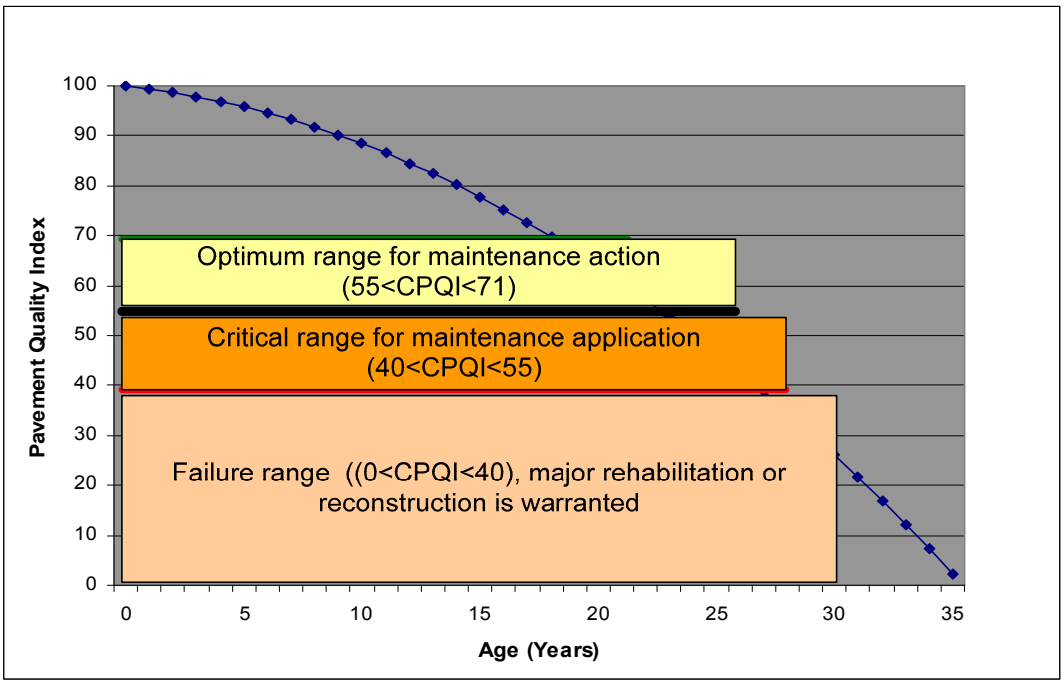


Figure 0.2: Various condition zones established for use in life cycle costing and planning process of maintenance work.

Models developed for predicting the OPQI curve can be used to predict the remaining life and various distress indices. Local modeling is the preferred method for predicting such an element as it depends on the data collected from the field in each region. In estimating the remaining life based on condition index calculated basically based on distress information, it is usually advised to take the failure limits set in Table 6.1 into consideration.

The proper method for estimation the remaining life at any time x of evaluation using this method is to calculate both Long and Short term Rate

of Deterioration as established in the PAVER system described in chapter 1 of this study.

Short Term Rate of Deterioration (STRD), which is the wanted parameter in the remaining life calculations, is estimated from the general trend models as being the reduction in the distress based condition index such as PQI each year. i.e. **,** STRD is calculated as follows:

$$\mathbf{STRD=PQI_i-PQI_x/Age} \qquad 6.1$$

Where:

STRD= Short Term Rate of Deterioration i.e. reduction in PQI points per year

PQI_i.= Initial Pavement Quality Index directly after construction or the application of an overlay (i.e. =100)

$PQI_{x=}$ Pavement Quality Index calculated at the year x after construction or the application of an overlay

Age= Age of the pavement since construction or last overlay in years.

The rating for STRD can be grouped as shown in the Table 6.2 below:

Table 0.2: Table 6.2: The Short Term Rate of Deterioration STRD rating groups

STRD Range	**Rating**
$\leq$ 4 points	Low
4<STRD$\leq$6	Medium
6<STRD	High

Based on the above information, the pavement remaining life can be estimated as follows:

$$\mathbf{RL_{PQI}= PQI_X - PQI_t/STRD} \qquad 6.2$$

Where:

STRD= Short Term Rate of Deterioration

PQI_X= Pavement Quality Index at any time x

PQI_t= Terminal Pavement Quality Index (i.e. PQI=30)

These limits can be used extensively in building the decision trees for selecting maintenance and rehabilitation options. Such limits will be used as a trigger values for optimizing the treatment types selection through a comprehensive look insight the condition elements and distress types. These decision trees will be later extended to include roughness, skid resistance and deflection parameters so that the treatment will be able to restore rideability, safety, and structural integrity of the pavement under rehabilitation.

Another approaches suggested by AASHTO to estimate the remaining life based on visual condition evaluation in which the overall condition for pavement section is evaluated by assigning a condition factor (CF) which is roughly estimated by adjusting the layer coefficient of each pavement layer according to the defects observed in the pavement. Then by using a series of charts and figures, the remaining life is estimated. This approach is considered a bit subjective and the condition factor is assigned based on no precise measurements. This approach has been investigated by Robert Elliott for the purpose of determining the practicality of this approach in overlay design (AASHTO 1993). It was concluded that this approach is not appropriate for overlay design and must be revised again to overcome the exiting deficiencies.

Estimation pavement remaining life based on traffic

According to the AASHTO, pavement serviceability decreases with time and traffic from an initial value designated as P_0 until it reaches a failure level designated as P_f providing no major rehabilitation work such as an overlay or partial reconstruction is applied. In this case, the total number of traffic application to failure would be N_f . At any point before reaching the failure condition, where there is a need for applying major rehabilitation work at

traffic level that already applied to the pavement of N_x , then the remaining life would be RL_x. The remaining life (RL_x) is expressed in terms of traffic by the following general equation:

$$[RL_x] = (N_f - N_x / N_f) \qquad 6.3$$

where:

RL_x = The remaining life (RL_x) expressed in terms of traffic ESAL at time t

N_f = The total number of traffic application to failure

N_x = Traffic level in terms of ESAL that is already been applied to the pavement at any point before reaching the failure condition

Therefore, the remaining life from traffic loading point of view is defined as being "*The additional load applications that could have been applied to reach the failure point expressed as a fraction of the total possible load applications* (AASHTO,1993). i.e. , it is the amount of traffic expressed in 18-kips (80kn) ESAL that the pavement could be expected to carry to failure which is according to AASHTO limits is PSI=1.5. This definition is based on the rational idea that pavement structural and functional capacity decreases with the increasing load applications.

Based on the above definitions , the accuracy of predicting the remaining life depends mainly on the accuracy in estimating the actual amount of traffic carried by the pavement up to date of evaluation and, the amount of traffic that the pavement is expected to carry up to failure (which is in this case at PSI=1.5). Therefore, the above equation can be re-written as described in the following equation.

$$[RL] = 100 \left[1 - \left(\frac{Np}{N_{1.5}} \right) \right] \qquad 6.4$$

where:

RL: Pavement remaining life in percent.

N_p= Total accumulated traffic in ESAL to date.

$N_{1.5}$=Total design traffic in ESAL to failure limit (i.e. when PSI=1.5).

The failure limit may be expressed in terms of PSI or any other condition indicator. According to the AASHTO, the terminal serviceability limits of 2.0 and 2.5 are the failure limits for major and local roads. This figure can be determined form the design nomographs or from the design equations.

<u>The above mentioned procedure for determining the remaining life based on traffic data has few deficiencies such as the accuracy of the past traffic estimation, inability to account for pre-overlay repairs, and the accuracy of the AASHTO empirical design method itself at high level of traffic</u>

It is possible that using this procedure, especially for the pavements that have been subjected to extensive maintenance repairs, the resultant value for the remaining life is too low though very little load associated distresses is present or the opposite can also be observed. If the above cases were observed in the reporting, this method should not be used and other evaluation methods based on different condition indicators should be practiced.

In determining the remaining life based on traffic estimates, the inputs from traffic data can be used to determine the condition of the pavement. In this regards, AASHTO propose the condition factor as a representation for the pavement condition. The condition factor=Effective structural capacity after N of ESAL repetitions (**SC_{eff}**) to the original pavement structural capacity (**SC_o**) I.e. :

$$CF = SC_{eff}/SC_o \qquad 6.5$$

Where:

CF= The pavement condition factor

SC_{eff}= Effective structural capacity after N of ESAL repetitions

SC_o= Design or Original pavement structural capacity.

The effective structural capacity can be estimated using various methods and procedures developed for this purpose based on either structural number, modulus of elasticity, deflections measurements and pavement thickness. The details of these procedures have been outlined in the previous chapters.

In PMS system, the remaining life can be computed based on many criteria related to the pavement condition and operational characteristics. Whichever, takes place first, then it will be the control criterion that should be considered in rehabilitation decision making. Engineering judgement should be practiced along with the empirical calculations for deciding the optimum method for evaluating the effective pavement condition.

Structural data in the pavement management systems

Monitoring the structural condition on periodic basis can be accomplished easily within the scope of a network monitoring plan (Hass et al 1993). As described in the previous chapters, structural data can be used effectively to report about the pavement condition at network level by using the available advanced tools.

Many researches discussed the best ways to use the collected structural data in the pavement management systems and the benefits gained from the incorporation of such data. In this regards, there are few conditions on using structural data in PMS; among these is; the ability of the data to determine the pavement structural deterioration, the ability of such data to help in life cycle costing analysis and finally to be a reliable source of the decision making by selecting the most optimum pavement rehabilitation option at network level. The ability of using structural data will enhance greatly the treatment selection process (Pain 1989).

Falling Weight Deflectometer (FWD) and Rolling Weight Deflectometer (RWD) can be used nowadays effectively to accomplish this task at network

level easily and quickly. Structural capacity parameters proposed in this study to be adopted in pavement management systems to represent the effective structural capacity of the pavement and the relevant prediction models will enable the structural capacity condition indicator to be an integral component in life cycle costing analysis and in the main skeleton of the treatment selection decision trees.

To evaluate the effective structural capacity and predicting the remaining structural life, there are various analytical approaches and methods to analyze the measured deflection data, among these are the followings:

- Deflection measurements using sensors readings at different stations (D_0, D_0-D_2, D_0-D_3, D_0-D_4)
- Curvature Function (D_0-D_{200})
- Back calculated Modulus of Elasticity
- Structural Capacity Index

Estimation pavement remaining life based on deflection method

Deflection is considered as one of the main parameters that indicates the structural pavement capacity of the flexible pavements. It is used successfully at project level and to a certain level at network level. Generally, it is well know that deflection has a trend to decrease at the early age of the pavement after opening to traffic (usually first 2 years of service), then the deflection value increases as traffic increases. The initial decrease in deflection value is mainly due to the effect of what is called the “densification” of the asphalt mix after being subjected to traffic action.

This trend and the time period in which the deflection value decreases with traffic is controlled by many factors such as loading rate, tire pressure, asphalt layer thickness and traffic level. After this stage, the deflection values starts to increase as traffic increase. This point is considered as the commencement point of the structural deterioration of the pavement . therefore, for pavement management system uses, the deflection values

should be recorded after few years of being subjected to traffic action. The study area, this period is selected to be 3 years for the major roads and 5 years for the minor roads respectively.

It has been argued that peak deflection values are that all we need in PMS to give an idea about the structural capacity of the pavement (Hass 1995). Pain indicated that this saying may lack accuracy and the use of the peak deflection as an integral tool in the treatment selection process will be limited if deflection is not related to expected traffic loading.(Pain 1989).

In this study an effort was directed to achieve this condition by developing a relationship between the peak deflection and the traffic level estimated by ESAL repetitions. As described before in chapter 4 and shown in the Figure 6.5 below (shown again here for easy reference) at network level, the remaining pavement capacity in terms pf ESAL and the number of years to reach the structural failure can not be predicted using only deflection measurements as the same deflection values can be obtained at different RSAL values.

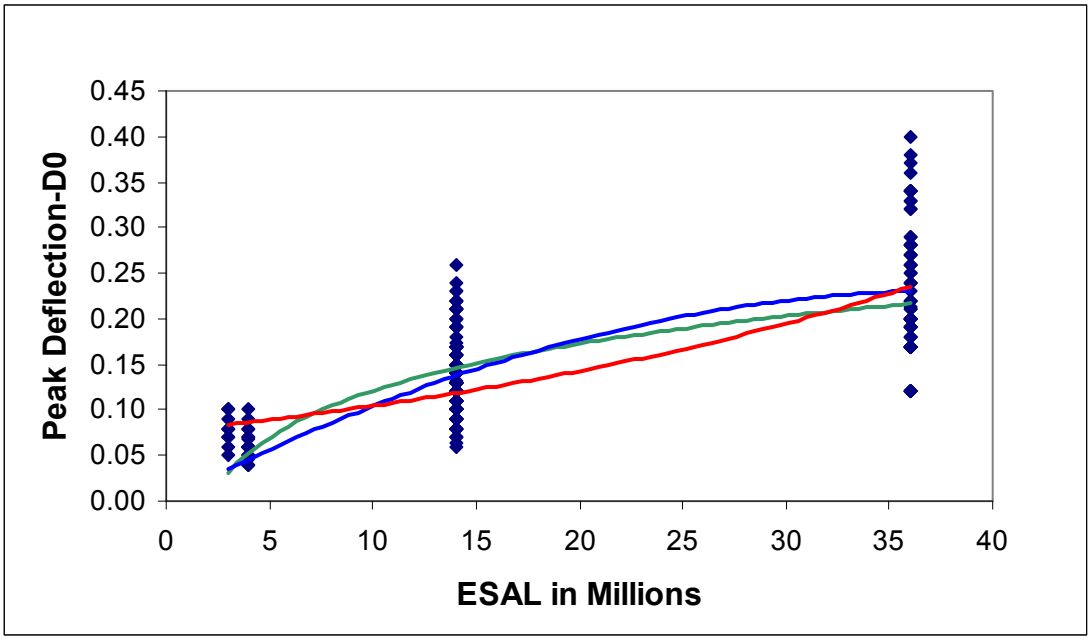


Figure 0.3: Peak deflection against traffic loading in terms of ESAL

Based on the above justification, the deflection measurements can not be used directly for predicting the structural capacity. Alternatively, these measurements can be used effectively to calculate other parameters that can be used in pavement management systems.

In the Australian Guide to the Structural Design of Roads Pavement (Austroads 1992) a simplified approach for rehabilitation (overlay) design has been developed based on both deflection measurements and traffic data. In principal, the deflection parameter can be used effectively as an indication of the rate permanent pavement deformation that is expected to occur as a resultant of the traffic loading. The design deflection for any particular traffic loading can be estimated from the relationship between the cumulative number of ESAL and the measured surface deflection. If the actual deflection is kept below the design deflection, permanent deformation should be kept to an acceptable level.

Based on the above, it is clear that deflection measurement can be used effectively to predict the permanent deformation but it will not be a reliable source of prediction for the fatigue cracking. *The measured deflection and the calculated curvature for the deflection bowel in conjunction with traffic data in terms of ESAL can be used in this case as predicting parameters for the structural capacity and the needed overlay.* The curvature is defined as the difference between the deflection measured at the central point of the loading plate (D_0) and the deflection measured at 200mm from the center of the load (D_{200}):
i.e.

$$\textbf{Pavement Curvature}=D_0-D_{200} \qquad 6.6$$

The curvature can be calculated using different formulas based on the asphalt or pavement layers thickness.

Despite the fact that structural number concept is acceptable in the field of pavement structural evaluation, but it is not the most appropriate method. Therefore, a more appropriate method with less complexity may need to be

used(Pain 1998). In Australia, more simplified approaches based on deflection measurement have been developed and used.

Remaining life based on deflection and curvature

The best models developed for the purpose of estimating the remaining life of the in service pavement at any time are these proposed by the Austroad (Austroad,1992).

They are basically based on direct deflection measurements and traffic in terms of ESAL calculations. The remaining life (**RL**) for the pavement in terms of ESAL at any time is proposed to be calculated using the following equation (Pain 1998):

$$\mathbf{RL = Min\ ((0.207*Deflection)^{-9.8}, (0.29*Curvature)^{-4.7})} \qquad 6.7$$

The overlay thickness is calculated based on two parameters:

Using deflection measurements: The overlay thickness can be estimated using the following equation:

$$\mathbf{OL_{AC} = Max\ (-334*(\ Tolerable\ Deflection/Actual\ Deflection)+320,0)} \qquad 6.8$$

Using Curvature parameter

The thickness of the overlay required after x years of service can be estimated based on the deflection data from which the curvature of the asphalt layer can be calculated. The base models found to be used in this regards is in the following form:

$$\mathbf{OL_{AC}=Max(-189*(\ Tolerable\ Curvature/Actual\ Curvature)+174,0)} \qquad 6.9$$

Where**:**

Tolerable Deflection (TD)=Maximum Deflection value allowed before failure. It is calculated as being:

$$\mathbf{TD=4.6\ *(ESAL_{Design})^{-0.102}}$$

Tolerable Curvature(TD) =Maximum curvature value allowed before failure. It is calculated as being:

$$TC=3.43 *(ESAL_{Design})^{-0.211}$$

Where :

$ESAL_{design}$= The cumulative standard axle load during the design life of the pavement.

Actual Curvature (AC) = Curvature value measured after X year of service.

The measured deflection and curvature should be corrected to both temperature and load. The above equation represents the two modes of failure for the flexible pavements. The term that includes deflection predicts the permanent deformation that might occur under traffic loading. The term that encompasses curvature parameter predicts the likely fatigue cracking that might occur in the flexible pavement as a result of pavement flexing resulted form traffic loading repetitions. The minimum of the two values will control the general performance of the pavement. In this case, the remaining life and the overlay thickness for the pavement will be calculated based on the minimum value of the two terms.

It is worth to indicate here that the deflection data used to derive the above formula, as well as other Austroads equations, were obtained using Benkelman Beam and Lacroix Deflectograph. The Benkelman Beam deflection test is usually carried out using a loaded truck - typically 80 kN (18,000 lb) on a single axle with dual tires inflated to 0.480 to 0. 550 Mpa (70 to 80 psi). Lacrois Deflectograph deflection test is almost similar to the Benkelman Beam in method of testing and loading level (Austroads 1992). Therefore, deflection data obtained using Falling Weight Deflectometer should be corrected or standardized to calculate the " Beam Deflection " value. The Beam Deflection is calculated using the following equation:

$$\textbf{Beam Deflection}= (0.055+(1.19(D0)*550) / (Load/1000) \qquad 6.8$$

where:

D0: Deflection measured at the central point of the loading plate

Load= Actual Load at which the FWD deflection is measured.

The Curvature Function is calculated using the following equation:

Curvature= 0.007+1.2(D0-D200)*550/(Load/1000) 6.9

Calculating the remaining life based on deflection data can be considered as a supplement to the decision taken based on other condition indicators. The identification of the required maintenance and rehabilitations and the optimum treatment can should be also judged by an experienced engineer. The above equation can be converted in to years by simply dividing the total remaining life in terms of ESAL repetitions by the number of traffic repetition in terms of ESAL per year.

Austroads proposes no approaches for predicting the future remaining life as no model were developed for this purpose. The values of these parameters will be helpful in identifying the pavement structural capacity and calculating the remaining life.

Austroads formulas can be used effectively in pavement management system since it depends on variables that can be easily measured. At network level, each pavement section will have a value for the remaining life and the overlay thickness.

Using models to predict the pavement deterioration based on deflection and ESAL , the structural capacity and the needed strengthening can be available in PMS at any time. In this case. the decision makers will be able to decide the level of the intervention and allow and propose the recommended rehabilitation treatment based on structural capacity evaluation as well as other pavement condition indicators mentioned elsewhere in this study.

In this regards, the effective structural capacity assessment will be either effective structural capacity calculated based on Structural number and E

values, or the remaining life estimated based on measured deflection, curvature and traffic data.

Uses of Deflection characteristic value (DCV) in PMS

Similar to the proposed characteristic value method for roughness, this approach is also applicable to the output from deflection analysis. The concept of "the characteristic value" is considered as one of the methods available to decision makers and maintenance engineers that allows to apply statistics to current data and analyze the risk inherent in such data (Austroad, 2000). As mentioned in chapter 4, this approach can be applied to all data pertaining to all condition indicators collected for condition assessment in order to establish the condition for each road and the network as a whole. This concept is particularly useful where the data is to be used for selection and allocation of candidate sections at network level as well as at project level. The characteristic value is defined as :

Characteristic value = $\mu \pm f\,\sigma$ 6.10

where;

μ = The population mean of deflection parameter

σ = The standard deviation of the population and f is given by Table 6.3 below.

Table 6.3: Recommended design values for the deflection parameters

Parameter	**f**	**% of all values that will be covered by the characteristic value**
Back calculated Modulus of Elasticity Value (E)	-2.0	97.5
Remaining Life (RL)	- 2.0	95
Overlay Thickness (OL)	+2.0	90

Provided the mean and standard deviation for any set of data can be easily determined, the chance of the characteristic value being representative of the true population is increased when the factor is set equal to 2.0 . The above table is proposed to be applied on the deflection output to minimize the risk inherent in such data and to be always on the safe side. Underestimating the values of both modulus of elasticity and Remaining life and increasing the overlay thickness will generate a design that can be considered strong enough to serve the intended design life.

Predicting pavement remaining life using other developed mechanistic models

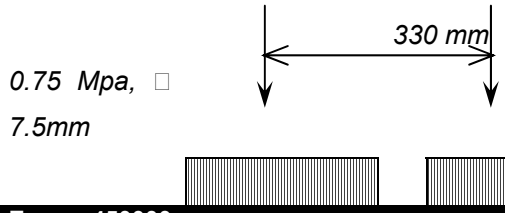
Part of the study objectives was to investigate the reliability of the developed models to predict functional and structural remaining life. The main problem associated with these models is that they yield variant remaining life estimates. The accuracy of the formulas for calculating the overlay thickness and the remaining life using deflection and traffic data can be verified by engineering judgment, close pavement condition monitoring and by a continuous feed back from field.

To explore this issue, a pavement model was built as a test model and the remaining lives based on fatigue, Rutting and roughness modes of failure were calculated. All the models proposed in Chapter 2 methodology were tried and the remaining lives based on different failure modes were calculated.

When a mechanistic design procedure is being developed, the distress mode should be identified to predict the pavement performance. Without first identifying the modes of concern, the performance indicators and, in turn, the pavement responses (stress and strain) cannot be selected. The distress modes that are considered in this study are the permanent deformation (rutting), fatigue cracking and roughness.

Therefore, two types of strains have been considered most critical for the design of flexible pavement as suggested in this methodology. One is the vertical compressive strain on the surface of sub-grade soil that causes the rutting and roughness. The other is the horizontal tensile strain at the bottom of asphalt mixture that causes the fatigue cracking. These two strains are used as failure criteria in the mechanistic design methods. The pavement performance is predicted and quantified in terms of the number of repetitions to failure due to each selected distress mode.

The analysis of the data was carried out to estimate the remaining life in terms of ESAL and the number of years to reach complete failure.



45mm	AC Wearing Course	E = 450000 psi	(3100 Mpa)	ISOTROPIC	Eh/Ev = 1
90mm	AC Binder course	E = 450000 psi	(3100 Mpa)	ISOTROPIC	Eh/Ev = 1
90mm	AC Base course	E = 39000 psi	(270 Mpa)	ISOTROPIC	Eh/Ev = 1
130mm	80% wet mix macadam	E = 39000 psi	(270 Mpa)	ANISOTROPIC	Eh/Ev = .5
290mm	30% CBR sub-base	E = 27500 psi	(190 Mpa)	ANISOTROPIC	Eh/Ev = .5
Infinite	Subgrade CBR 15%	E = 12500 psi	(85 Mpa)	ANISOTROPIC	Eh/Ev = 1

Figure 0.4: Pavement model inputs built for Remaining Life (RL) comparison

The model was tested under Low, Medium and High traffic loading levels. The estimated traffic loading were ,0.177, 0.41 and 1.25 million ESAL/year for Low, Medium and High traffic loading levels respectively. The physical

and engineering properties of the mix used in construction of the asphalt layers are shown in Table 6.3.

The Maximum compressive strain at top of subgrade soil and Maximum tensile strain at bottom of asphalt layers under the AC layer were calculated using CIRCLY software as shown Table 6.4 below.

Table 0.3: Three Examples of pavement cross sections for three roads (low trafficked, medium trafficked, and high trafficked).

Layer	Thickness (mm)		
	Low	Medium	High
Bit. Wearing Course	45.0	45.0	45.0
Bit. Binder course	0.0	0.0	90.0
Bit. Base Course	60.0	80.0	90.0
Wet Mix Macadam	130.0	150.0	130.0
80% Road-base Aggregate	0.0	0.0	0.0
30% CBR Subbase Aggregate	290.0	300.0	150.0
Sub-grade CBR 15%	infinite	infinite	infinite

Wiffin, and Lister (1962) stated that most materials behave as elastic in case of low stress. At speed over 15 mph, the behavior of the pavement is essentially elastic. Therefore, elastic theory can be used to represent the pavement behavior with a reasonable degree of accuracy.

CIRCLY elastic program is used in modeling the pavement sections then calculating the pavement responses (stress, strain, deflection) which are required for the damage prediction models. In this methodology, the maximum tensile strain at bottom of asphalt layer and the maximum compressive strain at top of sub-grade are selected to represent the pavement responses. These responses are selected as they are the major influence in the pavement performance.

Table 6.4 and Table 6.5 summarize the pavement responses (output of CIRCLY program) for the three example pavements cross section. Although

the maximum tensile strain at bottom of asphalt layer is in yy direction (parallel to truck wheel path), the fatigue cracking begins as longitudinal hairlines cracks parallel to the wheel path i.e. caused by the tensile strain at bottom of asphalt layer in the xx direction (perpendicular to the wheel truck path) as shown in Figure 6.4 below.

This is may be because the tensile strain in the yy direction recovered by the load itself during motion i.e. study of the horizontal strain in yy direction at point 1 Figure 3b during the load motion from location 1 through location 3 shows that the horizontal strain when the load at location 1 is compression strain then tension while the load at location 2 then reflected to compression when the load reach location 3.

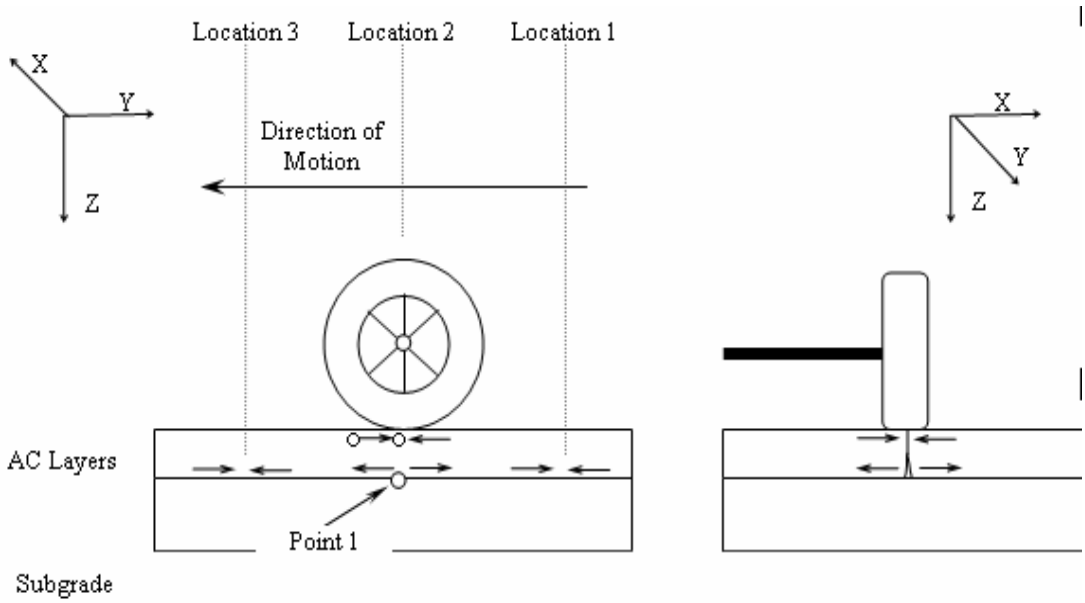


Figure 0.5: Critical Stress and Strains illustration under moving wheel load.

Due to the reasons discussed above the values of the maximum tensile strain in the ε_{xx} direction at bottom of asphalt layer and the maximum compressive strain ε_{yy} at top of the subgrade soil selected in this study as the pavement responses.

Table 0.4: Pavement responses for the three example pavements

Layer		Pavement cross section		
		Low Traffic	Medium Traffic	High Traffic
Max tensile strain at bottom of asphalt layer	ε_{xx}	0.1877 E-3	0.1516 E-3	0.07704 E-3
	ε_{yy}	0.2634 E-3	0.2189 E-3	0.1214 E-3
Max compressive strain at top of sub-grade soil		0.219 E-3	0.182 E-3	0.150 E-3

Table 0.5: Materials characteristics of the pavement model used in the comparison study.

Material data			
parameter	**Value**		**Definition**
Low Traffic)	**(Medium Traffic)**	**(High Traffic)**	
Et =0.0001877	0.0001516	0.0000770	Maximum tensile strain at bottom of asphalt layer
Ec =0.0002190	0.0001820	0.0001500	Maximum compressive strain at top of subgrade soil
E (asphalt)		450000	Dynamic modulus of asphalt mix (psi)
E (subgrade)	=	12500	subgrade resilient modulus (psi)
%AC	=	4.00	Effective asphalt content by weight
Vv	=	5.30	Percent air void
Peff	=	9.20	Effective asphalt content by volume
Traffic data		**Value**	**Definition**
parameter			
Gr		6.5%	Annual Growth rate "%" (Geometric growth)

The remaining life calculations shown in Table 6.6 to 6.8 have shown that different models has generated different values. None of the above mentioned models has yielded a close figure to the others. One common result was found to exist between these models which is the agreement that

mode of failure will be fatigue cracking. The model developed by asphalt institute was found to be the best model was found to calculate the remaining life .

Table 0.6: Number of repetition to failure (fatigue Cracking) for various developed models.

Model	N_f to failure (Fatigue Cracking) & Life before fatigue cracking (Years)		
	Low Trafficked	Medium Trafficked	High Trafficked
Asphalt Institute MS-I (Collop et al, 1995)	1,171,000 (6 Years)	2,364,000 (5 Years)	21,932,000 (12 Years)
Model suggested by Fin et al, 1977 (Hisham, 1997)	1,582,000 (7 Years)	3,196,000 (7 Years)	29,647,000 (15 Years)
Asphalt Institute MS-II (Kenis et al, 1982)	752,000 (4 Years)	2,185,000 (5 Years)	64,236,000 (23 Years)
Model suggested by Shell Oil (Kenis et al, 1982)	4,077,000 (14 Years)	13,691,000 (18 Years)	636,183,000 (56 Years)
Model suggested by Anderson (Anderson, 1990)	3,776,000 (14 Years)	5,787,000 (10 Years)	22,409,000 (12 Years)
Illinois Department of Transportation model (Thompson et al, 1984)	757,000 (4 Years)	1,436,000 (3 Years)	10,936,000 (7 Years)
U.K Transport and Road Research laboratory (powel et al, 1982)	2,084,000 (9 Years)	5,242,000 (10 Years)	97,610,000 (29 years)
Model suggested by Shell, 1978 (Austroads final draft, 2001)	2,032,000 (9 Years)	5,911,000 (11 Years)	174,398,000 (37 Years)

Table 0.7: Number of repetition to failure (Roughness) for various developed models.

Model	No. of load repetitions and Life before roughness failure		
	Low Trafficked	Medium Trafficked	High Trafficked
Model suggested by Anderson, 1990	7,715,000 (21 Years)	10,383,000 (15 years)	14,031,000 (9 years)

Table 0.8: Number of repetition to failure (Rutting) for various developed models

Model	N_f to failure Rutting) &Life before Rutting (Years)		
	Low Trafficked	**Medium Trafficked**	**High Trafficked**
Model suggested by santucci, 1977	136,398,000 (62 Years)	334,050,000 (63 years)	851,677,000 (61 Years)
Asphalt Institute permanent deformation model	31,143,000 (42 Years)	71,222,000 (40 Years)	169,046,000 (36 Years)
Model suggested by Shell	267,362,000 (73 Years)	560,519,000 (71 Years)	1,214,815,000 (66 Years)
Model suggested by Skook, et al, 1982	34,432,000 (41 Years)	78,955,000 (41 Years)	187,919,000 (38 Years)
Model suggested by Shell Oil	1,741,272,000 (103 Years)	4,213,533,000 (103 Years)	10,608,439,000 (100 Years)
Model suggested by Anderson, 1990	9,257,000 (23 Years)	12,219,000 (17 Years)	16,330,000 (10 Years)
U.K Transport and Road Research laboratory	17,630,000 (32 Years)	36,620,000 (30 Years)	78,601,000 (26 Years)
Belgium Road Research Center (Verstraeten et al,1982)	25,315,000 (37 Years)	56,623,000 (37 years)	131,312,000 (33 Years)
Model suggested by Austroads 2001	$2.E^{+11}$ (181 Years)	909,663,825, (189 Years)	3,521,614,607, (192 Years)

Asphalt Institute permanent deformation model has been found to give the optimum realistic estimation for the rutting life before failure at different traffic levels. On the other hand, Model suggested by Shell, 1978 (Austroads final draft, 2001) was found to give the best estimate for the remaining life based on fatigue cracking failure criterion at various traffic levels. <u>It is therefore, recommended always to use the models that include both the strain (ε_t)and modulus (E) variables when estimating fatigue cracking failure mode and to use models that include only compressive strains when rutting is considered as a failure criterion.</u>

Modulus of elasticity based method

Modulus as a basic material property has gained an increased attention to be used in the mechanistic analysis of the multi-layered pavement systems and also for predicting the pavement performance by predicting the pavement distress such as cracking and rutting. It is now a property that has been internationally recognized by the material and highways agencies as a basic characterization of the construction material. Therefore, it is the aim of this study to proof the various good uses of such property. In addition to all the aforementioned properties, the modulus is easy to be estimated using the non destructive testing equipments.

The analysis conducted on data collected from the field showed that elastic modulus can be used effectively to deduct the structural deterioration based on either traffic or age variable or both. Therefore, it may be used as a sound structural condition indicator in PMS reporting tools. It is well acknowledged that, the elastic modulus for the asphalt material is considered the most sensitive to the temperature and loading rate. Therefore, to obtain valuable and consistent evaluation results, the measurement of this property should be consistent across the time and procedure. In order to have such consistency in the modulus value along the successive evaluation cycles, the measurement procedure should be same and should be conducted at the same time period each year.
Generally, the elastic modulus may range in value from 1500Mpa to 3000Mpa and may reach sometimes to 7000 Mpa for certain types of mixes especially those designed with modifiers Based on the results of an intensive laboratory testing carried out through a pioneered asphalt research project in the study area for the available mixes used in pavement construction, this value was found to range from 1700Mpa to 2150 Mpa tested at 40° C as the area suffers from hot climate.

This typical values were measured at loading rate equivalent to typical vehicle speed 50Km/h. For most of the roads, the speed ranges from

40km/h for local roads to 130km/h for the freeways. The recommend elastic modulus for this range is ranges from 1700 to 6000Mpa measured at 40° C. The prediction of the remaining life using the aspect of modulus deterioration can be fulfilled by using lab tests and the developed deterioration curve. Many models have been developed for determination of the aged modulus based on material and bitumen characteristics.

Remaining life and Structural failure based on E value

The changes in the value of the asphalt modulus can be used effectively a structural failure criteria. Based on this trend, the effective structural capacity and the expected remaining life can be predicted with reasonable accuracy. In this regards, it is proposed that the value of the ESCI equal to 30, is to be selected as the *Failure Limit* at which a major intervention should be taken. Based on Ep/Ep_{eff} ratio determined based on the back calculation of the deflection data, the ESCI can be determined and the level of intervention can be established. At the value of ESCI equal to 30, the remaining life in years will be almost equal to zero. This conclusion is established based on the fact that the modulus of elasticity will loose more than 85% of its original value when cracking is started to be observable.

This conclusion is based on the analysis of the collected data for the cracked portions for hundreds of pavement sections tested by FWD which indicated that when the cracking becomes observable or visible on the pavement surface (i.e. the severity approaching medium or high), the effective asphalt or pavement modulus was found to be about 20 - 35 %of its original design value. This means that the pavement structure has already lost around 65 to 80% of its effective cross sectional area and the asphalt layer will perform as a loose aggregate layer. The value of Ep/Ep_{eff} ratio equal to 0.30 or the ESCI of value of 30 were chosen as a structural failure criteria used in treatment selection procedure as will be discussed in next chapter 7 and chapter 8 .

Based on the basic material engineering principals, the changes in asphalt modulus can be considered as a representation for the changes pavement modulus since the value of the elastic modulus for the unbound material do not vary considerably with time passing. The reduction in asphalt or pavement modulus will affect the performance significantly as it will result in either fatigue cracking or permanent deformation (rutting).

In order to evaluate the importance of the asphalt or pavement modulus on pavement performance, it is very crucial in this study to understand the philosophy of the asphalt mix design and the principles of pavement design. This is because, the asphalt mix is normally designed in balanced shape so that it can resist both fatigue cracking and permanent deformation. This means that the mix should be designed so that it has sufficient shear strength to resist rutting and high tensile strength to provide sufficient resistance to fatigue cracking.

In this regard, the aggregate particles distribution is considered as the most important property that prevents shear deformation (Rutting). The aggregate distribution should be designed in such a way, the asphalt mix is quickly stabilizes under traffic with further densification. This densification takes the form of either a secondary compaction under traffic loading or the reorientation of the aggregate particles. Further densification in the early ages of the pavement strengthen the pavement stability . but after that, this further densification may lead to permanent deformation or rutting.

On the other hand, Asphalt binder plays a decisive rule in providing the required fatigue life and preventing the durability problems. The percentage of asphalt used in the mix should be carefully selected so that enough binder will be available to guarantee enough flexibility and longer life for the pavement.

Air voids content in the asphalt mix is also very important to have good mix design. In general, the air void amount is a function of the gradation and the voids in the mineral aggregate. Proper selection of the air void percentage in

the mix will ensure the mix will not be flushed or rut under the traffic as a result of shear strength loss in case of low air void content. Proper selection of the air voids in the mix also ensure that the mix will not fail under the durability effect in case of high air void content. So, it is obvious that a good mix design is that after construction, a little change will take place in the air voids.

Many researches and lab tests indicated that the percentage of the air voids in the mix may be used as an indication to the pavement performance if destructive testing is to be undertaken for further assurance about the effective condition of the pavement. Based on many cores and field samples taken for many pavement section in failed mode i.e. rutted or cracked pavement, it was observed that if the percentage of the air voids is less that 0 to 30% of the design values, the pavement is considered as a failed pavement. In this case the voids filled with bitumen will be more that the acceptable limit and the mix will be flushed and rutted.

Permanent deformation is controlled by many factors related to the asphalt mix properties and the underneath materials. Aggregate properties such as gradation, surface texture and angularity are very significant in reducing the susceptibility of the asphalt mix to permanent deformation. Permanent deformation(rutting) was observed to occur frequently in the upper courses of the asphalt layer particularly the upper 100mm as a result of the repetitive plastic shear strains concentrated under the edges of the vehicle tires.

As shown in the Figures 6.6 below, which has been described thoroughly in chapter 4, it is envisaged that using age or traffic data, the effective condition of the pavement in service can be determined and the number of years to reach the failure point can be predicted.

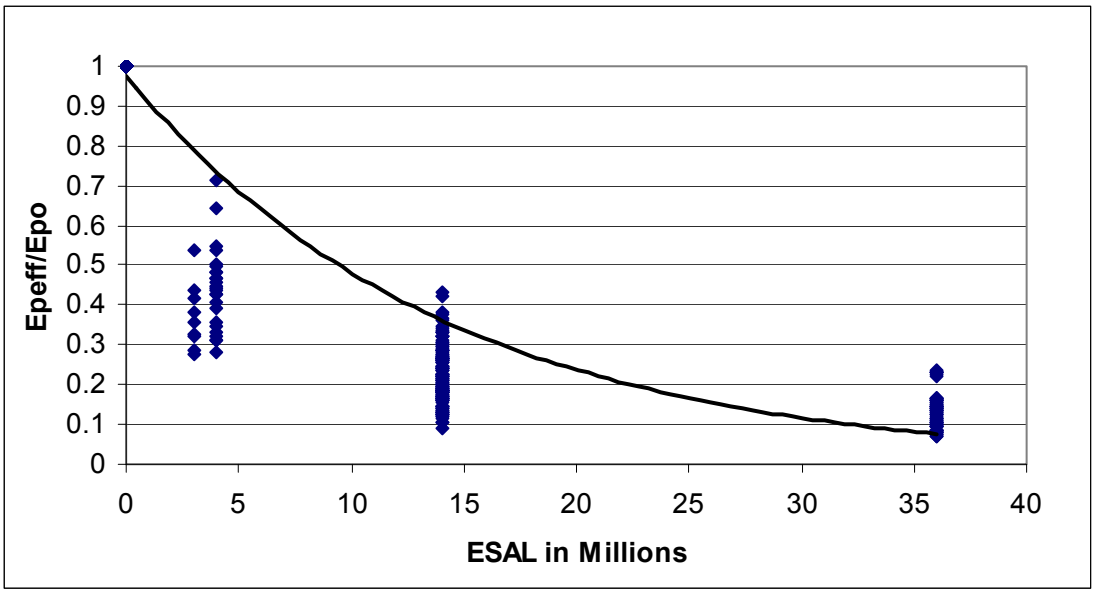


Figure 0.6: The application of the developed exponential model to predict the Pavement Surface Modulus (Ep/Epeff) value.

Prediction modulus of elasticity using simpler method can be easily fulfilled by using the modulus deterioration models developed in this study based data collected from the field. The model represent the deterioration curve takes the following form:

$$Ep_{eff}/Ep_o = 0.975e^{-0.0707ESAL} \qquad R^2 = 0.8488 \qquad 6.11$$

In conclusion, the concept of the relative value of effective pavement modulus to the original pavement modulus at the pavement age equal to zero (i.e zero ESAL) or the relative value of effective SN_{eff} to the original SNo has been proposed in this study as a condition indicators since it gives good representation about the ***exhausted*** and the ***remaining*** lives of the existing pavement structure.

As explained in Chapter 4 and Chapter 5, The ESCI can be determined easily using the following expression:

$$ESCI=100-((1-(SN_{eff}/SN_o))*100) \qquad 6.12$$

Where:

ESCI=Effective Structural Capacity Index (0-100)

SN_{eff} = Effective Pavement Structural Number.

SN_o =Original (Design) Pavement Structural Number

The remaining life for any pavement section can be estimated by considering the pavement failure limit based on the structural index set in the previous chapters. Using ESCI =30 as a failure limit, the number of years that might take to reach this limit can be calculated as follows:

$$STSD = ESCI_0 - ESCI_x / Age \qquad 6.13$$

Where:

STSD = Short Term of Structural Deterioration in terms of ESCI /year

$ESCI_0$= Structural index from the age equal to 0 (i.e. ESCI=100)

$ESCI_x$=The ESCI at the date x of testing calculated as in equation …….above.

Age= Age of the pavement section since construction or last overlay in years

Based on the above information, the remaining life in years can also be estimated using the following formula:

$$RL = ESCI_x - ESCI_t / STSD \qquad 6.14$$

Where:

$ESCI_x$=The ESCI at the date x of testing

$ESCI_t$=The effective structural capacity index at terminal value (i.e.=30)

STSD= Short Term of Structural Deterioration in terms of ESCI /year

Despite the decrease in ESCI is linear, it is estimated that this trend can be accepted at network level. The curve trend may become non linear as age advances and the deterioration will be faster.

Distress quantity based method (% of cracking and Rutting depth etc).

Various researches have been conducted to establish the failure limits of the flexible pavement using different condition indicators. In general, alligator cracking and rutting are the main structural distresses that can give a sound physical indication about the condition of the pavement and the cause of deterioration. Therefore those two types of distresses are usually referred to as the " Pavement Modes of Failure ". Huge efforts were paid to establish a well defined relationship between different condition indicators and the failure limits. In this regards, the relationship between deflection and pavement remaining life have been investigated by many researchers(Blight 1974, Lister 1972, and Lister and Kennedy, 1977).

In addition to that, the relationship between deflection and allowable deflection values under the effect of traffic and surface type have been also established. Moreover, the relationship between rutting and alligator cracking and the pavement failure limits have been studied and failure criteria were suggested. The following subsections describe in details the failure modes limits and its uses in PMS applications:

Fatigue Cracking,

This issue is very crucial in designing the decision trees for treatment selection at network level. The decision maker has to rely on an objective measures that can be applied with consistent implication across the pavement sections of the whole network. As indicated earlier in the previous chapters, cracking is the first symptom of the structural failure of the pavement. Upon the observance of the cracking in the pavement, the engineering properties of the pavement will start to decrease by time passing under the effect of traffic and environment.

In pavement management systems, it is very important to establish the critical limits for the percentage of cracking or cracking density that could be considered as the an indication of the structural failure and where the routine

maintenance work will be ineffective. Many researches have been conducted around the world by various highway agencies and interested individuals to specify the limits for such density where it can be said that the pavement is approaching its service life.

In this regards, It is recommended that overlay is warranted if the percentage of cracked pavement, including the patched area, to the total section area exceeding 15% (Nagumo et al ,1972). This percentage was investigated by others and found that the cracked amount should be around 10% to consider the pavement at the end of its structural life (Pedigo et al ,1982).

In order to establish the cracking limits in the study area, a pilot study has been conducted in which data bout the cracking and patching quantity has been investigated. 19 pavement sections pertain to a highly trafficked roads have been evaluated and the quantity of distress and patching were recorded as shown in Table 6.8a below.

In this study, the quantity and percentage of cracking in Low, Medium and High severity were considered. The quantity and percentage of the patching at different severities observed in these selected sections were also recorded. The reason behind including the patching into this percentage is due to the reason that fatigue cracking is the most frequent distress observed in these sections. Based on the historical records available in the PMS database, the patched areas were originally a badly cracked portions with medium or high severity alligator cracking.

Based on the outcomes of this pilot study, It is estimated that, for the pavement sections of PQI between 40 and 70 , which is considered the optimum range for rehabilitation, the pavement is in need for major rehabilitation, such as mill and inlay , partial reconstruction or reconstruction if the area of the cracking and patching are in the range between 10 to 17% of total area of the pavement section. In average, it was found that the pavement sections is considered failed from structural point of vie if the

percentage of cracking and patching exceeds 12% of the total section area. From economical point of view, it is impractical to patch all the cracked areas because this will increase the roughness level. In this case major rehabilitation is a necessity.

From the analysis of the data collected from these sections, it was found that a section with around 7% in average of High severity fatigue cracking and almost 5 % of mediums cracking can be considered as approaching the failure limit. Also, it was also observed that if percentage of the cracking at severities higher than low (i.e. medium and high severity cracking), exceeds 10% in average of the total pavement section area, then it is possible to consider it as it is at the end of the service life.

The above limits were established based on a representative data extracted from the field. These limits can be calibrated periodically based on the feedback from the field and by comparing the outputs generated from the PMS and the field condition for selected control sections.

Rutting

Rutting as a failure criteria was also investigated and different limits were proposed. In this regards, it was recommended that rutting in pavement should not be more than 12mm depth for major roads and it should be less than 18 mm for minor roads before major rehabilitation or reconstruction is deemed to be warranted(Veverka and Romain,1982). Other study indicated that major rehabilitation or reconstruction should be undertaken when rutting, which is considered as a major failure criterion, reaches a value of 20mm in the wheel track (Corney 1967). Pedigo et al, 1982 indicated that rut depth less than 5mm has no effect on the overall pavement condition (Pedigoetal,1982).

Table 6.8a: Percentage of cracking in the selected pavement sections fall within the range of PQI 40 to 70.

Section No	Area (Sq-m)	PQI	Allig L	Allig M	Allig H	Patch L	Patch M	Patch H	Total Allig	Total Patch	% Crack	% Patch	% Cr+Patch	PQI	% Alli-m	% Alli - H	Avg-Cr-m	Avg-Cr-h	Avg-all	Max-all	Min-all
section 1	2438	16.5	0	528	366	254	389	112	894	755	36.67	30.97	67.64	16.5	21.66	15.01					
section 2	5840	16.7	0	567	312	958	693	364	879	2015	15.05	34.50	49.55	16.7	9.71	5.34					
section 3	5840	24.1	10	955	631	930	150	150	1596	1230	27.33	21.06	48.39	24.1	16.35	10.80					
section 4	4380	25.1	0	380	647	638	0	0	1027	638	23.45	14.57	38.01	25.1	8.68	14.77					
section 5	4380	35	0	149	162	800	649	140	311	1589	7.10	36.28	43.38	35	3.40	3.70					
section 6	4380	37.6	16	201	124	806	585	316	341	1707	7.79	38.97	46.76	37.6	4.59	2.83					
section 7	5840	44.6	0	271	292	342	54	1	563	397	9.64	6.80	16.44	44.6	4.64	5.00					
section 8	4380	51.5	0	210	440	0	1	1	650	2	14.84	0.05	14.89	51.5	4.79	10.05	3.33	6.35	13.17	17.06	5.64
section 9	8001	54	5	203	553	1	226	377	761	604	9.51	7.55	17.06	54	2.54	6.91					
section 10	4380	53	22	182	321	155	3	0	525	158	11.99	3.61	15.59	53	4.16	7.33					
section 11	4380	59	0	126	287	0	0	0	413	0	9.43	0.00	9.43	59	2.88	6.55					
section 12	4380	66	105	42	100	0	0	0	247	0	5.64	0.00	5.64	66	0.96	2.28					
section 13	4380	75	0	12.5	120	0	0	0	133	0	3.03	0.00	3.03	75	0.29	2.74					
section 14	4380	81	0	10	0	0	0	0	10	0	0.23	0.00	0.23	81	0.23	0.00					
section 15	4380	85	5.6	0	0	0.64	0	0	5.6	0.64	0.13	0.01	0.14	85	0.00	0.00					
section 16	4380	90.7	8	0	0	0	0	0	8	0	0.18	0.00	0.18	90.7	0.00	0.00					
section 17	4380	100	0	0	0	0	0	0	0	0	0.00	0.00	0.00	100	0.00	0.00					
section 18	4380	57	0	126	287	1	1	1	413	3	9.43	0.07	9.50	57	2.88	6.55					

Studies undertaken by other researchers have indicated that 20mm rut depth is the most acceptable terminal value that indicates the complete failure of the pavement structure(Veeraragavan et al 1991 and reddy et al 1993).

In the study area, rutting of depth equal to 10 mm was adopted and used in both pavement design and rehabilitation. This value was chosen based on the Transportation Road Research Laboratory (TRRL) design criteria. In fact, the depth of rutting as a failure criterion varies a lot from road agency to another. A 10mm rut depth as a failure criterion is considered conservative as the number of load repetition to failure based on this criterion will be relatively low in comparison with 20mm suggested by other agencies.

The depth of rutting as a major failure mode is used widely in mechanistic-empirical pavement design software. In this regards, based on traffic characteristics and loading configuration, both rutting and fatigue cracking are used in the structural design algorithms to calculate the allowable number of traffic repetitions before the pavement reaches the failure point. This limit will be used in the decision trees in order to select the optimum repair option during generating the network maintenance and rehabilitation scenarios and treatments.

Finite element models developed by researchers in the University of California at Berkeley, has shown that the maximum stresses concentrations resulted from the traffic loading on the pavement system occurs at 50mm pavement depth(Deacon et al. 1994). Therefore, the temperature of pavement during structural capacity evaluation using deflection testing is taken in the asphalt layer at depth that ranges from 30 to 50mm.

In a study conducted in the study area with the asphalt research project, it was found that 8% of the truck traffic exceeds the axle load limit of 18 kip (8.2 tons) (ERES 1991). This 8% of overloaded trucks were found to cause more than 85% of the observed damage in pavement structure in the forms of rutting.

From the above studies , in is concluded that alligator cracking and rutting as a failure criteria differ from one area to another. It is envisaged that, the suggested value of the failure limit for rutting and cracking depends mainly on the following factors:

Targeted level of service differs from one region to another. It depend mainly on the available funds and resources, type of vehicle and traffic level and road class.

Estimation of pavement remaining life based on Roughness

Roughness information can be used for various purposes; such as the ride quality of the newly constructed pavement, assessment of pavement serviceability and performance and finally setting priorities for maintenance and repairs (Janoff 1996). Many studies have indicated that roughness deterioration is affected by many factors such as initial roughness value, traffic and environment. Initial roughness value was found to affect both long term roughness and long term cracking. The sue of roughness information are shown in Table 6.3 below.

Roughness-Present serviceability Index as terminal pavement condition limits.

The concept of Present Serviceability Index- PSI as an indicative measure of the pavement condition and the relationship between PSI and roughness measured by IRI has been thoroughly discussed in the previous chapters. The PSI concept has been and still being used as a main factor into the American Association of State Highway Officials –AASHTO design equations for both flexible and rigid pavements. PSI is an index measured on a scale ranges from 0-5 with 5 being an excellent.

This measure is subjective and it is a function of longitudinal surface characteristics such as slope variance, and certain types of distresses such as rutting, patching and cracking areas.

Table 0.9: Main roughness uses based on various pavement stages.

Area	Uses

New Construction	• Construction quality control • Rideability Level • Control specific Surface Profile Limit
Existing Pavement	• Predict loss of Serviceability • Identify serviceability –Performance Life Histories • Evaluate alternative design
Maintenance and Rehabilitation	• Establish maintenance and replacement criteria. • Locate abnormal highway changes and extreme construction deficiencies. • Evaluate road improvement costs • Prioritize maintenance and rehabilitation needs • Establish basis of allocation of maintenance resources.
Others	• Develop passenger comfort criteria • Determine effect of roughness on vehicle, braking steering etc. • Research into roughness, rideability and related pavement surface problems.

The diagram below shows an example of the remaining life concepts, using roughness as the basis of the pavement life. It should be noted that differing pavement conditions result in differing deterioration curves for each pavement section.

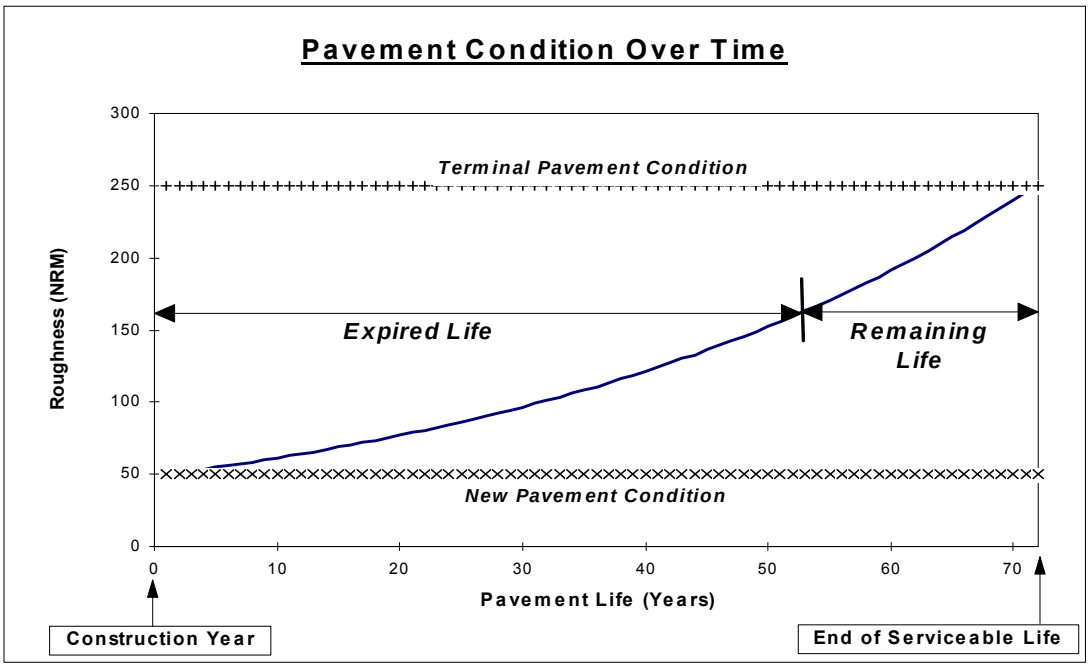


Figure 0.7: Pavement Roughness Deterioration over Time in NAASARA counts

PSI depends on both roughness (longitudinal profile characteristics) and distress data such as rutting, patching and cracking. In this regards, the roughness was found to contribute to the major part of the PSI value. The contribution of the roughness into PSI value was found to be around 95%. (Zaniewski, et al ,1985).

As many shortcoming were found to be inherent in the PSI method for pavement evaluation, it was the task for various highway agencies to develop an objective measurement method to evaluate the serviceability which was finally accomplished with the introduction of the International Roughness Index-IRI method for measuring roughness or serviceability.

The use of roughness data in terms of IRI as a trigger value of one of the main condition indicators for pavement rehabilitation is still referenced to the present serviceability concept because many pavement agencies such as AASHTO are still using this concept in pavement design of both flexible and rigid pavements. Therefore, roughness data is incorporated in the decision process by converting this value into the equivalent PSI value on a scale form 0 to 5.

As stated in the AASHTO pavement design and rehabilitation manuals, the major roads will reach the end of service life when the PSI equal to 2.5 for major roads and a PSI

equal to 2.0 for local roads. This concept will be discussed in details n the following sub sections.

Some arguments have been raised regarding the problems associated with using such simple models, that IRI-Age models assume that traffic levels will not change over time and that they are the same for all roads. Presumably the real parameter affecting the IRI is the traffic loading, as the pavement gets older it has also received more load. A pavement that is old but non trafficked will have a lower IRI than a pavement that is very heavily trafficked but not too old.

This can not be consider as a main drawback in using the IRI–Age models as they are normally developed using large number of pavement sections pertain mainly to major and heavily trafficked roads. Low Volume or Non-trafficked roads (i.e. local roads) are not usually tested for IRI. Such models are usually account for the general trend of the IRI deterioration line with time. They are basically used to predict the roughness level for major roads at network level. Age variable is a preferable predictor as it accounts for the cumulative effect of both traffic and environmental loading. The IRI-ESAL models could be more accurate, but still, the estimation of the traffic loading accurately is usually questionable and not easy to obtain.

The exponential function was found to have best fitness with the general data trend as indicated by the curvature coefficients and sign and the value of the Coefficient of Simple Determination R^2. Also, it is observed that Age variable accounts for around 93% of the deterioration models which indicates the relative importance of such factor on pavement performance. The models also show that, as the age increases, the roughness increases accordingly. At the early ages of the pavement service life, the rate of IRI

The Remaining Service Life (RSL) is needed for the decision maker in the field of maintenance to decide whether the road section is still functioning well or not. RSL based on age variable is estimated using Equation 8 and 9 below and it is taken as the difference between the age at which the roughness reaches its terminal value, calculated based on the models correlate between the PSI and the IRI, and the current age.

$$\mathbf{RSL = Age_{IRIt} - Age_{Current}} \qquad 6.15$$

According to the AASHTO publications, the pavement will be in need for rehabilitation when it reaches of terminal PSI=2.5 and PSI=2.0 for major and secondary roads respectively. The IRI values correspond to the aforementioned terminal PSI values are referred to as the Terminal IRI values.

Therefore, pavement sections with an existing IRI greater than 3.4 (Major Roads) and 4.4 (Secondary Roads) which correspond to a PSI less than 2.5 and 2.0 , are considered to be no more serviceable (refer to Table 4.7., chapter 4) At this time, it will be in need for a treatment to reduce the roughness, such as milling the surface and/or applying a thin or structural overlay and it may in some cases go far beyond these options to require partial or full reconstruction. For application purposes, *the exponential model* (Equation 6) was chosen as the best model that represents IRI-PSR relationship .

The terminal values for the roughness which indicate that the road is no more serviceable have been established in study area using the exponential model as being equal to IRI_t=3.4 and 4.4 for major and secondary roads respectively. Substituting the age value at the terminal IRI, this yields to,

$$\mathbf{RSL = 1/b \ ln\{IRI_t/a\} - Age_{Current}} \qquad 6.16$$

Where:

IRI_t= Terminal International Roughness Index of the pavement (mm/m or m/km).

Current Age = Age of the pavement section since original construction or last overlay in years.

a = The initial IRI value at age equal zero.

b = Measures the curvature of the performance line.

In this case, the RSL is taken as the difference between the age at which the roughness of the section would become equal to the terminal IRI and the current age since construction or last overlay.

The RSL in Equation 6.16 can be estimated for major and local roads as follows:

- Major Roads **(**Freeways, Expressways, Arterial, collector):

$$\mathbf{RSL\text{-}_{Mf} = 1/0.0507 \ \ln \{3.3/0.744\} - Current\ Age} \qquad 6.17$$

$$\mathbf{RSL\text{-}_{MS} = 1/0.059 \ \ln \{3.3/0.769\} - Current\ Age} \qquad 6.18$$

- Secondary Roads (Residential collector and Local roads):

$$\mathbf{RSL\text{-}_{sf} = 1/0.0507 \ \ln \{4.4/0.7442\} - Current\ Age} \qquad 6.19$$

$$\mathbf{RSL\text{-}_{ss} = 1/0.059 \ \ln \{4.4/0.769\} - Current\ Age} \qquad 6.20$$

Where:

RSL, **, RSL$_f$** = Remaining service life of a pavement section in years for slow and fast lanes, respectively.

Current Age = Age of the pavement section since original construction or last overlay in years.

The application of Equations 6.17 to 6.20 is shown in Figure 6.8. As can be seen in this figure, the trend of prediction is almost linear for both classes and for both lanes with relatively considerable difference in the predicted values between them. This can be seen also from the scatter grams of the IRI and pavement age and the value of the line curvature (b=0.059 for the slow lane and b=0.0507 for the fast lane) which is equal approximately the slope of the straight line, i.e. exponent value approaching to zero.

It is clear that the slow lane has more deterioration and has, of course, shorter service life. The reasons behind this linear relationship can be traced to the fact that most of the pavement sections included in the study were extracted from major rehabilitation projects as mentioned before and subjected approximately to equal levels of traffic loading, environment and maintenance. The linear trend for the pavement performance in terms of PSI was found suitable for pavement subjected to frequent overlays (Chiu Liu 2000).

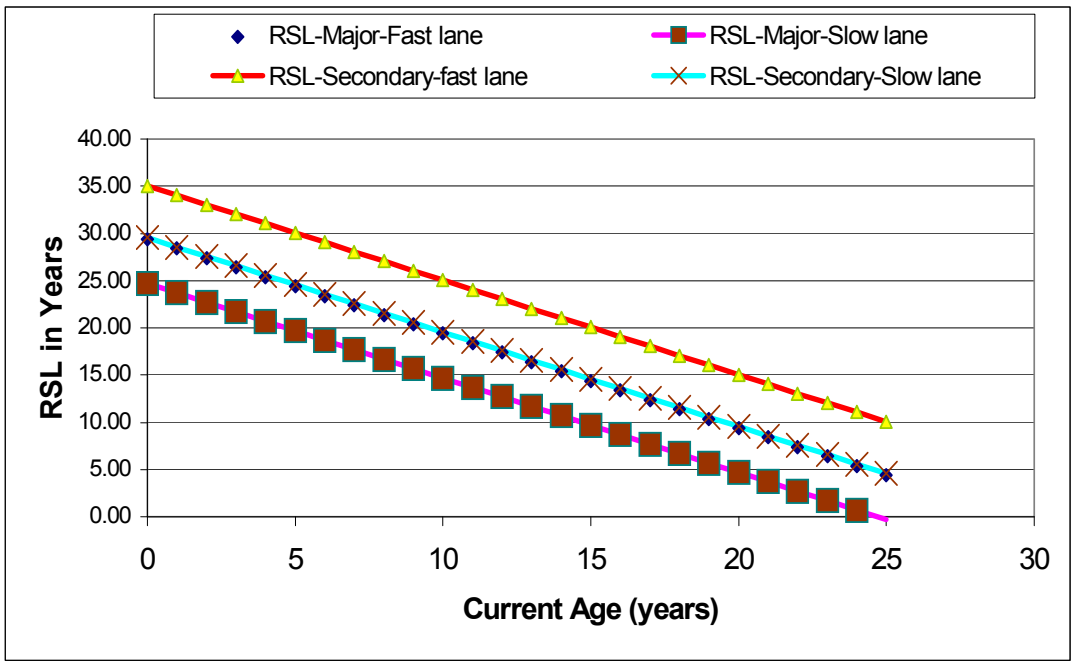


Figure 0.8: Predicted RSL for both major and secondary roads by lane .

Most of the pavement sections included in this study have received the same treatment in the last 20 years (i.e. overlay or mill and inlay). In addition to that, and as indicated earlier, the structural distresses such as rutting do not contribute much to pavement roughness. Therefore, the existing roughness will be a result of built -in construction irregularities and the effect of climatic factors that have almost the same effect on the selected pavement sections.

Roughness data, by itself, is not always a good indicator of the required M&R needs. It is a measure of user riding comfort and safety indicator. Roughness data in combination with other condition data and structural data will result in the most cost effective and suitable treatment option that restore the pavement structural and functional integrity (Shahin 1998).

At project level, roughness information are mainly used to locate road segment with critical roughness and it is a tool of road construction quality control, However, roughness data can be used to estimate the RSL of pavement section or as a decision making indicator.

Estimation of RSL Using IRI

One of the major benefits of the roughness data is the ability to predict the residual service life of a pavement section for the purpose of making decision on the required Maintenance and Rehabilitation (M&R) options and time of application. Applying Equations 6.19and equation 6.20 the RSL for various pavement sections have been calculated as given in Figure 6.11 Those pavement sections with predicted remaining service lives of less than 5, 10 or 15 years can be identified at network level. Attention is given mainly to those sections with remaining service life of less than 5 years. The use of IRI as well as other condition indicators as a decision making criteria will be investigated in details in chapter 7 and chapter 8.

Skid Resistance remaining life models

The major objective for measuring and maintaining the skid resistance data is to monitor the safety level provided by each pavement section so that the number of traffic accidents is kept minimum. Therefore, good safety record for each road section is essential in an effective PMS so that pavement sections with low skid resistance and high number of traffic accidents can be identified and the remedial rehabilitation actions can be applied. Predicting the skid resistance can be achieved using many parameters and measurements procedures.

The maximum value of friction coefficient is measured at the point of critical slip located between 10 to 20 % slip (Shahin 1998). Pavement of a macrotexture with a mean depth of about 1 mm is considered acceptable. Pavement micro-texture is dependent on the aggregate type and angularity. Material such as mountain crushed rock are of good quality while other materials such as limestone are normally give very low micro-structure and the pavement constructed such low quality material may become very slippery, especially when it is in wet condition.

In PMS , the effective value of the Skid Resistance Number (SRN) or any other skid resistance indicators such as, Friction factor , Texture Depth (TD) or the Polished Stone Value(PSV) are needed. These parameters are normally decrease with time and need to be measured periodically. It should be incorporated in the condition report to account for the functional or safety performance trend of the pavement under consideration.

Similar to the other pavement condition indicators, skid resistance is deteriorating with time under the effect of both traffic and environmental effects. Each agency should establish terminal values for the skid resistance level below which, a repair action has to be applied.

In the study area, the wet conditions as a direct result of the rain fall are rarely occurs due to the lack of rainfall precipitation across the entire year. Alternatively, roads surface in this region is affected by high levels of humidity, sand deposits due to the frequent sand storm and high surface temperatures which may create asphalt bleeding in he summer days. In this regards, the sand deposits may be considered the most affecting factor on friction level for the pavement surfaces. Sand deposits can fill the macrotexture cavities.

Under traffic application, these deposits tend to stick with the asphalt surfaces and polish the surface texture creating what is called the aquaplaning or hydroplaning that generates a thin film that separates between the vehicle tire and the pavement surface. On the other hand, the microtexture of the aggregate that constitute the pavement surface mix is significantly damaged as a result of the sun, heat and sand deposits under the effect of traffic loading.

Based on the above mentioned reasons, t is suggested that certain criteria can be applied to the study area as a guide for the maintenance and safety engineers to take the optimum decision regarding the safety level of the roads in service and the required repair action based on this indicator along with the consideration of the other condition indicators. These criteria are based on the following proposed figures:

Table 0.10: Proposed Friction factor values and the corresponding treatment priority level in the study area.

Longitudinal Friction Factor	Skid Resistance Number at speed 64km/h	Rating	Recommended Treatment	Priority
>0.75	75	Excellent	Do Nothing	Level 5
0.65 – 0.75	65 to 75	V. Good	Do Nothing, remove contaminants	Level 4
0.55-0.65	55 to 65	Good	Light treatment	Level 3

			needed (SSD or DSD	
0.40 –0.55	40 to 55	Fair	Surface treatment, (Milling),Grooving	Level 2
<0.40 (Terminal value)	<40	Poor	Resurfacing, Full Mill and Inlay.	Level 1

For the airports, the minimum limit for the friction factor for the runway normalized to 64km/h testing speed should be 0.50 according to the International Civil Aviation Organization (ICAO and Federal Aviation Administration(FAA) standards). This limit may reach up to o.60 in Japan. The limit for roads always lower than the value required on the runways because of the high rolling speed for the airplane which may reach up to 350km/h.

Prediction models that estimate the loss of skid resistance for the roads under traffic effect must contains variables that can be easily measured at network level. Also, skid resistance and speed relationship should be defined so that the skid resistance pertains to each road can be measured based on the posted speed practiced by the vehicle drivers. In general, ***as the speed increases, the loss of friction between the pavement surface and the tire is reduced and consequently the skid resistance is decreased***.

Skid resistance change is expected to depend mainly on the mean texture depth, the initial skid resistance value and the traffic loading. It is speculated that , the reduction in service life as a result of low initial skid resistance or friction factor can be estimated. As shown in the table above, it is estimated that any pavement section of friction factor of 0.40 (i.e. Skid Number SN=40)or less will be failed from frictional point of view.

The initial standard friction factor for the new roads is estimated based on survey done on hundreds of the pavement sections on Dubai is around 0.75. The actual service life in years N for any road can be calculated a s follows:

Original Design life for the pavement(assumed) N_{Des} = 15 years.

- Actual Service Life=N_{Act}

The actual service life N_{act} based on the initial roughness value can be estimated using the following formula:

$N_{Act} = N_{Des} * \{[F_{fAct} - F_{ft}]/[F_{fstd} - F_{ft}]\}$……1

Where:

N_{Act} = Actual Service Life in years

N_{Des} = Design life for the pavement assumed to be =15 years

F_{fAct} = Actual measured friction factor

F_{ft} =Terminal friction factor value

F_{fstd} =Standard (initial) friction factor measured directly after construction.

Substituting the terminal F_{ft} value as=0.4 , the formula can be re-written as follows:

$N_{Act} = N_{Des} * \{[F_{fAct} - F_{ft}]/[F_{fstd} - F_{ft}]\}$

$N_{Act} = 15 * \{[F_{fAct} - 0.4]/[0.75 - 0.4]\}$

The application of this equation is shown in Figure 6.13 below. As shown in this figure, the expected life decreases as the initial friction value increases. This indicates the effect of the pavement surface characteristics during the initial construction works on the performance of the pavement under traffic and environment loading.

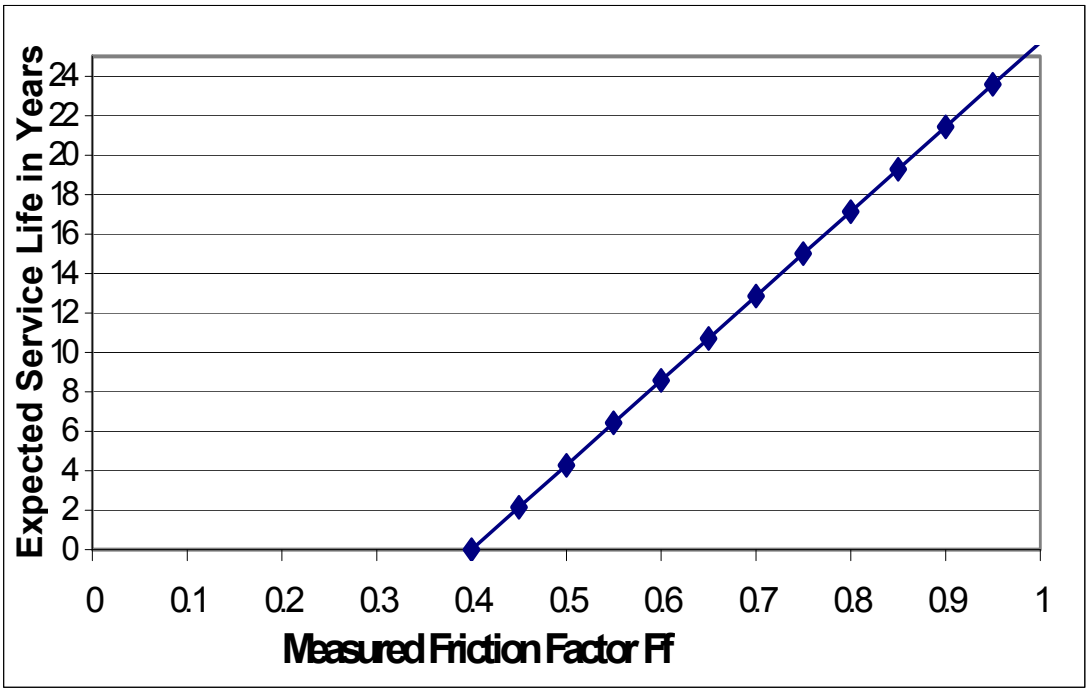


Figure 0.9 Reduction in service life as a result of low initial friction level.

Using the above figure, and based on the terminal values set above, the remaining life can be estimated for the in-service roads at any time of evaluation. The remaining life in this case will be the difference between the estimated life based on friction measurements and the age of the pavement section. Other models will be developed in the coming period of time based on many variables such as texture depth, aggregate or surface type, traffic levels and any others.

DEVELOPMENT OF FEASIBLE MAINTENANCE AND REHABILITATION OPTIONS

Introduction

Modern pavement management systems usually encompass certain modules that include a per-designed pavement maintenance and rehabilitation (M&R) alternatives. It also usually includes deterministic models, developed based on the collected local data, used for predicting the future condition, assigning the required treatment by choosing the proper maintenance category established by each roads agency. Other modules are also available in each PMS such as decision trees used for determining the most cost effective treatment for each section and the time of the application.

The determination of the required treatment depends on many performance classes such as; road class, surface type, condition index ,distress index values, failure mode and district (i.e. Rural or Urban). One of the main shortcomings of the existing systems is the ability of these systems to comprehensively analyze individual projects and determine the proper timing and the cost of treatment (Hicks et al 1999).

This study introduces various methods of treatment selection and planning based on selected performance variables. *In fact, the main purposes of establishing a pavement management system is to propose the proper treatment for the proper pavement section at proper time. This is because, the improper timing will lead to further substantial costs*. This case usually takes place when the pavement is left without applying proper preventive or corrective (routine) maintenance until it reaches a condition where a major rehabilitation or reconstruction is needed.

To achieve the above purposes, it is required that each agency to develop a systematic approach to determine what is the right treatment, which is the right road, and what is the right time in order to generate a cost effective program and provide the benefits of longer service life and customer

satisfaction that is expected. Based on the above, it is envisaged that each PMS should incorporate the following elements:

- **A well -established system guidelines**

A Pavement system is used to provide the overall objectives and strategies and includes, for example, identifying the program coordinator and any environmental or safety issues. It is important to note that such a system must be established to be capable of measuring progress.

- **Determine maintenance and rehabilitation needs**

This element requires that the transportation agency should have a complete inventory such as the pavement identification data, length, type of pavement surface, layer thickness, materials information, traffic, existing condition, roughness etc, pertaining to all the roads and streets under their jurisdiction. For determining the maintenance and rehabilitation needs, Threshold values should be established so that when the condition of a section of a roadway reaches that value, it triggers a pavement preservation activity. These threshold values can be based on whatever criteria related to the main condition indicators each road agency desires.

- **Establish the general framework for treatment selection**

This element should guarantee that the maintenance treatment selected is the “right” one for the type and level of distress, climate, and expected level of service of the candidate project. Understanding the expected performance of the potential treatments for each road agency is very important in selecting the most cost effective treatment.

- **Develop cost effective analysis methods**

There are a number of different procedures to determine the most cost effective treatment as it will be detailed later in the following chapter. Some of these methods are simple and straightforward while some are very complex. Each road agency should determine the best cost effective analysis approach that fits their needs.

- **Establishing a mechanism to determine master work program effectiveness.**

This feature of a comprehensive pavement management system is designed to provide a mechanism for the road agency to determine how effective the program is in accomplishing their highway improvement goals. This can be accomplished through generating the master work program that includes roads to be maintained, priority index, treatment type, level of improvement for each distress types and condition indicators. The effectiveness of the work program can be judged by using various cost effectiveness methods and the overall road performance detailed in the chapter to come.

An early course of preventive maintenance will prevent excessive damage to the pavement and minimize the future maintenance cost. This situation is depicted in Figure 7. 1. The cost values in this figure are just indicative.

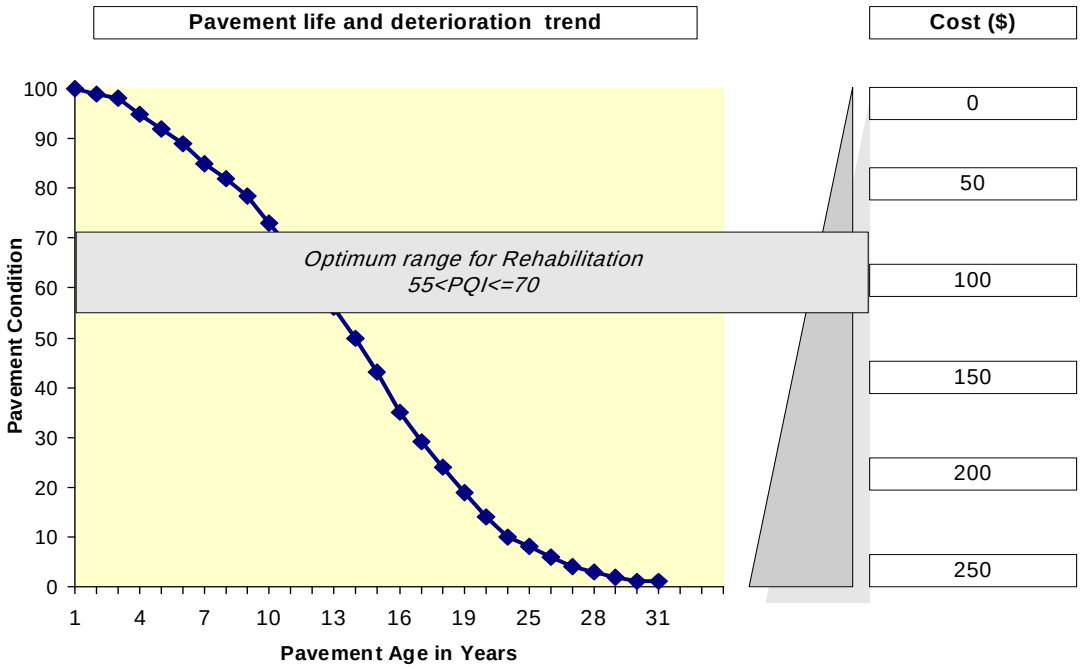


Figure 0.1: Conceptual illustration for the effect of maintenance timing and the corresponding maintenance cost

The trend of the cost is somehow realistic. I.e. the cost of the delayed maintenance if the optimum range of the Performance index (PI) value is

exceeded may reach up to 5 times if the maintenance was carried out within the optimum limits of the maintenance band.

The above figure indicates that the maintenance cost for certain pavement section is not increasing geometrically over time. This means that the cost increase is not steady. As indicated by the same figure, if preventive maintenance is not applied on time for the pavement section, the cost of maintenance for this pavement when it reaches a condition below the routine maintenance will be much more higher than the cost at the initial stages of the deterioration.

Based on the researcher experience, if a particular pavement section is not maintained on time by applying the necessary preventive or routine maintenance, the cost of maintenance at later stage, where major rehabilitation or reconstruction schemes are to be applied, will be at least 15 times more than of the cost of the same section if preventive or routine maintenance is applied on time. This indicates the size of the saving that can be achieved when the right maintenance type is applied at the right timing at both project and network levels. In applying the maintenance program, the main emphasis should be placed on preventing the pavement from reaching the condition in which major rehabilitation is required, as the cost associated with this approach can be substantial.

In selecting the optimum option for rehabilitation of a pavement section, the decision maker should rely on a sound and adequate information. **The optimum option is that one which maximizes the benefits and minimizes the costs.** This option is not always attainable due to the constraints and limitation that usually arise while verifying the available feasible solutions (AASHTO1993).

Selecting the optimum or the preferred option in considered as one of the most complex engineering tasks due to the vast number of variables included in the process of the decision making and limitations that may face the

selection procedure . Once the optimum option is selected, the detailed design can be carried as briefly outlined in the following main steps:

- Determine the main causes of pavement deterioration
- Suggest the feasible options to rectify the situations
- Filter these options against constraints and limitations
- And finally, select the optimum or the preferred option that can restore the riding quality, enhance the structural capacity and satisfies the safety requirement of the pavement taking the existing constraints and limitations into consideration.

The step by step pavement rehabilitation selection process suggested by this thesis writer is shown in Figure 7:2 below.

In PMS, the selection of the feasible alternatives should be based on the results of pavement evaluation using various condition indicators (Shahin 1990).

According to many international references in the field of maintenance and rehabilitation, the most common feasible treatment or repair alternatives that are used in flexible pavement at network level are listed below (Shahin 1998, Shahin et al 1990, Asphalt Institute,1983, AASHTO 1993):

Fog seal

It is spraying of a light coat or diluted emulsion normally 1 to 1 of a bituminous material on the surface of a pavement by means of machine distributor.

Uses: It is used to enrich the pavement surface and binder suffering from raveling and oxidization. It prolongs the service life and improves the waterproofing characteristics of the badly **weathered or raveled** pavement surface pertaining to non –heavily trafficked areas.

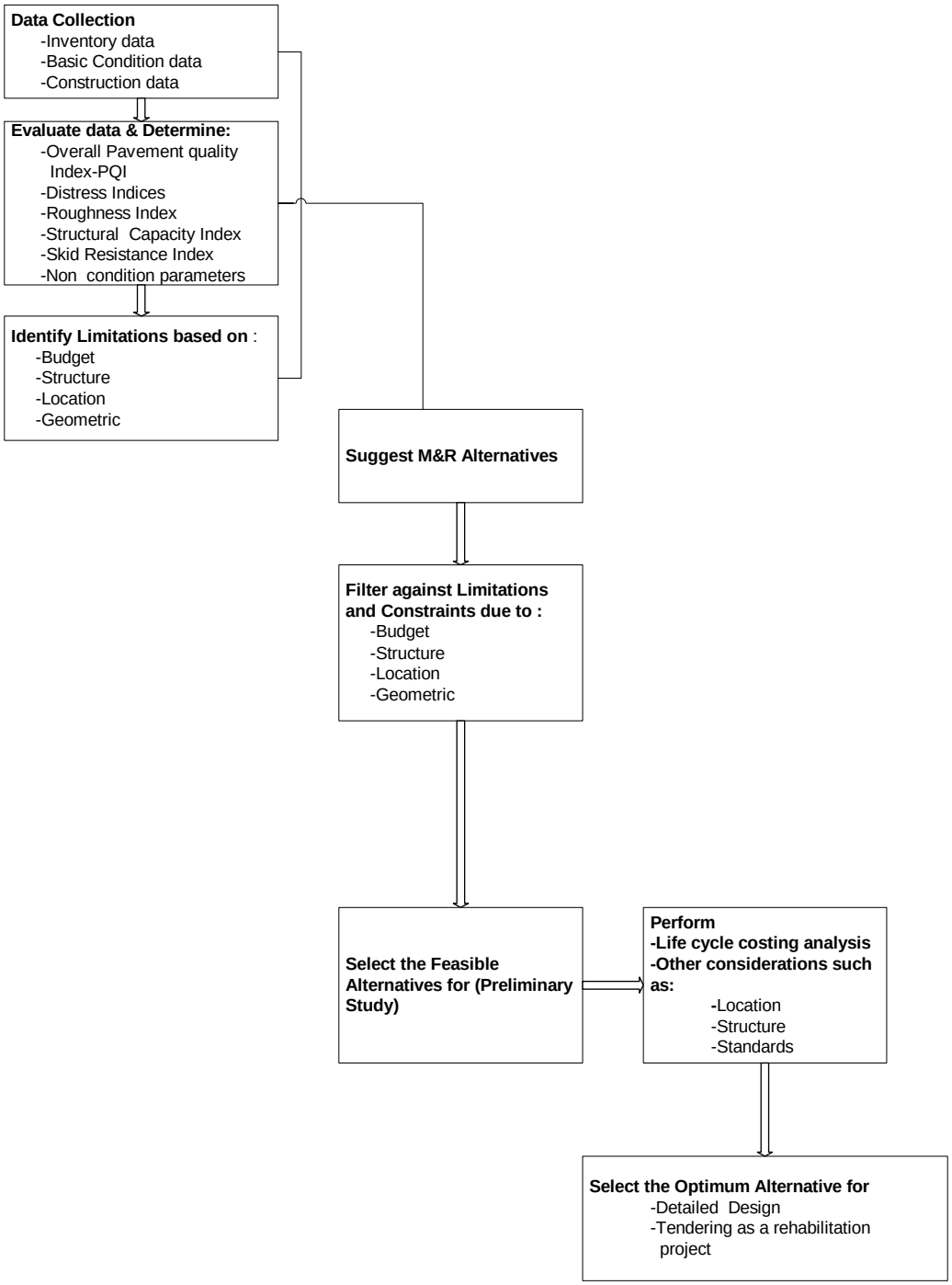


Figure 0.2: Major steps in pavement rehabilitation selection procedure.

Slurry Seal

This method includes applying a thin layer of asphalt mixture that is composed of fine aggregate, mineral filler and water.

Uses: It is used to improve the skid resistance characteristics of a structurally sound pavement. It also fills the hair line cracks (less than 3mm width) and

improves the waterproofing characteristics of the badly **weathered or raveled** pavement surface of the low trafficked areas. In certain case, it is used also on heavily trafficked roads to improve the surface operational characteristics.

Surface Treatment (Chip Seal, Sand Seal)

It involves the application of the asphalt binder and then cover it with a layer of aggregate rolled into the binder to give a new surface layer of a thickness less than 1 inch. If sand is used instead of the aggregate, then it is called Sand Seal.

Uses: This technique is used for low volume traffic roads to provide a skid resistant surface to a structurally sound pavement.

Cold Milling and Inlay

In this technique, part of the asphalt surface of the pavement, cracked or defected, is removed by milling machine.

Uses: This technique involves removing the deteriorated road surface (cracked, rutted, raveled or corrugated). It aims mainly at providing good bonding with the overlay, bringing the road grade to an acceptable level, and rectifying the badly cracked or deteriorated asphalt surfaces.

Hot recycling

This method involves using only the reclaimed asphalt materials from the cold milling along with new asphalt material, new aggregate material, and a recycling agent if needed to produce recycled hot asphalt paving mixture.

Uses: This method is used to rectify or rehabilitate a low or moderately cracked or raveled pavements or any other applications for which the conventional mix can be used.

Cold recycling

This method involves using the reclaimed asphalt materials and additional asphalt or water without the use of heat to produce paving mixture.

Uses: This method is used to rectify or rehabilitate badly cracked or raveled pavements. In this technique, the cracked layer is removed and remixed with

asphalt and water or cement to produce a new mixture which is normally used as a base course layer.

Overlay:

It involves the application of a new asphalt layer to an existing sound pavement to enhance the structural capacity and or improve riding quality. Overlay can be classified into two types:

Functional (Thin) overlay:

This type of overlay is used for improving the functional characteristics of the pavement and repair non-load associated distresses found to occur along the entire area of the existing pavement section. The thickness of such an overlay is usually $\leq$5cm. This case can be represented by a pavement section suffer from medium to high severity bleeding or raveling. Overlay can not be applied on any cracked surface without prior milling the existing surface to remove the cracked surface.

Uses of the thin overlay: This type of overlay is used for pavement sections that suffer from functional deficiencies. Functional deficiencies can be manifested through many condition aspects usually not related to load bearing capacity factors. Cases where thin overlay is suitable to apply include:

- High surface roughness
- Insufficient surface friction
- Weathering and raveling
- Settlements
- Bleeding
- and many other functional defects.

Structural (Thick) Overlay

This type of overlay is used to enhance both the structural capacity and functional characteristics of the pavement and to repair load -associated distresses found to occur along the entire area of the existing pavement section. The thickness of such an overlay is usually $\geq$5cm. This case can be

represented by a pavement section suffers from Low to high severity Cracking, Rutting ,Corrugation or any other distress caused by traffic loading. Similar to the functional overlay, structural overlay can not be applied on any cracked or defected surface without prior milling the existing surface to remove the cracked surface and reach to a sound foundation.

Uses of the structural overlay:

This type of overlay is used for pavement sections that suffer from major structural deficiencies and inadequate load bearing capacity. Cases where thick overlay is suitable to apply include:

High severity load related distresses such as rutting.

Insufficient bearing capacity (alligator cracking).

Reconstruction: It involves the removal of the existing pavement structure and replacing it with a new one.

Uses: This alternative is applied for the purpose of rectifying the badly deteriorated pavement and restoring both structural integrity and riding quality.

Each treatment can improve certain functional and structural characteristics of the roads in service. The expected service life for each treatment applied at the entire area of the pavement section (i.e. overall rehabilitation schemes) and the corresponding aim are summarized in Table 1 below:

The details of the repair actions and the definition of various full scale maintenance and repair alternatives can be found in details elsewhere (Asphalt Institute –MS-17, 1983).

According to Shahin, the above mentioned M &R alternatives are called the Overall Repair Actions. It is usually applied on a lengthy pavement sections of the roads under rehabilitation (Shahin 1998).

Table 0.1: Common Maintenance and Rehabilitation (M&R) alternatives, their uses and the expected service life.

Treatment	Main uses	Expected service life in years
Fog seal	To enrich the pavement surface and improve the waterproofing of **weathered or raveled** pavement surface	3-4
Slurry Seal	To improve the skid resistance, filling the hair line cracks (less than 3mm width) and improves the waterproofing of **weathered or raveled** pavement.	4-6
Chip Seal.	To provide a skid resistant surface to a structurally sound pavement	4-6
Cold Milling and Inlay	To provide good bonding with the overlay, bringing the road grade to an acceptable level, and rectifying the badly cracked or deteriorated asphalt surfaces.	7-10
Cold recycling	To rectify or rehabilitate badly cracked or raveled pavements.	12-15
Hot recycling	To rectify or rehabilitate a low or moderately cracked or raveled pavements or any other applications for which the conventional mix can be used.	7-10
Thin overlay $T \leq 5$cm.	To improve the functional properties of the pavement and repair non-load associated distresses found to occur along the entire area of the existing pavement section.	7-12
Structural Overlay $T \geq 5$cm.	To enhance both the structural capacity and functional characteristics of the pavement and to repair load -associated distresses.	12-15
Reconstruction	To rectifying the badly deteriorated pavement and restoring both structural integrity and riding quality.	15-20

Maintenance and Rehabilitation Categories in PMS

The above described overall Maintenance and rehabilitation activities and schemes in the study area can be classified differently based on the experience gained from practices and the basic aims of each M&R policies.

The selected treatment or repair action is usually falls under one of the following categories (Figure 7.3):

Preventive Maintenance Activities (PMA)

According to the AASHTO, Preventive Maintenance (PM) is defined as " A planned strategy of cost-effective treatments to an existing roadway system and its appurtenance that preserves the system, retards future deterioration, and maintains or improves the functional condition of the system without increasing the structural capacity".

Preventive maintenance is considered as a tool for pavement management system in which non-structural treatments are applied early in the life of a pavement in order to prevent them from further deterioration.

This can be achieved by applying the right treatment to the right pavement at the right time. The following activities or repair options fall within this category:

- **Fog Seal**
- **Chip seal (single or double surface dressing -SSD or DSD)**
- **Crack sealing**
- **Slurry seal**

<table>
<tr><th>PQI/OPQI Range</th><th>Pavement Condition Rating</th><th>Major Roads (Freeway, Expressway, Arterials, Collector, Ind/com Local)</th><th>Residential Local Roads</th></tr>
<tr><td>100</td><td>Excellent</td><td>Preventive Maintenance (86≤PQI≤100)</td><td rowspan="2">Preventive Maintenance (71≤PQI≤100)</td></tr>
<tr><td>85</td><td>Very Good</td><td rowspan="2">Routine Maintenance (56≤PQI<86)</td></tr>
<tr><td>70</td><td>Good</td><td rowspan="2">Routine Maintenance (41≤PQI<71)</td></tr>
<tr><td>55</td><td>Fair</td><td>Major Rehabilitation (Partial Reconstruction (41≤PQI<56)</td></tr>
<tr><td>40</td><td rowspan="2">Poor</td><td rowspan="2">Reconstruction (0≤PQI<41)</td><td rowspan="2">Reconstruction (0≤PQI<41)</td></tr>
<tr><td>0</td></tr>
</table>

Figure 0.3: Various maintenance categories based on the PQI value only.

- **Micro surfacing**
- **Surface treatment**
- **Thin overly**
- **Mill and Inlay of AC surface**

The expected service life for each repair action applied at limited area of the pavement section and the corresponding aims are summarized in the table below:

Table 0.2: The expected service life for preventive maintenance repair actions and the corresponding main uses.

Treatment	Main uses	Expected service life in years
Crack Sealing including Routing	To prevent water , debris and air intrusion inside the pavement layers and to improve the waterproofing of **cracked** pavement surface	1-3
Fog seal (localized area)	To enrich the pavement surface and improve the waterproofing of **weathered or raveled** pavement surface	3-4
Slurry Seal	To improve the skid resistance, filling the hair line cracks (less than 3mm width) and improves the waterproofing of **weathered or raveled** pavement.	4-6
Chip Seal	To provide a skid resistant surface to a structurally sound pavement	4-5
Micro surfacing – polymer modified.	To establish skid resistance, filling the hair line cracks (less than 3mm width) and ruts and improves the waterproofing and improve **weathered or raveled** pavement.	**4-6**
Thin overlay T≤5cm.	To improve the functional properties of the pavement and repair non-load associated distresses found to occur along the entire area of the existing pavement section.	7-12
Mill and Inlay of AC surface	To improve the functional properties of the pavement and repair non-load associated distresses found to occur along the entire area of the existing pavement section taking into consideration the existing geometric dimensions and standards.	7-12

Routine (Corrective) Maintenance Activities (RMA)

It is a planned strategy of cost -effective treatments to an existing roadway system and its appurtenance after failure occurrence to restore the functional condition of the system and improves the pavement riding quality without increasing the structural capacity.

In other words, routine maintenance is comprised of the day-to-day maintenance activities that are scheduled or their timing is within the control of maintenance personnel.

This policy applies to sections with critical PQI/OPQI ranging from 71 to 85 for roads classified as Freeway, Expressway, Arterial, Collectors and Local industrial/commercial. It also applies for sections with Critical PQI/OPQI ranging from 40 to 70 for roads classified as Local roads in residential areas. The following activities or repair options fall within this category:

- **Do Nothing**
- **Crack Sealing**
- **Partial Depth Patching or Mill and Inlay <than 50% of the surface depth)**
- **Full Depth Patching**
- **Skin (Surface) Patching**
- **Heat Sand and Rolling**
- **Fog Seal**
- **Rejuvenator Application**
- **Aggregate Sealing Coat**
- **Surface Seal Emulsion**
- **Replace Patching**
- **Painting Pavement Markings**
- **Cleaning Side Ditches**

Delays in routine maintenance increase the quantity and severity of pavement defects which increase the costs of the routine maintenance work.

Consequently, maintenance delays cause the cost of the life cycle of the pavement to be increased considerably. Table 7.3 below list the recommended routine maintenance repair actions at various severity level (Shahin et al 1979).

Table 0.3: The recommended routine maintenance and repair actions for flexible pavement distresses at various severity levels.

Severity Level → / Distress Type ↓	Low	Medium	High
Alligator Cracking	REJ,SSE	PDP,FDP,REJ	PDP,FDP
Bleeding	DN		
Block Cracking	DN,CS,REJ,ASC	CS,ASC	PDP
Corrugation	DN	PDP	PDP
Depression	DN	PDP,FDP,SP	PDP,FDP,SP
Jet blasting	ASC,DN,PDP		
Joint Reflection Cracking	DN,CS	CS	PDP,CS
Oil spillage	DN,PDP,FDP		
Patching	DN,CS,PDP,RP	FDP,RP	FDP,RP
Polished Aggregate	DN,ASC		
Shoving	DN	CS,PDP	CS,PDP
Slippage Cracking	DN,CS,PDP		
Swell	DN	FDP	FDP
L&T Cracking	DN,CS,REJ,ASC	CS,ASC	CS,PDP
Weathering and Raveling	DN,FS,REJ	FS,ASC	CS,ASC

where:

DN=Do Nothing
CS=Crack Sealing
PDP=Partial Depth Patching
FDP=Full Depth Patching
SP=Skin (Surface) Patching
HSR=Heat Sand and Rolling
FS=Fog Seal
REJ=Rejuvenator Application
ASC=Aggregate Sealing Coat
SSE=Surface Seal Emulsion
RP=Replace Patching

Based on the field experience of the researcher, each distress severity can be classified into 3 levels. Some of the routine maintenance works can be applied to a very limited area. Others can be considered effective if the

density is higher. This proposed set up can be used in PMS's to generate the required routine maintenance report on weekly or monthly basis.

It provide a great help in managing sever and sudden defects that take place as a result of man made actions or as a result of factors not related to normal pavement performance. For each individual distress type, there are many repair action that can be applied *on a limited scale* to rectify only the existing defect without enhancing the structural integrity or the functional capacity. The recommended routine repair actions for each distress at various severity level or threshold values are detailed as follows:

Distress type : Alligator Cracking

For the purposes of the development of the routine maintenance work program, which is usually applied on limited area of density less than 5% of the section area, and the PQI is more than 71 and less than 85 . The set up for the routine repair actions of this distress in PMS can be in the following format:

Table 0.4: The Severity Levels & routine Repair actions for Alligator Cracking

Severity Level	Distress Density % out of the section area	**Repair options**
L1	$0<D\leq1$	SSE (Allig L1)
L2	$1<D\leq2.5$	PDP(Allig L2)
L3	$2.5<D<5$	PDP (Allig L3)
M1	$0<D\leq1$	PDP (Allig M1)
M2	$1<D\leq2.5$	FDP (Allig M2)
M3	$2.5<D<5$	FDP (Allig M3)
H1	$0<D\leq1$	FDP (Allig H1)
H2	$1<D\leq2.5$	FDP (Allig H2)
H3	$2.5<D<5$	FDP (Allig H3)

where:

DN=Do Nothing CS=Crack Sealing PDP=Partial Depth Patching (equiv. to Mill and Inlay <than 50% of the surface depth) FDP=Full Depth Patching SP=Skin (Surface) Patching HSR=Heat Sand and Rolling	FS=Fog Seal REJ=Rejuvenator Application ASC=Aggregate Sealing Coat SSE=Surface Seal Emulsion RP=Replace Patching

The applications of the preventive maintenance and routine maintenance in PMS systems will be shown in chapter 8. The recommended repair actions apply only to the sections that still in sound condition but suffer from few localized defected areas. Such routine maintenance report is considered an integral part of the modern PMS and the required maintenance needs can be budgeted and reported to the maintenance engineers on regular basis.

Distress type : Longitudinal and transverse Cracks

For the purposes of the development of the routine maintenance work program, which is usually applied on limited area of density less than 2.5% of the section area , and the PQI is more than 71 and less than 85. The set up for the routine repair actions of this distress in PMS can be in the following format:

Table 0.5: The Severity Levels & routine Repair actions for L&T Cracking

Severity Level	Distress Density % out of the section area	**Repair options**
L1	$0<D\leq0.25$	DN (L&T L1)
L2	$0.25<D\leq0.5$	CS(L&T L2)
L3	$0.5<D<2.5$	ASC (L&T L3)
M1	$0<D\leq0.25$	CS (L&T M1)
M2	$0.25<D\leq0.5$	CS (L&T M2)
M3	$0.5<D<2.5$	ASC (L&T M3)
H1	$0<D\leq0.25$	PDP (L&T H1)
H2	$0.25<D\leq0.5$	PDP (L&T H2)
H3	$0.5<D<2.5$	PDP (L&T H3)

Distress type: Patching and utility cut patching

For the purposes of the development of the routine maintenance work program, which is usually applied on limited area of density less than 5% of the section area , and the PQI is more than 71 and less than 85. The set up for the routine repair actions of this distress in PMS can be in the following format:

Table 0.6: The Severity Levels & routine Repair actions for Patching and utility cut patching

Severity Level	Distress Density % out of the section area	**Repair options**
L1	$0<D\leq1$	DN (Patch L1)
L2	$1<D\leq2.5$	DN(Patch L2)
L3	$2.5<D<5$	DN (Patch L3)
M1	$0<D\leq1$	RP (Patch M1)
M2	$1<D\leq2.5$	RP (Patch M2)
M3	$2.5<D<5$	FDP (Patch M3)
H1	$0<D\leq1$	FDP (Patch H1)
H2	$1<D\leq2.5$	FDP (Patch H2)
H3	$2.5<D<5$	FDP (Patch H3)

Distress type: Potholes

For the purposes of the development of the routine maintenance work program, which is usually applied on limited area of density less than 0.025% of the section area , and the PQI is more than 71 and less than 85. The set up for the routine repair actions of this distress in PMS can be in the following format:

Table 0.7: The Severity Levels & routine Repair actions for Potholes

Severity Level	Distress Density % out of the section area	Repair options
L1	0<D≤0.01	PDP (POt L1)
L2	0.01<D≤0.025	PDP POt L2)
L3	0.25<D<0.05	PDP (POt L3)
M1	0<D≤0.01	FDP (POt M1)
M2	0.01<D≤0.025	FDP (POt M2)
M3	0.25<D<0.05	FDP (POt M3)
H1	0<D≤0.01	FDP (POt H1)
H2	0.01<D≤0.025	FDP (POt H2)
H3	0.25<D<0.05	FDP (POt H3)

Distress type: Rutting

For the purposes of the development of the routine maintenance work program, which is usually applied on limited area of density less than 5% of the section area and the PQI is more than 71 and less than 85. The set up for the routine repair actions of this distress in PMS can be in the following format:

Table 0.8: The Severity Levels & routine Repair actions for Rutting

Severity Level	Distress Density % out of the section area	Repair options
L1	0<D≤1	PDP (Rut L1)
L2	1<D≤2.5	PDP(Rut L2)
L3	2.5<D<5	PDP (Rut L3)
M1	0<D≤1	PDP (Rut M1)
M2	1<D≤2.5	FDP (Rut M2)
M3	2.5<D<5	FDP (Rut M3)
H1	0<D≤1	FDP (Rut H1)
H2	1<D≤2.5	FDP (Rut H2)
H3	2.5<D<5	FDP (Rut H3)

Distress type: Weathering and Raveling

For the purposes of the development of the routine maintenance work program, which is usually applied on limited area of density less than 25% of the section area, and the PQI is more than 71 and less than 85. The set up for the routine repair actions of this distress in PMS can be in the following format:

Table 0.9: The Severity Levels & routine Repair actions for Weathering and Raveling

Severity Level	Distress Density % out of the section area	**Repair options**
L1	$0<D\leq5$	DN (W&R L1)
L2	$5<D\leq10$	DN(W&R L2)
L3	$10<D\leq25$	DN (W&R L3)
M1	$0<D\leq5$	FS(W&R M1)
M2	$5<D\leq10$	FS(W&R M2)
M3	$10<D\leq25$	ASC (W&R M3)
H1	$0<D\leq5$	ASC W&R H1)
H2	$5<D\leq10$	ASC (W&R H2)
H3	$10<D\leq25$	ASC (W&R H3)

Distress type: Bleeding

For the purposes of the development of the routine maintenance work program, which is usually applied on limited area of density less than 25% of the section area, and the PQI is more than 71 and less than 85.

The set up for the routine repair actions of this distress in PMS can be in the following format:

Table 0.10: The Severity Levels & routine Repair actions for L&T Cracking

Severity Level	Distress Density % out of the section area	**Repair options**
L1	0<D≤5	DN (Bleed L1)
L2	5<D≤10	DN(Bleed L2)
L3	10<D≤25	DN (Bleed L3)
M1	0<D≤5	PDP(Bleed M1)
M2	5<D≤10	PDP(Bleed M2)
M3	10<D≤25	PDP (Bleed M3)
H1	0<D≤5	PDP Bleed H1)
H2	5<D≤10	PDP (Bleed H2)
H3	10<D≤25	PDP (Bleed H3)

Distress type: Depression

For the purposes of the development of the routine maintenance work program, which is usually applied on limited area of density less than 5% of the section area, and the PQI is more than 71 and less than 85. The set up for the routine repair of this distress in PMS can be in the following format:

Table 0.11: The Severity Levels & routine Repair actions for L&T Cracking

Severity Level	Distress Density % out of the section area	**Repair options**
L1	0<D≤1	DN (Bleed L1)
L2	1<D≤2.5	DN(Bleed L2)
L3	2.5<D≤5	DN (Bleed L3)
M1	0<D≤1	PDP(Bleed M1)
M2	1<D≤2.5	PDP(Bleed M2)
M3	2.5<D≤5	PDP (Bleed M3)
H1	0<D≤1	FDP (Bleed H1)
H2	1<D≤2.5	FDP (Bleed H2)
H3	2.5<D≤5	FDP (Bleed H3)

Distress type: Shoving

For the purposes of the development of the routine maintenance work program, which is usually applied on limited area of density less than 5% of the section area, and the PQI is more than 71 and less than 85. The set up for the routine repair actions of this distress in PMS can be in the following format:

Table 0.12: The Severity Levels & routine Repair actions for L&T Cracking

Severity Level	Distress Density % out of the section area	**Repair options**
L1	$0<D\leq1$	PDP (Shov L1)
L2	$1<D\leq2.5$	PDP(Shov L2)
L3	$2.5<D\leq5$	PDP (Shov L3)
M1	$0<D\leq1$	PDP(Shov M1)
M2	$1<D\leq2.5$	PDP(Shov M2)
M3	$2.5<D\leq5$	PDP (Shov M3)
H1	$0<D\leq1$	FDP (Shov H1)
H2	$1<D\leq2.5$	FDP (Shov H2)
H3	$2.5<D\leq5$	FDP (Shov H3)

The details of the repair action and the definition of various partial or full scale maintenance and repair alternatives can be found elsewhere (Asphalt Institute –MS-17, 1983)

Major Rehabilitation and Partial Reconstruction (MRR)

It is a planned strategy applied to an existing roadway system (mainly the upper layers) and its appurtenance after failure occurrence to improve the functional condition of the system and to increase the structural capacity. This policy applies to sections with critical PQI/OPQI ranging from 40 to 55

for Freeway, Expressway, Arterial, Collector and Local roads in industrial /commercial areas).

According to the Asphalt Institute (MS-17) major rehabilitation is warranted if the distress exceeded 40% of the surface area of the pavement. This option is recommended for sections suffer mainly from sever alligator and block cracking, Patching, Raveling and bleeding. The following activities or repair options fall within this category:

Overlay

Surface Large Patching

Full Depth Small Patching

Full Depth Large Patching

Thin Overly

Mill and Inlay of AC surface

Partial Reconstruction (asphalt removal , base course is, sometimes included)

Reconstruction-RCN

It is a planned strategy applied to an existing roadway system (It includes all layers) and its appurtenance after major failure occurrence for the purpose of improving the functional properties of the pavement system, enhance safety and increase the structural capacity. This policy applies for the sections with PQI less than 40 for all roads (Freeway, Expressway, Arterial, Collector and local roads in industrial /commercial and residential areas). The application of this policy indicates that the road has approached the end of its structural and functional lives. According to the Asphalt Institute (MS-17), reconstruction is warranted if the problem is extensive enough. This option is recommended for sections suffer from surface or subgrade problems such as; severe alligator cracking, rutting, corrugation, settlement and depressions. The following activities or repair options fall within this category:

Full reconstruction (Asphalt removal , base and subbase courses are included in this repair action.

The above mentioned M &R alternatives are called the Overall repair actions. It is usually applied on a lengthy pavement sections of the roads under rehabilitation (Shahin, 1998).

Treatment selection optimization methods

Treatment selection process should be a tree like structure with selections continually made from left side to right side incorporating various decision variable related to various condition indicators and others until the optimum treatment is selected. This procedure allows investigating several parameters and then selecting the most appropriate one that rectify both the functional and structural defect at once.

In this study, a step by step procedure will be presented starting by only one variables related to the distress data and PQI values as trigger values for selecting the suitable repair option. Then another condition indicator such as roughness trigger values will be incorporated, then a structural condition indicator using the overall effective structural capacity index (ESCI) will be also incorporated.

Finally, safety indicator parameter relating to skid resistance condition indicator will be incorporated. An examples demonstrating how the type of repair is changing by incorporating addition condition indicator will be highlighted in the computerized applications detailed in chapter 8 of this thesis.

In this study, the following methods for treatment selection using various parameters pertaining to various condition indicators are proposed:

Using PQI Graphical based method

The first step in selecting the treatment alternative that should be applied to a pavement section is to identify the maintenance and rehabilitation category in which the optimum treatment falls.

In this regards, the researcher was able to establish four main categories that can be used to select the optimum treatment for any pavement section.

This procedure is considered the simplest method to determine the maintenance and rehabilitation categories. It does not give any information about the existing distress, quantity and the cause of failure. The proper treatment is then identified at project level by the maintenance engineer. This method is mainly used to allocate the fund and establishing a rough estimate for the annual maintenance budget.

Figures 7.4 & 7.5 show the schematic illustration of the proposed Critical PQI Ranges for different maintenance actions for High and Low class roads, respectively.

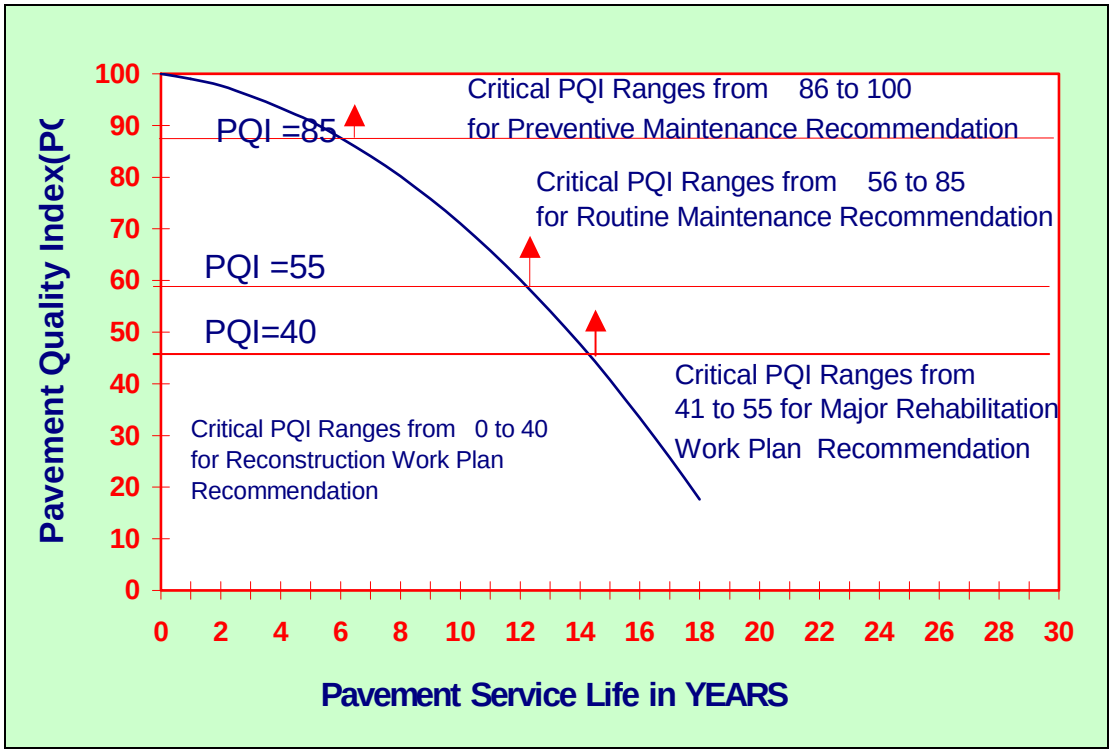


Figure 0.4: Various maintenance categories based on the critical PQI value for major roads.

The above mentioned method is similar to the present serviceability method described hereinbefore where a group of engineers and other individuals

drive over pavement section and decide the riding quality in terms of the present serviceability rating (PSR).

Based on a pre-set intervention trigger level, each section is selected to be maintained under one of the maintenance categories i.e. (PM, RM, MR or RCN). The only difference is that PQI is an objective measuring index calculated based on distress type, severity and quantity.

To optimize the process of the treatment selection, the selection process should always incorporate more than one pavement condition indicators such as distress types and the trigger limits for each distress.

The treatment type is selected according to the intervention or failure limits established for each distress types. As will be verified at later stage in this chapter, the selection process will include other condition indicators such as roughness index, structural capacity index, and skid resistance number designed on a pre-selected scale and based on the intervention levels established for each parameter.

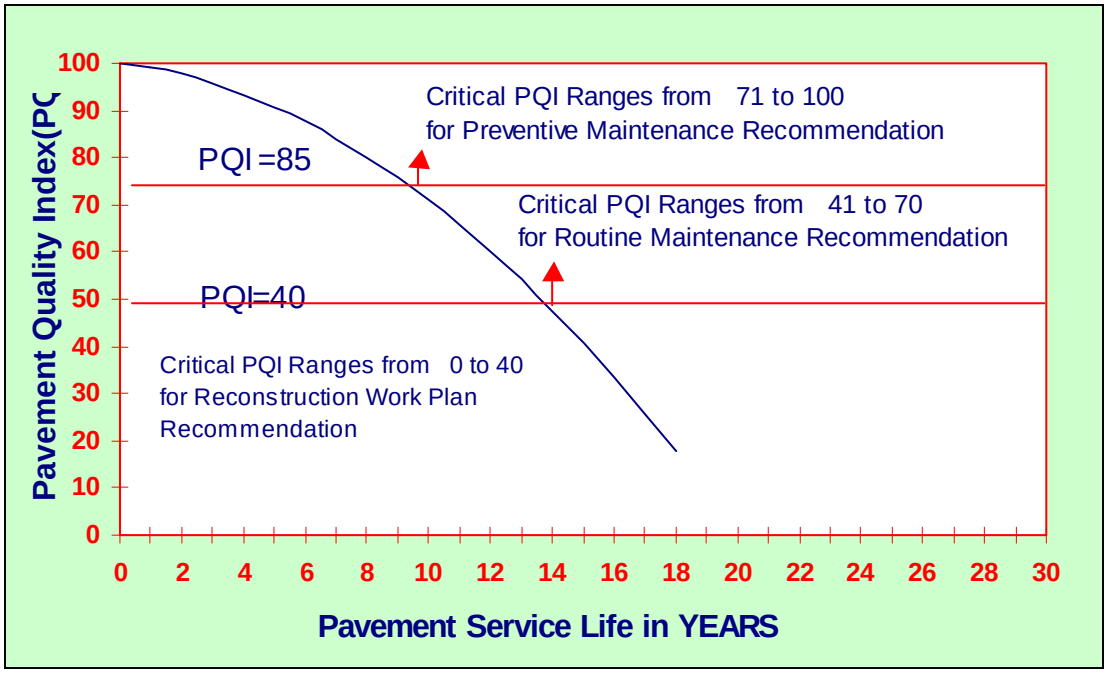


Figure 0.5: Various maintenance categories based on the critical PQI value for local roads.

Using PQI Graphical and distress data based method

The above maintenance and rehabilitation schemes can be applied on a full scale for the pavement section to come over the existing deficiencies in the pavement and to restore either the structural integrity or the functional capacity or both at the same time.

In order to depend on more reliable procedure for selecting the proper treatment, more information about the existing defects should be considered. Therefore, in this method, information about distress type, severity level and density are incorporated.

The majority of the above list of the distress were chosen to be used as a decision variables for the treatment selection process using PQI value and the distress indices trigger limits established for each distress type.

Based on the above detailed information about the distress types and the corresponding repair actions, the optimum treatment selection trees for various road classes can be established as shown in Figures 7.6 and 7.7 below.

The decision trees shown in these figures are dependent on PQI trigger values and the corresponding distress trigger values. In these trees, the main distress types were only included.

Such types could be found in wide areas so that the treatment types selected can be used to rectify the whole entire area upon application. Other distresses such as potholes, slippage cracking, depression, shoving, pumps and sags, rail –road crossings, swell and corrugation occur only in very limited cases and as a localized defected area.

Such distress types can be rectified using the preventive or corrective (routine) maintenance repair actions easily.

In most of PMS, treatment selection process depends solely on distress data if no other condition indicators criterion are available.

Using detailed distress data along with the PQI trigger values will give the decision maker better chance to select more appropriate treatment options that can eliminate the distresses in the pavement completely.

List the intervention levels for each distress types used in the decision tree for treatment selection process are shown in Table 7.13 below.

Table 0.13: List the intervention levels for each distress types used in the decision tree for treatment selection process.

Distress Types	**The recommended intervention levels based on distress index**			
	Level 1	Level 2	Level 3	Level 4
Alligator Cracking	30	60	90	>90
Rutting	30	60	90	>90
Bleeding	60	>60		
Patching	90	>90		
L&T cracking	30	60	90	>90
Shoving	30	60	90	>90
Potholes	30	60	90	>90
Depression	30	60	90	>90
Weathering and Raveling	60	>60		

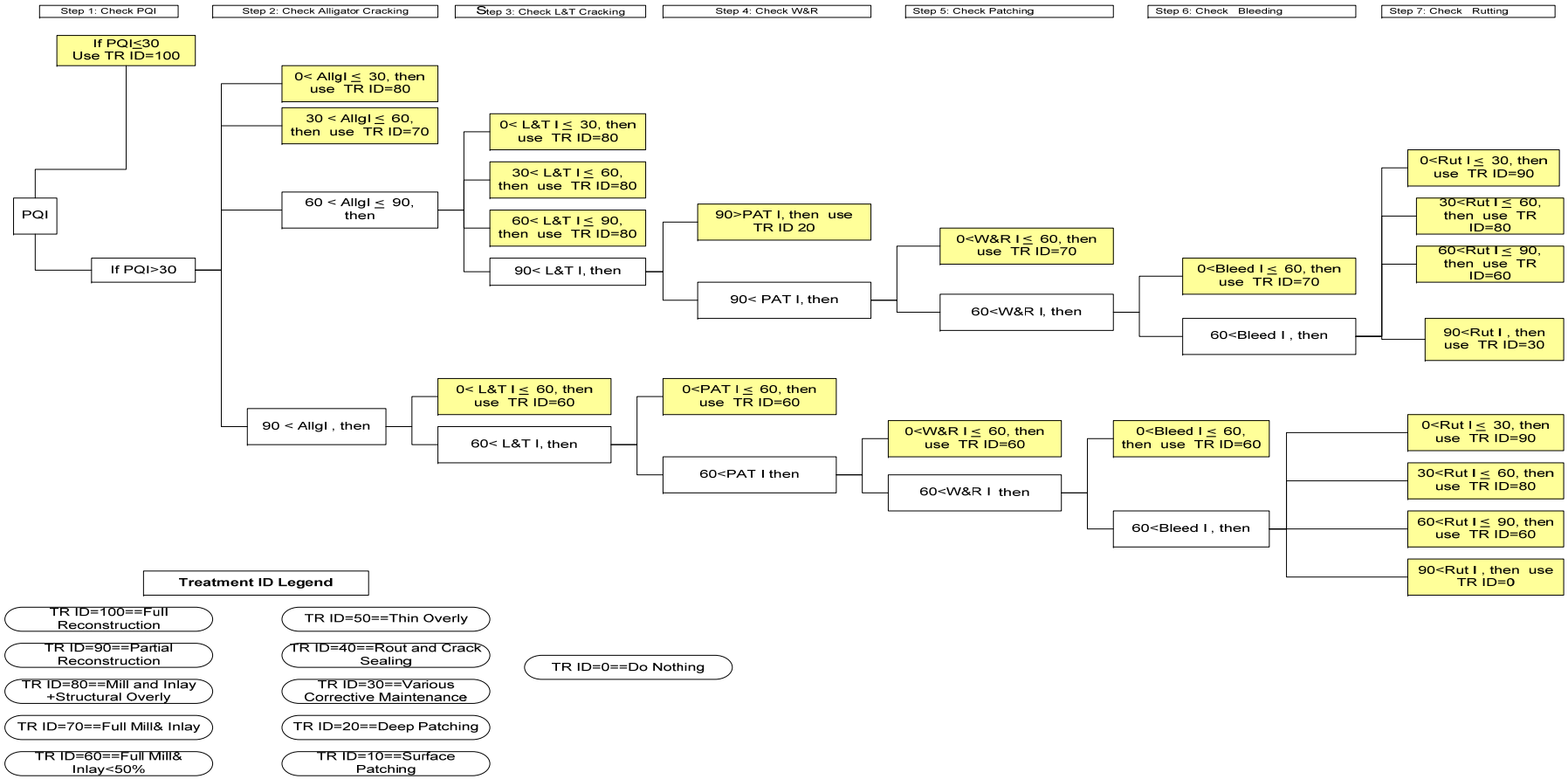


Figure 0.6: Treatment selection tree based on PQI value and the relevant distress data using Alligator (Fatigue) Cracking failure mode.

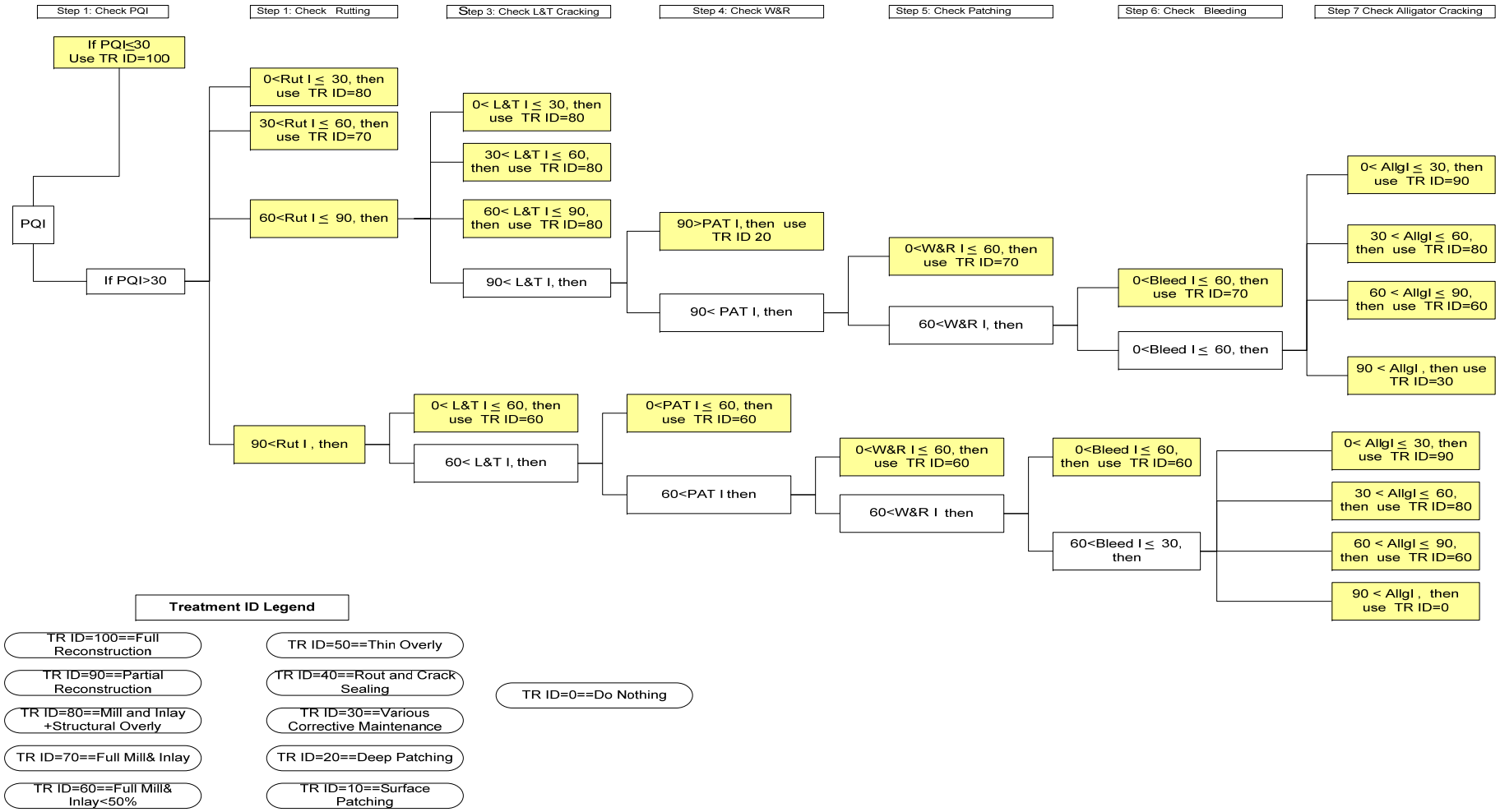


Figure 0.7: Treatment selection tree based on PQI value and the relevant distress data using Rutting (Permanent deformation) failure mode

Using OPQI , distress data, and roughness in the treatment selection process.

Incorporating roughness parameters in the treatment selection process at network level is becoming of great value as it is concerned with the road user satisfaction. The ability to use roughness data as an intervention limit is now possible with the huge advances in the functionality and modeling process of the computerized pavement management systems. Roughness in terms of the International Roughness Index (IRI) values or Remaining Service Life (RSL) in years can be used effectively in the treatment selection process based on intervention values which describes various levels of rideability.

Roughness in terms of IRI as a Decision Making Indicator

The developed models described in the previous chapters can be used to estimate or predict the roughness and the remaining functional life in years based on various independent variables such as Age, Traffic and PSI values. Using these models, it is possible to determine the level of treatment needed for each pavement section. In this study, the decision regarding Maintenance and Rehabilitation (M & R) options based on roughness data was formulated as follows:

- Where the assessed RSL is over 10 years (IRI<1.5), it is unwarranted to apply any treatment to extend the service life.
- Where the assessed RSL is less than 10 years (IRI>1.5), consideration was given to applying a suitable treatment to extend the service life.
- Pavement sections with an existing roughness greater than IRI of 3.4 (major roads) and 4.4 (minor roads) (and thus a PSI value less than 2.5 and 2.0 respectively) were considered of inadequate service level and requiring a major treatment such as milling the surface and/or applying an overlay or partial /full reconstruction.

As previously describe in chapter 6, the values between the initial IRI value and the terminal IRI value can be grouped so that a specific treatment type can be assigned for the each road section based on these limits. The following table is proposed to be used in the PMS treatment selection process for both major and minor roads.

Table 0.14: Intervention criteria and treatment selection trigger values for roughness condition indicator.

Road Class	IRI value	Max PSI value	Estimated Max. RSL (Years)	Treatment ID	Treatment type
Major Roads (Freeways, Expressways, and Arterials)	<1.5	>3.5	>10	0	Do nothing
	1.5<IRI<2.0	3.25	8.5	30	Corrective Maint.
	2.0<IRI<2.5	2.9	5	50	Mill &Inlay<50%
	2.5<IRI<3.0	2.7	2.5	60	Full Mill&Inlay
	3.0<IRI<3.4	2.5	0	70	Full Mill&Inlay
Minor Roads (collectors and local)	<2.0	2.9	>10	0	Do nothing
	2.0<IRI<3.0	2.7	6	30	Corrective Maint.
	3.0<IRI<4.0	2.5	2	50	Mill &Inlay<50%
	>4.0	<2.0	<2.0	60	Full Mill&Inlay

Based on the above intervention criteria and the IRI trigger limits, it proposed that pavement sections with remaining service lives of less than 5 years for both major and minor roads should be given a priority in repair and the proper major treatment should be assigned . Suitable treatments include *an asphalt overlay with or without prior full or partial milling of the surface*. Where the current roughness exceeds terminal IRI value of 3.4 and 4.4 for the major and minor roads respectively, either two layers of asphalt should be placed or the existing surface should be milled prior to the overlay to ensure that the required smoothness or riding quality is achieved.

Sections with remaining service lives between 5 and 10 years should be considered for corrective treatment on limited scale (i.e. short lengths with noticeable high roughness level) to reduce roughness if they are not in need for a treatment for other reasons (rutting or fatigue cracking). Otherwise, it is considered economically sound to postpone treating such areas for up to 5 years. Sections with remaining service lives over 10 years (including those with remaining service lives between 10 years and 15 years) are not in need for a treatment to reduce roughness.

Figure 7.8 shows the modified decision tree after incorporating the roughness parameters into the treatment selection process. In order to reduce the time of analysis and number of variable included in the selection process, few types of distresses were removed from the decision tree since the treatment selected based on roughness will handle the repair needs for these types. Therefore, Patching, Weathering and Raveling and Bleeding were removed and replaced by roughness parameters.

Using OPQI , distress, roughness and structural data in the treatment selection process.

Incorporating the structural data into the treatment selection process at network level will necessitate an establishment of well designed intervention levels for the major rehabilitation treatment based on the structural capacity parameters as well as other parameters pertain to the various condition indicators.

The most crucial step in incorporating the structural data is the wise selection of the parameters that can provide real assessment for the pavement structural capacity. In this regard, the structural capacity of the pavement section can be achieved through using either the measured and back calculated parameters such as *Deflection*, *Curvature*, or the *Remaining Structural Life (RSL)* in terms of traffic loading represented by the ESAL repetitions or the number of the years to complete pavement failure.

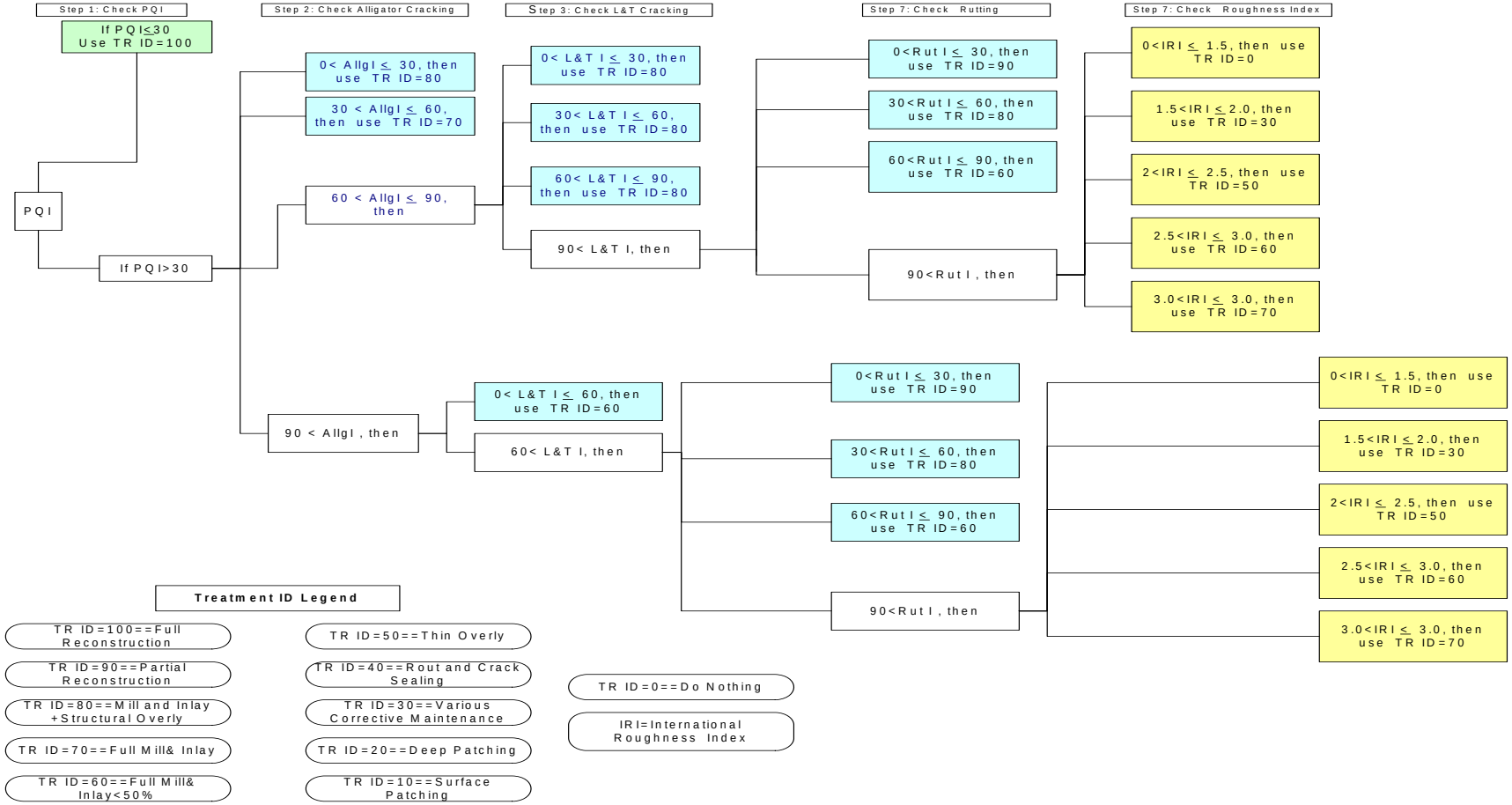


Figure 0.8: The modified decision tree after incorporating the roughness parameters into the treatment selection process.

Adopting one or more of the above mentioned parameters is the task of the PMS engineers who can assess the reliability of the information given by each parameter on the treatment selection process and its effect on the decision making process , maintenance and rehabilitation master work plan and the required budget.

Selecting the deflection measured at the center of the loading plate or any other deflection value is successful at project level but it will not give an accurate estimation for the structural capacity of the pavement *at network level* as discussed and verified in the previous chapters. This is because two pavement section may have the same deflection value but different structural capacity as shown in figure 4.30 which shows the relationship between the measured deflection value and the traffic level in terms of ESAL. Therefore, other parameter should be investigated.

Remaining life parameter in terms of the remaining ESAL repetitions or years to failure can be used at network level as it will give an acceptable assessment for the effective structural capacity and the remaining life at network level.

In addition to the remaining life value, The changes in the value of the asphalt modulus can be used effectively as a structural failure criteria as discussed and verified in chapter 4 (Figures 4.32-4.35), the reduction in asphalt modulus is a function of the applied traffic loading represented by the value of past ESAL. The value of the Ep_{eff} or the relative value of Ep_{eff} to the original modulus value (Ep_{eff}/Epo) decreases linearly with the number of ESAL up to the value of 15-20 millions ESAL. Then it decreases exponentially at the remaining potion of the figure until it reaches the complete failure at the end of pavement service life.

As indicated in chapter 4, the analysis of the collected data for the cracked portions for many pavement sections tested by FWD has indicated that when

the cracking becomes observable or visible on the pavement surface (i.e. the severity ranges from low to high), the effective asphalt or pavement modulus was found to be about 20 - 35 %of its original design value. This means that the pavement structure has already lost around 65 to 80% of its effective cross sectional area. *This value was chosen as a structural failure criteria to be used in treatment selection procedure as shown below.*

Based on the above, it is decided in this study to use both the Remaining Structural Life (RSL) and Structural Capacity Index (SCI) , which is estimated based on the Ep_{eff} value for the pavement, as an intervention parameters for treatment selection process. In this regard, similar to other condition indicators such as PQI, roughness, trigger values for these structural parameters should be established. Table 7.4 below shows the trigger values suggested for treatment selection process using structural data in terms of SCI and RSL.

Table 0.15: The trigger values suggested for treatment selection process using structural data.

Ep_{eff}/Epo	**SCI**	**Remaining structural life (years)**	**Treatment ID**	**Treatment type**
>0.9	>90	>15	30	Corrective Maintenance
0.6-0.9	60-90	10-15	60	Mill &Inlay<50%
0.3-0.6	30-60	5-10	80	Full Mill&Inlay+ Overlay
<0.30	<30	<5	90	Reconstruction

Using OPQI , distress, roughness, structural l and skid resistance data in the treatment selection process.

As indicated in chapter 6, the effective value of the Skid Resistance Number (SKN) or any other skid resistance indicators such as, Friction factor ,

Texture Depth (TD) can be used In PMS to report about the skid resistance condition. The incorporation of this indicator will give sufficient information about the functional or safety performance trend of the pavement under consideration. Similar to the other pavement condition indicators, skid resistance is deteriorating with time under the effect of both traffic and environmental effects. Each agency should establish intervention and terminal values for the skid resistance level below which, a repair action has to be applied. Under traffic application, the macro, micro and mega texture is deteriorating and the safety hazards and the potentiality for accidents is highly increased. The microtexture of the aggregate that constitute the pavement surface mix is significantly damaged as a result of the sun, heat and sand deposits under the effect of traffic loading. The trigger values for skid resistance condition indicator has been established in this study to assess the effective pavement condition at any time. The measurements are easy to conduct and straight forward. The levels of condition indicators are listed in table 6.3 and re grouped in different format as shown in the following table.

Table 0.16 : The trigger values suggested for treatment selection process using Skid Resistance data.

Longitudinal Friction Factor	**Skid Resistance Number at speed 64 km/h**	**Repair ID**	**Treatment Description**
>0.75	75	0	Do Nothing
0.55– 0.75	65 to 75	30	Do Nothing, remove contaminants
0.40 –0.55	40 to 55	60	Surface treatment, (Milling),Grooving
<0.40 (Terminal value)	<40	70	Resurfacing, Full Mill and Inlay

Skid resistance change is expected to depend mainly on the mean texture depth, the initial skid resistance value and the traffic loading. As shown in the table above, it is estimated that any pavement section of friction factor of

0.40 (i.e. Skid Number SN=40)or less will be failed from frictional point of view and an immediate repair action should be undertaken.

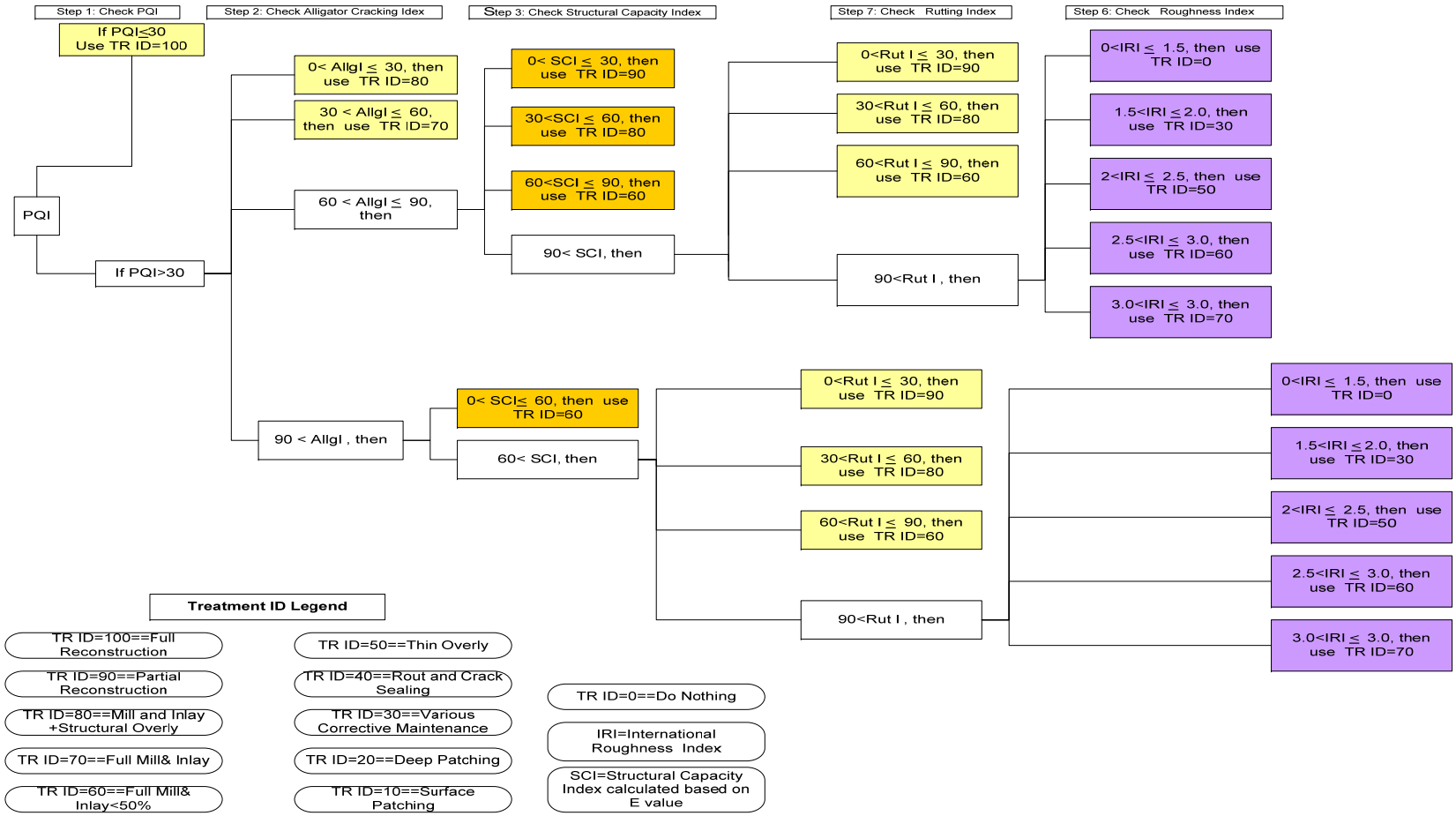


Figure 0.9 : The modified decision tree after incorporating the roughness and deflection parameters into the treatment selection process.

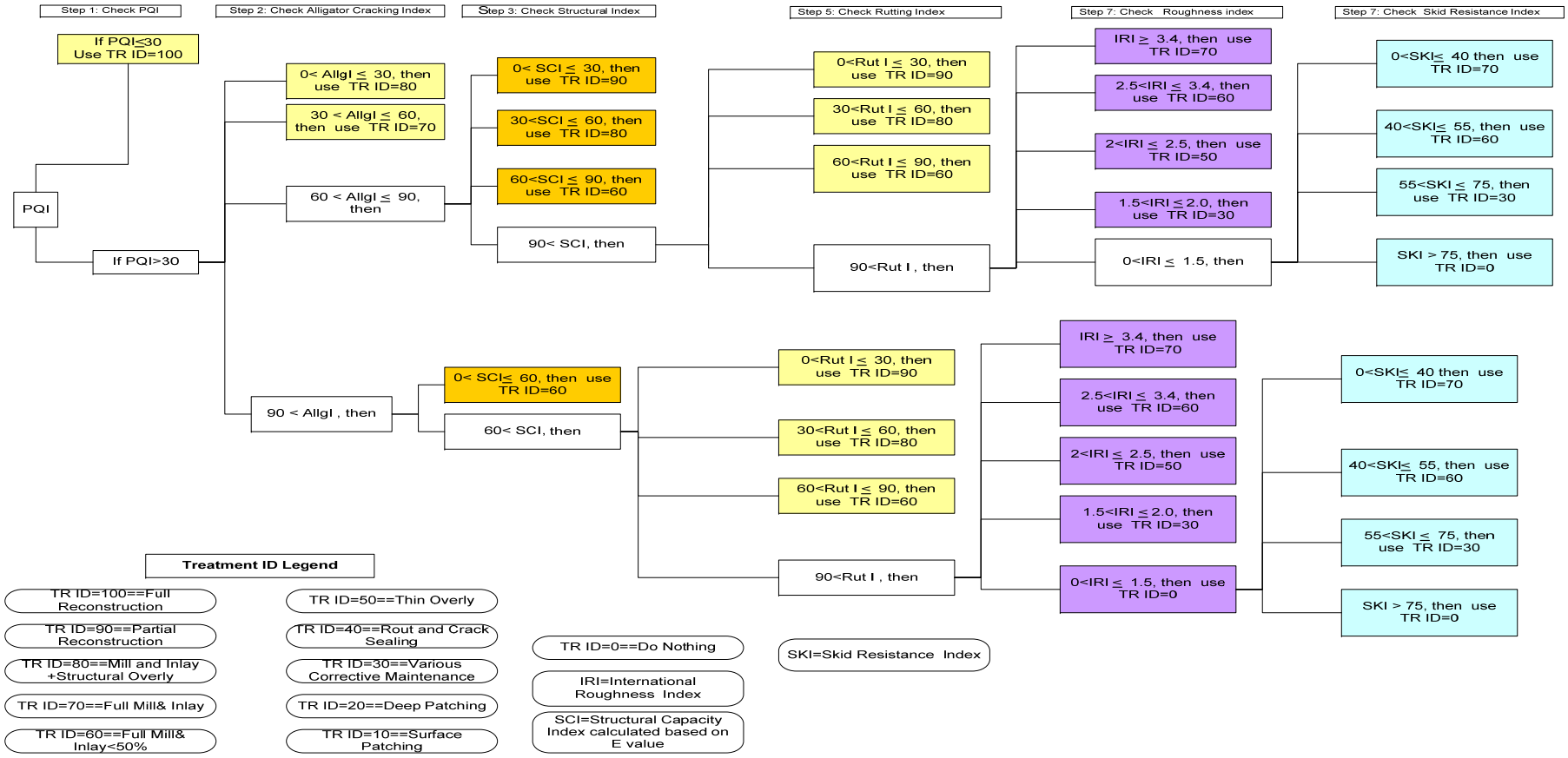


Figure 0.10: The modified decision tree after incorporating the roughness, deflection and skid resistance parameters into the treatment selection process (Alternative 1).

General Guidelines for determining the rehabilitation treatment

In the recently developed pavement management systems, the rehabilitation treatment options are considered from different perspectives as illustrated in Figures 7.6 to 7.11 and discussed in the following paragraphs:

Functional Service Adequacy

One of the main objectives for all roads agencies around the world is to give an acceptable serviceability to road users. Thus, sections with Present Serviceability Index (PSI) below 2.5 for major roads and 2.0 for minor roads (the corresponding terminal IRI is equal to 3.4 and 4.4 for major and minor roads respectively) should be treated to bring the PSI to a value above 2.5. Typically this would be through either an asphalt overlay, Mill and inlay, or, not very frequent, partial reconstruction . If there are no major structural defects, there are no other conditions on the road section requiring treatment, the overlay/ Mill and Inlay would be a nominal minimum thickness of 40 to 50mm.

Functional Remaining service life (RSL)

The remaining service life is the assessed time (in years) that it will take for the road section to reach the terminal PSI value of 2.5 for major roads and 2.0 for minor roads (the corresponding terminal IRI is equal to 3.4 and 4.4 for major and minor roads respectively). RSL can be also estimated using IRI-Age relationships developed based on performance data under various level of traffic. If this is short (i.e. set under 5 years), then consideration of a treatment to extend the remaining service life is warranted. Typically this would be an asphalt overlay Mill and inlay, or, not very frequent, partial reconstruction. Provided there are no other conditions on the road section requiring treatment, the overlay Mill and Inlay would be a nominal minimum thickness of 40-50mm. If the remaining service life is over 5 years, the service life is not considered as a primary justification for considering a treatment now. However, other conditions may warrant a treatment on such sections.

Roughness

The roughness is a primary criterion for determining the desirability of treating a road section as it is directly related to PSI level of the road. It can be used as a replacement for the PSI as it can be objectively measured. However, roughness affects the appropriateness of treatment options. If the IRI is greater than terminal IRI of 3.4 and 4.4 for major and minor roads respectively) then a nominal minimum 40-50mm overlay/mill and inlay will have difficulty producing the required roughness of below IRI 3.4 or 4.4 after treatment. In this case, for major roads, sections with a roughness over IRI 3.4, thicker or multi-layer overlays or full mill and inlay are required.

Rutting

The rut depth affects safety and indicates whether the pavement structure is giving adequate cover to the subgrade. If the rut depth is excessive, then consideration of a treatment to reduce the rut depth is warranted. Typically, this would be an asphalt overlay , mill and inlay, partial reconstruction or full reconstruction. Provided there are no other conditions on the road section requiring treatment, the overlay thickness is estimated so that it can reduce the rut depths after treatment to less than 2mm.

If the rut depth is greater than 10mm then a nominal minimum 40-50mm overlay/mill and inlay will have difficulty reducing the rut depth to below 2mm after treatment. In this case, for sections with rut depth greater 25mm, a two layer overlay or milling smooth prior to overlay is required to ensure there should be no difficulty producing the required rut depth after treatment. This would normally comprise a regulation or leveling course of sufficient depth to fill the ruts plus a surface course. For ruts of about 30mm to 45mm the asphalt overlay could comprise a nominal 60mm regulating course and a 40mm surface course.

Instead of a two-layer overlay, the existing surface could be milled smooth prior to placing an asphalt overlay. If the milling is controlled to produce the

required surface profile, then a 40-50mm minimum thickness overlay could be used.

Remaining structural life

The remaining structural life is the assessed time (in years) that it will take for the road section to reach the end of the fatigue life of the asphalt surface and/or the end of the deformation life of the subgrade. It can be also estimated using other various methods and approaches as described in chapter 6 of this study. Structural Remaining Life (SRL) can be estimated also based on traffic expected to the end of service life, or the deterioration ratio of the SN or E value in relative to the original design values. RSL can be also estimated using PQI-Age relationships developed based on performance data under various level of traffic.

If this is short (set in this Study at under 5 years), then consideration of a treatment to extend the remaining structural life is warranted. If the remaining structural life is over 5 years, it is considered sufficiently distant to allow consideration of deferment of treatments. Then structural life is not considered as a primary justification for considering a treatment now.

If the existing asphalt is sound with no cracks, the most appropriate treatment to extend the structural life is an asphalt overlay unless other restrictions exist. The thickness of overlay required is dependant on the structural life to be provided. That is normally set as 15 or 20 years.

If the existing pavement structure is cracked then it will be only a few years after an asphalt overlay before reflective cracks occur in the overlay. The number of years is dependant on the crack width and overlay thickness. For thin overlays (40mm and 50mm) this is usually 2 year to 5 years. For thicker overlays the time can be extended to some 5 to 7 years.

The preferred option for cracked pavements requiring a treatment is to remove the cracked asphalt, replace it with sound asphalt, and apply an

asphalt overlay of adequate thickness to give the required service and structural life. Figure 7.11 and Table 7.17 below summarize another selection procedure for recommended treatment options based on various pavement condition indicators and combinations .

Table 0.17: Selection procedure (Alternative 2) for the recommended treatment options (Refer to Figure 7-11):

IF	**THEN**
A + E + G	treatment required for next 5 years
A + E + F B + E + F	Overlay with asphalt
A + D + G	Patch and crack seal, consider surface treatment
A + D + F B + D + F	Patch and crack seal, then asphalt overlay
A + C + G B + C + G	Mill of rutted asphalt layer, then replace with same thickness of asphalt
A+C+F	Mill of rutted asphalt layer and inlay the same thickness and then add the required structural overlay thickness.
B + E + G	Mill smooth and apply thin surface treatment
B + D + G	Patch and crack seal, mill smooth and apply thin surface treatment

Rules for selecting the optimum treatment option

This following sub-Section discusses the rules and the guidelines used for selecting various pavement maintenance and rehabilitation options which may be considered suitable for rehabilitating pavement section of certain condition.

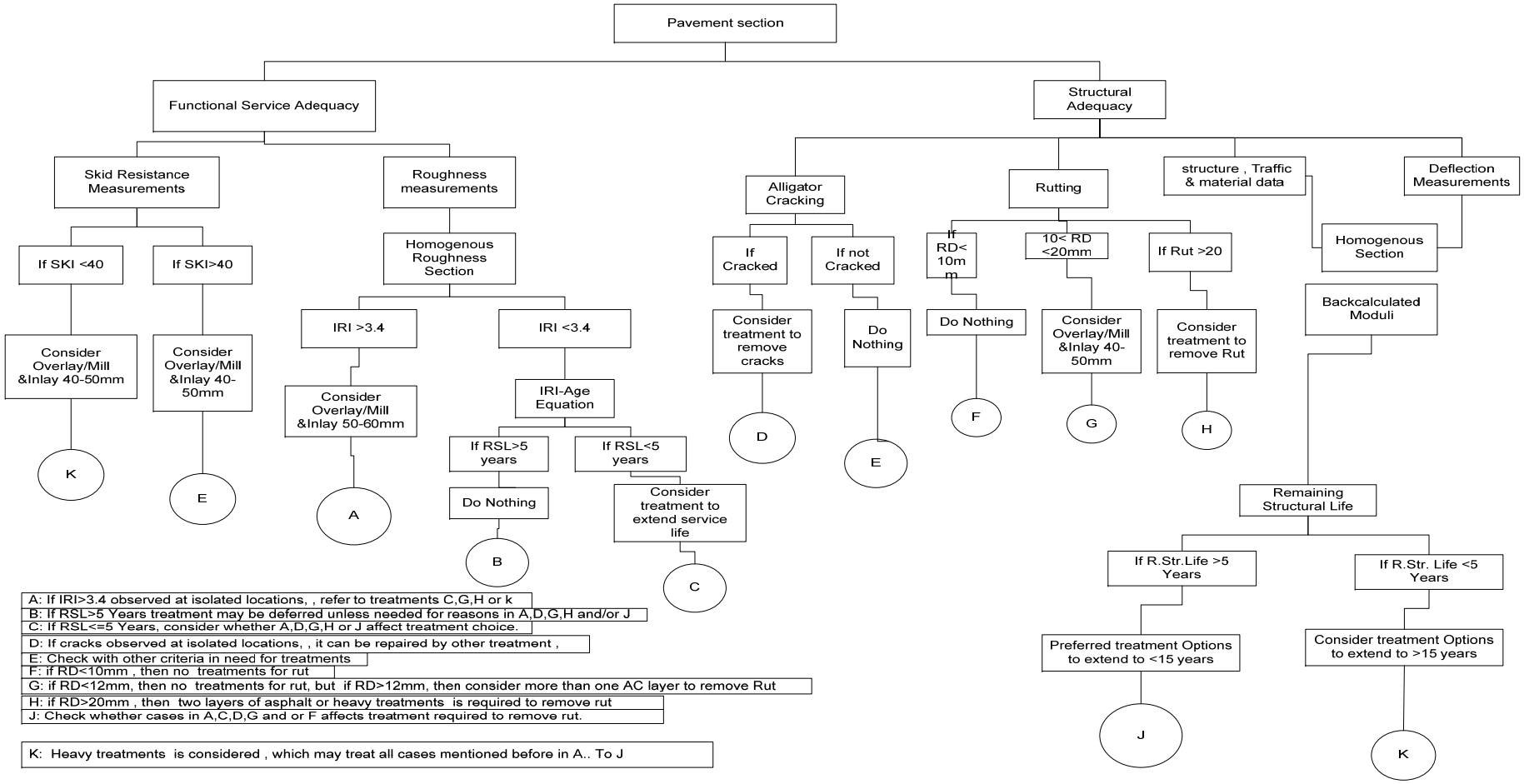


Figure 0.11 : An alternate decision tree based on alligator cracking, rutting , roughness, deflection and skid resistance parameters for treatment selection process (Alternative 2).

Overlay with asphalt

An asphalt overlay can be used to restore shape, protect the surface and add strength to give an extended structural life. An overlay is suitable where the existing asphalt *is not cracked*. If an overlay is placed on a cracked pavement, the cracks will reflect through in a relatively short time. For thin overlays (40mm to 50mm) this is usually 1 year to 5 years depending on the width of the cracks. In cases where milling is deemed to be difficult to be carried out, due to the unavailability of the proper machines and milling tools ,or placing on cracked PCC slabs, Stress relieving inter-layers (geotextile or SAMI layers) can be incorporated in the overlay to delay the reflective cracking. However, geotextile are some times difficult to lay and if not laid correctly result in premature failure. SAMI under thin asphalt layers can reduce the horizontal shear strength between asphalt layers allowing the surface to slide.

Thus, overlay with asphalt on traffic lanes is usually proposed on un-cracked surfaces requiring rut depth or roughness reduction, surface shape changes and/or pavement strengthening. On pavement sections with only isolated areas of cracking, the cracked areas can be patched first and then the patched area overlaid with asphalt.

Mill out existing asphalt and replace with asphalt

Where the asphalt surface is cracked, due to oxidation, ultra-violet light and traffic, an asphalt overlay is not recommended. Rather, to obtain a service life of 15 years, the cracked asphalt should be rejuvenated and/or replaced. Then any pavement strengthening required should be by additional asphalt placed on the rejuvenated and/or replaced asphalt. The recommended rehabilitation procedure in these cases is as follows:

- **Cracked asphalt surfaced pavements**

  Where the cracking is narrow and does not penetrate the full depth of the asphalt - rejuvenate and/or replace the asphalt to a depth below the cracking. Typically this should be to a depth of about 60mm to 90mm. This depth will leave a working layer of asphalt and protect the underlying base. Where there is wide cracking (over 2mm to 3mm), remove the asphalt to the full depth and replace

with new asphalt. As necessary, overlay the rejuvenated and/or replaced asphalt with an asphalt surface layer to give the required pavement structure.

- **For un-cracked pavement sections**
  Where the existing shape requires the surface to be reshaped (due to ruts and overlays) and an overlay is unsuitable, then the asphalt should be rejuvenated and/or replaced to a depth sufficient to remove the out of shape and/or unsuitable asphalt. Either of the preceding methods may be suitable.

Hot In-place Asphalt Recycling –HIPAR

Hot in-place recycling is a process where the asphalt pavement surface is heated in place, scarified, re-mixed, re-laid and rolled. New binder, binder rejuvenating agents, additional new asphalt, new aggregates or combinations of these may be added to obtain desirable asphalt characteristics and/or to produce a thicker asphalt layer.

Typically the HIPAR equipment can remove up to 60mm in one pass. For recycling thickness greater than 60mm, the surface is first milled off to the depth required less 60mm. The milled material may be recycled through the asphalt mixing plant.

Hot in-place recycling rectifies surface course asphalt material that is distressed due to oxidation, ultra violet light, errors in aggregate or asphalt proportioning in the original mix and restores the surface shape. It also removes all cracks. In most cases, the resulting asphalt is adequate for light to medium traffic loads. However, for heavily loaded sections, it is advisable to place a new surface layer over the recycled material.

Adopting the above proposed comprehensive criteria and limits to select the required treatment option could provide enough structural capacity, improve riding quality and at the same time enhance the safety measures on the rehabilitated roads.

COMPUTERIZED APPLICATION OF THE DEVELOPED PERFORMANCE INDEX

OPQI Calculation

In many developed and developing countries, the pavement maintenance needs are determined based on the results of visual inspection , and very often , the decision to perform certain types of maintenance is based on a single condition index such as pavement condition index, which in most of the cases does not give an adequate representation of pavement condition. As described in details in the previous chapters, the developed Overall Pavement Quality Index (OPQI) can be used effectively to report about the pavement condition and to determine the required maintenance and rehabilitation needs with relatively high accuracy at both network and to a certain extent at project level.

The use of such an index which include all condition indicators indices or the use of the other individual indices (PQI, IRI, SCI, and SKN can take various forms and combinations. The optimum use for the overall and the individual indices can be best decided and judged by the maintenance planning engineer.

In general, pavement engineers and maintenance managers appreciate the considerable value and benefits gained from implementing an effective PMS in managing the roads network maintenance activities and setting annual work programs based on both actual needs and priorities.

Quantifying the benefits of using such a new index in the new PMS can be understood or appreciated by the pavement maintenance engineers in particular when it comes to the treatment selection tasks. Selecting the right pavement section at right time to be treated using the right treatment option is the optimum goal for the maintenance engineer in order to maximize the benefits gained from spending the allocated maintenance budget in proper manner.

In this chapter, the researcher will also examine how an alteration in the performance condition indicators and trigger levels affects the evaluation results of the pavement network performance, maintenance programming and network investment decision taking process.

The subsequent sections and paragraphs detail the proposed implementation for the newly developed overall pavement index and the other condition indicators indices in PMS for the purpose of developing various budgeting scenarios and the optimum determination for the maintenance and rehabilitation needs at network level.

Overall Pavement Quality Index (OPQI) Set Up

Based on the experience, and for the purpose of simplifying the calculation procedure. Distress data, in addition to the data pertaining to the other condition indicators, were normalized using linear conversion manner to a number between 0 and 100 over the ranges listed below:

Distress data Ranges and normalization: Different distresses were normalized on a scale between 0-100. The range on which the normalization was applied are shown in the Table 8.1 below:

- The above ranges indicate the limits beyond which the distress area and severity has no additional effect on the overall condition rating and the deduct value. These limits depends mainly on the distress type and maintenance policies.

- International Roughness Index (IRI)= 0.9-12 : This range is selected based on the IRI scale established by the world bank (see Figure 4.18, Chapter 4). Practically, all roads of IRI beyond 5 are considered impassable and worthy of major rehabilitation or even reconstruction.

- Effective Structural Number (SN_{eff}) =1.8- 7.5 ; SN=1.8 applies for minimum structure strength for local roads where SN=7.5 applies for maximum structure strength for freeways roads.

Table 0.1: Distress data Ranges and normalization

Distress	Unit	Range
Rutting	Area	0-25%
Alligator Cracking	Area	0-25%
Bleeding	Area	0-50%
Patching	Area	0-25%
Potholes	Area	0-1%
Depression	Area	0-1%
Longitudinal and Transverse Cracking	Area	0-25%
Weathering and raveling	Area	0-50%
Shoving	Area	0-1%

- Skid Resistance (Friction factor)= 0.28-0.75. the limit o.75 applied for the friction level for the newly constructed pavement while the limit of 0.28 applied for the roads of slippery and polished aggregate surface.

The general formula proposed for calculating OPQI was mentioned in chapter
5 and it is repeated here for easy referencing:

$$OPQI_k = 100\sum_{i=1}^{i=n}\left[1-\left(1-\frac{CI_i}{100}\right)*W_{i,k}\right] \qquad 8.1$$

Substituting each part developed for each condition indicator will yield the general equation that encompasses the following terms:

$$OPQI = 100\left[1-\left(1-\frac{\mathrm{RI}}{100}\right)*1\right]*\left[1-\left(1-\frac{\mathrm{ACI}}{100}\right)*1\right]*\left[1-\left(1-\frac{\mathrm{BI}_i}{100}\right)*0.40\right]*$$
$$\left[1-\left(1-\frac{\mathrm{PCHI}}{100}\right)*0.65\right]*\left[1-\left(1-\frac{\mathrm{PTI}}{100}\right)*0.10\right]*\left[1-\left(1-\frac{\mathrm{DEPI}}{100}\right)*0.40\right]*$$
$$\left[1-\left(1-\frac{\mathrm{LTI}}{100}\right)*0.30\right]*\left[1-\left(1-\frac{\mathrm{WRI}}{100}\right)*0.40\right]*\left[1-\left(1-\frac{\mathrm{SHI}_i}{100}\right)*0.10\right]*$$
$$\left[1-\left(1-\frac{ARI_i}{100}\right)*WT_{Roughness}\right]*\left[1-\left(1-\frac{ESCI}{100}\right)*WT_{Structure}\right]*$$
$$\left[1-\left(1-\frac{SKRI}{100}\right)*WT_{Skid-\mathrm{Resistance}}\right] \qquad 8.2$$

Where:

OPQI: Overall Pavement Quality Index (selected to be on a scale from 0-100).

CI: Condition Indicator Index (also selected to be on scale from 0-100). CI includes the followings:

Rutting Index, (RI)

Alligator Cracking Index (ACI)

Bleeding Index (BI)

Patching Index (PCHI)

Potholes Index (PTI)
Depression Index (DEPI)
Longitudinal and Transverse Cracking Index (LTI)
Weathering and raveling index (WRI)
Shoving Index (SHI)
Average International Roughness Index (AIRI)
Effective Structural Capacity Index (ESCI)
Skid Resistance Index (SKRI)

K= K^{th} Pavement Performance Index.
i= i^{th} Distress or condition indicator out of the total number of the "n distresses or condition indicators
n= Total number of distress types or condition indicators included in the performance index.
Wt i, k=The impact or the relative weight of each distress type or condition indicator determined based on engineering judgment.

The combinations detailed in chapter 5 will be shown in the outputs of the computerized application of the developed index as described below:

$$\mathbf{OPQI_4 = \{(PQI\text{-}_{Distress})\ (Wt_{14})\} + \{(PQI\text{-}_{Roughness})*(Wt_{24})\} + \{(PQI\text{-}_{Structure})\ (Wt_{34})\} + \{(PQI\text{-}_{Skid})*(Wt_{44})\}} \quad 8.3$$

$$\mathbf{OPQI_3 = \{(PQI\text{-}_{Distress})\ (Wt_{13})\} + \{(PQI\text{-}_{Roughness})*(Wt_{23})\} + \{(PQI\text{-}_{Structure})\ (Wt_{33})\}} \quad 8.4$$

$$\mathbf{OPQI_2 = \{(PQI\text{-}_{Distress})\ (Wt_{12})\} + \{(PQI\text{-}_{Roughness})*(Wt_{22})\}} \quad 8.5$$

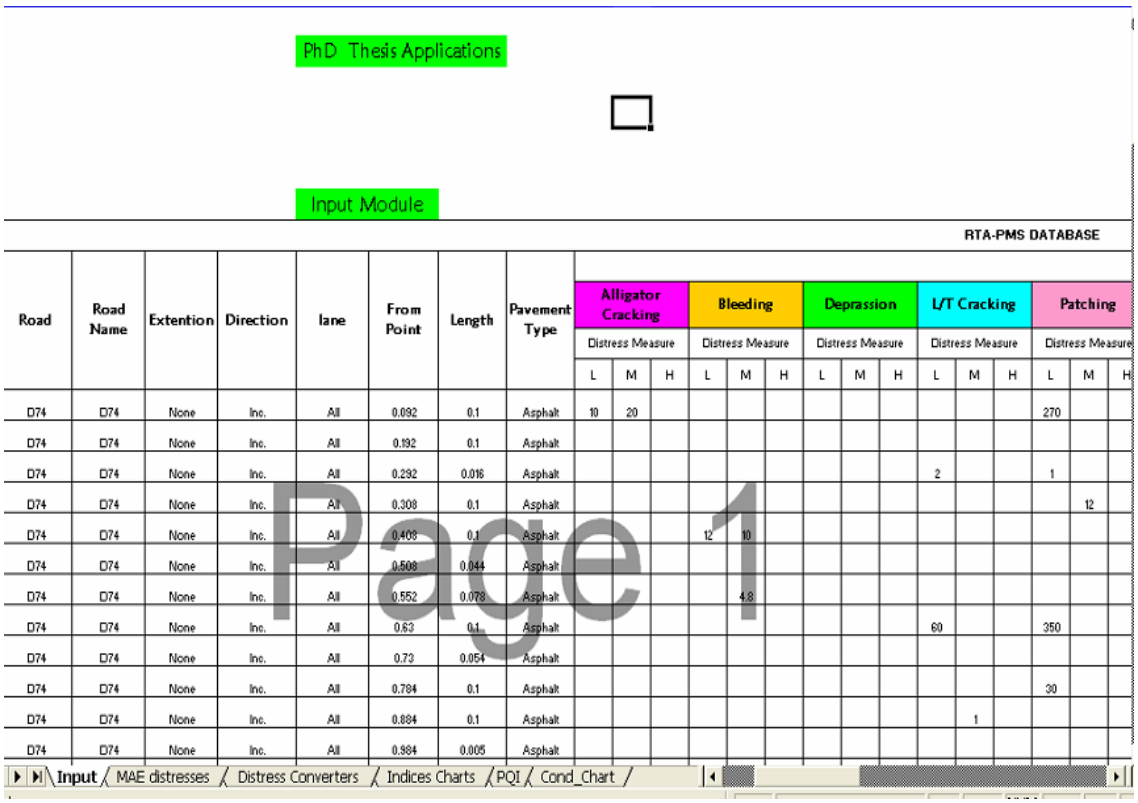
$$\mathbf{OPQI_1 = \{(PQI\text{-}_{Distress})\ (Wt_{11})\}} \quad 8.6$$

Since further modifications for the existing Dubai PMS was found to take long time due to the contractual complications, a Spreadsheet program was designed using Excel Visual Basic to show the effect of incorporating the

structural capacity index and the skid resistance index on the network budgeting and the optimum treatment selection process. The spread sheet designed for this purpose encompasses the following modules:

1. **Raw database *Input* module**: It includes all the raw data pertaining to each pavement section such as;
   - Inventory data such as Section ID's , From , To , Road Name, length, width, area, Functional Class etc.
   - Distress quantities at various severity levels (Low, medium, High).
   - Year of survey
   - Additional data related to the roughness, Deflection and skid resistance of each pavement section.

The input module designed for PQI/OPQI calculations is shown in Figure 8.1 below.



PhD Thesis Applications

Input Module

RTA-PMS DATABASE

Road	Road Name	Extention	Direction	lane	From Point	Length	Pavement Type	Alligator Cracking			Bleeding			Depression			L/T Cracking			Patching		
								Distress Measure			Distress Measure			Distress Measure			Distress Measure			Distress Measure		
								L	M	H	L	M	H	L	M	H	L	M	H	L	M	H
D74	D74	None	Inc.	All	0.092	0.1	Asphalt	10	20											270		
D74	D74	None	Inc.	All	0.192	0.1	Asphalt															
D74	D74	None	Inc.	All	0.292	0.016	Asphalt										2			1		
D74	D74	None	Inc.	All	0.308	0.1	Asphalt														12	
D74	D74	None	Inc.	All	0.408	0.1	Asphalt				12	10										
D74	D74	None	Inc.	All	0.508	0.044	Asphalt															
D74	D74	None	Inc.	All	0.552	0.078	Asphalt					4.8										
D74	D74	None	Inc.	All	0.63	0.1	Asphalt										60			350		
D74	D74	None	Inc.	All	0.73	0.054	Asphalt															
D74	D74	None	Inc.	All	0.784	0.1	Asphalt													30		
D74	D74	None	Inc.	All	0.884	0.1	Asphalt											1				
D74	D74	None	Inc.	All	0.984	0.005	Asphalt															

Input / MAE distresses / Distress Converters / Indices Charts / PQI / Cond_Chart /

Figure 0.1: Input Module used in the PQI/ OPQI calculation program

2. **Set Up Module**: This includes the set up parameters and conversion rules adopted to convert the raw distress data into distress density on scales from 0-100. This part includes distress conversion approaches which are briefed below (Figures 8.2 and 8.3):

- ***Maximum Allowable Extent***

  This approach is used to convert distresses of two dimension (Type and Severity) into distress index on a scale from 0-100. this module include also the relevant weight that represent the impact of the distress on the pavement condition.

- ***Distress Converter***:

  This method is used to convert distress or measurement of one dimension (single number on different scales), into an index on a scale from 0-100.

Distress Indices Formula - MAE

	Distress Type	Weight Factor	Severity level	Low Range Value	High Range Value	Minimum Score	Possible Causes			
							Load	Climate / Durability	Moisture / Drainage	Other
1	Alligator Cracking	1	Low	0	50	20	1			
			Medium	0	25	15				
			High	0	15	10				
2	Bleeding	0.4	Low	10	100	70		1		
			Medium	6	70	40				
			High	0	70	15				
3	Deprassion	0.5	Low	1	40	50				1
			Medium	0	20	40				
			High	0	10	25				
4	L/T Cracking	0.8	Low	5	25	70				1
			Medium	2	20	50				
			High	0	10	25				
5	Patching	0.65	Low	5	50	65				1
			Medium	0	15	20				
			High	0	10	10				
6	Pothholes	0.3	Low	0	1	50	1			
			Medium	0	1	20				
			High	0	0.5	0				
			Low	0	30	60				

Input | MAE distresses | Distress Converters | Indices Charts | PQI | Cond_Chart

Figure 0.2: Maximum Allowable Extent Module used in the PQI/ OPQI calculation program

Distress Indices Formula - Converters

designned By: **Adnan M S H Shiyab** | **PhD Thesis Applications** | Date 6/9/2007 19:21

No.	Distress	Weight		Distress Conversion Curve							Possible Causes			
				point 1	point 2	point 3	point 4	point 5	point 6	point 7	Load	Climate / Durability	Moisture / Drainage	Other
11	IRI	0.25	Distress Measure (IRI)	0	0.9	1.2	1.5	2.5	3.5	5	1			
			Index (ARI)	100	90	82	78	65	54	38				
12	Skid Resistance	0.25	Distress Measure (Mu)	0.2	0.3	0.4	0.5	0.6	0.7	1				1
			Index	20	30	40	50	60	70	100				
13	Structural Index	0.75	Distress Measure	0.4	0.5	0.6	0.7	0.8	0.9	1				
			Index	40	50	60	70	80	90	100				
14	Distress 14	0	Distress Measure	0	2	4	6	8	10	12				
			Index	100	55	35	25	15	8	3				
15	Distress 15	0	Distress Measure	0	2	4	6	8	10	12				
			Index	100	55	35	25	15	8	3				
16	Distress 16	0	Distress Measure	0	2	4	6	8	10	12				
			Index	100	55	35	25	15	8	3				

point 1 to point 7 difines multi piecewise linear transformation curve (Distress Conversion Curve)

Input / MAE distresses \ Distress Converters / Indices Charts / PQI / Cond_Chart /

Figure 0.3: Distress converter Module used in the PQI/ OPQI calculation program

- ***Indices Chart***:

  This module includes the charts of the deduct values for each distress at three severity levels.

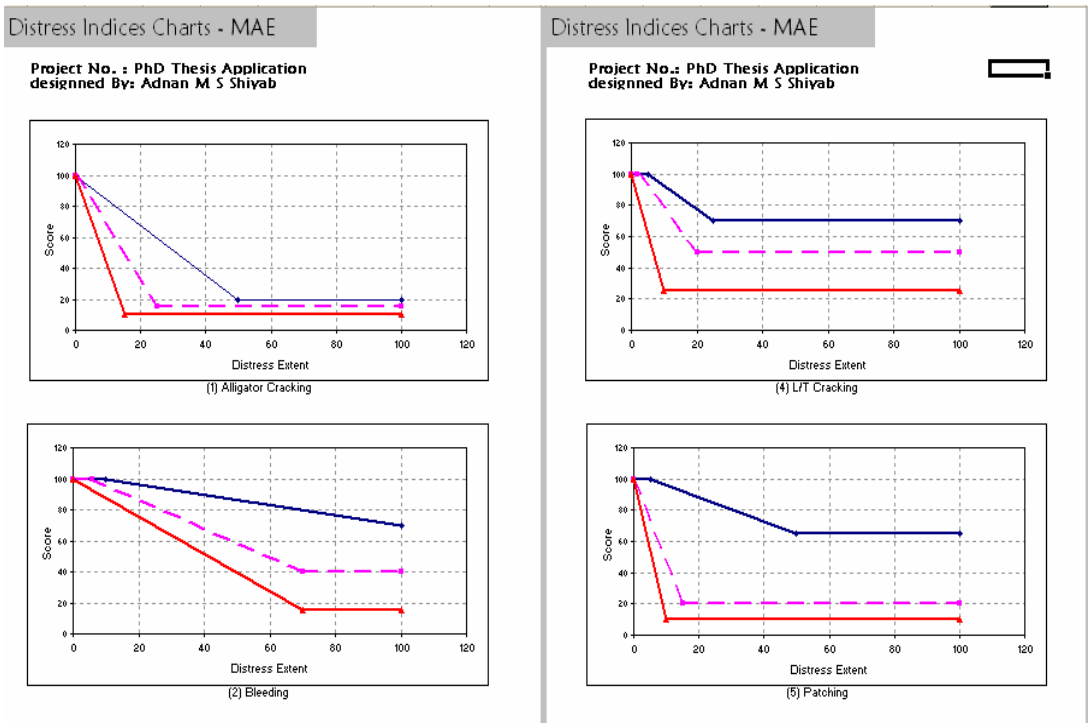


Figure 0.4: Distress Indices Chart Module for various distress types in the PQI/ OPQI calculation program

It also includes the deterioration curves for the other condition indicators of one dimensional distress such as roughness, skid resistance and deflection or structural index.

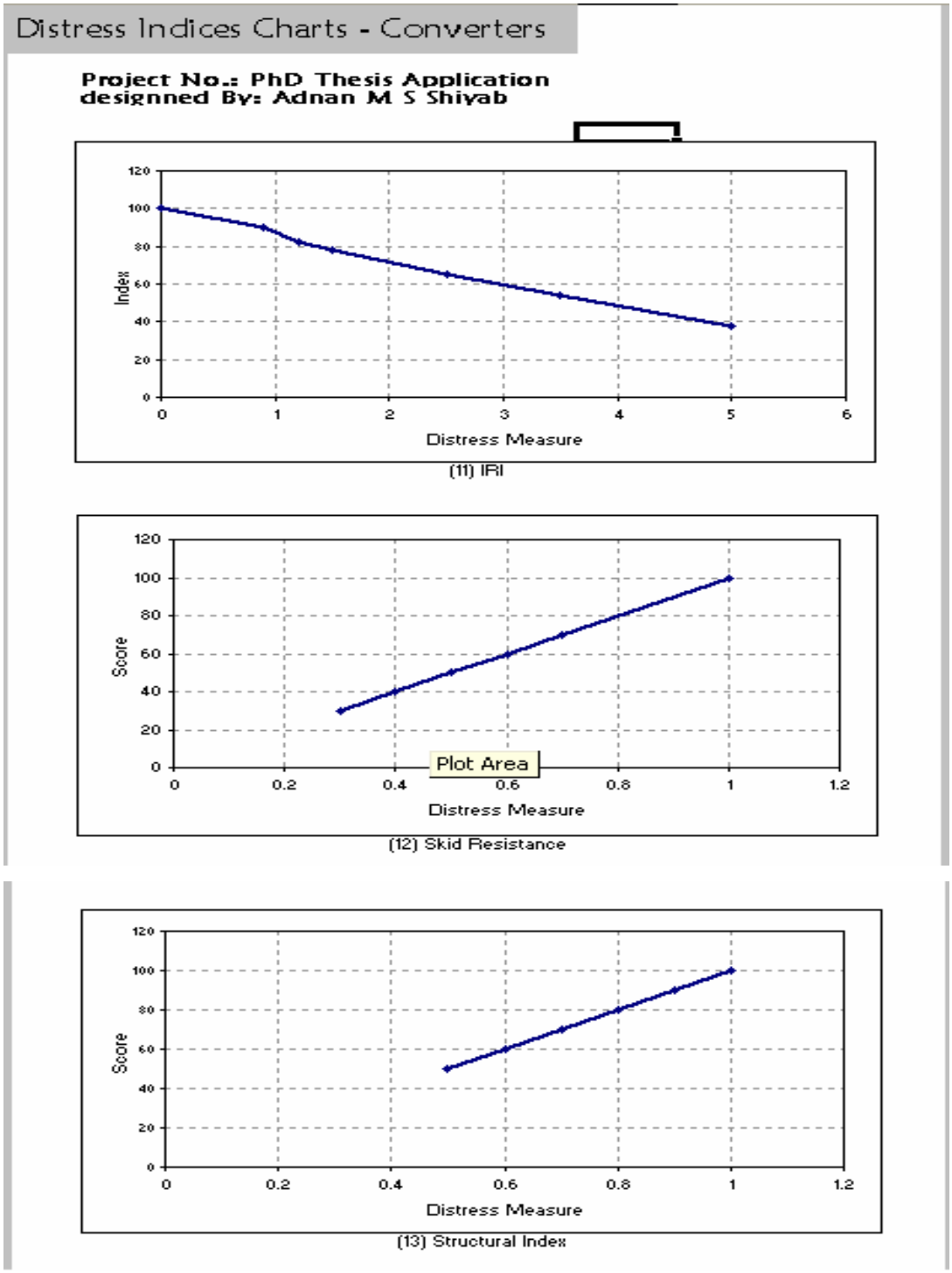


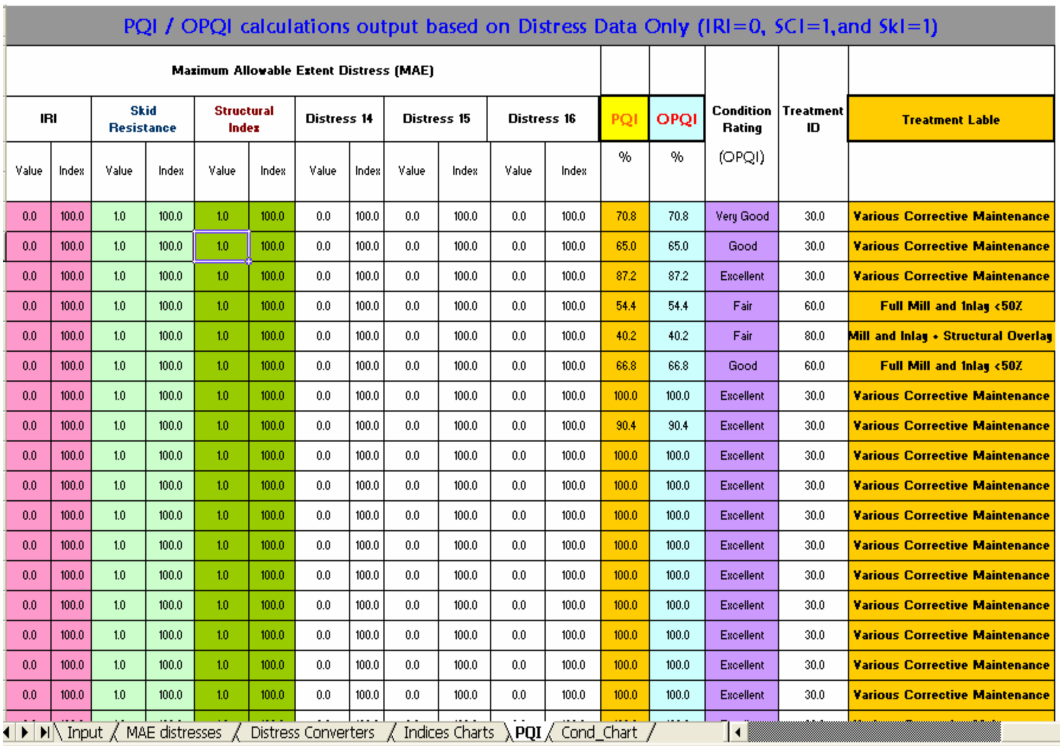
Figure 0.5: Distress Indices Chart Module for Roughness, Skid resistance and Deflection in the PQI/ OPQI calculation program

3. **Performance index module**: This includes the calculated performance index based on the aggregated processed data from all condition indicators. This module calculates four performance indices described below:

- ***The performance index ($OPQI_1$)*** using only distress information. The general formula is as follows:

$$\mathbf{OPQI_1 = \{(PQI\text{-}_{Distress})\ (Wt_{11})\}}$$

In this case, all other condition indicators are set to 100 (measured or due to the unavailability of the data).The selected treatment based on these information will be able to rectify the distresses exist in the surface of the inspected pavement. Any other issues related to the inherent roughness or structural defects may not be taken care of (Figure 8.6)



PQI / OPQI calculations output based on Distress Data Only (IRI=0, SCI=1,and SkI=1)

Maximum Allowable Extent Distress (MAE)																	
IRI		Skid Resistance		Structural Index		Distress 14		Distress 15		Distress 16		PQI	OPQI	Condition Rating	Treatment ID	Treatment Lable
Value	Index	Value	Index	Value	Index	Value	Index	Value	Index	Value	Index	%	%	(OPQI)		
0.0	100.0	1.0	100.0	1.0	100.0	0.0	100.0	0.0	100.0	0.0	100.0	70.8	70.8	Very Good	30.0	Various Corrective Maintenance
0.0	100.0	1.0	100.0	1.0	100.0	0.0	100.0	0.0	100.0	0.0	100.0	65.0	65.0	Good	30.0	Various Corrective Maintenance
0.0	100.0	1.0	100.0	1.0	100.0	0.0	100.0	0.0	100.0	0.0	100.0	87.2	87.2	Excellent	30.0	Various Corrective Maintenance
0.0	100.0	1.0	100.0	1.0	100.0	0.0	100.0	0.0	100.0	0.0	100.0	54.4	54.4	Fair	60.0	Full Mill and Inlay <50%
0.0	100.0	1.0	100.0	1.0	100.0	0.0	100.0	0.0	100.0	0.0	100.0	40.2	40.2	Fair	80.0	Mill and Inlay + Structural Overlay
0.0	100.0	1.0	100.0	1.0	100.0	0.0	100.0	0.0	100.0	0.0	100.0	66.8	66.8	Good	60.0	Full Mill and Inlay <50%
0.0	100.0	1.0	100.0	1.0	100.0	0.0	100.0	0.0	100.0	0.0	100.0	100.0	100.0	Excellent	30.0	Various Corrective Maintenance
0.0	100.0	1.0	100.0	1.0	100.0	0.0	100.0	0.0	100.0	0.0	100.0	90.4	90.4	Excellent	30.0	Various Corrective Maintenance
0.0	100.0	1.0	100.0	1.0	100.0	0.0	100.0	0.0	100.0	0.0	100.0	100.0	100.0	Excellent	30.0	Various Corrective Maintenance
0.0	100.0	1.0	100.0	1.0	100.0	0.0	100.0	0.0	100.0	0.0	100.0	100.0	100.0	Excellent	30.0	Various Corrective Maintenance
0.0	100.0	1.0	100.0	1.0	100.0	0.0	100.0	0.0	100.0	0.0	100.0	100.0	100.0	Excellent	30.0	Various Corrective Maintenance
0.0	100.0	1.0	100.0	1.0	100.0	0.0	100.0	0.0	100.0	0.0	100.0	100.0	100.0	Excellent	30.0	Various Corrective Maintenance
0.0	100.0	1.0	100.0	1.0	100.0	0.0	100.0	0.0	100.0	0.0	100.0	100.0	100.0	Excellent	30.0	Various Corrective Maintenance
0.0	100.0	1.0	100.0	1.0	100.0	0.0	100.0	0.0	100.0	0.0	100.0	100.0	100.0	Excellent	30.0	Various Corrective Maintenance
0.0	100.0	1.0	100.0	1.0	100.0	0.0	100.0	0.0	100.0	0.0	100.0	100.0	100.0	Excellent	30.0	Various Corrective Maintenance
0.0	100.0	1.0	100.0	1.0	100.0	0.0	100.0	0.0	100.0	0.0	100.0	100.0	100.0	Excellent	30.0	Various Corrective Maintenance
0.0	100.0	1.0	100.0	1.0	100.0	0.0	100.0	0.0	100.0	0.0	100.0	100.0	100.0	Excellent	30.0	Various Corrective Maintenance

Figure 0.6: Generated work program for sample roads based on distress data only.

- ***The performance index ($OPQI_2$)*** using distress data and roughness. The general formula for this index is as follows:

$$OPQI_2 = \{(PQI_{-Distress})\ (Wt_{12})\} + \{(PQI_{-Roughness})*(Wt_{22})\}$$

The selected treatment will handle both surface defects and riding or roughness problems. The selected option may or may not be able to rectify some structural deficiencies (Figure 8.7).

PQI / OPQI calculations output based on Distress Data and Roughness Measurements (SCI=1,and SKI=1)

Maximum Allowable Extent Distress (MAE)																
IRI		Skid Resistance		Structural Index		Distress 14		Distress 15		Distress 16		PQI	OPQI	Condition Rating	Treatment ID	Treatment Lable
Value	Index	Value	Index	Value	Index	Value	Index	Value	Index	Value	Index	%	%	(OPQI)		
2.7	62.6	1.0	100.0	1.0	100.0	0.0	100.0	0.0	100.0	0.0	100.0	70.8	64.2	Good	60.0	Full Mill and Inlay <50%
0.9	90.3	1.0	100.0	1.0	100.0	0.0	100.0	0.0	100.0	0.0	100.0	65.0	63.4	Good	60.0	Full Mill and Inlay <50%
1.1	85.7	1.0	100.0	1.0	100.0	0.0	100.0	0.0	100.0	0.0	100.0	87.2	84.1	Very Good	60.0	Full Mill and Inlay <50%
0.9	90.4	1.0	100.0	1.0	100.0	0.0	100.0	0.0	100.0	0.0	100.0	54.4	53.1	Fair	60.0	Full Mill and Inlay <50%
0.8	91.4	1.0	100.0	1.0	100.0	0.0	100.0	0.0	100.0	0.0	100.0	40.2	39.4	Poor	80.0	Mill and Inlay + Structural Overlay
0.9	90.4	1.0	100.0	1.0	100.0	0.0	100.0	0.0	100.0	0.0	100.0	66.8	65.2	Good	60.0	Full Mill and Inlay <50%
1.0	88.1	1.0	100.0	1.0	100.0	0.0	100.0	0.0	100.0	0.0	100.0	100.0	97.0	Excellent	60.0	Full Mill and Inlay <50%
0.8	91.6	1.0	100.0	1.0	100.0	0.0	100.0	0.0	100.0	0.0	100.0	90.4	88.5	Excellent	30.0	Various Corrective Maintenance
1.0	88.7	1.0	100.0	1.0	100.0	0.0	100.0	0.0	100.0	0.0	100.0	100.0	97.2	Excellent	60.0	Full Mill and Inlay <50%
0.9	89.2	1.0	100.0	1.0	100.0	0.0	100.0	0.0	100.0	0.0	100.0	100.0	97.3	Excellent	60.0	Full Mill and Inlay <50%
0.9	90.1	1.0	100.0	1.0	100.0	0.0	100.0	0.0	100.0	0.0	100.0	100.0	97.5	Excellent	60.0	Full Mill and Inlay <50%
0.7	91.9	1.0	100.0	1.0	100.0	0.0	100.0	0.0	100.0	0.0	100.0	100.0	98.0	Excellent	30.0	Various Corrective Maintenance
1.1	85.5	1.0	100.0	1.0	100.0	0.0	100.0	0.0	100.0	0.0	100.0	100.0	96.4	Excellent	60.0	Full Mill and Inlay <50%
0.7	91.9	1.0	100.0	1.0	100.0	0.0	100.0	0.0	100.0	0.0	100.0	100.0	98.0	Excellent	30.0	Various Corrective Maintenance
0.7	92.6	1.0	100.0	1.0	100.0	0.0	100.0	0.0	100.0	0.0	100.0	100.0	98.1	Excellent	30.0	Various Corrective Maintenance
0.7	92.4	1.0	100.0	1.0	100.0	0.0	100.0	0.0	100.0	0.0	100.0	100.0	98.1	Excellent	30.0	Various Corrective Maintenance
1.4	79.1	1.0	100.0	1.0	100.0	0.0	100.0	0.0	100.0	0.0	100.0	100.0	94.8	Excellent	60.0	Full Mill and Inlay <50%
0.9	88.9	1.0	100.0	1.0	100.0	0.0	100.0	0.0	100.0	0.0	100.0	100.0	97.2	Excellent	60.0	Full Mill and Inlay <50%

Input / MAE distresses / Distress Converters / Indices Charts \ PQI / Cond. Chart /

Figure 0.7:Generated work program for sample roads based on distress data and roughness measurements.

- ***The performance index ($OPQI_3$)*** using distress data, roughness and deflection. The general formula for this index is as follows:

$$OPQI_3 = \{(PQI_{-Distress})(Wt_{13})\} + \{(PQI_{-Roughness})*(Wt_{23})\} + \{(PQI_{-Structure})\ (Wt_{33})\}$$

The selected treatment will handle both surface defects and riding or roughness problems and will be able to rectify all structural and deficiencies. The selected option may lack the ability to overcome

safety or skidding problems which sometimes necessitate new mix type or to use some additives.

PQI / OPQI calculations output based on Distress Data and Roughness Measurements (SCI=1,and SKI=1)

Maximum Allowable Extent Distress (MAE)																
IRI		Skid Resistance		Structural Index		Distress 14		Distress 15		Distress 16		PQI	OPQI	Condition Rating	Treatment ID	Treatment Lable
Value	Index	Value	Index	Value	Index	Value	Index	Value	Index	Value	Index	%	%	(OPQI)		
2.7	62.6	1.0	100.0	0.7	70.7	0.0	100.0	0.0	100.0	0.0	100.0	70.8	50.1	Fair	60.0	Full Mill and Inlay <50Z
0.9	90.3	1.0	100.0	0.6	64.9	0.0	100.0	0.0	100.0	0.0	100.0	65.0	46.7	Fair	60.0	Full Mill and Inlay <50Z
1.1	85.7	1.0	100.0	0.9	87.0	0.0	100.0	0.0	100.0	0.0	100.0	87.2	75.9	Very Good	60.0	Full Mill and Inlay <50Z
0.9	90.4	1.0	100.0	0.5	54.3	0.0	100.0	0.0	100.0	0.0	100.0	54.4	34.9	Poor	60.0	Full Mill and Inlay <50Z
0.8	91.4	1.0	100.0	0.4	40.2	0.0	100.0	0.0	100.0	0.0	100.0	40.2	21.7	Poor	60.0	Full Mill and Inlay <50Z
0.9	90.4	1.0	100.0	0.7	66.6	0.0	100.0	0.0	100.0	0.0	100.0	66.8	48.9	Fair	60.0	Full Mill and Inlay <50Z
1.0	88.1	1.0	100.0	1.0	99.8	0.0	100.0	0.0	100.0	0.0	100.0	100.0	96.9	Excellent	60.0	Full Mill and Inlay <50Z
0.8	91.6	1.0	100.0	0.9	90.2	0.0	100.0	0.0	100.0	0.0	100.0	90.4	82.0	Very Good	30.0	Various Corrective Maintenance
1.0	88.7	1.0	100.0	1.0	99.8	0.0	100.0	0.0	100.0	0.0	100.0	100.0	97.0	Excellent	60.0	Full Mill and Inlay <50Z
0.9	89.2	1.0	100.0	1.0	99.8	0.0	100.0	0.0	100.0	0.0	100.0	100.0	97.2	Excellent	60.0	Full Mill and Inlay <50Z
0.9	90.1	1.0	100.0	1.0	99.8	0.0	100.0	0.0	100.0	0.0	100.0	100.0	97.4	Excellent	60.0	Full Mill and Inlay <50Z
0.7	91.9	1.0	100.0	1.0	99.8	0.0	100.0	0.0	100.0	0.0	100.0	100.0	97.8	Excellent	30.0	Various Corrective Maintenance
1.1	85.5	1.0	100.0	1.0	99.8	0.0	100.0	0.0	100.0	0.0	100.0	100.0	96.2	Excellent	60.0	Full Mill and Inlay <50Z
0.7	91.9	1.0	100.0	1.0	99.8	0.0	100.0	0.0	100.0	0.0	100.0	100.0	97.8	Excellent	30.0	Various Corrective Maintenance
0.7	92.6	1.0	100.0	1.0	99.8	0.0	100.0	0.0	100.0	0.0	100.0	100.0	98.0	Excellent	30.0	Various Corrective Maintenance
0.7	92.4	1.0	100.0	1.0	99.8	0.0	100.0	0.0	100.0	0.0	100.0	100.0	98.0	Excellent	30.0	Various Corrective Maintenance
1.4	79.1	1.0	100.0	1.0	99.8	0.0	100.0	0.0	100.0	0.0	100.0	100.0	94.6	Excellent	60.0	Full Mill and Inlay <50Z
0.9	88.9	1.0	100.0	1.0	99.8	0.0	100.0	0.0	100.0	0.0	100.0	100.0	97.1	Excellent	60.0	Full Mill and Inlay <50Z

Input / MAE distresses / Distress Converters / Indices Charts / PQI / Cond. Chart /

Figure 0.8:Generated work program for sample roads based on distress data, roughness and structural capacity measurements.

- ***The performance index (OPQI$_4$)*** using distress data, roughness, deflection and skid resistance. The general formula for this index is as follows:

$$OPQI_4 = \{(PQI\text{-}_{Distress})\ (Wt_{14})\} + \{(PQI\text{-}_{Roughness})*(Wt_{24})\} + \{(PQI\text{-}_{Structure})\ (Wt_{34})\} + \{(PQI\text{-}_{Skid})*(Wt_{44})\}$$

The selected treatment will handle both surface defects and roughness problems and will be able to rectify all structural and functional deficiencies. The selected option will be able also to overcome safety or skidding problems which sometimes necessitate new mix type or to use some additives .

PQI / OPQI calculations output based on Distress Data and Roughness Measurements (SCI=1,and SKI=1)

Maximum Allowable Extent Distress (MAE)																
IRI		Skid Resistance		Structural Index		Distress 14		Distress 15		Distress 16		PQI	OPQI	Condition Rating	Treatment ID	Treatment Lable
Value	Index	Value	Index	Value	Index	Value	Index	Value	Index	Value	Index	%	%	(OPQI)		
2.7	62.6	0.6	60.0	0.7	70.7	0.0	100.0	0.0	100.0	0.0	100.0	70.8	45.1	Fair	60.0	Full Mill and Inlay <50%
0.9	90.3	0.6	60.0	0.6	64.9	0.0	100.0	0.0	100.0	0.0	100.0	65.0	42.0	Fair	60.0	Full Mill and Inlay <50%
1.1	85.7	0.6	60.0	0.9	87.0	0.0	100.0	0.0	100.0	0.0	100.0	87.2	68.3	Good	60.0	Full Mill and Inlay <50%
0.9	90.4	0.6	60.0	0.5	54.3	0.0	100.0	0.0	100.0	0.0	100.0	54.4	31.4	Poor	60.0	Full Mill and Inlay <50%
0.8	91.4	0.6	60.0	0.4	40.2	0.0	100.0	0.0	100.0	0.0	100.0	40.2	19.5	Poor	60.0	Full Mill and Inlay <50%
0.9	90.4	0.6	60.0	0.7	66.6	0.0	100.0	0.0	100.0	0.0	100.0	66.8	44.0	Fair	60.0	Full Mill and Inlay <50%
1.0	88.1	0.6	60.0	1.0	99.8	0.0	100.0	0.0	100.0	0.0	100.0	100.0	87.2	Excellent	60.0	Full Mill and Inlay <50%
0.8	91.6	0.6	60.0	0.9	90.2	0.0	100.0	0.0	100.0	0.0	100.0	90.4	73.8	Very Good	30.0	Various Corrective Maintenance
1.0	88.7	0.6	60.0	1.0	99.8	0.0	100.0	0.0	100.0	0.0	100.0	100.0	87.3	Excellent	60.0	Full Mill and Inlay <50%
0.9	89.2	0.6	60.0	1.0	99.8	0.0	100.0	0.0	100.0	0.0	100.0	100.0	87.4	Excellent	60.0	Full Mill and Inlay <50%
0.9	90.1	0.6	60.0	1.0	99.8	0.0	100.0	0.0	100.0	0.0	100.0	100.0	87.6	Excellent	60.0	Full Mill and Inlay <50%
0.7	91.9	0.6	60.0	1.0	99.8	0.0	100.0	0.0	100.0	0.0	100.0	100.0	88.0	Excellent	30.0	Various Corrective Maintenance
1.1	85.5	0.6	60.0	1.0	99.8	0.0	100.0	0.0	100.0	0.0	100.0	100.0	86.6	Excellent	60.0	Full Mill and Inlay <50%
0.7	91.9	0.6	60.0	1.0	99.8	0.0	100.0	0.0	100.0	0.0	100.0	100.0	88.0	Excellent	30.0	Various Corrective Maintenance
0.7	92.6	0.6	60.0	1.0	99.8	0.0	100.0	0.0	100.0	0.0	100.0	100.0	88.2	Excellent	30.0	Various Corrective Maintenance
0.7	92.4	0.6	60.0	1.0	99.8	0.0	100.0	0.0	100.0	0.0	100.0	100.0	88.2	Excellent	30.0	Various Corrective Maintenance
1.4	79.1	0.6	60.0	1.0	99.8	0.0	100.0	0.0	100.0	0.0	100.0	100.0	85.2	Excellent	60.0	Full Mill and Inlay <50%
0.9	88.9	0.6	60.0	1.0	99.8	0.0	100.0	0.0	100.0	0.0	100.0	100.0	87.4	Excellent	60.0	Full Mill and Inlay <50%

Input / MAE distresses / Distress Converters / Indices Charts / PQI / Cond_Chart /

Figure 0.9: Generated work program for sample roads based on distress data, roughness structural capacity and skid resistance measurements.

The above spread sheets show that the treatment type based on OPQI is considerably affected by the introduction of the values of the other condition indicators. Consequently, the selected treatment option was found to vary based on the values and limits of the other condition indicators.

Figure 8.10 below shows the distribution of the treatment type resulted from the introduction of the condition indicators one each time for one of the major roads investigated for this purpose in the study area.

In this example, the distribution of the treatment types based on $OPQI_1$ (Distress data only) was 93% of the pavement sections were in need for corrective maintenance while 4% were in need for Mill and Inlay <50%, and 2% of the pavement sections were in need for full mill and inlay +structural Overly.

On the other hand, the distribution of the treatment types based on $OPQI_2$ (Distress data +Roughness) was 41% of the pavement sections were in need for corrective maintenance while 59% were in need for Mill and Inlay <50%,

and 2% of the pavement sections were in need for full mill and inlay +structural Overly.

The distribution of the treatment types based on $OPQI_3$ (Distress data +Roughness+ Structural capacity Index) was 41% of the pavement sections were in need for corrective maintenance while 59% were in need for Mill and Inlay <50%, and 2% of the pavement sections were in need for full reconstruction. The enclosure of skid resistance values has resulted in no change to the treatment type distribution as the surface of the road was of good skid resistance level.

The above comparison has shown that IRI has considerable impact on condition rating and consequently in the selection of the optimum treatment option.

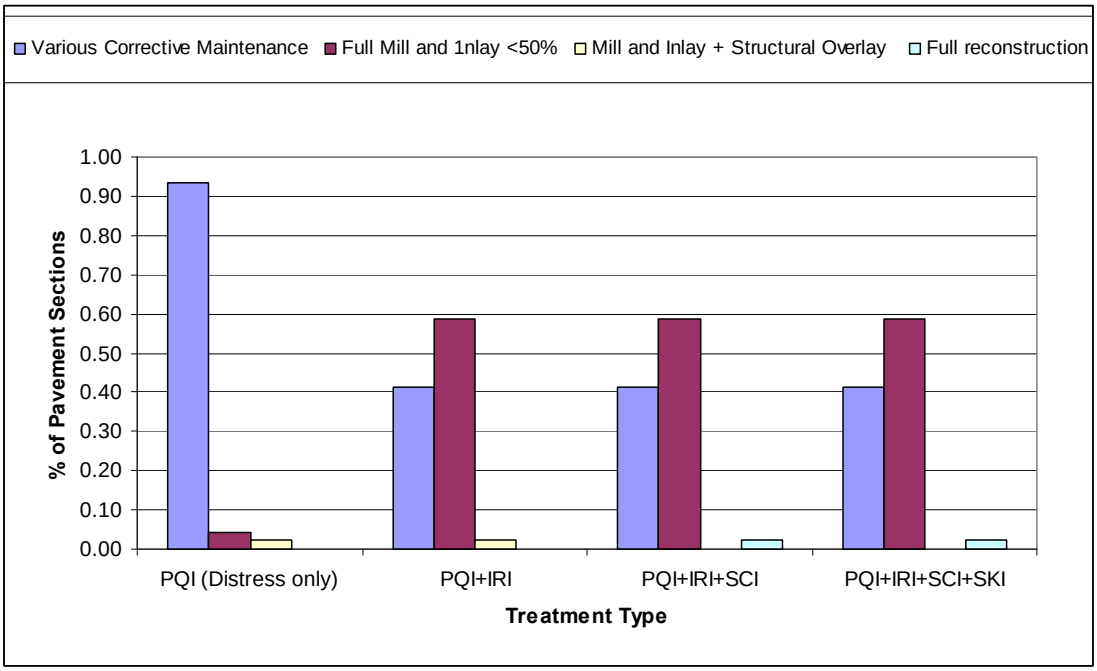


Figure 0.10: Change in treatment distribution as a result of introduction of various condition indicators

Common Treatment selection methods in PMS.

Cost/Benefit Ratio

In the treatment selection process based on the Cost/Benefit ratio or the Incremental Benefit /Cost methods, the main objective of the function is summarized as follows:

"For every year in the analysis period, suggest a work plan which encompasses of the preferred treatment options for roads sections so that the overall benefit is maximized and the cost within a limited budget is minimized".

The method is very complicated as it generates a work plan which Improves the condition of the road network under the condition of the limited budget. The method of maximizing the benefit is directly or indirectly controlled by the user method of configuration. The ***Area Under the Curve (AUC)*** technique is normally used in the optimization process. In this optimization process, the changes in Pavement Quality Index (PQI), Roughness (IRI), Structural Capacity (SCI), and Skid Resistance Index (SkI) resulted from the implementing of the optimum treatment on the selected pavement sections are used to calculate the benefits gained for the respective investment strategies.

This concept is illustrated in Figure 8.1 below. ***The benefit*** in this procedure represented by the area designated C, is estimated by calculating the area under the performance curve for the new treatment minus the remaining area under the performance curve for the existing performance model (Area B). The remaining life area must be removed because there is still some remaining life (salvage) in the pavement which is considered as a benefit in case the pavement is not repaired. On the other hand, ***the cost*** is calculated as the unit cost for the applied treatment. Therefore, the benefit/cost calculations are theoretically correct in selecting the pavement section which yield if treated the maximum benefit in comparison to the cost.

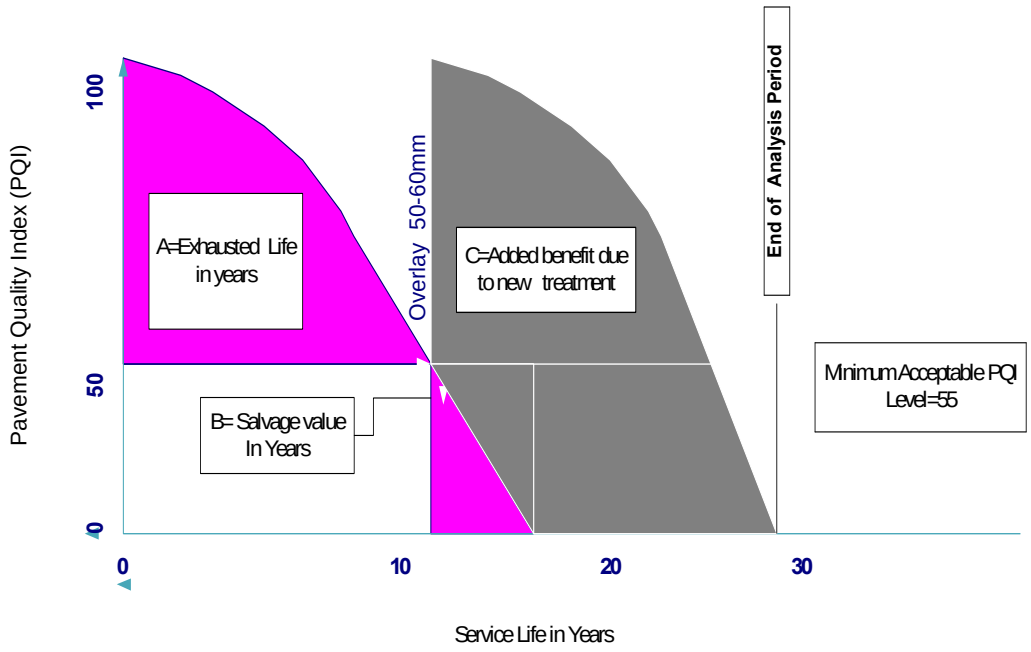


Figure 0.11: Illustration shows Benefit calculation using Cost/ Benefit method.

Priority Method (Limited Budget Scenario)

The other well known method used in PMS treatment selection analysis, is the priority method. This method is designed *to generate a work plan which encompasses of the preferred treatment options for roads sections so that the overall importance of the selected treated road sections is maximized within a limited budget.*

This method may be a single or multiple years prioritization and it is considered the most common method for developing work programs as it is simple to understand and explain. The results of the analysis using this method is greatly influenced by the priority coefficients established by each agency according to their needs and visions (Table 8.2, as an example). The priority coefficients are always established for the main performance classes that expected to play a rule in the decision making process. The priority variables may include, but not limited to, the following performance classes:

- Pavement condition
- Traffic loading
- Road functional class
- Location (Urban /Rural)

The above listed performance classes, used in priority analysis, are user defined variables as the priorities differ from one road agency to another.

Table 0.2: Priority coefficients proposed for the main performance classes used in priority analysis method

Performance Class	Variable Description	Priority Coefficient (PC)	Priority limit Value
Pavement Condition	Excellent	1	
	Very Good	2	
	Good	3	
	Fair	4	
	Poor	5	
Traffic Loading	Low	1	< 1500 VPD
	High	5	>10,000 VPD
Road Functional Class	Freeways	5	
	Expressways	4	
	Arterials	3	
	Collectors	2	
	Local industrial	2	
	Local/Residential	1	
Location	Urban	1	
	Rural	2	

Whereas the treatment scope variables are used to define what type of treatment to be selected for certain pavement sections, the priority variables are used to define the order in which these sections are to be treatment. For the purpose of the computerized application of the treatment selection based

on priority method, the priority values listed in Table 8.2 above were suggested.

The overall pavement priority index for a pavement section is calculated using the multiplicative procedure by multiplying the variables by the each others .

For example, the priority index for a pavement section pertaining to a road classified as Arterial (PC=3) of a Very Good condition (PC=2) and located in Rural area(PC=2) and heavily trafficked of 80,000 VPD (PC=5) would be 3*2*2*5=60.

Needs Method (Open Budget Scenario)

Another method used in PMS treatment selection analysis in addition to the above described two methods, is the ***Need method***. This method is designed to generate a work plan which encompasses of the preferred treatment options with minimum possible cost for roads sections so that the overall network condition is to reach the targeted condition threshold leaving a predefined percentage of the road network below the targeted level.

This method is used to generate an open budget scenario in which no consideration to the priority factor is given. The above three method are the most common that used in PMS to generate the work plans and scenarios.

Table 8.3 below includes the proposed Maximum Treatment Improvement (MTI) on distress condition and main condition indicators indices proposed by the researcher.

These values can be used by the pavement management system to calculate the benefit gained as a result of applying certain treatment type to rectify certain defects. The new deterioration curve resulted from applying such a treatment is compared against the relevant cost. If the benefit/cost ratio is more than 1, then the selected treatment is considered satisfactory.

Table 0.3: Maximum treatment Improvement on distress condition and main condition indicators indices.

Treatment Code	Treatment Description	Unit Cost	Treatment Priority	Exclusion Priority	Exclusion Years	Alligator Imp	Bleeding Imp	L&T Imp	Patch Imp	Rut Imp	W/R Imp	SCI Imp	(IRI) Imp	SKN Imp	PQI Imp	OPQI Imp	
0	DN	0	0	0	0	0	0	0	0	0	0	0	0		0	0	0
5	SP	3900	5	5	3	15	0	10	15	10	0	0	10		15	10	5
10	DP	7500	10	15	4	50	0	20	0	30	0	0	20		20	20	10
15	DMSW	3900	15	20	5	40	40	40	40	40	40	40	40		40	40	15
20	CS	9000	20	10	2	10	0	50	0	0	0	0	0		20	5	20
30	TO	30000	30	30	6	50	50	50	50	50	50	50	35		50	40	30
35	MI<50%	20000	35	35	7	50	50	100	50	50	50	50	50		50	50	35
40	FMI	75000	40	40	12	100	100	100	100	100	100	100	100		100	100	40
45	FR	75000	45	45	10	100	100	100	100	100	100	100	100		100	100	45
50	DN	150000	50	50	15	100	100	100	100	100	100	100	100		100	100	50

	Treatment Description		
		CS	Cr.Seal (Rout+Seal)
DN	Do Nothing	TO	Thin Overlay
SP	Surf. Patching	MI<50%	Mill & Inlay < 50%
DP	Deep Patching	FMI	Full Mill & Inlay
DMSW	Do Minimum Surface Work	FR	Full Reconstruction

Consolidation of various condition indicators

Maintenance and rehabilitation schemes developed in this research effort are designed not only based on a single index but also taking other factors into consideration. These schemes are designed based on inputs which include distress types, distress severity, distress density or distress index, roughness index , structural capacity index , skid resistance number , location, and other inventory data.

The outputs from the analysis of various condition indicators should be consolidated on one graph and studied carefully for the purpose of identifying the segment which suffer from severe structural, or functionally/structurally or only functional deterioration. The aim of this action is to divide the road or network into section of homogenous condition for further course of action or to receive a repair action.

Figure 8.12 below shows an example of the road segmentation based on various condition indicators. The segmentation process helps in reducing the cost and optimizing the treatment selection process.

There is always interference areas between the structural and functional condition of the pavement. The road which is structurally damaged will have always high roughness. The selected treatment should be able to restore the structural integrity and improve the riding quality.

This consolidated chart is used also for further network segmentation based on condition as most of the new generation of the pavement management systems offer an option for dynamic segmentation based on different criteria such as condition, widening of the road by constructing additional lanes, etc.

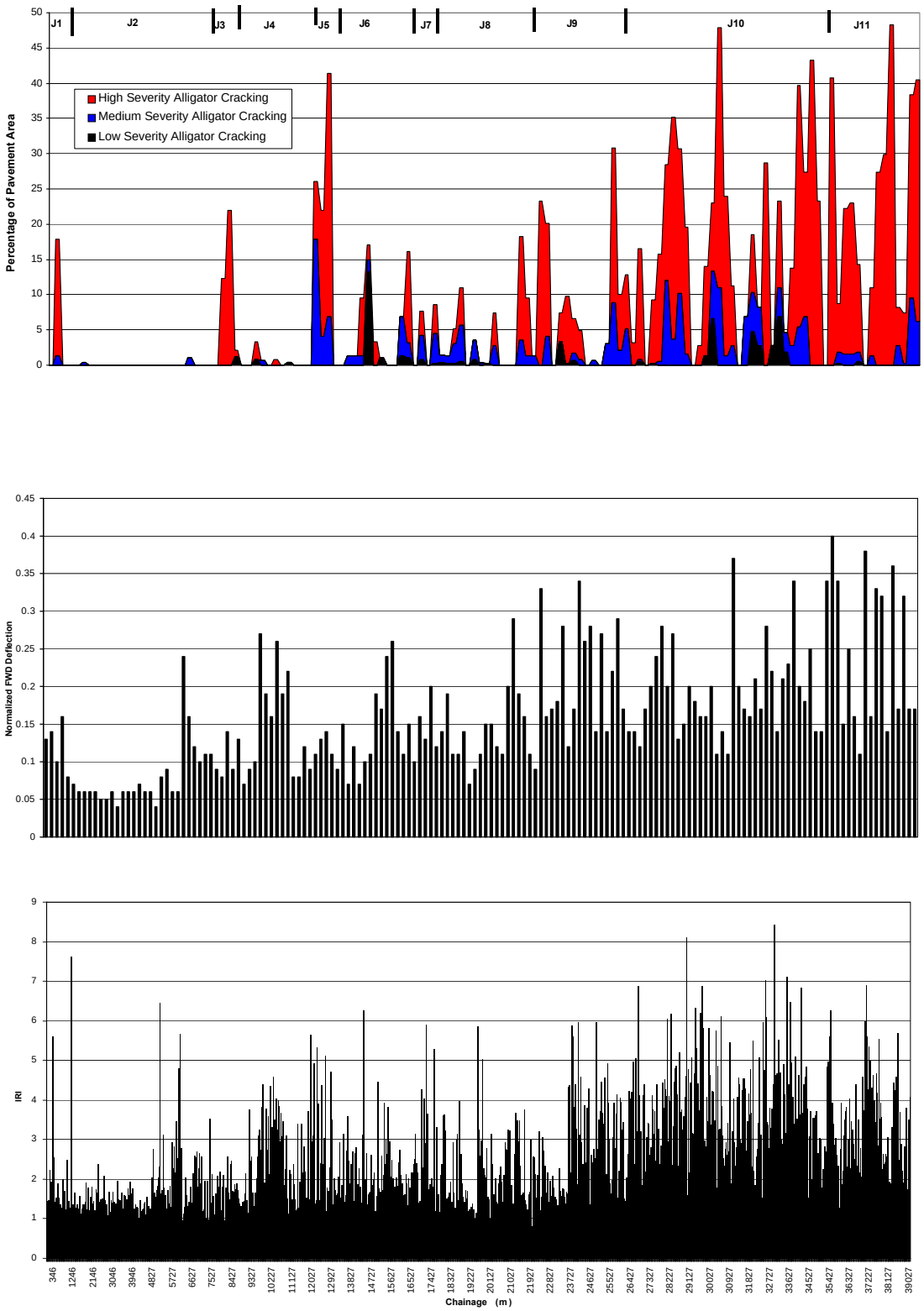


Figure 0.12: Example of Pavement segmentation and Treatment selection based on consolidated pavement condition indicators

Individual pavement condition indicators are consolidated to the most critical condition indicator. The critical indicator is the one which necessitates the heaviest treatment i.e.

which can restore structural, functional and safety integrity of the pavement section. The treatment selected to handle structural deficiencies always has the capability to restore riding quality and skid resistance problems. Such illustrative figures are designed to help managers to make informed decisions regarding maintenance requirements based on solid engineering analyses.

Life cycle cost analysis for feasible treatment alternatives

In order to determine the most cost effective treatment, many procedures were developed. Each method generates certain outputs which help in optimizing the treatment selection process by maximizing the benefits and minimizing the cost. These methods range in complexity from direct, straight forward cost benefit evaluation to a most sophisticated methods which needs many inputs to calculate the effectiveness of the selected treatment. The following table summarizes the most common methods in cost effectiveness calculations in pavement management systems.

Life cycle costing analysis is normally used in PMS to select the most cost effective option among a list of feasible alternatives. This list is usually selected based on the detailed pavement evaluation and analysis.

Life cycle costing techniques are used in the following engineering applications:

- To evaluate the implications for pavement or asset financing and the investment strategies by examining the trade-off between the capital and the maintenance costs.
- To explore the effect of traffic loading on pavement maintenance strategies and the corresponding costs.
- Evaluate the relative characteristics, performance and costs of adopting various pavements types such as asphalt , concrete or modified asphalt pavements.
- To evaluate various pavement design alternatives and practices by highlighting the costs at various traffic loading levels and fleet spectrum.

The cost for the selected alternatives can be calculated by many procedures, in general, there are many types of *cost* parameters needed to perform life cycle calculations, these parameters include:

- ***Initial Cost (C Initial):*** It is the present year cost for an alternative. No future costs are considered.

- ***Present Worth Value(PW):*** This method is the most common one used in life cycle cost analysis to compare among various feasible maintenance and rehabilitation alternatives. It is the simplest and most straightforward of the methods since it represents the current value of all costs that will be incurred over the lifetime of the project. If the M&R alternative are to compared using this procedure, then all alternatives should be evaluated over the same analysis period (Sharaf *et al*. 1987).

The effect of the interest and inflation rate on the alternative cost is considered in this calculation. The future cost has to be adjusted taking into consideration the inflation rate .

The future cost of the M&R option (i.e. after t years in the future) can be calculated as follows (Shahin,1998)

$$C_{m\&R\text{-}t} = C_p(1+r)^t \qquad 8.7$$

Table 0.4: Most common cost -effective analysis methods in PMS

Analysis Method	Main Inputs	Main Outputs
1-Life Cycle Costing.	• Analysis period • Treatment Unit Cost • Treatment expected life • Interest rate • Inflation rate	Determine Equivalent Uniform Annual Cost for each treatment Lowest EAUC is selected.
2-Equivalent Annual Cost	• Daily cost for Manpower • Consumed materials	Unit cost for the treatment over the

	• Tools	expected life.
3- Cost - Effectiveness	• Predicted Performance Curve	Treatment effectiveness represented by the area under the performance curve.
4-longevity cost index	• Treatment Unit Cost • Treatment expected life • Present value of the unit cost over expected life of the treatment • Traffic loading	Compare the present value against life and traffic

Where:

$C_{m\&R-t}$= The cost of the Maintenance and Rehabilitation alternative at t years in the future.

C_p=The cost of the maintenance and rehabilitation alternative in the present day .

r=Annual inflation rate.

t=The time in the future in years.

Then, the present value (PV) of the future cost for any M&R alternative, which represent the total cost of all the money that would need to be invested now to cover the funding over the design period, can be calculated by applying a discount rate i. i.e. :

$$\mathbf{PV= C_{m\&R-t} / (1+i)^t} \qquad 8.8$$

Where:

Pv=The present value of the money. I.e. the amount of the money that would have to be placed in an interest bearing account now to be in a value of $C_{m\&R-t}$ in t years.

i= The annual interest rate in decimals.

The present value of the M&R option expected to serve t years in the future can be calculated

$$PV = C_p * \{ (1+r)^t / (1+i)^t \} \qquad 8.9$$

Where:

Pv= The present value of the money. I.e. the amount of the money that would have to be placed in an interest bearing account now to be in a value of $C_{m\&R-t}$ in t years.

C_p= The cost of the maintenance and rehabilitation alternative in the present day .

r= Annual inflation rate in decimals.

i= The annual interest rate in decimals.

t= The time in the future in years.

If the maintenance and rehabilitation alternative encompasses more than one activity, then the present value of the this alternative would be calculated by summing up the initial costs of various activities after adjusting the future costs for the inflation and interest rates. Therefore, the present value of the M&R alternative which includes more than one activity can be calculated using the following formula:

$$PV = C_i + \sum^{t=n} C_p * \{ (1+r)^t / (1+i)^t \} \qquad 8.10$$

If the pavement under assessment has a salvage value, the net present worth can take the following form

$$PV = C_i + \sum^{t=n} C_p * \{ (1+r)^t / (1+i)^t \} - S(1+R)^{-n} \qquad 8.11$$

where

PV=The present value of the money. I.e. the amount of the money that would have to be placed in an interest bearing account now to be in a value of $C_{m\&R-t}$ in t years.

C_i=The summation of the initial costs for all activities incorporated in the M&R alternative.

C_p=The cost of the maintenance and rehabilitation alternative in the present day .

r= Annual inflation rate in decimals.

i= The annual interest rate in decimals.

t= The number of years from the present to the ith maintenance or rehabilitation application time

R=Discount rate i.e inflation rate minus interest rate (r-i)

S = Salvage value of pavement at the end of the analysis period expressed in terms of present values

n= Number of years in the analysis period.

Equivalent Uniform Annual Cost : This method is used to compare various M&R alternatives of different performance lives. This procedure combines all costs and all expenses into a single annual sum that is equivalent to all disbursements during the pavement's service life (Sharaf *et al*. 1987). A cost recovery factor should be applied to the present value of each M&R alternative to obtain the EUAC value. The EUAC can be calculated as follows:

$$\mathbf{EUAC=PV*CRF} \qquad 8.12$$

Where:

PV= The present value of the money. I.e. the amount of the money that would have to be placed in an interest bearing account now to be in a value of $C_{m\&R\text{-}t}$ in t years.

CRF= Cost recovery factor and it is calculated as:

$$\mathbf{CRF=I(1+I)^n\ /(1+I)^n\ -1} \qquad 8.13$$

In the life cycle costing comparisons, the EUAC is divided by the surface area of the pavement section to be treated to get the unit cost per squared meter or yard.

The EUAC is suggested by many researchers as it is simple and straight forward(Hicks et al1999) . It can be used in the cost effective evaluation of various M&R alternatives. The output from maintenance and the rehabilitation point of view is the *unit cost per expected service life* of the suggested treatment. I.e.

EAC= Unit Cost / Expected life of the M&R option in Years 8.14

The EAC for any treatment options can be determined directly taking advantage of the expected service life for each treatment which were outlined in Table 7.1 and Table 7.2 in the previous chapter.

In most of the road agencies, present worth of cost is the preferred procedure to compare various maintenance and rehabilitation alternative as it takes the cost of all activities included in each option and bring it back to the present time taking into consideration the expected service life for each treatment.

Figure 8.1 shows a schematic illustration example for the treatment options and the corresponding performance trend along the analysis period.

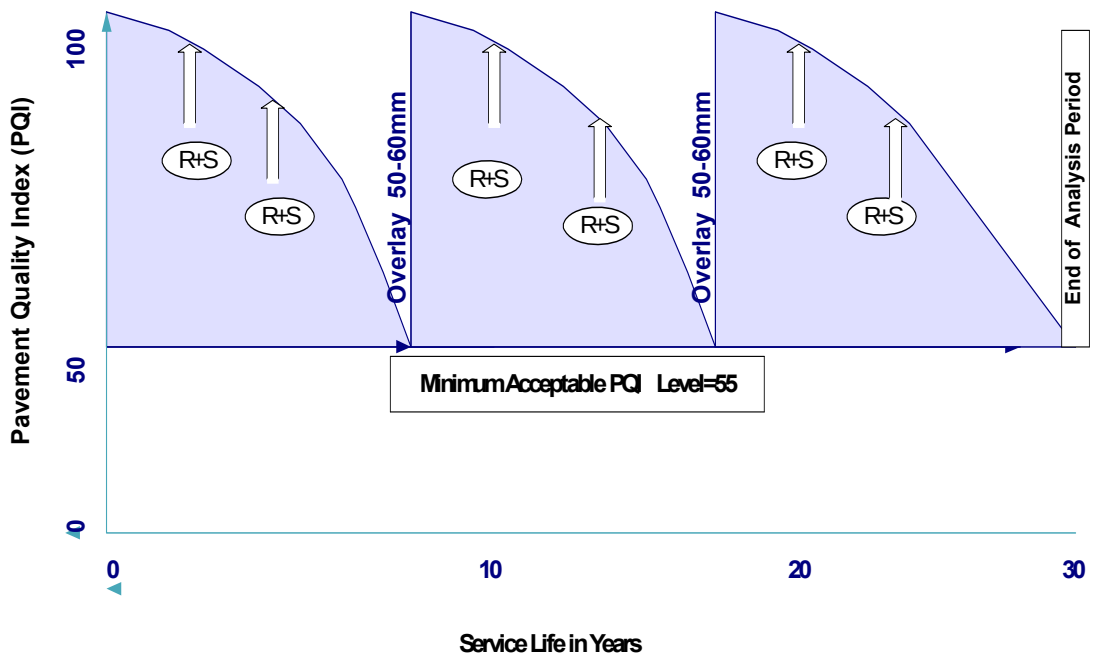


Figure 0.13: Life cycle costing analysis for treatment options adopted in PMS

In transportation , the decision makers are always keen to know about the effectiveness and the cost effectiveness of the proposed treatment option.

The effectiveness of the selected treatment can be determined based on the performance curves. It is calculated as being the total area under the performance

curve, or curves if the rehabilitation scheme is composed of a series of activities, over the analysis period. Then this area is multiplied by the length of the pavement section and the average traffic volume. i.e. :

Effectiveness= Area under PQI-Age Curve*L*ADT 8.15

Where:

Area= Total area under the performance curve, or curves if the rehabilitation scheme is composed of a series of activities, over the analysis period.

L=Length of the pavement section under rehabilitation in Km or Mile.

ADT=Average daily Traffic volume between the 0 and end of the analysis period.

The ***Cost-Effectiveness*** value for the selected treatment option can be determined as follows:

Cost-Effectiveness=(Effectiveness/Total PW Cost)
–Terminal cost (salvage) value 8.16

The preferred solution is the one that has the highest cost –effectiveness value. The cost effectiveness ratio for the compared treatment option is similar to the cost/benefit ratio used in the economical analysis to compare two candidate solutions. This ratio will give an idea about the percentage of the effectiveness for one solution against the other one.

The evaluation period should be based on the economic life of the asset i.e. the pavement. It should be long enough to capture all the significant costs required to make an effective comparison of the alternatives. Usually, the whole life cycle costing is calculated over of period of 20-30 years.

The final aim of selecting the optimum treatment is to decrease the percentage of the structural cracking, decrease roughness to the target level, increase the skid resistance to the targeted level and finally to increase the structural capacity of the pavement.

CONCLUSIONS AND RECOMMENDATIONS.

This study effort aimed primarily at investigating the optimum way to use the outputs pertaining to the main condition indicators of flexible pavement. Current PMS's use these outputs in various forms and at different management levels to report about pavement condition, estimate the remaining life and estimate the required financial funds for maintenance work programs.

Data pertaining to thousands of pavement sections were used in the analysis to prove some of the results and to answer the main question; *Which parameters to use in PMS and in which form*. Various available analysis approaches were investigated and discussed.

New methods were suggested as an alternatives to the current available methods which found to be impractical in certain cases. Based on the outcomes of this study, the following conclusions were drawn and a number of recommendations were raised:

1. Current pavement management systems available around the world are considered one–index based system as it depend mainly on the pavement distress data only.
2. Rutting was found to be one of the basic parameters in pavement management system as it can be used to report about pavement condition at *network level*. It can be used to assessing the pavement serviceability, Monitoring the progression of rutting, Scheduling and plan rehabilitation treatment.
3. Rut data can be used effectively in pavement management systems to select the optimum maintenance and rehabilitation option to restore the pavement structural capacity and improves the existing functional characteristics.
4. Rutting , in addition to the alligator cracking information incorporated in the pavement quality index (PQI) , were found useful in predicting the effective structural capacity at network level.
5. Roughness was found to be a crucial pavement condition indicator right across the world. It is the most widely used condition parameter adopted by most of the road agencies to report about pavement functional performance.
6. Roughness is to be reported in terms of International Roughness index (IRI). The units for this index can be in terms of mm/m , m/km , Inch/mile and NAASRA counts/km. (1 count=15.2mm).
7. The IRI can also be reported at network level in the following formats:
    1. Lane IRI
    2. Yaw (Difference between the left and right wheel path measurements.
    3. Cumulative distribution curves to report network health at different risk levels.
8. Roughness-Age Models which describe the roughness progression rate based on the current age or current roughness value can be used efficiently in pavement asset management for preparing the multi-years master work plans and estimation of the maintenance and rehabilitation budget.
9. The terminal value of the roughness should be selected based on road class and the posted speed. The study found that an IRI of 3.4 can be adopted as a terminal value for major roads where speed is usually more than 100k/h.

10. Deflection data are helpful to report about pavement condition at both project and network levels. At network level, the back calculated parameters from deflection basin such as pavement modulus, pavement curvature and the deflection basin characteristics are much more appropriate for reporting about pavement structural conditions in PMS.
11. The importance of the structural capacity indicator stems from the fact that pavements are often rehabilitated or reconstructed due to the inherent structural capacity weaknesses. Therefore, structural capacity information can be used to filter the pavement sections that should be selected to work on at project level.
12. The ability to use structural data as well as other pavement condition indicators will enhance considerably the reliability of the decision making process and rationalize the treatment selection procedure. .
13. Austroads formulas can be used effectively in pavement management system since it depends on variables that can be easily measured. At network level, each pavement section will have a value for the remaining life and the overlay thickness.
14. The overlay thickness calculated by the mechanistic or mechanistic approaches is a misleading option as it is always impractical. Mill and Inlay or partial reconstruction should be introduced instead as most of the roads under rehabilitation will be cracked of suffer from defects.
15. Using the models developed to predict the pavement deterioration based on deflection and ESAL, the structural capacity and the needed strengthening can be made available in PMS at any time.
16. The Pavement Quality Index (PQI), IRI, SN_{eff} parameters are good predictors for the pavement remaining life and can be used effectively in PMSs to guide engineers and decision makers in the treatment selection process and the scheduling of maintenance and rehabilitation works.
17. The effective structural number was found to have good correlations with the overall pavement thickness of the pavement and value of the deflection measured at the center of the loading plate. The two variables were found to account for more than 92% of the structural capacity prediction model
18. Traffic variable in terms of the accumulated standard repetitions (ESAL) was found to account for more than 60% of the deflection model predictability. Other variables such as E value, compaction, asphalt and base layer thickness can improve the predictability of the model if included.

19. The concept of the relative value of effective pavement modulus to the original pavement modulus gives a reliable representation about the exhausted and the remaining life of the in-service pavement structure.
20. The pavement is reported to be structurally failed, when the effective asphalt or pavement modulus is about 20 - 35 % of its original design value.
21. It was found that when the area of the cracking and the patching distresses exceeds 17% of the total pavement section area, or the depth of rutting is more than 15mm , the pavement is reported to be structurally failed and major rehabilitation or reconstruction should be applied.
22. Skid resistance can be reported in the form of friction factor Mu. Also, Mean texture depth was found to have good predictor of the skid resistance.
23. The most accurate assessment of the pavement performance can be achieved through the use of maximum deflection in combination with an indicator of the radius of curvature of the pavement under load.
24. The characteristic design value (CDV) of deflection parameters are recommended to be in PMS as follows:

Parameter	**f**	**% of all values that will be covered by the CDV**	**Recommended Design value**
Back calculated Modulus of Elasticity Value (E)	-2.0	97.5	Mean - 2 *(ST Dev)
Remaining Life (RL)	- 2.0	97.5	Mean - 2 *(ST Dev)
Overlay Thickness (OL)	+2.0	97.5	Mean + 2 *(ST Dev)

.

25. Mean Texture Depth was found to be a good predictor of the skid resistance level of the roads. It explains more that 70 % of the skid resistance performance. Others may be explained by the microtexture and speed.
26. Skid resistance can be reported in the form of International Friction Index (**IFI)**, as a well defined universal index, along with other two numbers ;**F60** Friction (Microtexture) related number measured at 60 km/h velocity and Macrotexture related number and **Vp,** which constitute the IFI.
27. IFI index can be used in Pavement management system applications to report about skid resistance characteristics and the network level of safety. These three figures can be used to report about pavement condition, accidents, airports operations, and maintenance management surveys.

28. It was found that using multi–indices condition indictors is much more helpful in selecting the appropriate treatment to fully restore the riding quality and the surface structural integrity.
29. Without incorporating all of the pavement condition indicators into the pavement management systems, there will be obviously many limitations and deficiencies in selecting the optimum maintenance and rehabilitation alternative of pavement section at network level, setting priorities, and selecting the optimal time for carrying out the required repair.
30. The suitable weights that should be given to each index are proposed to be; 0.50 for physical condition, 0.25 for structural index and 0.2 for roughness and 0.05 for skid resistance index.
31. Further research effort is deemed necessary to develop more predictive models using other deflection basin parameters, such as cross sectional area, and remaining life in terms of ESAL in PMS at network level.

CONCLUSIONS AND RECOMMENDATIONS.

This study effort aimed primarily at investigating the optimum way to use the outputs pertaining to the main condition indicators of flexible pavement. Current PMS's use these outputs in various forms and at different management levels to report about pavement condition, estimate the remaining life and estimate the required financial funds for maintenance work programs.

Data pertaining to thousands of pavement sections were used in the analysis to prove some of the results and to answer the main question; *Which parameters to use in PMS and in which form*. Various available analysis approaches were investigated and discussed.

New methods were suggested as an alternatives to the current available methods which found to be impractical in certain cases. Based on the outcomes of this study, the following conclusions were drawn and a number of recommendations were raised:

1. Current pavement management systems available around the world are considered one–index based system as it depend mainly on the pavement distress data only.
2. Rutting was found to be one of the basic parameters in pavement management system as it can be used to report about pavement condition at *network level*. It can be used to assessing the pavement serviceability, Monitoring the progression of rutting, Scheduling and plan rehabilitation treatment.
3. Rut data can be used effectively in pavement management systems to select the optimum maintenance and rehabilitation option to restore the pavement structural capacity and improves the existing functional characteristics.
4. Rutting , in addition to the alligator cracking information incorporated in the pavement quality index (PQI) , were found useful in predicting the effective structural capacity at network level.
5. Roughness was found to be a crucial pavement condition indicator right across the world. It is the most widely used condition parameter adopted by most of the road agencies to report about pavement functional performance.
6. Roughness is to be reported in terms of International Roughness index (IRI). The units for this index can be in terms of mm/m , m/km , Inch/mile and NAASRA counts/km. (1 count=15.2mm).

7. The IRI can also be reported at network level in the following formats:
    1. Lane IRI
    2. Yaw (Difference between the left and right wheel path measurements.
    3. Cumulative distribution curves to report network health at different risk levels.
8. Roughness-Age Models which describe the roughness progression rate based on the current age or current roughness value can be used efficiently in pavement asset management for preparing the multi-years master work plans and estimation of the maintenance and rehabilitation budget.
9. The terminal value of the roughness should be selected based on road class and the posted speed. The study found that an IRI of 3.4 can be adopted as a terminal value for major roads where speed is usually more than 100k/h.
10. Deflection data are helpful to report about pavement condition at both project and network levels. At network level, the back calculated parameters from deflection basin such as pavement modulus, pavement curvature and the deflection basin characteristics are much more appropriate for reporting about pavement structural conditions in PMS.
11. The importance of the structural capacity indicator stems from the fact that pavements are often rehabilitated or reconstructed due to the inherent structural capacity weaknesses. Therefore, structural capacity information can be used to filter the pavement sections that should be selected to work on at project level.
12. The ability to use structural data as well as other pavement condition indicators will enhance considerably the reliability of the decision making process and rationalize the treatment selection procedure. .
13. Austroads formulas can be used effectively in pavement management system since it depends on variables that can be easily measured. At network level, each pavement section will have a value for the remaining life and the overlay thickness.
14. The overlay thickness calculated by the mechanistic or mechanistic approaches is a misleading option as it is always impractical. Mill and Inlay or partial reconstruction should be introduced instead as most of the roads under rehabilitation will be cracked of suffer from defects.
15. Using the models developed to predict the pavement deterioration based on deflection and ESAL, the structural capacity and the needed strengthening can be made available in PMS at any time.

16. The Pavement Quality Index (PQI), IRI, SN_{eff} parameters are good predictors for the pavement remaining life and can be used effectively in PMSs to guide engineers and decision makers in the treatment selection process and the scheduling of maintenance and rehabilitation works.
17. The effective structural number was found to have good correlations with the overall pavement thickness of the pavement and value of the deflection measured at the center of the loading plate. The two variables were found to account for more than 92% of the structural capacity prediction model
18. Traffic variable in terms of the accumulated standard repetitions (ESAL) was found to account for more than 60% of the deflection model predictability. Other variables such as E value, compaction, asphalt and base layer thickness can improve the predictability of the model if included.
19. The concept of the relative value of effective pavement modulus to the original pavement modulus gives a reliable representation about the exhausted and the remaining life of the in-service pavement structure.
20. The pavement is reported to be structurally failed, when the effective asphalt or pavement modulus is about 20 - 35 % of its original design value.
21. It was found that when the area of the cracking and the patching distresses exceeds 17% of the total pavement section area, or the depth of rutting is more than 15mm , the pavement is reported to be structurally failed and major rehabilitation or reconstruction should be applied.
22. Skid resistance can be reported in the form of friction factor Mu. Also, Mean texture depth was found to have good predictor of the skid resistance.
23. The most accurate assessment of the pavement performance can be achieved through the use of maximum deflection in combination with an indicator of the radius of curvature of the pavement under load.
24. The characteristic design value (CDV) of deflection parameters are recommended to be in PMS as follows:

Parameter	**f**	**% of all values that will be covered by the CDV**	**Recommended Design value**
Back calculated Modulus of Elasticity Value (E)	-2.0	97.5	Mean - 2 *(ST Dev)
Remaining Life (RL)	- 2.0	97.5	Mean - 2 *(ST Dev)
Overlay Thickness (OL)	+2.0	97.5	Mean + 2 *(ST Dev)

.

25. Mean Texture Depth was found to be a good predictor of the skid resistance level of the roads. It explains more that 70 % of the skid resistance performance. Others may be explained by the microtexture and speed.
26. Skid resistance can be reported in the form of International Friction Index (**IFI)**, as a well defined universal index, along with other two numbers ;**F60** Friction (Microtexture) related number measured at 60 km/h velocity and Macrotexture related number and **Vp,** which constitute the IFI.
27. IFI index can be used in Pavement management system applications to report about skid resistance characteristics and the network level of safety. These three figures can be used to report about pavement condition, accidents, airports operations, and maintenance management surveys.
28. It was found that using multi–indices condition indictors is much more helpful in selecting the appropriate treatment to fully restore the riding quality and the surface structural integrity.
29. Without incorporating all of the pavement condition indicators into the pavement management systems, there will be obviously many limitations and deficiencies in selecting the optimum maintenance and rehabilitation alternative of pavement section at network level, setting priorities, and selecting the optimal time for carrying out the required repair.
30. The suitable weights that should be given to each index are proposed to be; 0.50 for physical condition, 0.25 for structural index and 0.2 for roughness and 0.05 for skid resistance index.
31. Further research effort is deemed necessary to develop more predictive models using other deflection basin parameters, such as cross sectional area, and remaining life in terms of ESAL in PMS at network level.

References

Chapter 1

1. Wotring, Donald C., Baladi,Gilbert Y., Buch Neeraj, and Bower Steve (1998) " Pavement Distress and Selection of Rehabilitation Alternatives" in Transportation Research Record 1629, TRB, National Research Council, Washington D.C., pp 214-225.
1. Hass, R., Hudson, W. R., Zaniewski, J. P, 1994. Modern Pavement Management , Krieger Publishing Company, Malabar, Florida.
2. Zaniewski, J,P., and Mamluk M.S.,1996 pavement maintenance effectiveness – preventive maintenance treatment participants Handbook. FHWA-SA-96-127.FHWA, US Department of Transportation.
3. Graham Smith and Clell Harral, 1987. Road deterioration in developing countries, in Transportation Research Record 1128, TRB, National Research Council, Washington D.C., pp 28-35.
4. AASHTO (1993) AASHTO Guide for the Design of Pavement Structures. American Association of State Highway and Transportation Officials, Washington, D.C, PP 1-31.
5. Shahin M.Y.(1998) Pavement Management for Airports, Roads, and Parking lots. Third Edition, Chapman and Hall, kluwer Academic Publisher, Norwell, Massachusetts, USA.
6. AASHTO (1993) AASHTO Guide for the Design of Pavement Structures. American Association of State Highway and Transportation Officials, Washington, D.C, PP 1-31.
7. Transportation Research Board 1994, Third International Conference on Managing Pavements, National Academy Press, Washington D.C.
8. Y.H Huang, 1993. Pavement Analysis and Design, Prentice-Hall, Inc. a Paramount Communication Company, Englewood, New Jersey ,USA pp406 and 473.
9. Shahin M Y , Darter M. , Star D, Kohn. 1986 "Evaluation of airfield pavement condition and determination of rehabilitation needs' in Transportation Research Record 1680, TRB, National Research Council, Washington D.C., pp 1-20.

10. Graham Smith and Clell Harral, 1987. Road deterioration in developing countries, in Transportation Research Record 1128, TRB, National Research Council, Washington D.C., pp 28-35.
11. B.B. Reddy and A.Veeraragavan, 1997. Structural Performance of in service flexible pavements., in Journal of Transportation Engineering, ASCE, vol. 122,no.1, USA, Pp.156-167.
12. Zaniewski J., and Mamlouk M., 1999 . " Pavement Preventive Maintenance , Key to quality highway "in Transportation Research Record 1680, TRB, National Research Council, Washington D.C., pp 26-29.
13. R. Gary Hicks, J.S.Moulthrop, and J. Dalieden, 1999. Selecting preventive maintenance for flexible pavement. In Transportation Research Record 1025, TRB, National Research Council, Washington D.C., pp 1-12.
14. Eltahan A.A, Dalieiden F. J, and Simpson L. A. Effectiveness of maintenance treatment of flexible pavement. in Transportation Research Record 1680, TRB, National Research Council, Washington D.C., pp 19-25.
15. Wijk, AJ and Sadzik. E,1998. Use of axle load information in pavement management systems, in Proceeding 4th International Conference on Managing Pavements, 4, International Convention Center-ICC, Durban, South Africa, PP 84-96.
16. Paine, D. (1998). The incorporation of structural data in a pavement management System. Proceeding 4th International Conference on Managing Pavements, 4, International Convention Center-ICC, Durban, South Africa, PP 97-113.
17. Hass , R., Hudson, W.R. and Zaniewski, J., (1993). Modern pavement management System. Krieger Publishing, Florida. USA.
18. Hass , R., (1995). Principles and applications of pavement management. Proceeding 2nd International Conference on Roads and Airfield Pavement Technology. Vol 1 , Singapore.
19. Paine, D. (1998). The incorporation of structural data in a pavement management System. Proceeding 4th International Conference on Managing Pavements, 4, International Convention Center-ICC, Durban, South Africa, PP 97-113.
20. Hajek, J.J ,Musgrove G and kazmeirowski T. J , 1998. Measurement management and utilization of rutting data. In Proceeding 4th International

Conference on Managing Pavements, 4, International Convention Center -ICC, Durban, South Africa, PP 226-237.

21. Hand , A.J., Sebaaly, P.E., and Epps J. (1998) " Development of performance models based on department of transportation pavement management system data" in Transportation Research Record 1684, TRB, National Research Council, Washington D.C., pp 215-222.
22. Wotring, Donald C., Baladi, Gilbert Y., Buch Neeraj, and Bower Steve (1998) " Pavement Distress and Selection of Rehabilitation Alternatives" in Transportation Research Record 1629, TRB, National Research Council, Washington D.C., pp 214-225.
23. Dixon E, O'brien III, Kohn S.D, Shahin, M Y 1983 . Prediction of pavement performance using nondestructive test results. in Transportation Research Record 943, TRB, National Research Council, Washington D.C., pp 13-17.
24. Sameh M. Zaghloul and , J.Brian Kerr, 1999 . Reduced rehabilitation cost from use of Falling Weight Deflectometer. In Transportation Research Record 1655, TRB, National Research Council, Washington D.C., pp 16-23.
25. Daly, B 2004 . A review of the prediction of pavement remaining life. Proceeding 6th International Conference on Managing Pavements, Brisbane Convention &Exhibition Center, Queensland, Australia. Paper #174.
26. Archilla A.R., madanat S. " Statistical Model of pavement rutting in asphalt concrete mixes "in Transportation Research Record 1764, TRB, National Research Council, Washington D.C., pp 71-77.
27. Ningyuan L., kazmierowski T., and sharma B., " Verification of network –level pavement roughness measurements". in Transportation Research Record 1764, TRB, National Research Council, Washington D.C., pp 128-138.
28. Shafizadeh, k. ,F. Manerring , and L.M Pierce (2002). A statistical Analysis of Factors associated with Driver-perceived road Roughness on Urban highways. Washington state department of transportation, WA-Rd 538.1, Olympia, Washington.
29. Pierce , L.M Sivaneswaran, N., Willoughby, K, and Mahoney, J. 2004. Impact of Smoothness on Hot Mix Asphalt Pavement Performance in Washington State. Proceeding 6th International Conference on Managing Pavements, Brisbane Convention &Exhibition Center, Queensland, Australia.

30. Kelvin C.P Wang, Weiguo Gong, Robert P Elliott 2004. A Feasibility Study on data automation for comprehensive pavement condition survey. Proceeding 6th International Conference on Managing Pavements, Brisbane Convention &Exhibition Center, Queensland, Australia. Paper #174.
31. Rohd, T.G, Fritz J., Elizabeta S. and Theinis. H. (1998). The calibration and use of the HDM-IV Performance models in a PMS. Proceeding 4th International Conference on Managing Pavements, 4, International Convention Center-ICC, Durban, South Africa, PP 1312-1332.
32. Martin, T., and Roberts, J., 1996. Recommendation for monitoring pavement performance. ARRB Transport Research ARR 293, AARB TR Vermont South, Victoria, Australia, Pp69.
33. AUSTROAD, 1994. AUSTROAD Pavement design guide –Interim Version of Revised Overlay Design Procedures. AUSTROAD Pavement Research Group, APRG, Report No. APRG 94/10(DA), ARRB Transport Research, VIC, Australia.
34. AUSTROAD, 1987a). Guide to the Visual Assessment of Pavement Condition. AUSTROAD Pavement Research Group, APRG, ARRB Transport Research, VIC, Australia.
35. AUSTROAD, 1997. Pavement Design, A Guide to the Structural Design of Road Pavement . AUSTROAD Pavement Research Group, APRG, Report No. APRG 94/10(DA), ARRB Transport Research, VIC, Australia.
36. Paine, D. (1998). The incorporation of structural data in a pavement management System. Proceeding 4th International Conference on Managing Pavements, 4, International Convention Center -ICC, Durban, South Africa, PP 97-113.
37. Pologruto M., " Procedure for use of Falling weight Deflectometer to determine AASHTO layer coefficients" in Transportation Research Record 1629, TRB, National Research Council, Washington D.C., pp 11-19.
38. Hajek, J.J ,Musgrove G and kazmeirowski T. J , 1998. Measurement, management and utilization of rutting data. In Proceeding 4th International Conference on Managing Pavements, 4, International Convention Center-ICC, Durban, South Africa, PP 226-237.
39. Stolle. E Dieter, and F.W.Jung.,1994 " Simplified, rational Approach to Falling weight Deflectometer data interpretation" in Transportation Research Record 1355, TRB, National Research Council, Washington D.C., pp 82-89.

40. Visser AT, Kannemeyer. L and Tekie S. 1998. Experience with roughness measurements for pavement management in southern Africa. In Proceeding 4th International Conference on Managing Pavements, 4, International Convention Center -ICC, Durban, South Africa, PP 238-253.
41. VonQuintis, H. l, Altahan A., and Yau A., “Smoothness model for hot- mix asphalt –surfaced pavements” in Transportation Research Record 1764, TRB, National Research Council, Washington D.C., pp 139-156.
42. Shahin M.Y.(1998) Pavement Management for Airports, Roads, and Parking lots.. Third Edition, Chapman and Hall, kluwer Academic Publisher, Norwell, Massachusetts, USA.
43. Jayawickrama . P W. , R . Prassanna and S. P Senadheera, 1996 . Survey of state practices to control skid resistance on Hot-Mix asphalt Concrete pavements. in Transportation Research Record 1536, TRB, National Research Council, Washington D.C., pp 52-58.
44. Anderson. D. A, W. Meyers and J. L. Rosenberger, 1986. Development of procedure for correcting skid resistance measurements t a standard end of season value. in Transportation Research Record 1084, TRB, National Research Council, Washington D.C., pp 40-48.
45. Skerritt , D.H., 1993. Aggregate type and traffic volume as a controlling factors in bituminous pavement friction. in Transportation Research Record 1418, TRB, National Research Council, Washington D.C., pp 22-29.
46. Judith B Corley Lay, 1998. Friction and surface texture characterization of 14 pavement test sections in Greenville, north Carolina. in Transportation Research Record 1639, TRB, National Research Council, Washington D.C., pp 155-161.
47. Shahin M.Y.(1998) Pavement Management for Airports, Roads, and Parking lots.. Third Edition, Chapman and Hall, kluwer Academic Publisher, Norwell, Massachusetts, USA.
48. Carey ,Jr, W, and Irick, P,e. (1975) The pavement serviceability performance concept” Highway Research Bulletin, Vol.250,HRB,Washington,DC.,40-58.
49. Chiu Liu and Robert herman,1996. New approach to roadway performance indices. In Journal of Transportation Engineering, Vol. 122 , No.5 Pp(329-335).
50. M.Y Shahin , and J A . Walter, 1990. Pavement Maintenance management for Roads and Streets using PAVER system. Technical Report M-90/05.

Construction Engineering Research Laboratory. US Army Corps of Engineers, Champaign, Illinois.

51. Ministry of Transportation and communication. 1982. Manual for condition rating of flexible pavements. Ontario, Canada. .
52. Ministry of Transportation and communication. 1980. Pavement Design & Rehabilitation Manual. Ontario, Canada..
53. The Asphalt Instritute,1983. Asphalt overlays for highway and street rehabilitation, Manual Series No. 17 (MS-17), Maryland, U.S.A. pp16-17.
54. U S Department of Transportation, Federal Highway Administration 1998. Long Term Pavement Performance Program-LTPP, The Next Decade" Publication no. FHWA-RD-98-109, USA.
55. The Asphalt Instritute,1983. Asphalt overlays for highway and street rehabilitation, Manual Series No. 17 (MS-17), Maryland, U.S.A. pp16-17.
56. PMS Unit (2006). Maintenance budget report for the year 2007. Road Department, Dubai Municipality, Dubai, pp.2-10.

Chapter 2

1. Sayers, M.W, t.D .Gillespie and W.D.O. Paterson (1986). Guidelines for conducting and calibration of roads roughness. Technical Report No. 46, World Bank, Washington. D.C.
2. International Standards Organization (ISO) . Mechanical vibration -Road Surface Profiles-Reporting of Measured Data. Organization for Standardization ,1995.
3. M.Y Shahin , and J A . Walter, 1990. Pavement Maintenance management for Roads and Streets using PAVER system. Technical Report M-90/05. Construction Engineering Research Laboratory. U.S Army Corps of Engineers, Champaign, Illinois..
4. Shahin M.Y.(1998) Pavement Management for Airports, Roads, and Parking lots. Third Edition, Chapman and Hall, kluwer Academic Publisher, Norwell, Massachusetts, USA..
5. Y.H Huang, 1993. Pavement Analysis and Design, Prentice-Hall, Inc. a Paramount Communication Company, Englewood, New Jersey ,USA pp9-11.
6. Y.H Huang, 1993. Pavement Analysis and Design, Prentice-Hall, Inc. a Paramount Communication Company, Englewood, New Jersey ,USA p547.
7. M.Y Shahin , and J A . Walter, 1990. Pavement Maintenance management for Roads and Streets using PAVER system. Technical Report M-90/05. Construction Engineering Research Laboratory. U.S Army Corps of Engineers, Champaign, Illinois.
8. Texas Research and development incorporated(TRDI). Technical report, PMS Unit –roads department –Dubai municipality -2001
9. Shahin 1989 and Shahin 1998
10. Cheetam ,A , 1998. Auto-adoptative pavement performance prediction methodology. In Proceeding 4th International Conference on Managing Pavements, 4, International Convention Centre-ICC, Durban, South Africa, PP 552-567.
11. A.C. Collop, D. Cebon, and M. S. A. Hardy, 1995. “ Viscoelastic Approach To Rutting in Flexible Pavement “, Journal of Transportation Engineering, Vol. 121, No. 1, pp. 82-93.

12. Anderson, D. A., R. Luhr, and C. E. Entle,1990. " Framework for Development of Performance – related Specifications for Hot – Mix Asphaltic Concrete," NCHRP Report No. 332, Transportation Research Board.
13. David K. H. Chua,1996. " Framework for PMS Using Mechanistic Distress Submodels ", Journal of Transportation Engineering, Vol. 122, No. 1, pp. 82-93.
14. Hisham A. Ali and Shiraz D. Tayabji, 1997. " Evaluation of Mechanistic – Empirical Performance Prediction models for Flexible Pavement ", Transportation Research Record.
15. Kenis, W. J., Sherwood, J. A., and McMahon, T. F.,1982 "Verification and Application of the VESYS Structural Subsystem", 5th International Conference on the Structural Design of Asphalt Pavement, the Delft University of Technology, The Netherlands, Proc. Vol. 1, pp 333-346.
16. Manuel Ayres Jr. and Matthew W. Witczak, 1998. " AYMA – A Mechanistic Probabilistic System to Evaluate Flexible Pavement Performance", Transportation Research Record, Paper Prepared for Presentation at 77th Annual Meeting.
17. M. W. Witczak. And O. A. Fonseca,1996. " Revised Predictive Model for Dynamic (Complex) Modulus of Asphalt Mixture ", Transportation Research Record, Paper Prepared for Presentation.
18. Paul, N. Khosla, and Sachiyo Kawaguchi,2000. " Comparative Evaluation of Design and Performance of Marshal and Superpave Mixes ", Transportation Research Record, National Research Council, Washington. D. C., Paper Prepared for Presentation.
19. Powell, W.D, et al, 1984. "The Structural Design Bituminous Pavement", TRRL Laboratory Report 1132, Transportation and Road Research Laboratory, U.K.
20. S. Nazarian, D. Yaun and V. Tandom,1998. " Structural Field Testing of Flexible Pavement Layers with Seismic Methods for Quality Control ", Transportation Research Record, Paper Prepared for Presentation.
21. Thompson, M. R., Dempseyy, J., Hill, H. and Vogel, J.,1987. "Characterizing Temperature Effects For Pavement Analysis and Design", Transportation Research Record 1121, Transportation Research Board, pp14-22.

22. Austroads project No. T&E.P.N.023,2001. “2001 Austroads pavement design guide (final draft), pp70.
23. Wiffin, A.C., and Lister, N.W., 1962. “The Application of Elastic Theory to Flexible pavement,” 1st International Conference on The Structural Design of Asphalt Pavements, The University of Mich., , pp. 499-521.

Chapter 3

1. AASHTO (1993). AASHTO Guide for the Design of Pavement Structures. American Association of State Highway and Transportation Officials, Washington, D.C, PP 87-91.
2. Youder, J and Witczak, M. principle of pavement design, Jone Wiley and sons Inc, 1975.
3. K. P. George, A. S. Rajagopal, and L. K. .Lim, 1989. Models for predicting pavement deterioration. In Transportation Research Record 1215. TRB, National Research Council, Washington D.C., pp 1-7.
4. M.Y Shahin, and J. M. Becker, 1984. Development of performance prediction models for airfield pavement. In Transportation Research Record 985. TRB, National Research Council, Washington D.C., pp 25-33.
5. Y. H Huang, 1993. Pavement Analysis and Design, Prentice-Hall, Inc. a Paramount Communication Company, Englewood, New Jersey ,USA p282-285.
6. Turki I. Al suleiman, Nasser A. Saeed, Adnan M. Shiyab and Sultan Al Khafaili, 1999. Maintenance and rehabilitation of Low volume trafficked Roads, case study in Dubai, Jordan 2nd civil engineering international conference, Amman, Jordan.
7. Geometric Design Manual for Dubai roads, 1999. roads department, Dubai Municipality, UAE.
8. AUSTROADS (1988). Roadway capacity - a guide to traffic engineering practice, part 2, Sydney, AP - 11.2/88.
9. AUSTROADS (1992). Pavement design - a guide to the structural design of road pavements, Sydney, AP - 17/92.
10. Federal Highway Administration , Highway Statistics Division, USA reproduced from Thickness Design-Asphalt Pavements for highways and streets, Manual Series No.1, the Asphalt Institute, College park, 1981.
11. Zhanmin Zhang, Joseph P. Leidy , Izydor Kawa, and Ronald Hudson, 2000. Impact of changing traffic characteristics and environmental condition on flexible pavement.. In Transportation Research Record 1730. Paper no.00-1218, TRB, National Research Council, Washington D.C., pp 125-131.
12. Hudson,S.W.S. B.Seed, F.N.Finn and R.F Carmichael, 1988. Evaluation of the increased pavement loading. Volume I-research results and findings. Final report, FHWA-AZ88-219,.I ARE Inc., Austin TX,:Arizona Department of transportation, Phoenix, Za; FHWA USA Department Of Transportation.

13. Turki Al Suleiman, Adnan M Shiyab, and Hashem R .Al Massaied. 1996 . Prediction of airfield pavement deterioration , in the Road and Transport Research Journal, Volume 5, No. 3, Australia.
14. S.M. Herrin, M.I Darter E. J. Barenberg and M.Y Shahin,1986. Airfield pavement condition evaluation and management at Dulles international airport. In Transportation Research Record 1060. TRB, National Research Council, Washington D.C., pp 53-60.
15. Wijk, AJ and Sadzik. E,1998. Use of axle load information in pavement management systems, in Proceeding 4th International Conference on Managing Pavements, 4, International Convention Center- ICC, Durban, South Africa, PP 84-96.
16. S.H. Shiau and H.S.YU (2000). Load and displacement prediction for shakedown analysis of layered pavement. . in Transportation Research Record 1730, TRB, National Research Council, Washington D.C., pp 117-124.
17. AASHTO (1993). AASHTO Guide for the Design of Pavement Structures. American Association of State Highway and Transportation Officials, Washington, D.C, PP 87-91
18. Watanatada, T et al 1987. The highway design and maintenance standards model, volume 1 and 2. the john Hopkins university press, Baltimore, USA.
19. Y. H Huang, 1993. Pavement Analysis and Design, Prentice-Hall, Inc. a Paramount Communication Company, Englewood, New Jersey ,USA p282-285.

20. AASHTO (1993). AASHTO Guide for the Design of Pavement Structures. American Association of State Highway and Transportation Officials, Washington, D.C, PP 87-91.
21. H Huang, 1993. Pavement Analysis and Design, Prentice-Hall, Inc. a Paramount Communication Company, Englewood, New Jersey ,USA p282-285.
22. Jayawickrama . P W. , R . Prassanna and S. P Senadheera, 1996 . Survey of state practices to control skid resistance on Hot-Mix asphalt Concrete pavements. in Transportation Research Record 1536, TRB, National Research Council, Washington D.C., pp 52-58.
23. Zaniewski J., and Mamlouk M., 1999 . “ Pavement Preventive Maintenance , Key to quality highway “in Transportation Research Record 1680, TRB, National Research Council, Washington D.C., pp 26-29.

24. Paterson, D. E. 1985. NCHRP synthesis of highway Practice 122: Life cycle cost analysis for pavement. TRB, National Research Council, Washington, D,C.
25. R. Hassan, K McManus, and J. Holden, 1999. Predicting pavement deterioration modes using waveband analysis. In Transportation Research Record 1652. TRB, National Research Council, Washington D.C., pp 181-187.

Chapter 4

1. Shahin, M Y, and Kohn. S.D. (1981) . Pavement maintenance management for roads and parking lots, Champaign, Illinois, Construction engineering research laboratory, Technical Report , M-294. USA.
2. Shahin M.Y.(1998) Pavement Management for Airports, Roads, and Parking lots. Third Edition, Chapman and Hall, kluwer Academic Publisher, Norwell, Massachusetts, USA.
3. AASHTO (1993) AASHTO Guide for the Design of Pavement Structures. American Association of State Highway and Transportation Officials, Washington, D.C, PP 1-31.
4. PMS Unit (2003). Maintenance budget report for the year 2003. Road Department, Dubai Municipality, Dubai, pp.2-10.
5. M.Y Shahin , and J A . Walter, 1990. Pavement Maintenance management for Roads and Streets using PAVER system. Technical Report M-90/05. Construction Engineering Research Laboratory. US Army Corps of Engineers, Champaign, Illinois.
6. M.Y Shahin , and J A . Walter, 1990. Pavement Maintenance management for Roads and Streets using PAVER system. Technical Report M-90/05. Construction Engineering Research Laboratory. US Army Corps of Engineers, Champaign, Illinois.
7. Shahin M.Y.(1998) Pavement Management for Airports, Roads, and Parking lots. Third Edition, Chapman and Hall, kluwer Academic Publisher, Norwell, Massachusetts, USA.
8. Carey ,Jr, W, and Irick, P,e. (1975) The pavement serviceability performance concept" Highway Research Bulletin, Vol.250,HRB,Washington,DC.,40-58.
9. Y.H Huang, 1993. Pavement Analysis and Design, Prentice-Hall, Inc. a Paramount Communication Company, Englewood, New Jersey ,USA. pp426 and 427.
10. Zaniewski, J.P., W R Hudson, R.High, and S.W Hudson,1985. Pavement rating procedures, contract No.DTFH61-83C-00153,Federal Highway Administration.
11. Carey ,Jr, W, and Irick, P,e. (1975) The pavement serviceability performance concept" Highway Research Bulletin, Vol.250,HRB,Washington,DC.,40-58.

12. Shahin M.Y.(1998) Pavement Management for Airports, Roads, and Parking lots.. Third Edition, Chapman and Hall, kluwer Academic Publisher, Norwell, Massachusetts, USA.
13. Sayers, M W and Thomas D.Gillespi and William D.O Paterson. 1986. Guidelines for conducting and calibrating road roughness measurements. World Bank Technical Report No. 46, the World Bank, Washington D.C .USA. PP 1-87.
14. FHWA, 1987b. Highway performance monitoring system ,field manual for the continuous analytical and statistical database, Appendix JFHWA order M5600.1AOMB No.2125-0028,Federal highway administration.
15. Y.H Huang, 1993. Pavement Analysis and Design, Prentice-Hall, Inc. a Paramount Communication Company, Englewood, New Jersey ,USA pp436 -437.
16. Shahin M.Y.(1998) Pavement Management for Airports, Roads, and Parking lots.. Third Edition, Chapman and Hall, kluwer Academic Publisher, Norwell, Massachusetts, USA.
17. Sayers, M W and Haramihas, S.M 1996. the little book of profiling. Basic information and interpreting road profile. UMTR, Ann Arbor, USA.
18. Waheed Uddin and Victor Torres-Verdin. 1998. Service life analysis for managing road pavement in Mexico. Proceeding 4th International Conference on Managing Pavements, 4, International Convention Center-ICC, Durban, South Africa, PP 882-898.
19. Austraods, 2000. Guidelines for Roads Condition Monitoring, Part 1, Pavement Roughness. Sydney, Australia.
20. B.B Reddy, N. Sivath Kumar and A.Veeraragavan.1998. Development of maintenance intervention criteria for in-service flexible pavement. Proceeding 4th International Conference on Managing Pavements, 4, International Convention Center-ICC, Durban, South Africa, PP 986-997.
21. Paterson, W.D. and Ottoh –Okine, B..1992. Simplified models of paved road deterioration based on HDM-III. Paper presented at the 71st Annual meeting of transportation Research Board, Washington, D.C.
22. Australian Road Reaserach Board (ARRB),1996 TR, research report ARR 279. ,pp67. (AARB Vermont South, Vic)
23. B.B Reddy, N. Sivath Kumar and A.Veeraragavan.1998. Development of maintenance intervention criteria for in-service flexible pavement. Proceeding 4th

International Conference on Managing Pavements, 4, International Convention Center-ICC, Durban, South Africa, PP 986-997.

24. Shahin M.Y.(1998) Pavement Management for Airports, Roads, and Parking lots.. Third Edition, Chapman and Hall, kluwer Academic Publisher, Norwell, Massachusetts, USA.
25. Patterson, WDO. (1986) International Roughness Index: Relationship to other measures of roughness and ride quality. Transport Research Record 1084. Transportation Research Board-TRB, Washington. D.C., pp. 397-406.
26. Charles E Dougan, Lisa Aultman, Soon Nam, Bradley Overturf and Christine Hobson,2003. Variation of pavement smoothness between adjacent lanes, implications for performance based contracting, TRB annual meeting, TRB, Washington Dc.
27. Chiu Liu and Robert herman,1996. New approach to roadway performance indices. In Journal of Transportation Engineering, Vol. 122 , No.5 Pp(329-335).
28. AASHTO (1993) AASHTO Guide for the Design of Pavement Structures. American Association of State Highway and Transportation Officials, Washington, D.C, PP III-30-III-44.
29. S. kameyama, A.kasahara,K.Himeno and T.Ikeda 1998. Optimization of maintenance and rehabilitation strategy using structural capacity index and surface distress index. In Proceeding 4th International Conference on Managing Pavements, 4, International Convention Center-ICC, Durban, South Africa, PP 839-853.
30. Michael Pologruto., 2001. Procedure for use of falling weight deflectometer to determine the AASHTO layer coefficients. In Transportation Research Record 1764, TRB, National Research Council, Washington D.C., pp 11-20.
31. AASHTO (1993) AASHTO Guide for the Design of Pavement Structures. American Association of State Highway and Transportation Officials, Washington, D.C, PP III-30-III-44.
32. Shahin M.Y.(1998) Pavement Management for Airports, Roads, and Parking lots.. Third Edition, Chapman and Hall, kluwer Academic Publisher, Norwell, Massachusetts, USA.
33. Charles E Dougan, Lisa Aultman, Soon Nam, Bradley Overturf and Christine Hobson,2003. Variation of pavement smoothness between adjacent lanes,

implications for performance based contracting, TRB annual meeting, TRB, Washington Dc.

34. Jameson, G. W. 1992. Development of Philippines asphalt overlay procedure-Pavement management system. Technical Assistance project TA 1426-PHL. Asian Development Bank 1992.
35. Rohde, G.W, 1994. Determining structural number from FWD testing. In Transportation Research Record 1448,TRB, National Research Council, Washington, D.C., pp61-68.
36. AASHTO (1993) AASHTO Guide for the Design of Pavement Structures. American Association of State Highway and Transportation Officials, Washington, D.C, PP III-30-III-44.
37. Jayawickrama . P W. , R . Prassanna and S. P Senadheera, 1996 . Survey of state practices to control skid resistance on Hot-Mix asphalt Concrete pavements. in Transportation Research Record 1536, TRB, National Research Council, Washington D.C., pp 52-58.
38. Anderson. D. A, W. Meyers and J. L. Rosenberger, 1986. Development of procedure for correcting skid resistance measurements t a standard end of season value. in Transportation Research Record 1084, TRB, National Research Council, Washington D.C., pp 40-48.
39. Skerritt , D.H., 1993. Aggregate type and traffic volume as a controlling factors in bituminous pavement friction. in Transportation Research Record 1418, TRB, National Research Council, Washington D.C., pp 22-29.
40. Odoki,JB, and kerali,H.R (2000). Highway Development and Management (HDM-4).Volume4-HDM-4,Technical Reference Manual Guide, PIARC, Paris.
41. Anderson. D. A, W. Meyers and J. L. Rosenberger, 1986. Development of procedure for correcting skid resistance measurements t a standard end of season value. in Transportation Research Record 1084, TRB, National Research Council, Washington D.C., pp 40-48.

Chapter 5

1. S. kameyama, A.kasahara,K.Himeno and T.Ikeda 1998. Optimization of maintenance and rehabilitation strategy using structural capacity index and surface distress index. In Proceeding 4th International Conference on Managing Pavements, 4, International Convention Center-ICC, Durban, South Africa, PP 839-853.
2. Fwa, T. F., Sinha K.C and reverson J., 1988. Highway routine maintenance programming at network level. Journal of transportation Engineering,ASCE,Vol.114, No.5 pp539-554.
3. Feighan k.J., Shahin M Y and Sinha K.C,1987. A dynamic approach to optimization for pavement management system. Proceeding Second American Conference o managing pavements. Sponsored by Ontario Ministry of Transportation and US federal highway Administration Vol.2 Pp195-206.
4. Texas Research and Development Incorporated(TRDI). Technical Report, PMS Unit –Roads Department –Dubai municipality -2001.
5. T.F. Fwa, W.T. Chan and K Z hoque (1998). Network level programming for pavement management using generic algorithms. In Proceeding 4th International Conference on Managing Pavements, 4, International Convention Center -ICC, Durban, South Africa, PP 824-825.
6. Neil C. Krutz and Mary Stroup-Gardiner, 1993. Relationship between permanent deformation of asphalt concrete pavement and moisture sensitivity. In Transportation Research Record: Journal of the Transportation Research Board, No.1259 TRB, National Research Council, Washington D.C., pp 181-187.
7. Hans Ertman Larsen and per ullidtz (1998). Development of improved mechanistic deterioration models for flexible pavement. In Proceeding 4th International Conference on Managing Pavements, 4, International Convention Center -ICC, Durban, South Africa, PP 583-596.
8. Paterson, W.D. and Ottoh –Okine, B.1992. Simplified models of paved road deterioration based on HDM-III. Paper presented at the 71st Annual meeting of transportation Research Board, Washington, D.C.
9. ARRB 1996, TR, research report ARR 279. March, PP67. AARB Vermont South, Vic, Australia.

10. Hans Ertman Larsen and per ullidtz (1998). Development of improved mechanistic deterioration models for flexible pavement. In Proceeding 4th International Conference on Managing Pavements, 4, International Convention Center -ICC, Durban, South Africa, PP 583-596.
11. ERES Consultants, 2004. Rolling Weight Deflectometer (RWD) demonstration and comparison to other devices in Texas. Texas Department of Transportation & Federal Highway Administration (FHWA) . Final Report, Project No. 15874. Austin, TX, USA. PP 1-3.
12. P. Ullidtz. and R.N.Stubstad(1990) . Why bother about layer Moduli. Third International Conference on Bearing Capacity of Roads and Airfields. Thondheim, Norway. Pp 1-9.
13. Jameson, G. W. 1992. Development of Philippines asphalt overlay procedure-Pavement management system. Technical Assistance project TA 1426-PHL. Asian Development Bank 1992.
14. Rohde, G.W, 1994. Determining structural number from FWD testing. In Transportation Research Record 1448, TRB, National Research Council, Washington, D.C., pp61-68.
15. AASHTO (1993) AASHTO Guide for the Design of Pavement Structures. American Association of State Highway and Transportation Officials, Washington, D.C, PP III-30-III-44.
16. Stefan Romanoschi, and John. B.Metcalf. Simple approach to estimation of the pavement structural Capacity. In Transportation Research Record: Journal of the Transportation Research Board, No.1652. TRB, National Research Council, Washington D.C., 1999, pp 198-205.
17. Per Ullidtz. . Deterioration modes for managing flexible pavements. In Transportation Research Record: Journal of the Transportation Research Board, No.1655. TRB, National Research Council, Washington D.C., 1999, pp 181-187.
18. Anderson. D. A, W. Meyers and J. L. Rosenberger, 1986. Development of procedure for correcting skid resistance measurements t a standard end of season value. In Transportation Research Record 1084, TRB, National Research Council, Washington D.C., pp 40-48.
19. Hass, R., W.R.Hudson, and J.zaniewski. Modern pavement management .Kriegler publishing company, Malabar, Fla., 1994.

Chapter 6

1. AASHTO (1993) AASHTO Guide for the Design of Pavement Structures. American Association of State Highway and Transportation Officials, Washington, D.C, PP 1-31.
2. T.S.Vepa, K.P.George, and A. Raja Shekhran. Prediction of pavement remaining life. In Transportation Research Record: Journal of the Transportation Research Board, No.1524. TRB, National Research Council, Washington D.C., 1996, pp 137-144.
3. Dixon E.O'brien III, Starr D.kohn and Mohamed .Y Shahin 1983. Prediction of pavement performance using non-destructive testing result. In Transportation Research Record: Journal of the Transportation Research Board, No.943. TRB, National Research Council, Washington D.C., 1999, pp 13-16.
4. Xin Chen, Stuart Hudson, Gaylord Cumberledge and Eric Perrone 1995. Pavement performance modelling program for Pennsylvania. In Transportation Research Record: Journal of the Transportation Research Board, No.1508. TRB, National Research Council, Washington D.C., 1995, pp 1-9.
5. R.M. Anderson, McGennis.R.B, T. Kennedy, and m. Solaimanian 1995. Background of superpave asphalt mixtures design and analysis. National Asphalt Training Centre, Demonstration Project 101. Federal Highway Administration (FHWA), Report No. FHWA-SA-94-069, Washington, D.C. USA.
6. AASHTO (1993) AASHTO Guide for the Design of Pavement Structures. American Association of State Highway and Transportation Officials, Washington, D.C, pp84-85).
7. T.S.Vepa,K.P.George, and A.Raja Shekhran. Prediction of pavement remaining life. In Transportation Research Record: Journal of the Transportation Research Board, No.1524. TRB, National Research Council, Washington D.C., 1996, pp 137-144.
8. AASHTO (1993) AASHTO Guide for the Design of Pavement Structures. American Association of State Highway and Transportation Officials, Washington, D.C, Appendix M, ppM1-M9.
9. AASHTO (1993) AASHTO Guide for the Design of Pavement Structures. American Association of State Highway and Transportation Officials, Washington, D.C, PP 1-31.

10. Hass, R., Hudson, W .R. and Zaniewski, J. ,1993. Modern pavement management .Krieger Publishing, Florida, USA.
11. PAINE D., “The incorporation of structural data in a Pavement Management System”, proceedings of the 4th international Conference on Managing Pavements, Vol. 1, Durban, South Africa, pp. 97-114,1998.
12. Hass, R., Hudson,1995. Principles and Application of Pavement Management. Proceeding of the 2nd international Conference on Roads and airfield Pavement Technology, Vol. 1, Singapore.
13. PAINE D., “The incorporation of structural data in a Pavement Management System”, proceedings of the 4th international Conference on Managing Pavements, Vol. 1, Durban, South Africa, pp. 97-114,1998.
14. Austraods 1992. Pavement design, A Guide to the structural Design of Roads pavements. Sydney, Australia.
15. Statistical Package for Social Science (SPSS), 2003. Version 11, SPSS Inc. Headquarters, 233 S. Wacker Drive, Chicago, Illinois 60606,USA.
16. Paine D.,1998. “The incorporation of structural data in a Pavement Management System”, Proceedings of the 4th international Conference on Managing Pavements, Vol. 1, Durban, South Africa, pp. 97-114.
17. Austraods 1992. Pavement design, A Guide to the structural Design of Roads pavements. Sydney, Australia.
18. Paine D.,1998. “The incorporation of structural data in a Pavement Management System”, Proceedings of the 4th international Conference on Managing Pavements, Vol. 1, Durban, South Africa, pp. 97-114.
19. Blight, G.E 1974. Permanent deformation in asphalt materials. Journal of Transportation engineering. ASCE,100 (1), pp 263-278.
20. Lister N.W 1972. Deflection criterion for flexible pavements and design of overlays. Proceeding, 3rd International Conference on Structural Design of Pavement, University of Michigan, Ann Arbor, Mich., 1206-1226.
21. Lister N.W. and Kennedy, C.K 1977. A system for the prediction of pavement life and design of pavement strengthening. Proceeding, 4th International Conference on Structural Design of Pavement. , University of Michigan, Ann Arbor, Mich., pp 629-648.

22. Nagumo, S., Takiunari, M., and Taniomoto, S. 1972. Deflection criteria for asphalt pavement. Proceeding, 3rd International Conference on Structural Design of Pavement, University of Michigan, Ann Arbor, Mich., 670-691.
23. Pedigo, R. D., Robert, R. L., and Finn, F.N.1982. Distress-performance relationships and overall performance concepts for flexible pavement. Proceeding, 5th International Conference on Structural Design of Pavement, Delft University of Technology., Delft, The Netherlands, pp367-375. , Ann Arbor, Mich., 1206-1226.
24. Veverka ,V.,and romain,J.1982. Pavement evaluation and overlay design. Practical method of Belgian Road Research Center. Proceeding, 5th International Conference on Structural Design of Pavement, Delft University of Technology., Delft, The Netherlands, pp367-375. , Ann Arbor, Mich., 695-706.
25. Corney, D. 1967. Failure criteria for flexible pavements. Proceeding, 2nd International Conference on Structural Design of Pavement, University of Michigan, Ann Arbor, Mich.,608-612.
26. Pedigo, R. D., Robert, R. L., and Finn,F.N.1982. Distress-performance relationships and overall performance concepts for flexible pavement. Proceeding , 5th International Conference on Structural Design of Pavement ,Delft University of Technology., Delft, The Netherlands, pp367-375. , Ann Arbor,Mich.,1206-1226.
27. Veeraragavan, A, Dattareya,J.K.,and Prabhudeva,M.1991. Development of failure criteria for flexible pavement overlays for Indian conditions. IRCJ., New Delhi,India,52-1,pp 183-204.
28. Reddy,K.S.and Pandey,B.B.1993. Analytical design of flexible pavements. PhD thesis, Bangalore University., Bangalore, India.
29. Deacon ,J.A,J.S. Coplants.Taybali,and C.L.Monismith (1994). Temperature consideration in asphalt-aggregate mixture analysis and design. In Transportation Research Record: Journal of the Transportation Research Board, No.1454. TRB, National Research Council, Washington D.C..
30. ERES International, 1991. Asphalt Research Project .Final Investigative Report. Contract R360. Dubai, UAE.
31. Neil C.krutz and Mary Stroup-Gardenier 1993. Relationship between permanent deformation of asphalt concrete and moisture sensitivity. In Transportation

32. Michael S. Janoff 1996. Pavement smoothness, in Information Series No.111. National Asphalt Pavement Association (NAPA). Lanham, MD USA.
33. Zaniewski, J.P., W R Hudson, R.High, and S.W Hudson,1985. Pavement rating procedures, contract No.DTFH61-83C-00153, Federal Highway Administration.
34. Al Suleiman, T., Saeed, N. A., Shiyab, A.M., Al khafaili, S. (1999) Maintenance and rehabilitation of low -volume roads In hot climate, case study In Dubai. Jordan Second Civil Engineering Conference, 1, Amman-Jordan, pp. 81-103.
35. Adnan M. S. Shiyab and Turki I. Al- Suleiman (Obaidat) 2003. Predicting Pavement Remaining Service Life (RSL) using roughness data, Case Study in Dubai. In the International Journal of Pavement Engineering (JOPE). Vol.4 (2), U.K, PP121-129.
36. Patterson, WDO. (1986) International Roughness Index: Relationship to other measures of roughness and ride quality. Transport Research Record 1084. Transportation Research Board-TRB, Washington. D.C., pp. 397-406.
37. AASHTO (1993) AASHTO Guide for the Design of Pavement Structures. American Association of State Highway and Transportation Officials, Washington, D.C, PP III-30-III-44.
38. Adnan M. S. Shiyab and Turki I. Al- Suleiman (Obaidat) 2003. Ppredicting Pavement Remaining Service Life (RSL) using Roughness Data, Case Study in Dubai. In the International Journal of Pavement Engineering (JOPE). Vol.4 (2),U.K , PP121-129.
39. Chiu, L. (2000) Vehicle–road interaction, evolution of roads profiles and present serviceability index. Road Materials and Pavement Design, 1, pp.35-51.
40. Shahin M.Y.(1998) Pavement Management for Airports, Roads, and Parking lots.. Third Edition, Chapman and Hall, kluwer Academic Publisher, Norwell, Massachusetts, USA.

Chapter 7

1. R. Gary Hicks, J.S.Moulthrop, and J. Dalieden, 1999. Selecting preventive maintenance for flexible pavement. In Transportation Research Record 1025, TRB, National Research Council, Washington D.C., pp 1-12.

2. M.Y Shahin , and J A . Walter, 1990. Pavement Maintenance management for Roads and Streets using PAVER system. Technical Report M-90/05. Construction Engineering Research Laboratory. US Army Corps of Engineers, Champaign, Illinois.

3. Shahin M.Y.(1998) Pavement Management for Airports, Roads, and Parking lots.. Third Edition, Chapman and Hall, kluwer Academic Publisher, Norwell, Massachusetts, USA.

4. AASHTO (1993) AASHTO Guide for the Design of Pavement Structures. American Association of State Highway and Transportation Officials, Washington, D.C, PP 1-31.

5. The Asphalt Institute,1983. Asphalt overlays for highway and street rehabilitation, Manual Series No. 17 (MS-17), Maryland, U.S.A. pp16-17.

6. The Asphalt Institute,1983. Asphalt overlays for highway and street rehabilitation, Manual Series No. 17 (MS-17), Maryland, U.S.A. pp16-17.

7. M.Y Shahin , and J A . Walter, 1990. Pavement Maintenance management for Roads and Streets using PAVER system. Technical Report M-90/05. Construction Engineering Research Laboratory. US Army Corps of Engineers, Champaign, Illinois.

8. Shahin M.Y.(1998) Pavement Management for Airports, Roads, and Parking lots.. Third Edition, Chapman and Hall, kluwer Academic Publisher, Norwell, Massachusetts, USA.

9. AASHTO (1993) AASHTO Guide for the Design of Pavement Structures. American Association of State Highway and Transportation Officials, Washington, D.C, PP 1-31.

10. PAINE D., “The incorporation of structural data in a Pavement Management System”, proceedings of the 4th international Conference on Managing Pavements, Vol. 1, Durban, South Africa, pp. 97-114,1998.

Chapter 8

1. Essam Sharaf, Eric Reichelt, Mohammed .Y.Shahin,and Kumares C.Sinha 1987. Development of a methodology to estimate pavement maintenance and repair costs for different ranges of pavement condition index. In Transportation Research Record 1123, TRB, National Research Council, Washington D.C., pp 30-38.
2. Shahin M.Y.(1998) Pavement Management for Airports, Roads, and Parking lots.. Third Edition, Chapman and Hall, kluwer Academic Publisher, Norwell, Massachusetts, USA.
3. R. Gary Hicks, J.S.Moulthrop, and J. Dalieden, 1999. Selecting preventive maintenance for flexible pavement. In Transportation Research Record 1025, TRB, National Research Council, Washington D.C., pp 1-12.

Appendix A

Data Files used in models development (Digital Copy in Excel format attached to the thesis on a CD

Statistical Characteristics and correlation matrices for the developed models

Table A-1: Correlation matrix between various variables pertaining to physical condition indicator included in the study

Correlations

		PQI	Age	Alligator	BLEEDING	IRI	LT Cracking	POTHOLES	RUT	WR	Road Class
PQI	Pearson Correlation	1	-.549**	.509**	.147**	-.026*	.560**	.589**	.370**	.766**	-.226**
	Sig. (2-tailed)	.	.000	.000	.000	.015	.000	.000	.000	.000	.000
	N	8771	8771	8771	8771	8771	8771	8771	8771	8771	8771
Age	Pearson Correlation	-.549**	1	-.105**	-.064**	.017	-.286**	-.304**	-.091**	-.564**	.217**
	Sig. (2-tailed)	.000	.	.000	.000	.120	.000	.000	.000	.000	.000
	N	8771	8771	8771	8771	8771	8771	8771	8771	8771	8771
Alligator	Pearson Correlation	.509**	-.105**	1	.118**	-.006	.082**	.113**	.344**	.178**	.005
	Sig. (2-tailed)	.000	.000	.	.000	.591	.000	.000	.000	.000	.638
	N	8771	8771	8771	8771	8771	8771	8771	8771	8771	8771
BLEEDING	Pearson Correlation	.147**	-.064**	.118**	1	-.004	.003	.032**	.183**	.072**	.016
	Sig. (2-tailed)	.000	.000	.000	.	.724	.811	.003	.000	.000	.127
	N	8771	8771	8771	8771	8771	8771	8771	8771	8771	8771
IRI	Pearson Correlation	-.026*	.017	-.006	-.004	1	-.005	-.015	-.006	-.030**	.080**
	Sig. (2-tailed)	.015	.120	.591	.724	.	.638	.154	.562	.005	.000
	N	8771	8771	8771	8771	8771	8771	8771	8771	8771	8771
LT Cracking	Pearson Correlation	.560**	-.286**	.082**	.003	-.005	1	.273**	.057**	.267**	-.121**
	Sig. (2-tailed)	.000	.000	.000	.811	.638	.	.000	.000	.000	.000
	N	8771	8771	8771	8771	8771	8771	8771	8771	8771	8771
POTHOLES	Pearson Correlation	.589**	-.304**	.113**	.032**	-.015	.273**	1	.058**	.230**	-.163**
	Sig. (2-tailed)	.000	.000	.000	.003	.154	.000	.	.000	.000	.000
	N	8771	8771	8771	8771	8771	8771	8771	8771	8771	8771
RUT	Pearson Correlation	.370**	-.091**	.344**	.183**	-.006	.057**	.058**	1	.133**	.058**
	Sig. (2-tailed)	.000	.000	.000	.000	.562	.000	.000	.	.000	.000
	N	8771	8771	8771	8771	8771	8771	8771	8771	8771	8771
WR	Pearson Correlation	.766**	-.564**	.178**	.072**	-.030**	.267**	.230**	.133**	1	-.269**
	Sig. (2-tailed)	.000	.000	.000	.000	.005	.000	.000	.000	.	.000
	N	8771	8771	8771	8771	8771	8771	8771	8771	8771	8771
Road Class	Pearson Correlation	-.226**	.217**	.005	.016	.080**	-.121**	-.163**	.058**	-.269**	1
	Sig. (2-tailed)	.000	.000	.638	.127	.000	.000	.000	.000	.000	.
	N	8771	8771	8771	8771	8771	8771	8771	8771	8771	8771

**. Correlation is significant at the 0.01 level (2-tailed).

*. Correlation is significant at the 0.05 level (2-tailed).

Table A-2: Statistical characteristics of the models developed for Freeways and Expressways

Correlations

		PQI	Age
PQI	Pearson Correlation	1	-.561**
	Sig. (2-tailed)	.	.000
	N	760	760
Age	Pearson Correlation	-.561**	1
	Sig. (2-tailed)	.000	.
	N	760	760

**. Correlation is significant at the 0.01 level

```
Dependent variable.. PQI                    Method.. QUADRATI

Listwise Deletion of Missing Data

Multiple R            .75136
R Square              .56454
Adjusted R Square     .56339
Standard Error       7.26233

             Analysis of Variance:

                 DF   Sum of Squares      Mean Square

Regression        2        51760.117        25880.058
Residuals       757        39925.233           52.741

F =     490.69731       Signif F =  .0000
```

```
-------------------- Variables in the Equation --------------------

Variable                  B        SE B       Beta         T  Sig T

V35                1.762291     .139478    .983596    12.635  .0000
V35**2             -.122736     .005886  -1.623288   -20.852  .0000
(Constant)        94.701946     .552811              171.310  .0000
_


Dependent variable.. PQI               Method.. CUBIC

Listwise Deletion of Missing Data

Multiple R           .87137
R Square             .75929
Adjusted R Square    .75834
Standard Error      5.40301

            Analysis of Variance:

                DF   Sum of Squares      Mean Square

Regression       3        69615.814        23205.271
Residuals      756        22069.535           29.193

F =     794.90506       Signif F =  .0000

-------------------- Variables in the Equation --------------------

Variable                  B        SE B       Beta         T  Sig T

V35               -3.646811     .242080  -2.035413   -15.064  .0000
V35**2              .403867     .021738   5.341473    18.579  .0000
V35**3             -.013075     .000529  -4.220112   -24.732  .0000
(Constant)       105.725419     .606482              174.326  .0000
_


Dependent variable.. PQI               Method.. COMPOUND

Listwise Deletion of Missing Data

Multiple R           .55455
R Square             .30753
Adjusted R Square    .30662
Standard Error       .14770

            Analysis of Variance:

                DF   Sum of Squares      Mean Square

Regression       1         7.343777        7.3437767
Residuals      758        16.536053         .0218154

F =     336.63309       Signif F =  .0000

-------------------- Variables in the Equation --------------------
```

Variable	B	SE B	Beta	T	Sig T
V35	.984093	.000860	.574328	1144.216	.0000
(Constant)	105.837490	.830443		127.447	.0000

—

Dependent variable.. PQI Method.. POWER

Listwise Deletion of Missing Data

Multiple R	.38693
R Square	.14972
Adjusted R Square	.14859
Standard Error	.16367

Analysis of Variance:

	DF	Sum of Squares	Mean Square
Regression	1	3.575210	3.5752095
Residuals	758	20.304621	.0267871

F = 133.46759 Signif F = .0000

---------- Variables in the Equation ----------

Variable	B	SE B	Beta	T	Sig T
V35	-.076602	.006631	-.386932	-11.553	.0000
(Constant)	106.763652	1.228619		86.897	.0000

—

Dependent variable.. PQI Method.. S

Listwise Deletion of Missing Data

Multiple R	.23632
R Square	.05585
Adjusted R Square	.05460
Standard Error	.17247

Analysis of Variance:

	DF	Sum of Squares	Mean Square
Regression	1	1.333644	1.3336441
Residuals	758	22.546186	.0297443

F = 44.83695 Signif F = .0000

---------- Variables in the Equation ----------

Variable	B	SE B	Beta	T	Sig T
V35	.147140	.021974	.236322	6.696	.0000
(Constant)	4.508548	.009535		472.860	.0000

—

```
Dependent variable.. PQI                Method.. GROWTH

Listwise Deletion of Missing Data

Multiple R           .55455
R Square             .30753
Adjusted R Square    .30662
Standard Error       .14770

            Analysis of Variance:

                DF   Sum of Squares      Mean Square

Regression       1         7.343777        7.3437767
Residuals      758        16.536053         .0218154

F =     336.63309       Signif F =  .0000

-------------------- Variables in the Equation --------------------

Variable                  B        SE B       Beta         T  Sig T

V35                -.016035     .000874   -.554554   -18.348  .0000
(Constant)         4.661905     .007846              594.146  .0000
_
Dependent variable.. PQI                Method.. EXPONENT

Listwise Deletion of Missing Data

Multiple R           .55455
R Square             .30753
Adjusted R Square    .30662
Standard Error       .14770

            Analysis of Variance:

                DF   Sum of Squares      Mean Square

Regression       1         7.343777        7.3437767
Residuals      758        16.536053         .0218154

F =     336.63309       Signif F =  .0000

-------------------- Variables in the Equation --------------------

Variable                  B        SE B       Beta         T  Sig T

V35                -.016035     .000874   -.554554   -18.348  .0000
(Constant)       105.837490     .830443              127.447  .0000
_
```

Table A-3: statistical characteristics of the models developed for Arterials

Correlations

		PQI	Age
PQI	Pearson Correlation	1	-.894**
	Sig. (2-tailed)	.	.000
	N	1157	1157
Age	Pearson Correlation	-.894**	1
	Sig. (2-tailed)	.000	.
	N	1157	1157

**. Correlation is significant at the 0.01 level

Curve Fit

```
MODEL:  MOD_1.
_


Dependent variable.. PQI                Method.. LINEAR

Listwise Deletion of Missing Data

Multiple R           .89399
R Square             .79921
Adjusted R Square    .79904
Standard Error      6.72517

            Analysis of Variance:

                 DF   Sum of Squares      Mean Square

Regression        1        207928.36        207928.36
Residuals      1155         52238.23            45.23

F =    4597.34675       Signif F =  .0000

-------------------- Variables in the Equation --------------------

Variable                   B        SE B       Beta          T  Sig T

V31                -2.103080     .031017   -.893987   -67.804  .0000
(Constant)        106.780192     .277549              384.726  .0000
_


Dependent variable.. PQI                Method.. LOGARITH

Listwise Deletion of Missing Data


Multiple R           .70121
R Square             .49169
Adjusted R Square    .49125
Standard Error     10.70036

            Analysis of Variance:
```

```
                 DF   Sum of Squares      Mean Square

Regression        1        127921.79        127921.79
Residuals      1155        132244.80           114.50

F =    1117.24372       Signif F =  .0000

-------------------- Variables in the Equation --------------------

Variable                   B        SE B       Beta         T  Sig T

V31              -11.751203     .351567   -.701207   -33.425  .0000
(Constant)       110.419025     .594117              185.854  .0000
_


Dependent variable.. PQI               Method.. INVERSE

Listwise Deletion of Missing Data

Multiple R           .39253
R Square             .15408
Adjusted R Square    .15335
Standard Error     13.80381

            Analysis of Variance:

                 DF   Sum of Squares      Mean Square

Regression        1         40087.03        40087.035
Residuals      1155        220079.56          190.545

F =     210.38085       Signif F =  .0000

-------------------- Variables in the Equation --------------------

Variable                   B        SE B       Beta         T  Sig T

V31               18.818723    1.297439    .392533    14.505  .0000
(Constant)        87.030528     .606741              143.439  .0000
_


Dependent variable.. PQI               Method.. QUADRATI

Listwise Deletion of Missing Data

Multiple R           .90597
R Square             .82078
Adjusted R Square    .82047
Standard Error      6.35649

            Analysis of Variance:

                 DF   Sum of Squares      Mean Square

Regression        2        213539.20        106769.60
Residuals      1154         46627.40            40.41
```

```
F =    2642.48338       Signif F =  .0000

-------------------- Variables in the Equation --------------------

Variable                  B        SE B       Beta         T  Sig T

V31                -.904729     .105834   -.384586    -8.549  .0000
V31**2             -.047695     .004047   -.530146   -11.784  .0000
(Constant)       103.073463     .409589              251.651  .0000
_


Dependent variable.. PQI               Method.. CUBIC

Listwise Deletion of Missing Data

Multiple R           .90693
R Square             .82251
Adjusted R Square    .82205
Standard Error      6.32839

            Analysis of Variance:

                 DF   Sum of Squares      Mean Square

Regression        3        213990.70        71330.234
Residuals      1153         46175.89           40.048

F =    1781.09739       Signif F =  .0000

-------------------- Variables in the Equation --------------------

Variable                  B        SE B       Beta         T  Sig T

V31                -.282916     .213068   -.120263    -1.328  .1845
V31**2             -.111243     .019351  -1.236512    -5.749  .0000
V31**3              .001609     .000479    .460163     3.358  .0008
(Constant)       101.805740     .555729              183.193  .0000
_


Dependent variable.. PQI               Method.. COMPOUND

Listwise Deletion of Missing Data

Multiple R           .80047
R Square             .64076
Adjusted R Square    .64044
Standard Error       .15012

            Analysis of Variance:

                 DF   Sum of Squares      Mean Square

Regression        1         46.423087        46.423087
Residuals      1155         26.027414          .022535

F =    2060.08426       Signif F =  .0000
```

```
-------------------- Variables in the Equation --------------------

Variable                  B        SE B       Beta         T  Sig T

V31                 .969064     .000671    .449117  1444.363  .0000
(Constant)       111.496541     .690752              161.413  .0000
_


Dependent variable.. PQI               Method.. POWER

Listwise Deletion of Missing Data

Multiple R           .60262
R Square             .36315
Adjusted R Square    .36260
Standard Error       .19987

            Analysis of Variance:

                 DF   Sum of Squares      Mean Square

Regression        1        26.310475        26.310475
Residuals      1155        46.140026          .039948

F =     658.61686       Signif F =  .0000

-------------------- Variables in the Equation --------------------

Variable                  B        SE B       Beta         T  Sig T

V31                -.168529     .006567   -.602620   -25.664  .0000
(Constant)       116.541427    1.293309               90.111  .0000
_


Dependent variable.. PQI               Method.. S

Listwise Deletion of Missing Data

Multiple R           .32824
R Square             .10774
Adjusted R Square    .10697
Standard Error       .23658

            Analysis of Variance:

                 DF   Sum of Squares      Mean Square

Regression        1         7.805984        7.8059835
Residuals      1155        64.644518         .0559693

F =     139.46907       Signif F =  .0000

-------------------- Variables in the Equation --------------------

Variable                  B        SE B       Beta         T  Sig T

V31                 .262604     .022236    .328241    11.810  .0000
```

```
(Constant)         4.425354     .010399              425.568  .0000
_


Dependent variable.. PQI               Method.. GROWTH

Listwise Deletion of Missing Data

Multiple R           .80047
R Square             .64076
Adjusted R Square    .64044
Standard Error       .15012

            Analysis of Variance:

                 DF   Sum of Squares      Mean Square

Regression        1        46.423087        46.423087
Residuals      1155        26.027414          .022535

F =    2060.08426       Signif F =  .0000

-------------------- Variables in the Equation --------------------

Variable                   B        SE B       Beta         T  Sig T

V31                 -.031424     .000692   -.800472   -45.388  .0000
(Constant)          4.713994     .006195              760.901  .0000
_


Dependent variable.. PQI               Method.. EXPONENT

Listwise Deletion of Missing Data

Multiple R           .80047
R Square             .64076
Adjusted R Square    .64044
Standard Error       .15012

            Analysis of Variance:

                 DF   Sum of Squares      Mean Square

Regression        1        46.423087        46.423087
Residuals      1155        26.027414          .022535

F =    2060.08426       Signif F =  .0000

-------------------- Variables in the Equation --------------------

Variable                   B        SE B       Beta         T  Sig T

V31                 -.031424     .000692   -.800472   -45.388  .0000
(Constant)        111.496541     .690752              161.413  .0000
```

Table A-4: statistical characteristics of the models developed for Collectors

Correlations

		PQI	Age
PQI	Pearson Correlation	1	-.880**
	Sig. (2-tailed)	.	.000
	N	1536	1536
Age	Pearson Correlation	-.880**	1
	Sig. (2-tailed)	.000	.
	N	1536	1536

**. Correlation is significant at the 0.01 level

Curve Fit

```
MODEL:  MOD_2.

Variable:  PQI          Maximum value:           100.00
This variable contains values that are larger than the input upper
bound.
The LGSTIC model cannot be fitted for this variable.
_



Dependent variable.. PQI               Method.. LINEAR

Listwise Deletion of Missing Data

Multiple R           .87975
R Square             .77396
Adjusted R Square    .77381
Standard Error      4.98946

            Analysis of Variance:

                 DF   Sum of Squares      Mean Square

Regression        1        130753.61        130753.61
Residuals      1534         38188.47            24.89

F =    5252.26660       Signif F =  .0000

-------------------- Variables in the Equation --------------------

Variable                   B        SE B       Beta         T  Sig T

V35                -1.254399     .017309   -.879747   -72.473  .0000
(Constant)        105.078160     .191416              548.952  .0000
_



Dependent variable.. PQI               Method.. LOGARITH

Listwise Deletion of Missing Data

Multiple R           .67542
R Square             .45619
Adjusted R Square    .45583
Standard Error      7.73894
```

```
            Analysis of Variance:

                 DF   Sum of Squares      Mean Square

Regression        1        77069.069        77069.069
Residuals      1534        91873.011           59.891

F =    1286.81917       Signif F =  .0000

-------------------- Variables in the Equation --------------------

Variable                   B        SE B       Beta         T  Sig T

V35                -8.047442     .224336   -.675416   -35.872  .0000
(Constant)        108.797275     .439339              247.639  .0000
_


Dependent variable.. PQI               Method.. INVERSE

Listwise Deletion of Missing Data

Multiple R           .35371
R Square             .12511
Adjusted R Square    .12454
Standard Error      9.81598

            Analysis of Variance:

                 DF   Sum of Squares      Mean Square

Regression        1         21135.90        21135.905
Residuals      1534        147806.18           96.353

F =     219.35807       Signif F =  .0000

-------------------- Variables in the Equation --------------------

Variable                   B        SE B       Beta         T  Sig T

V35                14.117582     .953199    .353705    14.811  .0000
(Constant)         91.069528     .351338              259.208  .0000
_


Dependent variable.. PQI               Method.. QUADRATI

Listwise Deletion of Missing Data

Multiple R           .89769
R Square             .80585
Adjusted R Square    .80560
Standard Error      4.62560

            Analysis of Variance:

                 DF   Sum of Squares      Mean Square
```

```
Regression        2        136141.80        68070.899
Residuals      1533         32800.28           21.396

F =    3181.45714       Signif F =  .0000

-------------------- Variables in the Equation --------------------

Variable                   B        SE B       Beta         T  Sig T

V35                 -.374956     .057695   -.262968    -6.499  .0000
V35**2              -.030922     .001949   -.642114   -15.869  .0000
(Constant)        101.597097     .282152              360.079  .0000
_


Dependent variable.. PQI               Method.. CUBIC

Listwise Deletion of Missing Data

Multiple R           .90912
R Square             .82649
Adjusted R Square    .82615
Standard Error      4.37419

            Analysis of Variance:

                 DF   Sum of Squares      Mean Square

Regression        3        139629.54        46543.180
Residuals      1532         29312.54           19.134

F =    2432.54784       Signif F =  .0000

-------------------- Variables in the Equation --------------------

Variable                   B        SE B       Beta         T  Sig T

V35                 1.104960     .122441    .774941     9.024  .0000
V35**2              -.173232     .010700  -3.597291   -16.189  .0000
V35**3               .003360     .000249   1.984590    13.501  .0000
(Constant)         98.216351     .365913              268.415  .0000
_


Dependent variable.. PQI               Method.. COMPOUND

Listwise Deletion of Missing Data

Multiple R           .86864
R Square             .75454
Adjusted R Square    .75438
Standard Error       .06399

            Analysis of Variance:

                 DF   Sum of Squares      Mean Square

Regression        1         19.310145        19.310145
Residuals      1534          6.281792          .004095
```

```
F =    4715.49576       Signif F =  .0000

-------------------- Variables in the Equation --------------------

Variable                   B        SE B       Beta         T  Sig T

V35                  .984872     .000219    .419521  4504.660  .0000
(Constant)        106.623199     .261761              407.330  .0000
_


Dependent variable.. PQI                Method.. POWER

Listwise Deletion of Missing Data

Multiple R           .65862
R Square             .43378
Adjusted R Square    .43341
Standard Error       .09719

            Analysis of Variance:

                 DF   Sum of Squares      Mean Square

Regression        1        11.101352        11.101352
Residuals      1534        14.490585          .009446

F =    1175.20953       Signif F =  .0000

-------------------- Variables in the Equation --------------------

Variable                   B        SE B       Beta         T  Sig T

V35                 -.096584     .002817   -.658622   -34.281  .0000
(Constant)        111.316392     .614197              181.239  .0000
_


Dependent variable.. PQI                Method.. S

Listwise Deletion of Missing Data

Multiple R           .34198
R Square             .11695
Adjusted R Square    .11638
Standard Error       .12138

            Analysis of Variance:

                 DF   Sum of Squares      Mean Square

Regression        1         2.993029        2.9930286
Residuals      1534        22.598908         .0147320

F =     203.16495       Signif F =  .0000

-------------------- Variables in the Equation --------------------
```

Variable	B	SE B	Beta	T	Sig T
V35	.167998	.011786	.341982	14.254	.0000
(Constant)	4.499983	.004344		1035.830	.0000

—

Dependent variable.. PQI Method.. GROWTH

Listwise Deletion of Missing Data

Multiple R	.86864
R Square	.75454
Adjusted R Square	.75438
Standard Error	.06399

Analysis of Variance:

	DF	Sum of Squares	Mean Square
Regression	1	19.310145	19.310145
Residuals	1534	6.281792	.004095

F = 4715.49576 Signif F = .0000

---------- Variables in the Equation ----------

Variable	B	SE B	Beta	T	Sig T
V35	-.015244	.000222	-.868643	-68.669	.0000
(Constant)	4.669301	.002455		1901.946	.0000

—

Dependent variable.. PQI Method.. EXPONENT

Listwise Deletion of Missing Data

Multiple R	.86864
R Square	.75454
Adjusted R Square	.75438
Standard Error	.06399

Analysis of Variance:

	DF	Sum of Squares	Mean Square
Regression	1	19.310145	19.310145
Residuals	1534	6.281792	.004095

F = 4715.49576 Signif F = .0000

---------- Variables in the Equation ----------

Variable	B	SE B	Beta	T	Sig T
V35	-.015244	.000222	-.868643	-68.669	.0000
(Constant)	106.623199	.261761		407.330	.0000

—

```
Dependent variable.. PQI                Method.. LGSTIC

Listwise Deletion of Missing Data

Notes:
4  Some values bigger than specified upper bound; no equation
estimated.

Notes:
4  Some values bigger than specified upper bound; no equation
estimated.
```

Table A-5: statistical characteristics of the models developed for Locals

Correlations

		PQI	Age
PQI	Pearson Correlation	1	-.864**
	Sig. (2-tailed)	.	.000
	N	1593	1593
Age	Pearson Correlation	-.864**	1
	Sig. (2-tailed)	.000	.
	N	1593	1593

**. Correlation is significant at the 0.01 level

Curve Fit

```
MODEL:  MOD_2.
```

```
Variable:  PQI          Maximum value:           100.00
This variable contains values that are larger than the input upper
bound.
The LGSTIC model cannot be fitted for this variable.
```

```
Dependent variable.. PQI                Method.. LINEAR

Listwise Deletion of Missing Data

Multiple R           .86422
R Square             .74688
Adjusted R Square    .74672
Standard Error      6.00153

            Analysis of Variance:

                 DF   Sum of Squares      Mean Square
```

```
Regression        1        169086.10        169086.10
Residuals      1591         57305.27            36.02

F =    4694.43727       Signif F =  .0000

-------------------- Variables in the Equation --------------------

Variable                   B        SE B       Beta         T  Sig T

V36                -1.144803     .016709   -.864219   -68.516  .0000
(Constant)        103.818748     .232231              447.050  .0000
_


Dependent variable.. PQI               Method.. LOGARITH

Listwise Deletion of Missing Data

Multiple R           .78264
R Square             .61252
Adjusted R Square    .61228
Standard Error      7.42536

            Analysis of Variance:

                 DF   Sum of Squares      Mean Square

Regression        1        138670.04        138670.04
Residuals      1591         87721.33            55.14

F =    2515.05567       Signif F =  .0000

-------------------- Variables in the Equation --------------------

Variable                   B        SE B       Beta         T  Sig T

V36                -9.198503     .183419   -.782639   -50.150  .0000
(Constant)        109.224511     .396001              275.819  .0000
_


Dependent variable.. PQI               Method.. INVERSE

Listwise Deletion of Missing Data

Multiple R           .58136
R Square             .33798
Adjusted R Square    .33756
Standard Error      9.70580

            Analysis of Variance:

                 DF   Sum of Squares      Mean Square

Regression        1         76515.20        76515.196
Residuals      1591        149876.18           94.202

F =     812.24167       Signif F =  .0000
```

```
-------------------- Variables in the Equation --------------------

Variable                  B        SE B       Beta         T  Sig T

V36               30.577425    1.072898    .581358    28.500  .0000
(Constant)        84.391789     .353223              238.920  .0000
_


Dependent variable.. PQI               Method.. QUADRATI

Listwise Deletion of Missing Data

Multiple R           .86867
R Square             .75458
Adjusted R Square    .75427
Standard Error      5.91133

            Analysis of Variance:

                 DF   Sum of Squares      Mean Square

Regression        2        170830.61        85415.303
Residuals      1590         55560.77           34.944

F =    2444.35665       Signif F =  .0000

-------------------- Variables in the Equation --------------------

Variable                  B        SE B       Beta         T  Sig T

V36                -.659441     .070637   -.497817    -9.336  .0000
V36**2             -.018060     .002556   -.376771    -7.066  .0000
(Constant)       102.166615     .327104              312.336  .0000
_


Dependent variable.. PQI               Method.. CUBIC

Listwise Deletion of Missing Data

Multiple R           .87443
R Square             .76463
Adjusted R Square    .76419
Standard Error      5.79084

            Analysis of Variance:

                 DF   Sum of Squares      Mean Square

Regression        3        173106.10        57702.033
Residuals      1589         53285.27           33.534

F =    1720.71050       Signif F =  .0000

-------------------- Variables in the Equation --------------------

Variable                  B        SE B       Beta         T  Sig T
```

V36	.887110	.200091	.669685	4.434	.0000
V36**2	-.153545	.016637	-3.203306	-9.229	.0000
V36**3	.003116	.000378	1.715829	8.238	.0000
(Constant)	98.801035	.519237		190.281	.0000

—

Dependent variable.. PQI Method.. COMPOUND

Listwise Deletion of Missing Data

Multiple R	.82355
R Square	.67824
Adjusted R Square	.67803
Standard Error	.08459

Analysis of Variance:

	DF	Sum of Squares	Mean Square
Regression	1	23.996506	23.996506
Residuals	1591	11.384223	.007155

F = 3353.62721 Signif F = .0000

---------- Variables in the Equation ----------

Variable	B	SE B	Beta	T	Sig T
V36	.986455	.000232	.438870	4246.259	.0000
(Constant)	104.879221	.343292		305.510	.0000

—

Dependent variable.. PQI Method.. POWER

Listwise Deletion of Missing Data

Multiple R	.73787
R Square	.54445
Adjusted R Square	.54416
Standard Error	.10065

Analysis of Variance:

	DF	Sum of Squares	Mean Square
Regression	1	19.262934	19.262934
Residuals	1591	16.117796	.010131

F = 1901.45893 Signif F = .0000

---------- Variables in the Equation ----------

Variable	B	SE B	Beta	T	Sig T
V36	-.108414	.002486	-.737867	-43.606	.0000
(Constant)	111.607000	.599084		186.296	.0000

—

```
Dependent variable.. PQI               Method.. S

Listwise Deletion of Missing Data

Multiple R           .54402
R Square             .29596
Adjusted R Square    .29552
Standard Error       .12513

            Analysis of Variance:

                 DF   Sum of Squares      Mean Square

Regression        1        10.471247        10.471247
Residuals      1591        24.909483          .015656

F =     668.81169       Signif F =  .0000

-------------------- Variables in the Equation --------------------

Variable                  B        SE B       Beta         T  Sig T

V36                 .357706     .013832    .544021    25.861  .0000
(Constant)         4.422943     .004554              971.286  .0000
_


Dependent variable.. PQI               Method.. GROWTH

Listwise Deletion of Missing Data

Multiple R           .82355
R Square             .67824
Adjusted R Square    .67803
Standard Error       .08459

            Analysis of Variance:

                 DF   Sum of Squares      Mean Square

Regression        1        23.996506        23.996506
Residuals      1591        11.384223          .007155

F =    3353.62721       Signif F =  .0000

-------------------- Variables in the Equation --------------------

Variable                  B        SE B       Beta         T  Sig T

V36                -.013638     .000236   -.823551   -57.911  .0000
(Constant)         4.652809     .003273             1421.481  .0000
_


Dependent variable.. PQI               Method.. EXPONENT
```

Listwise Deletion of Missing Data

Multiple R	.82355
R Square	.67824
Adjusted R Square	.67803
Standard Error	.08459

Analysis of Variance:

	DF	Sum of Squares	Mean Square
Regression	1	23.996506	23.996506
Residuals	1591	11.384223	.007155

F = 3353.62721 Signif F = .0000

---------- Variables in the Equation ----------

Variable	B	SE B	Beta	T	Sig T
V36	-.013638	.000236	-.823551	-57.911	.0000
(Constant)	104.879221	.343292		305.510	.0000

—

Dependent variable.. PQI Method.. LGSTIC

Listwise Deletion of Missing Data

Notes:
4 Some values bigger than specified upper bound; no equation estimated.

Table A.6: Pearson Correlations of pavement condition indicator with traffic, Age, CBR and subgrade resilient modulus *

Correlations						
		PQI	AGE	ESAL	CBR	MRs
PQI	Pearson Correlation	1.000	-.573(**)	-.875(**)	.477(**)	.483(**)
	Sig. (2-tailed)	.	.000	.000	.000	.000
	N	73	73	73	73	73
AGE	Pearson Correlation	-.573(**)	1.000	.661(**)	-.584(**)	-.587(**)
	Sig. (2-tailed)	.000	.	.000	.000	.000
	N	73	73	73	73	73
ESAL	Pearson Correlation	-.875(**)	.661(**)	1.000	-.460(**)	-.468(**)
	Sig. (2-tailed)	.000	.000	.	.000	.000
	N	73	73	73	73	73
CBR	Pearson Correlation	.477(**)	-.584(**)	-.460(**)	1.000	1.000(**)
	Sig. (2-tailed)	.000	.000	.000	.	.000
	N	73	73	73	73	73
MRs	Pearson Correlation	.483(**)	-.587(**)	-.468(**)	1.000(**)	1.000
	Sig. (2-tailed)	.000	.000	.000	.000	.
	N	73	73	73	73	73
** Correlation is significant at the 0.01 level (2-tailed).						

*This table extracted from SPSS program output

TABLE A-7: Pearson correlation coefficient between the dependent variable SN_{eff} and independent variables included in the study.

Correlations

		SNEFF_MM	E1	E3	E2	E4	D0	D200	D300	D450	D600	D900	D1200	D1500	D1800	Hpt (Inches)	HP_T_MM	D0-D200	D0-D300	D0-D450	D0-D1200	D0-D1800	EH	LD0D450
SNEFF_MM	Pearson Correlation	1	-.499**	.135	-.074	-.217*	-.615**	-.436**	-.196*	.092	.267**	.399**	.386**	.292**	.188*	.973**	.963**	-.784**	-.832**	-.834**	-.716**	-.667**	.228*	-.839**
	Sig. (2-tailed)	.	.000	.126	.402	.013	.000	.000	.025	.300	.002	.000	.000	.001	.032	.000	.000	.000	.000	.000	.000	.000	.009	.000
	N	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130
E1	Pearson Correlation	-.499**	1	.083	.099	.064	.000	-.035	-.127	-.221*	-.268**	-.294**	-.298**	-.293**	-.271**	-.438**	-.440**	.090	.151	.162	.067	.041	.691*	.112
	Sig. (2-tailed)	.000	.	.347	.260	.472	1.000	.689	.151	.011	.002	.001	.001	.001	.002	.000	.000	.309	.087	.066	.446	.640	.000	.204
	N	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130
E3	Pearson Correlation	.135	.083	1	.636**	.357**	-.506**	-.570**	-.606**	-.551**	-.471**	-.331**	-.224*	-.162	-.179*	.154	.167	-.110	-.157	-.230**	-.468**	-.498**	.175*	-.320**
	Sig. (2-tailed)	.126	.347	.	.000	.000	.000	.000	.000	.000	.000	.000	.010	.065	.041	.080	.058	.213	.075	.009	.000	.000	.046	.000
	N	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130
E2	Pearson Correlation	-.074	.099	.636**	1	.424**	-.498**	-.584**	-.596**	-.515**	-.424**	-.319**	-.282**	-.281**	-.264**	.064	.079	-.049	-.153	-.245**	-.446**	-.477**	.176*	-.259**
	Sig. (2-tailed)	.402	.260	.000	.	.000	.000	.000	.000	.000	.000	.000	.001	.001	.002	.471	.371	.580	.082	.005	.000	.000	.045	.003
	N	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130
E4	Pearson Correlation	-.217*	.064	.357**	.424**	1	-.234**	-.424**	-.552**	-.638**	-.646**	-.567**	-.442**	-.310**	-.173*	-.212*	-.189*	.356**	.251**	.173*	-.140	-.217*	-.131	.121
	Sig. (2-tailed)	.013	.472	.000	.000	.	.007	.000	.000	.000	.000	.000	.000	.000	.049	.015	.032	.000	.004	.049	.112	.013	.136	.169
	N	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130
D0	Pearson Correlation	-.615**	.000	-.506**	-.498**	-.234**	1	.954**	.829**	.598**	.410**	.233**	.212*	.267**	.319**	-.688**	-.699**	.651**	.746**	.811**	.975**	.989**	-.535*	.868**
	Sig. (2-tailed)	.000	1.000	.000	.000	.007	.	.000	.000	.000	.000	.008	.015	.002	.000	.000	.000	.000	.000	.000	.000	.000	.000	.000
	N	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130
D200	Pearson Correlation	-.436**	-.035	-.570**	-.584**	-.424**	.954**	1	.952**	.794**	.643**	.470**	.416**	.419**	.433**	-.501**	-.522**	.395**	.521**	.611**	.882**	.925**	-.415*	.695**
	Sig. (2-tailed)	.000	.689	.000	.000	.000	.000	.	.000	.000	.000	.000	.000	.000	.000	.000	.000	.000	.000	.000	.000	.000	.000	.000
	N	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130
D300	Pearson Correlation	-.196*	-.127	-.606**	-.596**	-.552**	.829**	.952**	1	.938**	.837**	.694**	.611**	.559**	.539**	-.244**	-.273**	.132	.246**	.349**	.710**	.778**	-.298*	.464**
	Sig. (2-tailed)	.025	.151	.000	.000	.000	.000	.000	.	.000	.000	.000	.000	.000	.000	.005	.002	.135	.005	.000	.000	.000	.001	.000
	N	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130
D450	Pearson Correlation	.092	-.221*	-.551**	-.515**	-.638**	.598**	.794**	.938**	1	.971**	.886**	.789**	.691**	.626**	.071	.037	-.177*	-.080	.016	.433**	.525**	-.140	.149
	Sig. (2-tailed)	.300	.011	.000	.000	.000	.000	.000	.000	.	.000	.000	.000	.000	.000	.423	.675	.044	.367	.860	.000	.000	.112	.090
	N	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130
D600	Pearson Correlation	.267**	-.268**	-.471**	-.424**	-.646**	.410**	.643**	.837**	.971**	1	.955**	.863**	.738**	.656**	.263**	.224*	-.370**	-.285**	-.197*	.225*	.326**	-.035	-.058
	Sig. (2-tailed)	.002	.002	.000	.000	.000	.000	.000	.000	.000	.	.000	.000	.000	.000	.003	.010	.000	.001	.025	.010	.000	.693	.511
	N	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130
D900	Pearson Correlation	.399**	-.294**	-.331**	-.319**	-.567**	.233**	.470**	.694**	.886**	.955**	1	.954**	.849**	.754**	.392**	.350**	-.479**	-.422**	-.357**	.022	.126	.033	-.227**
	Sig. (2-tailed)	.000	.001	.000	.000	.000	.008	.000	.000	.000	.000	.	.000	.000	.000	.000	.000	.000	.000	.000	.800	.152	.709	.009
	N	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130
D1200	Pearson Correlation	.386**	-.298**	-.224*	-.282**	-.442**	.212*	.416**	.611**	.789**	.863**	.954**	1	.948**	.880**	.353**	.312**	-.405**	-.360**	-.311**	-.009	.086	-.014	-.203*
	Sig. (2-tailed)	.000	.001	.010	.001	.000	.015	.000	.000	.000	.000	.000	.	.000	.000	.000	.000	.000	.000	.000	.919	.332	.877	.021
	N	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130
D1500	Pearson Correlation	.292**	-.293**	-.162	-.281**	-.310**	.267**	.419**	.559**	.691**	.738**	.849**	.948**	1	.953**	.234**	.198*	-.243**	-.202*	-.171	.059	.132	-.116	-.072
	Sig. (2-tailed)	.001	.001	.065	.001	.000	.002	.000	.000	.000	.000	.000	.000	.	.000	.007	.024	.005	.021	.052	.505	.135	.189	.418
	N	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130
D1800	Pearson Correlation	.188*	-.271**	-.179*	-.264**	-.173*	.319**	.433**	.539**	.626**	.656**	.754**	.880**	.953**	1	.128	.094	-.116	-.088	-.059	.128	.179*	-.177*	.036
	Sig. (2-tailed)	.032	.002	.041	.002	.049	.000	.000	.000	.000	.000	.000	.000	.000	.	.146	.288	.187	.321	.506	.147	.042	.044	.681
	N	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130
Hpt (Inches)	Pearson Correlation	.973**	-.438**	.154	.064	-.212*	-.688**	-.501**	-.244**	.071	.263**	.392**	.353**	.234**	.128	1	.995**	-.843**	-.902**	-.910**	-.784**	-.734**	.332*	-.892**
	Sig. (2-tailed)	.000	.000	.080	.471	.015	.000	.000	.005	.423	.003	.000	.000	.007	.146	.	.000	.000	.000	.000	.000	.000	.000	.000
	N	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130
HP_T_MM	Pearson Correlation	.963**	-.440**	.167	.079	-.189*	-.699**	-.522**	-.273**	.037	.224*	.350**	.312**	.198*	.094	.995**	1	-.822**	-.886**	-.899**	-.785**	-.740**	.331*	-.884**
	Sig. (2-tailed)	.000	.000	.058	.371	.032	.000	.000	.002	.675	.010	.000	.000	.024	.288	.000	.	.000	.000	.000	.000	.000	.000	.000
	N	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130
D0-D200	Pearson Correlation	-.784**	.090	-.110	-.049	.356**	.651**	.395**	.132	-.177*	-.370**	-.479**	-.405**	-.243**	-.116	-.843**	-.822**	1	.972**	.942**	.758**	.694**	-.591*	.902**
	Sig. (2-tailed)	.000	.309	.213	.580	.000	.000	.000	.135	.044	.000	.000	.000	.005	.187	.000	.000	.	.000	.000	.000	.000	.000	.000
	N	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130
D0-D300	Pearson Correlation	-.832**	.151	-.157	-.153	.251**	.746**	.521**	.246**	-.080	-.285**	-.422**	-.360**	-.202*	-.088	-.902**	-.886**	.972**	1	.989**	.845**	.788**	-.573*	.952**
	Sig. (2-tailed)	.000	.087	.075	.082	.004	.000	.000	.005	.367	.001	.000	.000	.021	.321	.000	.000	.000	.	.000	.000	.000	.000	.000
	N	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130
D0-D450	Pearson Correlation	-.834**	.162	-.230**	-.245**	.173*	.811**	.611**	.349**	.016	-.197*	-.357**	-.311**	-.171	-.059	-.910**	-.899**	.942**	.989**	1	.900**	.851**	-.565*	.974**
	Sig. (2-tailed)	.000	.066	.009	.005	.049	.000	.000	.000	.860	.025	.000	.000	.052	.506	.000	.000	.000	.000	.	.000	.000	.000	.000
	N	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130
D0-D1200	Pearson Correlation	-.716**	.067	-.468**	-.446**	-.140	.975**	.882**	.710**	.433**	.225*	.022	-.009	.059	.128	-.784**	-.785**	.758**	.845**	.900**	1	.993**	-.544*	.934**
	Sig. (2-tailed)	.000	.446	.000	.000	.112	.000	.000	.000	.000	.010	.800	.919	.505	.147	.000	.000	.000	.000	.000	.	.000	.000	.000
	N	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130
D0-D1800	Pearson Correlation	-.667**	.041	-.498**	-.477**	-.217*	.989**	.925**	.778**	.525**	.326**	.126	.086	.132	.179*	-.734**	-.740**	.694**	.788**	.851**	.993**	1	-.528*	.896**
	Sig. (2-tailed)	.000	.640	.000	.000	.013	.000	.000	.000	.000	.000	.152	.332	.135	.042	.000	.000	.000	.000	.000	.000	.	.000	.000
	N	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130
EH	Pearson Correlation	.228**	.691**	.175*	.176*	-.131	-.535**	-.415**	-.298**	-.140	-.035	.033	-.014	-.116	-.177*	.332**	.331**	-.591**	-.573**	-.565**	-.544**	-.528**	1	-.591**
	Sig. (2-tailed)	.009	.000	.046	.045	.136	.000	.000	.001	.112	.693	.709	.877	.189	.044	.000	.000	.000	.000	.000	.000	.000	.	.000
	N	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130
LD0D450	Pearson Correlation	-.839**	.112	-.320**	-.259**	.121	.868**	.695**	.464**	.149	-.058	-.227**	-.203*	-.072	.036	-.892**	-.884**	.902**	.952**	.974**	.934**	.896**	-.591*	1
	Sig. (2-tailed)	.000	.204	.000	.003	.169	.000	.000	.000	.090	.511	.009	.021	.418	.681	.000	.000	.000	.000	.000	.000	.000	.000	.
	N	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130	130

**. Correlation is significant at the 0.01 level (2-tailed).

*. Correlation is significant at the 0.05 level (2-tailed).

Table A-8: Statistical characteristics and AVOVA tables for the models developed in chapter 4.

Model Summary [c]

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.963[a]	.928	.927	10.46651
2	.967[b]	.934	.933	10.01826

a. Predictors: (Constant), HP_T_MM

b. Predictors: (Constant), HP_T_MM, D0

c. Dependent Variable: SNEFF_MM

ANOVA [c]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	179771.2	1	179771.158	1641.028	.000[a]
	Residual	14022.134	128	109.548		
	Total	193793.3	129			
2	Regression	181046.9	2	90523.431	901.937	.000[b]
	Residual	12746.430	127	100.366		
	Total	193793.3	129			

a. Predictors: (Constant), HP_T_MM

b. Predictors: (Constant), HP_T_MM, D0

c. Dependent Variable: SNEFF_MM

Coefficients [a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	-1.616	3.784		-.427	.670
	HP_T_MM	.232	.006	.963	40.510	.000
2	(Constant)	-33.016	9.523		-3.467	.001
	HP_T_MM	.251	.008	1.042	32.767	.000
	D0	.185	.052	.113	3.565	.001

a. Dependent Variable: SNEFF_MM

A-9: Statistical characteristics of the model described in Equation

```
Dependent variable.. SNEFF_MM          Method.. POWER
Listwise Deletion of Missing Data
Multiple R =0.55977,  R Square= 0.31334,   Adjusted R Square=0.30798
Standard Error =0.24698

                        Analysis of Variance:
                 DF   Sum of Squares      Mean Square
Regression        1        3.5629119        3.5629119
Residuals       128        7.8078119        0.0609985
F =      58.40980       Signif F =  .0000

-------------------- Variables in the Equation --------------------

Variable                   B        SE B       Beta         T  Sig T
D0                  -.648487     .084851   -.559769    -7.643  .0000
(Constant)       2807.988748 1099.954682                2.553  .0119
```

```
Dependent variable.. SNEFF_MM          Method.. EXPONENT
Listwise Deletion of Missing Data
Multiple R = 0.59411, R Square = 0.35296, Adjusted R Square=0.34791
Standard Error       .23975

                        Analysis of Variance:
                 DF   Sum of Squares      Mean Square
Regression        1        4.0134503        4.0134503
Residuals       128        7.3572735         .0574787
F =      69.82500       Signif F =  .0000

-------------------- Variables in the Equation --------------------

Variable                   B        SE B       Beta         T  Sig T
D0                  -.007430     .000889   -.594107    -8.356  .0000
(Constant)        304.879345   28.777295               10.594  .0000
```

Table A-10: AIRI-ARI correlation matrix

Correlations

			IRI	ARI
Spearman's rho	IRI	Correlation Coefficient	1.000	-.999**
		Sig. (2-tailed)	.	.000
		N	61	61
	ARI	Correlation Coefficient	-.999**	1.000
		Sig. (2-tailed)	.000	.
		N	61	61

**. Correlation is significant at the .01 level (2-tailed).

The End