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ÉLABORATION D'UNE PROCÉDURE POUR PÉNALISER LES ENTREPRENEURS SUITE AUX MANQUEMENTS EN CONTRÔLE ET ASSURANCE DE LA QUALITÉ EN CONSTRUCTION ROUTIÈRE

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IL A FAIT L'OBJET D'UNE PRÉSENTATION DEVANT CE JURY ET UN PUBLIC

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À L'ÉCOLE DE TECHNOLOGIE SUPÉRIEURE

ÉLABORATION D'UNE PROCÉDURE POUR PÉNALISER LES ENTREPRENEURS SUITE AUX MANQUEMENTS EN CONTRÔLE ET ASSURANCE DE LA QUALITÉ EN CONSTRUCTION ROUTIÈRE

Laid Temacini

(Résumé)

Le développement économique durable de toute nation est essentiel pour l'amélioration continu du niveau et de la qualité de vie des citoyens. Ce développement est sans doute lié en grande partie à l'efficience, l'efficacité et la coordination des systèmes de transport dans son ensemble. Le transport terrestre joue un rôle prépondérant dans ce système global du fait qu'il assure la mobilité des biens et des personnes, le désenclavement des régions isolées et les échanges commerciaux de toute nature.

Le transport routier constitue l'épine dorsale de l'activité économique des nations. Pour la majorité des pays en voie de développement, le secteur du transport routier joue un rôle très important et assure à lui seul plus de 80 à 90 pour-cent du total des échanges commerciaux à l'intérieur du pays comme au delà des frontières. Ces infrastructures sont dispendieuses et les frais à encourir pour leur maintien dans un état acceptable durant leur cycle de vie sont fortement liés à la fiabilité et à la maîtrise des paramètres et variable de conception, à la caractérisation des matériaux disponibles ainsi qu'à leur mise en œuvre. L'assurance et le contrôle de la qualité sont cependant souvent négligés alors qu'ils jouent un rôle de premier plan quant à l'optimisation des coûts des défaillances qui pourront avoir des conséquences considérables sur les coûts aux usagés et aux coûts globaux à la société.

Malgré leur importance, la plupart des routes dans les pays en voie de développement sont mal gérées et mal entretenues. Une mauvaise construction et un entretien différé sont souvent la cause des dégradations prématurées. Les coûts élevés du transport causés par des chaussées en mauvais état sont principalement couverts par les usagers qui paient directement les coûts d'exploitation de leurs véhicules et indirectement, l'administration (à travers les taxes et les impôts) pour la construction et l'entretien.

Le contrôle et l'assurance de la qualité en conception et en construction routière sont des éléments vitaux qui prédéterminent la durée de vie des chaussées. Le contrôle de la qualité est requis pour assurer que la chaussée conçue et construite répond à un

minimum d'exigence associé au niveau de service et à la durée de vie désirée. L'objectif principal de cette étude est de mettre en évidence, d'une manière concrète, les conséquences économiques découlant du manquement en contrôle et assurance de la qualité en conception et construction des routières. Elle comprend la réalisation de trois objectifs spécifiques, soit : 1) la réalisation d'une recherche bibliographique sur le sujet, 2) la conception d'un ensemble de chaussées représentatives du trafic existant dans les pays en voie de développement, 3) la simulation de divers scénarios de manquement dans la conception et la construction de route à l'aide du programme HDM-III (Highway Design and Maintenance Standard) de la Banque Mondiale, et, 4) l'application des outils statistiques pour l'élaboration d'une procédure de quantification des coûts globaux à la société avec laquelle on sera en mesure de prédire l'impact économique des variations des facteurs de conception et des méthodes de construction des chaussées.

L'application rigoureuse des principes de l'économie dans la gestion des chaussées est une composante primordiale pour l'allégement des coûts globaux du transport. Ceci est particulièrement vrai pour les routes à faible trafic où une légère non-conformité des épaisseurs structurales des composantes de la chaussée par rapport à celles prescrites peut considérablement hausser le coûts global du transport.

Cette recherche aura pour but de mettre en évidence d'une part, la nécessité de la pratique de l'assurance et du contrôle rigoureux de la qualité en conception et en construction routière, et d'autre part, de dégager l'impact en terme de coûts globaux à la société des variables principales en conception selon leur poids ainsi que les impacts cumulatifs dus à leurs interactions. Afin de prévenir la corruption, la négligence et le manquement en contrôle et en assurance de la qualité dans les travaux routiers, l'objectif principal est de développer une procédure pour établir et calculer des pénalités aux entreprises œuvrants dans les pays en vois de développement à partir du logiciel d'évaluation technico-économique de la Banque Mondiale et des méthodes usuelles de conception des chaussées.

Dans les projets de construction de route, habituellement, la responsabilité de l'entrepreneur est limitée à la fourniture des matériaux et à leur mise en œuvre. Il doit exécuter les travaux en grande partie selon les prescriptions du donneur d'ouvrage et sous la direction et la supervision de l'ingénieur résidant. Les études de conception et d'ingénierie pertinentes au projet auront été préparées par l'administration ou son délégué. Les pays en voie de développement emploient encore cette approche traditionnelle pour la passation des marchés de construction routière. Avec ce type de prescription traditionnelle, l'entrepreneur doit quasiment suivre les instructions pour le choix des matériaux et des équipements ainsi que les méthodes de construction. Chaque pas est contrôlé et dans la majorité des cas, dirigé par l'ingénieur résidant. Si l'entrepreneur adhère pleinement aux spécifications prescrites, les prix contractuels en totalité lui sont accordés. Le problème majeur avec ce type de prescription est que les pénalités infligées aux entrepreneurs pour la non-conformité sont arbitraires. Elles sont basées uniquement sur l'habileté et le jugement du personnel résidant. Les travaux non conformes seraient soit repris aux frais de l'entrepreneur ou à la discrétion de l'ingénieur, acceptés au prix contractuel ou à un prix réduit. Quand une réduction est envisagée, elle est négociée au cas par cas après le fait. Dans ce type de prescription contractuelle, ni le comportement futur de la chaussée, ni les méthodes statistiques de contrôle de la qualité sont utilisées.

Avec l'augmentation de la charge à l'essieu des véhicules et les contraintes économiques, la préservation des infrastructures de transport constitue un défi majeur pour les administrations routières. Cependant, de nouvelles exigences, en matière de partage de responsabilité de la part de l'entrepreneur quand au comportement futur de la chaussée et par conséquent aux coûts à encourir sont incontournables. Ainsi de nouvelles méthodes de passation des marchés de construction de route basées sur le résultat final ou le comportement à long terme s'imposent. Ces nouvelles exigences forceront l'entrepreneur à étendre ses activités traditionnelles et à l'inciter à prendre ses responsabilités en matière de contrôle et assurance de la qualité.

Idéalement, les pénalités à infliger aux entrepreneurs sont fonction du niveau de qualité requis pour les variables contrôlable pendant la construction. La littérature propose plusieurs méthodes de calcul de ces pénalités. Ces méthodes peuvent être subdivisées en deux familles : les méthodes arbitraires ou subjective et les méthodes rationnelles. La différence entre ces deux méthodes réside dans l'existence ou non d'une correspondance entre les critères de conception ou de comportement futur de la chaussée et la construction de la chaussée. Actuellement, de nombreux projets de recherche liés au développement des liens entre ces facteurs contrôlables sont en cours dans la plupart des pays développés.

Pour la quantification des coûts globaux à la société dus au manquement en contrôle et assurance de la qualité en construction routière dans les pays en voie de développement, la procédure adoptée dans cette étude est basée sur les modèles technico-économiques de la Banque Mondiale (HDM-III) et la méthode AASHTO de conception des chaussées flexibles. Cette procédure intègre les coûts globaux à la société (les coûts de construction, de réhabilitation et aux usagers) pour une durée de vie utile de 20 ans. La méthodologie adoptée est basée sur les techniques statistiques de planification des expériences. Un plan expérimental complet à trois niveaux à été retenu pour l'analyse des effets des variables indépendantes et leurs interactions sur la réponse. Les variables indépendantes considérées dans cette analyse sont : 1) le trafic exprimé en équivalent de charge axiale simple (ÉCAS), 2) le CBR du sol support, et la perte relative de la capacité structurale de la chaussée, exprimée en pourcentage du nombre structural (ΔSN/SN). Ce plan expérimental à trois niveaux a été choisi du fait de la complexité des algorithmes empiriques du programme HDM-III et de l'AASHTO. Pour simuler le manquement en contrôle et assurance de la qualité, une réduction de 10 et 20 pour-cent sur le nombre structural requis (SN de L'AASHTO) à été retenue. Pour pouvoir supporter le trafic anticipé, les chaussées (telles que construites) sont réhabilitées selon la méthode AASHTO de conception du resurfaçage. Cette méthode est basée sur la durée de vie résiduelle des chaussées. Le comportement et les coûts associés à chacune des vingt-sept conditions expérimentales (sections de chaussées) sont simulés à l'aide programme HDM-III. La variable dépendante ou la réponse exprime le paiement dû à l'entrepreneur exprimé en pourcentage du prix total contractuel de construction de la chaussée.

Les coûts supplémentaires résultants d'une mauvaise construction sont déduits de la valeur présente de l'annuité équivalente uniforme des coûts totaux du transport résultant de la stratégie optimale de réhabilitation de la section de chaussée déficiente. La procédure employée est basée sur les concepts de la durée de vie économique. Les chaussées telles que conçues et celles telles que construites sont évaluées et comparée entre elles sur une période commune dans leur cycle de vie utile. Ce concept de durée de vie économique définit le temps, dans une période d'analyse donnée, où l'annuité équivalente des coûts est minimum. Il est couramment utilisé dans les études de remplacement d'équipement dans les procédés industriels.

L'analyse de la variance à été conduite pour l'étude des effets des variables principales (trafic, CBR, et ΔSN/SN) et leurs interactions sur la variable dépendante (pénalité PF). Dans cette analyse, la signification des effets principaux et des interactions doubles a été basée sur un niveau de confiance de 5 pour-cent. Les résultats obtenus montrent que pour un niveau de confiance de 5 pour-cent, les seules variables significatives sont le trafic, la perte relative de la capacité structurale de la chaussée (ΔSN/SN) et leurs interactions. Le CBR du sol support et toutes ses interactions avec les autres facteurs n'étaient pas significatifs à ce niveau de confiance et cependant ils étaient exclus de l'analyse.

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<u>**A Decision Making System to Penalise Road Contractors for Lack of QA/QC**</u>

Laid Temacini

(Summary)

The challenge of preserving the road asset, with increasing axle loading combined with decreasing capital and maintenance funding, makes it imperative that specification and tendering systems evolve to reflect end result requirements. This would be enroute to full performance based parameters such as rutting resistance, fatigue endurance and service life durability.

To assess a penalty scale to contractors for lack of conformance to specifications, a full three level factorial design is selected so that all interactions between the independent variables could be investigated and considered in the analysis. A set of pavement types and strengths constructed on weak, fair and strong subgrades and subjected to a range of traffic representing the conditions prevailing in most developing countries were selected. To simulate the lack of QA/QC in the construction phase, a reduction of 10 and 20 percent on the pavement structural number (SN) obtained from AASHTO design algorithm were assumed. To withstand the traffic loading for the entire analysis period, as-constructed pavements were rehabilitated using AASHTO overlay design method based on the remaining life approach. Pavement performance simulation procedures were based on HDM-III performance and economic prediction models. AASHTO outputs (SN) calculated for each pavement strength and the resulting simulated lack of conformance serve as direct inputs to HDM-III.

Analysis of variance (ANOVA) was conducted to investigate the main effects of traffic loading, subgrade CBR and pavement effective structural number, together with their two level interaction effects on the penalty factor. For this analysis we selected the 5 per-cent confidence level for testing the significance of the main effects and the two level interactions effects. The results show that the main effects that are significant at the 5 per-cent level are the traffic loading and the pavement effective SN. An appropriate penalty scale procedure was developed on the basis that it would be justifiable to deduce a sufficient amount of funds from the contract price a the time of construction for future road repairs resulting from a reduced road life.

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AASHO	American Association of Sate Highway Officials;
AASHTO	American Association of State Highway and Transportation Officials;
AC	Asphalt concrete;
CBR	California bearing ratio;
ERS	End-result specifications;
ESAL	Equivalent single axle load;
FHWA	Federal Highway Administration;
IRI	International road roughness index;
LCC	Life-cycle cost;
LTPP	Long-term pavement program;
M&C	Materials and construction;
NCHRP	National Cooperative Highway Research Program;
OC	Operating characteristic;
PF	Pay factor;
PRS	Performance-related specification;
PSI	Present serviceability index;
PSR	Present serviceability rating;
QA	Quality assurance;
QC	Quality control;
SHRP	Strategic Highway Research Program;
TQM	Total quality management;
A_C	annual cost at the end of economic life for as-constructed pavement;

L_C	economic life of as-constructed pavement;
A_T	annual cost at the end of economic life for as-designed pavement;
L_T	economic life of as-designed pavement.
Ty_{cr2}	expected (mean) age of surfacing at initiation of narrow cracking, in years;
SNC	modified structural number;
YE_4	annual traffic loading, million ESALs/lane/year.
K_{ci}	calibration Factor
F_c	The occurrence distribution factor for cracking initiation for the subsection (i.e., weak = 0.55, medium = 0.98, strong = 1.48)
CQ	construction quality indicator for surfacing and is equal to 1 if the surfacing has construction faults and 0 otherwise;
$\Delta ACRA_d$	predicted change in the area of all cracking during the analysis year due to road deterioration expressed in percent of the total area;
K_{cp}	calibration factor for cracking progression
CRP	retardation of cracking progression due to preventive treatment;
ΔTCRA	fraction of the analysis year in which narrow cracking progression applies, expressed in years, and given by :
ΔTCRW	fraction of the analysis year in which wide cracking progression applies, expressed in years, and given by :
RDM	mean rut depth of both wheelpaths, in mm;
AGER	age of the pavement since latest overlay or construction, in years;
COMP	compaction index of pavement;
RC_i	relative compaction, i.e., the ratio of the compaction measured in the field to the nominal compaction, as a fraction ;
Hi	thickness of layer, in mm;
NE_4	cumulative number of ESALs;

DEF	mean peak Benkelman beam deflection under 80 kN standard axle load of both wheelpaths, in mm;
RH	rehabilitation states of the pavement and is equal to null for original pavements and to one for overlaid pavements;
MMP	mean monthly precipitation, in m/month;
CRX	area of indexed cracking, in percent.
RDS	rut depth standard deviation of both wheelpaths, in mm;
RDM	mean rut depth of both wheelpaths, in mm;
COMP	compaction index of pavement relative to a standard fraction;
NE4	cumulative number of ESALs;
TYRAV	predicted number of years to ravelling initiation;
K_{vi}	calibration factor for ravelling initiation ;
Fr	occurrence distribution factor for ravelling initiation for the subsection;
YAX	total number of axles of all vehicle classes for the analysis year expressed in millions/lane;
RRF	raveling retardation factor due to maintenance;
$\Delta ARAV_d$	predicted change in the ravelling area expressed in percent
$\Delta TRAV$	fraction of the analysis year during which
TMIN	predicted time between the initiation of either wide cracking or raveling, whichever occurs earliest, and the probable initiation of potholing, expressed in years;
HS	thickness of the bituminous surfacing, in mm;
YAX	annual number of vehicle axles, in million axles/lane/year;
$\Delta APOT$	predicted change in the total area of potholes during the analysis year due to road deterioration, limited to 10%;
$\Delta APOT_{cr}$	predicted change in the area of potholes during the analysis year due to cracking;

$\Delta APOT_{rv}$	predicted change in the area of potholes during the analysis year due to raveling;
$\Delta APOT_{pe}$	predicted change in the equivalent area of potholes during the analysis year due to pothole enlargement;
CR_4	area of wide cracking, expressed in percent of pavement area;
ARAV	area of ravelling, expressed in percent of pavement area;
HS	thickness of bituminous surfacing;
YAX	annual number of vehicle axles, expressed in million axles/lane/year;
MMP	mean monthly precipitation, expressed in meters.
ΔQI_d	increase in roughness over time period t, expressed in QI;
K_{gp}	calibration factor for roughness progression;
K_{ge}	calibration factor for the environment related annual fractional increase in roughness;
SNCK	modified structural number adjusted for the effect of cracking;
ΔSNK	predicted reduction in the structural number due to cracking since the last pavement reseal, overlay or reconstruction;
ECR	predicted excess cracking beyond the amount that existed in the old surfacing layers at the time of the last pavement reseal, overlay or reconstruction;
PCRX	area of previous indexed cracking in the old surfacing and base layers;
LGTH	the length of the roadway in km;
V_u	the predicted vehicle speed for the uphill segment in m/s;
V_d	the predicted vehicle speed for the downhill segment in m/s;
UFC_u	the predicted unit fuel consumption for the uphill segment, in ml / s;
UFC_d	the predicted unit fuel consumption for the downhill segment, in ml/ s;
CRPM	the calibrated engine speed, in revolution per minute;

HP_d	the vehicle power on the downhill road segment, in metric hp,
NT	the number of tires per vehicle;
RREC	the ratio of the cost one retreading to the cost of one new tire,
NR	the number of retreadings per tire carcass predicted as:
NR_o	the base number of recaps;
C_{otc}	the constant term of the tread wear model;
C_{tctc}	the wear coefficient;
CF_2	the average squared circumferential force per tire, given as:
Cf_u	the average circumferential force per tire on the uphill road segment,
CF_d	the average circumferential force per tire on the downhill CF_d
L	the average force per tire in the direction perpendicular to the road
VOL	the average wearable rubber volume per tire for a given vehicle axle-wheel configuration and nominal tire size, in dm^3;
CKM	the average age of the vehicle group in km, defined as the average number of kilometers the vehicles have been driven since they were built,
$LIFE_o$	the average vehicle service life in years;
AKM_o	the average number of kilometers driven per year per vehicle type;
CKM^1	the ceiling on average cumulative kilometerage;
k	the age exponent, a fixed model parameter;
C_{osp}	the constant coefficient in the exponential relationship between spare parts consumption and roughness;
C_{spqi} parts	the roughness coefficient in the exponential relationship between spare consumption and roughness;
Q_{iosp}	the transitional value of roughness, in QI, beyond which the relationship between spare parts consumption and roughness is linear;
LH	predicted number of maintenance labor-hours per 1 000 vehicle-km;
PC	the standardized parts cost per 1 000 vehicle-km, expressed as a fraction of new vehicle price;

C_{olh} the constant coefficient in the relationship between labor hours and parts costs;

C_{lhpc} the exponent of parts cost in the relationship between labor hours and parts costs;

C_{lhqi} the roughness coefficient in the exponential relationship between labor hours and roughness;

INTRODUCTION

Sustainable economic development of all nations is essential for the continuing improvement of the standard of living and the quality of life of their people. It is underpinned by an efficient, effective and co-ordinated total transport system. In developing countries, however, the road system is a vital component in their total transport system and, indeed, it provides for both mobility and accessibility. In spite of their importance, most roads in these countries are poorly managed and badly maintained. The poor state of the road network is reflected in the large backlog of faulty construction and poor or deferred maintenance. The economic costs incurred from roads in poor condition are born primarily by road user who pays user cost directly and also pays indirectly, through taxes, the agency costs for construction and maintenance. Thus, pavement economics from the planning and design stages up to construction, maintenance and rehabilitation is vital component in the total transport system. This is particularly true for low volume roads where small changes in the thicknesses of the structural layers due to improper design or a faulty construction can significantly increase the total transport cost. Road user cost is the most significant part and count for up to 95 percent of the total transport cost and is greatly dependent on the changes in road condition with time. Because pavement design and construction are so important to the overall road life-cycle cost, every effort should be made in selecting the optimum thickness design and enforcing contractor reliability for proper implementation.

The quality of materials incorporated and work performed directly influence the service life of a pavement, maintenance costs, level of service and user costs. Materials are a key economic factor to pavements, as their total in-place cost is about 40 to 50 percent of a total cost of a highway project. Quality can be defined as the characteristics of a product (materials and methods of pavement construction, for instance) that provide a level of functional (i.e., serviceability) and structural performance.

In road construction project, the traditional responsibility of the contractor is to carry out the construction work, largely to the method and under the direction of the resident engineer, the design and all the relevant engineering studies having been carried out by or on behalf of the road agency. Developing countries still use this traditional approach to their road construction contract for specifying and accepting highway materials and construction. With this type of specifications, the contractor is directed to combine specific materials in definite proportions, use specific type of equipment, and place the material or product in a prescribed way. Each step is controlled and in many cases directed by a representative of the road agency. If the contractor adheres fully to the prescribed specifications, a hundred percent payment to the contractor is assured. A major deficiency of this method-type specifications is that penalties for contractor nonconformance are somewhat arbitrary and based solely on the skills and judgment of the inspector. Deficient work was either removed or, at the discretion of the resident engineer, accepted at full price or at a reduced price. When price reductions were applied, they were typically negotiated on a case-by-case basis after the fact. In this method-type specifications, pavement performance and statistical concepts are seldom employed.

The challenge of preserving the pavement asset, with increasing axle loading combined with decreasing capital and maintenance funding, makes it imperative that specification and tendering systems evolve to reflect end result requirements. This would be enroute to full performance based parameters such as rutting resistance, fatigue endurance and service life durability. End result specifications (ERS), based on statistical quality assurance concepts, appear to provide a framework to rationally define agency-contractor interaction and agency-contractor responsibilities for life-cycle cost effective methods and materials of pavement construction. The responsibilities of these contractors become more extensive than their previous duties in working directly for a road agency. It requires new and different personnel to manage Quality assurance systems (QA), quality control systems (QC) and testing requirements for acceptance criteria. Thus, QA systems has in many cases produced an environment where the contractor is continually aware and reporting upon the quality and progress of his work. The intention of QA/QC to improve and report upon quality within the work organization should produce clearer responsibilities and a greater diligence at site level. QC ensures that the specified materials are combined and placed so that the pavement will have the desired level of performance. QA is all of the activities necessary to verify, audit and evaluate quality. With the move to ERS, the contractor take on QC responsibility, while the agency uses QA for assessing the contractor's capacity to undertake responsibilities.

The development of adjusted pay schedules include the determination of appropriate pay levels for various levels of quality. Several methods were proposed in the literature, and where there was little or no information relating quality measures to performance, the methods have necessarily been quite arbitrary. Nowadays, interest in performance-related specifications (PRS) for both concrete and asphalt pavements is growing in developed countries. Extensive research and development programs have been underway for several years. A key aspect of PRS is that the quality characteristics that are measured during construction should be strongly related to the performance of the pavement. Conceptually, the aim of PRS is to improve the quality of construction by measuring the quality characteristics that are directly or indirectly related to performance.

In the current study, an attempt is made to develop a rational method of establishing pay schedules, based on the legal principle of liquidated damages. A comprehensive decision making procedure for assessing penalties to road contractors resulting from inadequate or faulty design and construction is developed, based on pavement performance and economic models incorporated in World Bank's HDM-III. These penalties are intended to prevent corruption, neglect or lack of QA/QC in pavement construction and rehabilitation. An appropriate pay factor procedure is developed on the basis that it would be justifiable to deduce a sufficient amount of funds from the contract price at the time of construction for future road repairs resulting from a reduced road life. The method adopted is applicable to all material characteristics and construction methods for which data is available to relate quality to performance.

CHAPITRE 1

QUALITY ASSURANCE / QUALITY CONTROL

1.1 Historical development and evolution

As concepts, quality assurance and quality control of products and services have been known for many centuries and it is only recently that they emerged as formal management tools. Traditionally, they were mostly reactive and based entirely on inspections (1). In our days, activities linked to these concepts are far more diversified and recognized as essential for the strategic success. Modern quality management and quality control have emerged gradually and followed a continuous evolution through intensive research and practical applications. Currently, they can be subdivided into four categories: inspection, statistical quality control, quality assurance and strategic quality management.

1.1.1 Inspection

In the eighteenth and nineteenth century, concepts of quality control as known today, did not exist. Most of manufactured products were produced by artisans, and skilled workers (2). A that time, produced quantities were small and matching of components to one another was manual. Well performing products were viewed as the natural result of well trained artisans and experimented workers (3). Formal inspection became necessary only with the emergence of large scale productions and the need for interchangeable spare parts. The rise in the volume of production rendered the manual fitting of components to one another very slow and hence costly to consumers. Such a situation gave rise to what has been called the American System of Manufacturing. This system consisted in the use of a specifically conceived machinery for the mass production of identical component following a pre-established sequence of operations (4). In industrial manufacturing, the key evolution in quality control (QC) concepts was the rational development of component adjustment systems and devices, gauges, etc. since the early 1800s. These adjustment devices, and gauges were conceived to position cutting tools and to properly adjust components to machinery such that manufacturing operations can be executed in identical and precise manner (5). The standardization of these devices ensured a high degree of similarity and interchangeability. However, produced parts may still deviate from one another. These sources of variation can be attributed to multiple factors, such as bad adjustment, deficient raw materials, used tools and so forth.

In 1819 a calibrating device based on a standard model of the manufactured product was produced. This new tool has revolutionized inspection methods because judgments once based on the naked eye were replaced by a more objective and verifiable process (6).

With the development of the American manufacturing system, calibration was continuously refined and inspection became even more important. In the early 1900s, Frederick W. Taylor (1919), considered as the father of the scientific management, suggested new approaches to the organization of the work and to its execution (7). At that time, labor force consisted particularly of new immigrants and farm workers without any experience of the manufacturing process. Taylor suggested that the work should be conceived by engineers who at the same time are responsible for the tools and the standardization procedures. With this new reorganization, the worker becomes himself part of the big production machine. The quality of the work becomes the responsibility of inspectors. Complex operations have been subdivided into smaller tasks and each of them was attributed to a worker. Taylor's principles have built a high degree of task specialization. The workers execute tasks to meet the engineers prescriptions, and managers and inspectors ensured that workers met the standards. Naturally, in such situations, inspectors and workers saw often each other as adversaries. Taylor's principles have been used in 1907 by Ford industrial enterprise for manufacturing complex cars to lesser costs. That has been possible through the division of the work into elementary tasks and to their execution by unqualified workers. To this stage of the technological evolution, the inspection was considered as an essential part of the process. Its main objective was to separate good from non-conforming products.

In 1920, inspection activities were formally recognized as a function of quality control. This function had the responsibility to develop tools, check for the product characteristics and dimensions, detect errors, and ensure the necessary good operations of execution. At that time, it was considered that the only way to ensure the quality of products is to inspect all manufactured items.

With the publications of G.S Radford (1922), inspection activities were more formally linked to quality control. For the first time quality was viewed as a distinct management responsibility and as an independent function (8). Nevertheless, quality control was limited to a few non-statistical inspection activities as for example counting, measuring, and repairs. It was necessary to wait several years before the statistical nature in manufacturing processes was fully recognized and the role of quality professionals redefined. The research undertaken by Bell Telephone Laboratories during the 20s has resulted in what is called today statistical quality control (1).

1.1.2 Statistical quality control

The publication of W.A. Shewhart (1931) has given a scientific foundation to the statistical quality control. His work remain an unavoidable reference tool for modern quality management. Shewhart has given precise definitions of the manufacturing quality control, has developed powerful techniques for its supervision and its daily application and has suggested multiple ways of continuously improving quality (9).

1.1.2.1 Process control

Shewhart was the first to recognize the natural variation of industrial processes. These variations can be understood and mastered with the use of statistics and probability tools. Shewhart, during his works on quality assurance for Bell Telephone had concluded that two products deriving from the same process can not conform exactly to the same specific·ions. Raw materials, operator skills and equipment competence are all subjected to variations, and influence the quality of the finished product. Shewhart observed that a series of identical parts produced by a unique operator on the same machine will tend to show variations with time. This awareness of the inherent variability in manufacturing processes has reoriented managers vision regarding quality concepts. The problem is no longer the existence of these variations but rather the distinction between acceptable variations and inadmissible fluctuations. The concept of statistical quality control developed by Shewhart (1931) stipulates that :

> A phenomenon will be said to be controlled when, through the use of past experience, we can predict, at least within limits, how the phenomenon may be expected to vary in the future. Here it is understood that prediction means that we can state, at least approximately, the probability that the observed phenomenon will fall within the given limits (9).

Shewhart has developed simple techniques for determining these limits, as well as graphical methods to illustrate production values and to assess if they are within acceptable limits. The results of his findings are among the most powerful tools used to day by quality professionals. The advantages of this technique are that external problems to processes can be dissociated from those inherent to processes.

1.1.2.2 Sampling

It is recognized that a hundred percent inspection is an inefficient way to separate the good products from bad. Checking a limited number of manufactured products then deciding on that basis if the entire lot is acceptable or not is clearly a more advantageous alternative. Because a sample can not be fully representative of a population, a lot can be accepted while in reality it contains a number of defective items. The opposite can also happen, and all of a production lot can be rejected while it is in perfectly acceptable quality. Harold Dodge and Harry Romig (1944) of Bell Laboratories have identified these problems and recognized them as the consumer and the producer risks. They have developed for the account of the company sampling plans that ensured that for a given level of defect, the probability to accept a defective lot would be limited to a certain percentage (10).

1.1.3 Quality assurance

Statistical quality control has played a significant role in manufacturing processes but with the growing complexity in manufactured products, problems of defect prevention have emerged. During the 40s, the American armament industries have invented and adopted new concepts of quality assurance (11). These new concepts allow to ensure at different stages of the manufacturing process that the operations have been correctly executed. Four major elements are involved in the prevention: 1) quantifying the costs of quality, 2) total quality control, 3) reliability engineering, and 4) zero defect.

1.1.3.1 Costs of quality

Until the 1950s, most efforts to improve quality were based on the assumption that defects were costly. In the absence of such sound scientific foundations, administrators and managers have the habit to take subjective decisions. For them, it remains always a critical question :How much quality is enough ? (1)

In 1951 Joseph Juran (1951) tackled the question in his publication. In the first chapter of his book, he discussed the economics of the quality (12). He observed that the costs for achieving a given level of quality could be grouped in two categories: avoidable costs and unavoidable costs. Unavoidable costs are those associated with prevention: inspection, sampling, sorting, and other quality control initiatives. On the other hand, avoidable costs are those related to product failures : scrapped materials, time required for rework and repair, complaint processing, and financial losses resulting from unsatisfied clients. The works of Juran have allowed administrators and managers to decide how much to invest in quality improvement. Juran sustains that additional expenditure on prevention would likely be justified as long as avoidable costs remained high. These concepts illustrated also an other important principle. This principle stipulates that decisions taken earlier in a production process for the manufacture of a new product had direct implications on the level of service incurred later on.

1.1.3.2 Total quality control

In 1956 Armand Feingenbaum (1956) took Juran's principles a step further by proposing total quality control. He argued that high quality products, can not be obtained if the manufacturing department was forced to work in isolation. In his publications he stated that :

> The underlying principle of this total quality view ... is that to provide a genuine effectiveness, control must start with the design of the product and end only when the product has been placed in the hands of a customer who remains satisfied...the first principle to recognize is that quality is everybody's job. (13)

According to Feingenbaum, all new products, from their design stage to market, involve more or less the same activities. From the quality viewpoint, they can be grouped into three categories: new design control, materials control, and product control. To be successful, these activities required the cooperation of multiple departments. In fact, as a product moved through the three principal stages, groups as varied as marketing, engineering, purchasing, manufacturing, shipping and customer service had to become involved. Otherwise, errors might be made early in the process that could cause problems to appear later, i.e., during assembly or, worse, after the product was in a customer hands.

1.1.3.3 Reliability engineering

This new discipline emerged at about the same time as total quality control. It relies heavily on probability and statistics concepts and is defined as the probability of a product's performing a specified function without failure, for a given period of time, under specified conditions (14). Supported by modern probability tools, reliability engineering become a formal method for predicting equipment and products performance over time (14). Its widespread applications started with the postwar growth of aerospace and electronics industries in the United States (15). Like total quality control, reliability engineering was aimed at preventing defect from happening in the first place. It too emphasized engineering skills and attention to quality throughout the design process.

1.1.3.4 Zero defects

Zero defects was the last significant development in the quality assurance era. It focused on management expectations and the human relations rather than on the process alone. It has its origin at the martin company in 1961-62 while building Pershing missiles for the American Department of Defense (16). The idea started with awarding incentives to workers to lower the defect rate. These efforts led for the first time on December 12, 1961 to the delivery of the first missile with zero defect. The American Department of Defense (D.O.D) continued to demand for higher quality. One month later, the D.O.D ordered and received in time from the company a Pershing missile with zero defect. At that time, Philip Crosby (1979), another guru of the total quality management (TQM), was involved in activities related to control the cost of the quality for the Pershing. He proposed a larger approach to achieve zero defects. He believed and defended the inclusion of fundamental changes in the organization in addition to the use of statistical methods (17). Despite success with the Pershing, his ideas met resistance with other industries.

1.1.3.5 Strategic quality management

Many managers believe, in fact, that quality assurance was the last important development in the discipline. The quality programs they adopt now are very much similar to those emerged twenty years ago. They always believe on such well established principles as interfunctional coordination, statistical quality control, cost of quality, and zero defects. However, in a number of growing industries, a new vision has begun to emerge. It embodies a dramatic change in perspective. An ever increasing number of top level managers and chief executive officers, have expressed an interest in quality. They have linked it with profitability, defined it from the consumer's point of view, and required its inclusion in the global strategic planning process. From this departure, quality is viewed in many business activities as an aggressive competitive weapon.

1.2 QA/QC in the highway construction industry - Emergence and evolution

According to Tunnicliff et al., (1974), contractor control of quality is not a new concept. Early pioneers in bituminous paving such as Abbott, DeSmedt, and the Barber Asphalt Paving company had built their own quality control systems hundred years ago. To a large extent, they developed their own systems independently. A that time, they had learned that quality control was necessary in order to duplicate successes (18). According to the same author, (1973), Warren Brothers is no newcomer to quality control. Since the company began building patented pavements in 1901, Warren

Brothers had developed a quality control system that was used successfully for over 40 years (19). According to Tunnicliff et al., (1974), although early contractors had control systems, paving specifications that required certain controls were also in use before 1900 (18). By 1920, according to Blanchard (1919) referred by Tunnicliff, control by specifications rather than by contractor was preferred (20).

1.2.1 Traditional construction specifications

The literature reported that the evolution of written specifications as an element of construction contract was primarily linked to the emergence of contracting as a business enterprise from pools of laborers and craftsman that individually hired their services to owner-builders (21). In the United States, road construction contracts emerged during the mid 19^{th} century with the works of William M. Gillespie (1849), who in one of his famous publications states that :

> The actual construction of a road, after its location has been completed, may be carried on ... under the superintendence of the agents of the company, or town, by which it has been undertaken; but it will be more economically executed by contract. (22)

Gillespie's view of the contracting process includes drawings, specifications, advertisement, sealed bids, performance bonds, performance time, penalties, payment schedules and retainages, is exactly what is known today as method specifications or materials and methods specification. Nearly a century and half ago, he defined road contract specifications as :

> ... containing an exact and minute description of the manner of executing the work in all its details. (22)

The intent of such prescriptions in Gillespie's view was to convey to the contractor that which was necessary for him to do the work, in much the same manner that one would convey the same information to one's own employees. Under such a method of specifying work, it is assumed that the owner or the owner's representative knows exactly what he wants to be constructed and how the construction should be completed (21).

1.2.2 Statistically based end-result specifications

The evolution of traditional specifications into forms more appropriate to the complexity of contemporary highway construction has been gradual and continuing process. It has depended on the rapid development of the highway technology in which the quality of the completed work is linked to specific measurable attributes that can be determined by controlling selected materials and construction variables through the processes of design, inspection and testing at the time of construction (21). Method or prescription-type specifications rely heavily on the skill and the integrity of the constructor and the knowledge and the judgment of the inspectors and the engineers overseeing the work. In fact, method specifications are prescriptions where the essential characteristics of the finished work are not known, not measurable and no practical timely acceptance tests is available.

L. Greg Ohrn et al., (1998) reported that one of the problems with this approach is that over the past few decades construction has become more technically oriented and specialized. It appears that while it has been a significant evolution in technical construction knowledge, this information is possessed primarily by the contractors. Owners no longer seem to possess the specialized construction knowledge for building their own facilities (23). Engineers and constructors have followed similar path. Engineers have become technically specialized in the design, but not in the construction processes. Constructors have become technically oriented and specialized in the construction but not in the design and operation of these facilities

According to Willenbrock (1976), another major specification problem surfaced in the early 1960s with the American Association of State Highway Officials (AASHO) Road test program. The AASHO test was designed to determine the effect of variations of traffic loading on pavement structural sections To conduct the test, sections of highway pavements were constructed under a very tight material and construction method specifications. One of the primarily lessons learned from the test was that the materials and construction quality had an unexpectedly high variability. This variability appeared even under the tightly controlled conditions of the test research program (24). For the most part, deviations from the specified tolerances exceeded what was considered normal at that time. Cary et al., (1966) reported that one of the conclusion that emerged from the test was that sampling plans were not adequate for estimating the true characteristics of construction items for which the specifications were written, and therefore a hundred percent compliance with the specification limits could not be guaranteed (25). In essence, the specification cannot arbitrarily set an absolute tolerance without regard to the natural variability of the material and the process.

Given the problem with traditional specifications and the desire of owners to construct high quality facilities, there is a need to develop alternatives to traditional specification methods. According to Chamberlin (1995), one of the early alternatives was to treat construction like other manufacturing processes and to recognize the natural variability of the materials and construction processes. Statistical quality assurance (SQA) processes similar to those used in industrial manufacturing, were proposed for construction. Under this type of specifications, owner's acceptance is based on representative statistical sampling and acceptance procedures. The owner's acceptance procedure recognizes the inherent variability in the materials and processes, and realistic tolerances are allowed (21). While statistically based sampling procedures and end result acceptance criteria can, theoretically, be adopted independent of one another, they have been welded in the literature and practice of highway construction management since the

1960s and have come to be referred to collectively as statistical end-result specifications (ERS).

NCHRP 38 (1976) reported that, statistical specifications based on the end result place the entire responsibility for quality control on the contractor. Quality control includes all activities related to materials and methods of construction and equipment that are necessary to ensure that the final product has the characteristics and the quality desired (26). The owner has the entire responsibility for specifying quality assurance, those activities that ensure that the contractor's quality control is effective. This responsibility requires that the owner periodically spot-check the quality control activities of the contractor to see that sampling is random, that tests are performed correctly, that test results are reported and that documentation is up to date.

1.2.3 Performance-related specifications

While ERS may guarantee improved compliance and improved evidence of compliance, in themselves they do not guarantee improved performance, which depends on a better understanding of the relationship between the factors controlled during construction and the performance and worth of the finished product. The essential performance-related characteristics can only be identified if one knows the relative impact on performance of all of the characteristics thought to be performance-related. Performance related specifications (PRS) have been developed on the foundation of SQA specifications. They address one of the primary weakness of SQA specifications : the failure to quantify and relate the quality characteristics of the materials and construction to the performance of the constructed pavements.

The goal of PRS is to optimize the cost of the constructed pavement in relation to its performance. This can ideally be achieved by materials and construction prescriptions that reflect the best understanding of what defines the quality of the finished work and using these prescriptions in a contractual framework that maximizes cost-effectiveness. While the objectives of PRS were well known for some time, they have been considered as a major component of highway research only since the early 1980s when the Federal Highway Administration (FHWA) instituted a new research program called Performance-Related Specifications for Highway Construction and Rehabilitation. The objective of this research program as stated by Mitchell (1981) is :

> To identify those existing specifications for construction of flexible and rigid pavement structures that relate directly to performance and to develop additional specifications, as needed, to provide complete systems of performance-related specifications for such construction. (27)

An additional objective was to provide a more rational basis for payment adjustment plans, which had been based primarily on experience. Mitchell (1981) reported that an attempt was made to develop PRS for Portland Cement pavements and for Asphalt Concrete surface courses and overlays. Both systems were expected to include requirements that would assure rideability, skid resistance, structural capacity and durability. (27)

FHWA, along with the National Cooperative Highway Research Program (NCHRP) and the American Association of State Highway and Transportation Officials (AASHTO), began extensive research into developing the supporting data needed to PRS formulation. According to Welborn (1984) and Majidzadeh (1984), these projects initiated in 1984 involved extensive literature searches and syntheses of background information for the development of PRS for both AC and PCC pavements (28, 29). Another project of significant importance referenced by Chamberlin (1995), was NCHRP Project 10-26, *Data Bases for Performance-Related Specifications for Highway Construction* (21). The purpose of this project was to examine existing data bases to determine whether the available information could be used for the development of performance models. The idea behind this extensive search was to develop performance models that would be the mechanisms for relating the material and construction testing performed during construction to the performance of the completed work. Unfortunately, the conclusion of this study was that the existing data bases were inadequate to derive the needed performance models. Knowing the weakness of the existing data bases, the NCHRP Project 10-26 team concluded from their studies that further research on PRS should be within a general framework that clearly distinguished among the different classes of variables and would provide for a multi-stage derivation of the required PRS relationships.

According to Tuggle (1992), in 1990, a Transportation Research Board (TRB) task force on research and development needs in highway construction engineering and management identified the development of PRS as their highest priority. The result was that both the NCHRP and the FHWA have funded research projects to develop models to identify the relationships between the engineering properties of a material and the life-cycle performance of the constructed pavement (30). The current research has been focused primarily on surfacing material, Asphalt concrete (AC) and Portland cement concrete (PCC) pavements. The basic framework for PRS research was developed as a result of the AASHTO-sponsored NCHRP Project 10-26. The most important conclusion that emerged from this research reveal that while there exist a number of primary relationships between one or more performance indicators (such as load applications to failure) and known performance predictors (such as layer thickness), most of these relationships include variables that are not amenable to control during construction. As a result, Irick (1988, 1990) concluded that secondary relationships would be required to show the nature and extent of associations between performance predictors and other material and construction factors that are amenable to control (such as concrete mix proportions) (31,32). Much of the subsequent development work for

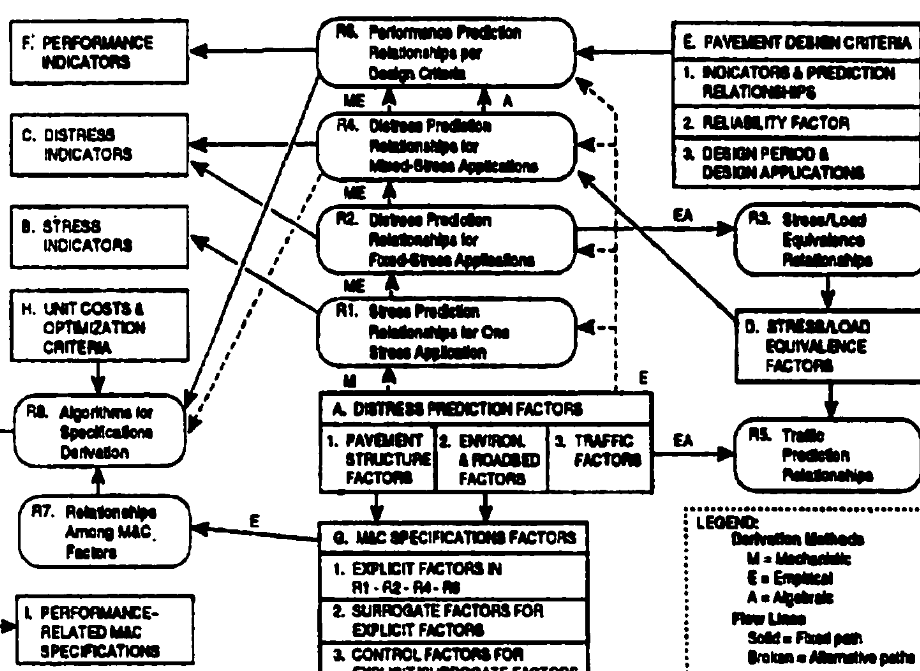
PRS consisted of identifying existing primary and secondary relationships and evaluating their usefulness.

Following NCHRP Project 10-26 both the NCHRP and the FHWA initiated parallel programs for the development of PRS for both AC and PCC pavements. The framework of research categories proposed by Irick et al. (1990) is shown in Figure 1.1 below.

Figure 1.1

Elements of performance related M&C specifications

After Irick et al.,(1990)



Developed and prepared as a working document for the advisory panel of NCHRP Report 332 (33), this framework describes the relationships among the variables that characterize the design, the construction and the service phases of pavements. It permits one to better visualize the relationships that must be developed to craft specification that are truly performance-related, that is specifications in which variations in the materials and construction factors controlled during construction have a known and quantifiable relationship to variations in the performance of the finished work. Greg Ohrn (1998) summarized this framework as follows (23) :

Explicit predictors for stress/Distress/Performance

This is a major portion of the database that will be used to develop the primary and secondary relationships. This database is composed of observational and experimental data, and is divided into four subcategories :

1. traffic factors, such as vehicle mix and rates and equivalent single axle loading (ESAL);
2. environmental factors, such as climate, drainage and geometry;
3. roadbed soil factors, such as composition, susceptibility, and response factors;
4. structural factors, such as surfacing, base and subbase.

Material and Construction Factors

This is the major portion of the database that will be used to develop the primary and secondary relationships. It is composed of three subcategories :

1. surrogates for explicit predictors;
2. control factors for explicit predictors and surrogates;
3. process control factors.

Primary Relationships

Primary relationships are those used to predict pavement stress, distress, and performance from a particular combination of predictors that represent traffic relationships, roadbed soil factors and pavement structural conditions. It is divided into three categories :

1. stress prediction relationships, such as stress indicators and stress predictors;
2. distress prediction relationships, such as distress indicators and distress predictors;
3. performance prediction relationships, such as performance indicators and performance predictors.

Secondary Relationships

Secondary relationships include all equations or algorithms that show interrelationships among material and construction factors. By definition, secondary relationships do not contain indicators for stress/distress/performance, but should account for all material and construction factors that are explicit predictors in the primary relationships.

Algorithms for pavement design and material and construction specifications

This is the combination of the primary and secondary relationships with pavement design criteria, material and construction resources, unit costs, and specification criteria. The results of this development process phase are performance/cost predictions and acceptance/payment plans.

Performance-related material and construction specifications

This is planned as the final phase of the development process where actual PRS are developed. This phase includes the development of target levels, specification limits, acceptance plans and payment plans.

1.2.4 Adjustable payment plans

Traditionally, highway agencies used method-type specifications for specifying and accepting highway materials and construction. With this type of specifications, the contractor is directed to combine specific materials in definite proportions, use specific type of equipment, and place the material or product in a prescribed way. Each step is controlled and in many cases directed by a representative of the highway agency. If the contractor adheres fully to the prescribed specifications, a hundred percent payment to the contractor is assured. A major deficiency of this method-type specifications is that penalties for contractor nonconformance are somewhat arbitrary and based solely on the skills and judgment of the inspector. Deficient work was either removed or, at the discretion of the engineer, accepted at full price or at a reduced price. When price reductions were applied, they were typically negotiated on a case-by-case basis after the fact. In this method-type specifications, statistical concepts are seldom employed.

1.2.4.1 Judgment plans

According to Moore et al., (1981) pay schedules were based initially on judgment, and then modified as the result of experience under actual contract conditions. Judgment plans are not supported by a relationship that quantitatively links the payment schedule to the anticipated performance of the as-constructed pavement and therefore are not considered as rational plans (34). They are typically keyed to the average value of the quality characteristic being measured, to the frequency of deficiencies, or to the percentage of the work within tolerance. In those instances where acceptance was based on more than one quality characteristic for the same item (i.e., pavement density and thickness), payment was based on the item with the lowest pay factor, on the average of the individual pay factors, or on their product.

1.2.4.2 Rational plans

These plans are intended to compensate contractors in proportion to the level of quality estimated to be achieved. They must reflect the actual diminished or enhanced worth of the completed work, or some identifiable costs associated with its construction. Willenbrock (1977) et al., investigating approaches on to the development of rational payment plans, concluded that the more logical approaches to establishing payment reduction are those based on quality characteristics with known and mathematically quantifiable relationships to the level of performance or serviceability anticipated. In such a plan, the adjusted unit price is related directly to the expected percentage loss or enhancement in performance or serviceability. For instance, if a correlation between the thickness of a new asphalt concrete pavement and its service life has been established, price adjustments would be applied that are in proportion to the reduction or enhancement in the expected service life by the difference between the as-built thickness and the design thickness (24). The use of such adjustment plans include a quality level below which the work is unacceptable, and the payment schedule applies only to quality levels between that value and the design value, or greater.

Since the early 1980s, the development of adjustable payment plans has focused on approaches that are based on some measure of the anticipated cost associated with diminished or enhanced performance, rather than on incremental differences in the performance itself. To do this, the material and construction variables related to performance, and over which the contractor has control, should be identified and dissociated from those over which the contractor has no control. Ideally, these selected material and construction variables must then be related to the pavement performance by some mathematical algorithms, ideally the same algorithms used for designing the pavement and for the specifications. Given the performance algorithms, the anticipated performance of the as-constructed pavement can be predicted from results of acceptance tests and compared to the as-designed pavement. The relative cost difference can be obtained using any appropriate engineering economic principles.

CHAPTER 2

EXPERIMENTAL DESIGN

2.1 Historical background

The literature reported that the basic concepts of design of experiments were developed in the 1920's in England by Sir Ronald A. Fisher. Primarily he used these new concepts to agricultural experimentation. He published his first classical book entitled, Design of Experiments, in 1935. Other scientists and statisticians followed Fisher's concepts through extensive theoretical research and development. Among the most interesting findings, the works published by Plackett and Burman in 1946, « Multifactorial Experiment ». These findings provide the bases of what is called screening experiments. This form of experimental design is used to identify the variables which most strongly affect the results of an experiment. Since then, American and English statisticians continued to develop experimental design theories and applications that were mainly oriented to the agricultural field. At the end of the twentieth century, it has been introduced to other disciplines, such as manufacturing and related industries (35). More recently, design of experiment techniques have gained wide applications in the transportation field, and predominantly in pavement deterioration modeling and pavement design.

2.2 Experimental design concepts and procedures

Irick (1988) defined an experimental design for the development of prediction equations as a well defined plan for assessing the effects of sets of independent variables

on one or more dependent variables. The experimental plan should specify the experimental units and the treatments applied to them. The experimental units consist of either road test sections,
laboratory test specimens of pavement layers or components or a computer simulation (31). Experimental design applications involve five major steps:

1. definitions for the dependent variables and their indicators;
2. identification and specifications of those independent variables and corresponding indicators that determine the physical makeup of the experimental units;
3. specification for other independent variables such as the traffic loading and environment;
4. specification of the prediction factor effects that must be assessed; and
5. specification of the inference space within which the derived prediction equation is to be valid.

To develop a factorial design, at least two levels must be specified for the independent variables to cover the range of interest for each prediction factor. Three or more levels may be necessary for attribute factors that have more than two classes or for variable factors whose effects are likely to be non-linear. If every possible combination for the independent variables is used, the result is a complete factorial design. This design makes it possible to observe every possible main effect and interaction effect of the prediction factors, and more important, to separate analytically each effect from all other effects. If the total number of factor level combinations is large, a partial or fractional factorial design may be used, but it then becomes impossible to separate certain effects from one another. In such cases, the fractional design generally produces certain confounding of higher order interaction effects with lower order interaction effects.

After the factorial combinations that are of most interest are used, the next phase of the experimental design is to specify the procedures that will be used to place and replicate the experimental units and treatments within the inference space. Proper placement of experimental units within the inference space requires a complete randomization procedure of the experimental units. This randomization provides assurance that all units are subject to all error variables, that the prediction factor effects are unbiased relative to error effects, and that unbiased estimates can be calculated for the error effects. To observe the error effects explicitly, it is necessary to replicate at least some factor-level combinations within the inference space. In any case, replications must provide a sufficient basis for the estimation of error variance, and therefore for the assessment of prediction equation precision.

After the experimental design has been implemented, the corresponding data provide values for all dependent variables that were set forth in the experimental design. Computer software for several types of statistical analysis are available for derivation of the required prediction equations. These computer software include methods such as analysis of variance (ANOVA), analysis of covariance and linear and nonlinear regression analysis. These analytical tools make it possible to attain the following objectives :

1. estimates of prediction factor (EPF) effects;
2. estimates of error effects;
3. significance assessments of prediction factor effects;
4. estimate of model constants;
5. precision estimates for individual constants;
6. precision estimates for predictions.

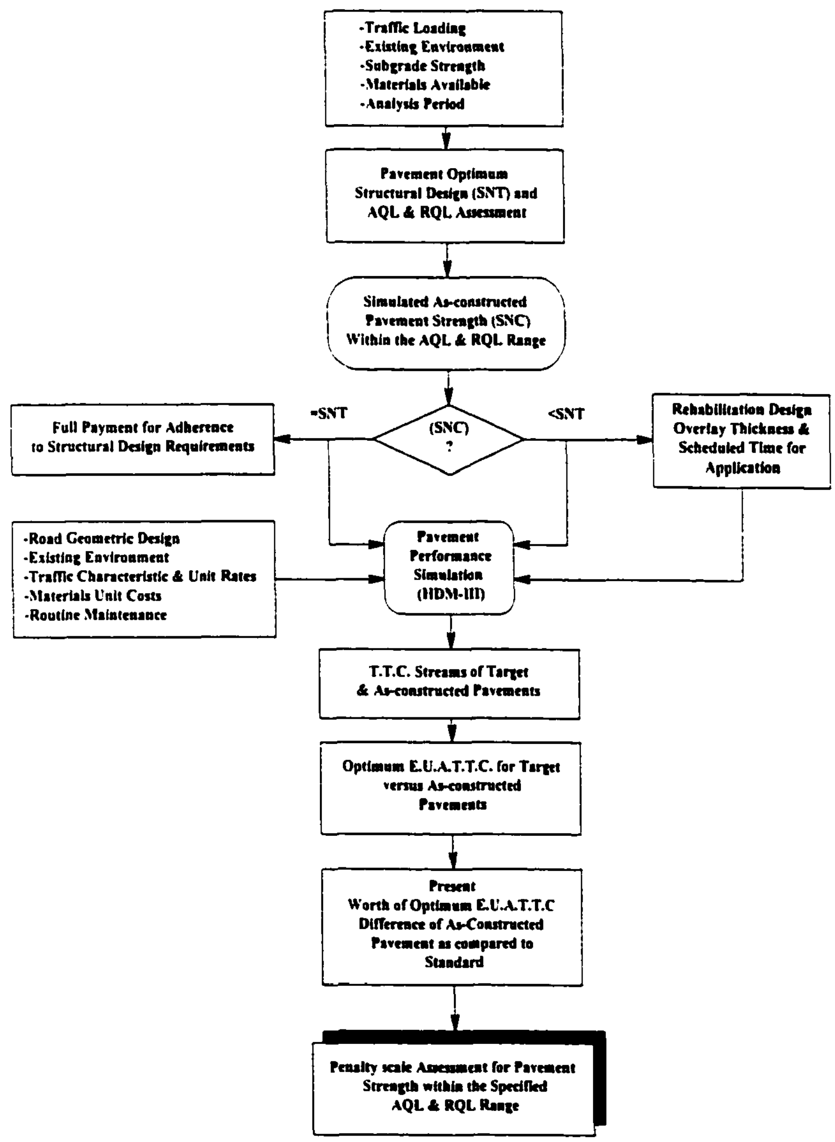
Objective 1 is to quantify the amount of variation in an observation that is attributable to each main effect and interacting effect of the predictor variables. Objective 2 is to quantify the amount of variation in an observation that is attributable to error variables. If the experimental design involves restricted randomization, then the objective 2 includes the partition of total error variation into subsets that correspond to various levels of error variation. Objective 3 is attained through statistical significance tests that compare each EPF effect with appropriate error effects. If any comparison produces a significant effect at a given probability level, it is inferred that the relevant EPF effect is real, that is, the effect truly exists within the inference space of the experimental data. The selected significance level represents the magnitude of the controlled chance that the inference is erroneous. Thus the results derived show not only which factor effects are significant, but also how much of the total variation in an observation can be attributed to factor effects and how much can be attributed to error variables. Analysis of variance and covariance therefore set the stage for regression analysis by determining just which factors and combinations thereof are statistically significant components of the regression models.

2.3 Penalty scale establishment procedure

To assess penalties to contractors for lack of adherence to design specifications, a full three level factorial design is selected so that all interactions between the independent variables could be investigated and considered in the analysis. A set of pavement types and strengths constructed on weak, fair and strong subgrades and subjected to a range of traffic representing the conditions prevailing in most developing countries will be selected. The unit material and labour costs and the traffic characteristics and unit prices used in the simulations are in accordance with those applicable in West Africa. The three level factorial design for the independent variables, i.e., traffic loading, subgrade CBR and the relative loss in pavement full strength ($\Delta SN/SN$) is chosen because of the non-linearity of the AASHTO design equation and the complexity of HDM-III empirical algorithms. Figure 2.1 below illustrates the penalty scale development procedure to be adopted in this study.

Figure 2.1

Penalty scale establishment procedure



where

SNC	=	as-constructed pavement structural number;
SNT	=	as-designed pavement structural number;
T.T.C	=	total transport cost;
E.U.A.T.T.C	=	equivalent uniform annual total transport cost.

This study will be conducted in two stages namely the design / construction stage and the operation stage, i.e., HDM-III simulations. For both stages, pavement strengths are expressed in terms of the commonly known AASHO structural number (SN) used in AASHTO flexible design equation and as input to the HDM-III program. The SN coefficient is described as a linear combination of coefficients of individual materials and thickness of pavement layers above the subgrade. Using the AASHTO flexible design equation, the AASHTO overlay design method based on the remaining life approach, and the performance and economic models incorporated in the Highway Design and Maintenance Standards of the World Bank (HDM-III), the procedure adopted in this study is based on total society cost which includes construction, maintenance and rehabilitation (M&R) and user costs for a twenty years analysis period. The discount rate is fixed at 12 percent. Contractor cost responsibility is calculated from the present worth of the discounted total cost difference that results from the contractor's inferior performance compared to standards.

2.3.1 Independent variables

The independent variables considered in the current study are: the traffic loading expressed in equivalent single axial load (ESAL), the subgrade CBR and the relative loss in pavement full strength expressed in percent effective structural number (ΔSN/SN).

The three levels design for the independent variables is chosen because of the non-linearity of the AASHTO design equation and HDM-III complex algorithms.

2.3.1.1 Traffic loading

Low volume roads in developing countries are subjected to a wide range of mixed traffic levels. According to Queiroz et al. (1987), typical traffic volumes and compositions are approximately 500 to 5000 vehicles per day in which commercial vehicles are in the general range of 20 to 60 percent (36).

In this study, three two-way average annual daily traffic levels are considered: 500 vpd to 1 000 vpd, 1 250 vpd to 2 500 vpd and 2 000 to 4 000 vpd. Heavy vehicles are set at 40 percent of the total annual average daily traffic (AADT). Annual traffic growth and composition are summarized in Table 2.1 below. Case 2 and 3 loading are taken as 2.5 and 4 times the traffic composition for case 1 loading, respectively. These vehicle attributes are typical of West African roads.

Table 2.1

Traffic levels and composition

Average Annual Daily Traffic	Car	Pickup	Bus	Light truck	Medium Truck	Heavy Truck	Articulated Truck
Traffic 1	150	100	40	30	60	70	50
Traffic 2	375	250	100	75	150	175	125
Traffic 3	600	400	160	120	240	280	200
Traffic Growth (%)	4	4	3.4	3.4	3.4	3.4	3.5

2.3.1.2 Roadbed soil resilient modulus

To illustrate the implication of constructions in different soils, three subgrade strengths of CBR of 6, 10 and 14 percent were selected.

2.3.1.3 Lack of pavement full strength

The Structural number, the output of the AASHTO design algorithm, is a measure of pavement strength. It summarizes the complex interactions between material type and stiffness, layer thickness and depth, subgrade stiffness and surface condition. It is expressed as follows :

$$SN = \Sigma(m_i * a_i * h_i) \tag{2.1}$$

where

m_i = drainage coefficient of the layer i taken as 1 for asphalt concrete and 0.8 for base;
a_i = structural (strength) coefficient of the layer i;
h_i = thickness of the layer i in inches.

The structural coefficient is correlated to the modulus of elasticity of each layer. Table 2.5 in the AASHTO guide (1993) provides a chart that may be used to estimate the structural layer coefficient of dense-graded asphalt concrete surface course based on its resilient modulus (E_{AC}) at 68^0F. For granular base layers, Figure 2.6 of the same guide provides a chart that may be used to estimate the structural layer coefficient, a_2, from one of the four different laboratory test results, including base resilient modulus, EBS. Granular base layer coefficient can also be obtained from the following AASHTO empirical relationship (37):

$$a_2 = 0.249(\log_{10} EBS) - 0.977 \quad (2.2)$$

where EBS is a function of the stress state within the layer.

In the current study, to simulate the lack of QA/QC in pavement construction phase, reductions in pavement full strength expressed in percent effective structural number ($\Delta SN/SN$) of 90 percent and 80 percent of the target SN obtained from AASHTO design equation were assumed. Reductions in allowable load applications resulting from reduced pavement strength are obtained from the same AASHTO flexible design equation. To withstand the traffic loading for the entire analysis period, as-constructed pavements were rehabilitated using AASHTO overlay design method based on the remaining life approach (37).

2.3.2 Dependent variable

The dependent variable is the resulting penalty to be assessed to the contractor (PF) expressed in percent of the initial contract bid price. Pavement performance simulation procedures are based on HDM performance and economic prediction models. DARWin (pavement Design, Analysis, and Rehabilitation for Windows) outputs (SN) calculated for each pavement strength serve as direct inputs to HDM-III. HDM-III performance and economic evaluation models are capable of predicting pavement deterioration over time and assigning costs to the various target (as-designed) and as-constructed pavement alternatives. These costs include the initial road construction and future maintenance and rehabilitation. Furthermore, user costs associated with the various alternatives (i.e., vehicle operating costs and user delay costs during maintenance and overlay) are also included.

2.3.3 Design matrix

Table 2.2 below shows the design matrix comprising all variable combinations for a total of twenty seven experiment simulations. These experiments are to be run on HDM-III model using the results obtained from the AASHTO design equation, the AASHTO overlay design method based on the remaining life approach, and the lack of pavement full strength as direct inputs. Factors combinations do not need to be randomized.

Table 2.2

Experimental matrix for
penalty function assessment

TRAFFIC LOADS (ESALs)	TRAFFIC 1			TRAFFIC 2			TRAFFIC 3		
SUBGRADE CBR (%)	6	10	14	6	10	14	6	10	14
PERCENT EFFECTIVE PAVEMENT STRENGTH	100 % 90 % 80 %	100 % 90 % 80 %	100 % 90 % 80 %	100 % 90 % 80 %	100 % 90 % 80 %	100 % 90 % 80 %	100 % 90 % 80 %	100 % 90 % 80 %	100 % 90 % 80 %

CHAPTER 3

PAVEMENT DESIGN

3.1 Introduction

Design of highways and airfield pavements involves selection of materials and determination of thickness of various layers that should be used in pavement construction so that the pavement layers are stable and carry the traffic during the design period without any major maintenance. Highway pavements and paved roads in most developing countries for moderate to low volume traffic consist essentially of granular materials in the form of crashed rock or laterites with thin bituminous surfacing. The use of laterites in road construction is generally limited to the use of the laterite as it is extracted and selected to their gradation. The surfacing consists generally of hot mixed asphalt concrete or a double surface treatment.

3.2 Pavement design

Bhandari et al., (1987) defines a normal full strength pavement as one designed on the bases of accepted pavement engineering principles to carry a specific number of cumulative equivalent standard axle loads (ESAL) until an acceptable level of functional serviceability is reached (38). Watanatada et al. (1987), reported that the HDM-III model is not intended to be used in final engineering design, but rather it is a tool for economic analysis of alternative road standards, either at the project or network levels (39). Two recognized general approaches to the design of pavement structures are in practice today :

(a) the empirical approach such as the AASHTO method of design, which relies on the experience of the user and subsequent correlation with performance and,

(b) the rational or mechanistic approach, which is primarily based on theoretical concepts of modeling structural behavior.

The AASHTO performance equation is a commonly used empirical performance model for predicting pavement deterioration in terms of the Present Serviceability Index (PSI). This equation was developed with data from AASHO Road Test and predicts the number of 18-kip equivalent single axle loads (ESALs) before the PSI reaches a specified terminal serviceability level. According to Richard M. Weed (1984), for a given pavement thickness and corresponding strength, the AASHTO design procedure provides a convenient way to relate as built quality to service life. Although it may eventually be supplanted by other design methods, the AASHTO procedure is widely used and provides basic guidance on the relative importance of key quality measures (40). The equation is empirically derived and relates the number of 18-kip ESALs to pavement layer thichnesses, soil resilient modulus, and environmental conditions. Rex (1964), also suggests the use of the AASHTO Guide for the Design of Flexible Pavement Structures as a method of determining penalties for a deficiency of thickness of asphalt pavements. Using a normal 3 in. (75 mm) surface course as an example, he estimates that the loss of serviceability, in terms of equivalent axle loads, of a 0.25 in. (6 mm) reduction in thickness would be 17 percent, and of a 0.5 in (13 mm) reduction would result in a loss of 33 percent (41).

The design procedure adopted in this study is based on the empirical AASHTO design algorithm incorporated in a computer program called DARWin. The input parameters for each pavement type to be designed comprise the traffic loading over the analysis period, expressed in ESALs, initial and terminal pavement serviceability index

(PSI), reliability level (R), overall standard deviation (S_0), roadbed soil resilient modulus (Mr) and material properties. The outputs are the thicknesses of base and surface layers.

3.3 Review of the AASHTO design procedure

Today, in most countries in the world, flexible pavements are designed or evaluated by using the procedure developed from the American Association of State Highway Officials (AASHO). David R. Luhr (1980) reported that the AASHTO pavement design method was developed by using the results from the AASHO road test conducted on October 1958 through November 1959 near Ottawa, Illinois. This carefully engineered experiment included 6 loops and 468 test sections of asphalt pavement that were subjected to traffic loads ranging from 9 kN (2 kip) single axles to 214 kN (48 kip) tandem axles. These test sections were monitored to determine how different pavement thicknesses and traffic loads affected pavement performance. Pavement performance was subjectively measured by a panel of raters by using a Present Serviceability Rating (PSR) that ranges from 0 for very poor to 5 for excellent. Correlation of the panel ratings of performance with measurement of cracking, rut depth, and roughness gave a more quantitative Present Serviceability Index (PSI), so that the condition of the pavement could be accurately determined by measurement, rather than by assembling a panel of raters. The analysis of road test data resulted in a design method that determines the strength required of a pavement structure to withstand a given number of load applications before the performance of the pavement reaches a given minimum or terminal PSI. The required strength of a pavement is given in terms of the structural number (SN), which is the thickness of each pavement layer, multiplied by a strength coefficient for each layer, summed over all the layers of the pavement (42).

In 1986 the American Association for State Highway and Transportation Officials (AASHTO) published its comprehensive Guide for Design of Pavement Structures containing major improvements from previous editions and reflecting research findings from the previous 20 years. The latest version revised in 1993 of the AASHTO design algorithms based on serviceability loss is expressed as (37) :

$$\log_{10} W_{18} = Z_r S_o + 9.36\log_{10}(SN+1) - 0.20 + \log_{10} [\Delta PSI/(4.2-1.5)]/[0.40 + 1094/(SN+1)^{5.19}] + 2.32\log_{10} M_r - 8.07 \quad (3.1)$$

where

W_{18} = estimated future traffic for the performance period expressed in ESALs ;

Z_r = number of standard deviations (from the standard normal distribution curve) corresponding to reliability level R ;

S_o = overall standard deviation ;

SN = required structural number to sustain traffic applications in the performance period ;

ΔPSI = design serviceability loss

M_r = effective resilient modulus of roadbed material.

3.4 DARWin input parameters

The input parameters required are the estimated total cumulative traffic loading expressed in equivalent single axial load (ESALs) that the pavement should sustain during the specified performance period without any major repair, the effective roadbed soil resilient modulus (M_r), the initial and terminal pavement serviceability index (PSI), the reliability level and the standard deviation.

3.4.1 Analysis period

It is the period of time for which the analysis is to be conducted, i.e., the length of time that any design strategy must cover. The analysis period is analogous to the design life used by pavement design engineers. AASHTO Design Guide for Pavement Structures (1993) suggests for low volume paved roads a design life between 15 and 25 years (37). In this study a 20 years analysis period is selected.

3.4.2 Traffic loading

Traffic loading can be converted in total cumulative equivalent single axial loads by using the following formula :

$$W_{18} = AADT * \%D * \%L * \% HV * TF * Nd * fa \tag{3.2}$$

where

W_{18}	=	total cumulative equivalent single axial load (ESALs) ;
AADT	=	average annual daily traffic;
%D	=	vehicles per direction in percent;
%L	=	vehicles per lane in percent;
%HV	=	heavy vehicles in percent;
TF	=	truck factor;
Nd	=	number of days in the year;
fa	=	$[(1+g)^n - 1]/g$
fa	=	heavy traffic growth factor;
g	=	compound traffic growth rate;
n	=	analysis period.

In this study, for consistency, the HDM traffic model has been used to estimate total ESALs for the analysis period. The cumulative one-way traffic loading in ESALs are :

Traffic 1 = 2 792 500

Traffic 2 = 6 981 050

Traffic 3 = 11 169 600

3.4.3 Roadbed soil resilient modulus

To adopt CBR values to AASHTO flexible design equation, Heukelom and Klomp (1962) have reported a correlation between CBR and roadbed soil modulus (Mr) using dynamic compaction and in situ soil modulus as follows (43):

$$Mr = 1\,500 * CBR \tag{3.3}$$

The coefficient of CBR in this relation varied from 750 to 3000. However, this relationship has been extensively used by design agencies and researchers and is considered reasonable for fine grained soils. The converted CBR values expressed in pounds per square inch (psi) are illustrated in Table 3.1 below.

Table 3.1

Soil resilient modulus

Soil CBR (%)	Soil Resilient Modulus (psi)
CBR 1 = 6	9,000
CBR 2 = 10	15,000
CBR 3 = 14	21,000

3.4.4 Serviceability

According to AASHTO Design Guide (1993), the serviceability of a pavement is defined as its ability to serve the traffic which use the facility. The primary measure of serviceability is the Present Serviceability Index (PSI), which ranges from 0 for impossible road to 5 for a perfect road. Selection of the lowest serviceability index (PSI_f) is based on the lowest index that will be tolerated before major rehabilitation, resurfacing or reconstruction becomes necessary. AASHTO Guide for Design Of Pavement Structures suggested a terminal serviceability index of 2.5 or higher for the design of major highways and 2 for highways of lesser traffic volumes and for relatively minor highways (37). In this study initial and terminal serviceability index (PSI) are fixed at 4 and 2 respectively.

3.4.5 Reliability and standard deviation

The 1986 AASHTO Guide (1993) define reliability as :

> The reliability of a pavement design performance process is the probability that a pavement section designed using the process will perform satisfactorily over the traffic and environmental conditions for the design period (37).

For a given reliability level (R), the reliability factor is a function of the overall standard deviation (S_0) that accounts for both chance variation in the traffic prediction and normal variation in pavement performance prediction for a given W_{18}. The 1993 AASHTO Guide outlines the procedure for the application of the reliability concepts. In this study, the reliability level and the overall standard deviation are fixed respectively at 75 percent and 0.44.

3.5 DARWin output results

In this study, the respective elastic modulus for the asphalt concrete surfacing and the granular base layer are fixed at 400 000 psi and 30 000 psi and the drainage coefficients are respectively 1 and 0.8. Using the above input data, DARWin output results expressed in terms of calculated SN for the set of pavement types, anticipated traffic loading and subgrade CBR are summarized in Table 3.2 below. Layer thicknesses above the subgrade were obtained using minimum surfacing thicknesses suggested by the AASHTO Interim Guide for Design of Pavement Structures (1993). The strength of the double bitumen surface treatment (DBST) associated with case 1 loading was neglected (37). These designed pavement structures are considered as-target pavements. They are used in this study as a reference to assess contractor's bid price for construction purposes.

Table 3.2

Target pavements structural strength

TRAFFIC	CBR 1 = 6%		CBR 2 = 10%		CBR 3 = 14%	
	STRUCT. LAYERS (in)	SN	STRUCT. LAYERS (in)	SN	STRUCT. LAYERS (in)	SN
T1 = 2 792 500 (ESALs)/ Lane	DBST 0.59 GB 29.91 Total 30.50	0.00 3.35 3.35	DBST 0.59 GB 24.91 Total 25.50	0.00 2.79 2.79	DBST 0.59 GB 21.96 Total 22.55	0.00 2.46 2.46
T2 = 6 981 050 (ESALs)/ Lane	AC 3.50 GB 21.34 Total 24.84	1.47 2.39 3.86	AC 3.50 GB 15.63 Total 19.13	1.47 1.75 3.22	AC 3.50 GB 12.32 Total 15.82	1.47 1.38 2.85
T3 = 11 169 600 (ESALs)/ Lane	AC 4.00 GB 21.88 Total 25.88	1.68 2.45 4.13	AC 4.00 GB 15.89 Total 19.89	1.68 1.78 3.46	AC 4.00 GB 12.32 Total 16.32	1.68 1.38 3.06

3.6 Sensitivity of AASHTO design equation

For the development of pay functions for performance related specifications (PRS) and end result specifications (ERS), it is recommended that a sensitivity analysis should be conducted for all the models to be used to evaluate the sensitivity of the performance prediction to various pavement design factors and to evaluate the effects of these pavement design factors and their interactions on the predicted pavement performance (33).

The AASHTO flexible design equation expresses the number of applications of an equivalent 18-kip load (W_{18}) that the pavement can sustain as a function of its structural strength, subgrade resilient modulus, initial and terminal pavement serviceability index (PSI) and environment. The pavement structural strength is expressed in terms of the structural number required for given combinations of soil support, total traffic expressed in equivalent single axle loads (ESALs), initial and terminal PSI and the environment. The required SN is related to the layer thicknesses by means of layer coefficients representing the relative strength of the construction materials. The layer coefficients are based on the elastic modulus and have been determined based on stress and strain calculations in a multilayered pavement system. According to AASHO empirical research, the SN is defined as a linear combination of the layer strength coefficients, thicknesses of the individual layers above the subgrade and the permeability of the construction materials.

To investigate how durability and design variables affect pavement strength in the context of the AASHTO procedure, a sensitivity study is conducted by varying one design variable at the time. It is recognized that because the AASHTO model is based on regression equations developed from AASHO Road Test Data, this sensitivity may not agree with other design procedures. Nonetheless, the analysis provides an idea of the impact that varying the input variables has on the load capability in the AASHTO design.

In this study, a sensitivity analysis is conducted for double bitumen surface treatment and asphalt concrete pavements. The two-way AADT for both as-designed pavements are 500/1 000 vpd and 2 000/4 000 vpd respectively and the subgrade CBR is fixed at 10 percent. For the asphalt concrete pavement, the independent variables are AC thickness (h_{ac}), AC strength coefficient (a_{ac}), base thickness (h_{gb}), base strength coefficient (a_{gb}) and base drainage coefficient (m_{gb}). The same independent variables are used for DBST pavement except for the surfacing which is neglected. The traffic loading, soil CBR, reliability and standard deviation are kept constant. To quantify sensitivity, the corresponding percentage of change in the structural strength ($\Delta SN/SN$) and in the load carrying capability per 10 percent change increment in the base values of the design variables are calculated and presented in Appendix A, Tables 1A through 10A. The results are summarized in Tables 3.3 (a), 3.3 (b), 3.4 (a), and 3.4 (b) below.

Table 3.3 (a)

Percentage change in pavement strength per 10 percent change increment in design variables base values for a two-way AADT of 500/1 000 vpd and a CBR of 10 percent

Design variables base values	Percent change in design variables base values										
	-50	-40	-30	-20	-10	0	+10	+20	+30	+40	+50
	Percent change in pavement strength ($\Delta SN/SN$) *100										
h_{gb} = 24.91 in.	-50	-40	-30	-20	-10	0	10	20	30	40	50
a_{gb} = 0.14	-50	-40	-30	-20	-10	0	10	20	30	40	50
m_{gb} = 0.8	-50	-40	-30	-20	-10	0	10	20	30	40	50

Table 3.3 (a) above shows that the percentage of change increment in the pavement strength is directly proportional to the corresponding percentage of change increment in the design base values (one percent change in material properties for one percent change in load carrying capability). It also shows, that deviations in any of the granular base properties (layer thickness, and structural and drainage coefficients) from the design base value have the same impact on the pavement structural strength.

Table 3.3 (b)

Percentage change in load carrying capacity per 10 percent change increment in design variables base values for a two-way AADT of 500/1 000 vpd and a CBR of 10 percent

Design variables base values	Percent change in design variables base values										
	-50	-40	-30	-20	-10	0	+10	+20	+30	+40	+50
	Percent change in load carrying capacity $(\Delta W_{18}/W_{18}) * 100$										
h_{gb} = 24.91 in.	-98	-95	-89	-75	-48	0	85	219	439	783	1312
a_{gb} = 0.14	-98	-95	-89	-75	-48	0	85	219	439	783	1312
m_{gb} = 0.8	-98	-95	-89	-75	-48	0	85	219	439	783	1312

Table 3.3 (b) above and Figure 1A (appendix A) summarize the impact of incremental changes in the pavement carrying capacity per 10 percent change increment in the design base values. These results show that the load carrying capability of the pavement drops drastically to almost half its designed value for 10 percent decrease in any material base characteristics. For 20 percent material deficiency, the pavement loses its strength by 75 percent. At 50 percent material deficiency, the pavement becomes unstable and cannot fulfill its intended function.

Table 3.4 (a)

Percentage change in pavement strength per 10 percent change Increment in design variables base values for a two-way AADT of 2 000/4 000 vpd and a CBR of 10 percent

Design variables base values	Percent change in design variables base values										
	-50	-40	-30	-20	-10	0	+10	+20	+30	+40	+50
	Percent change in pavement strength ($\Delta SN/SN$) *100										
h_{ac} = 4 in.	-24.3	-19.5	-14.6	-9.8	-4.9	0	4.9	9.8	14.6	19.5	24.3
a_{ac} = 0.42	-24.3	-19.5	-14.6	-9.8	-4.9	0	4.9	9.8	14.6	19.5	24.3
h_{gb} = 15.89 in.	-25.8	-20.6	-15.5	-10.3	-5.2	0	5.2	10.3	15.5	20.6	25.8
a_{gb} = 0.14	-25.8	-20.6	-15.5	-10.3	-5.2	0	5.2	10.3	15.5	20.6	25.8
m_{gb} = 0.8	-25.8	-20.6	-15.5	-10.3	-5.2	0	5.2	10.3	15.5	20.6	25.8

Table 3.4 (a) above shows that the percentage of change increment in the pavement strength is proportional to the percentage of change increment in the design base values (two percent change in material properties for one percent change in load carrying capability). It also shows, that deviations in any of the pavement material properties (layer thickness, and drainage and structural coefficients) from the design base value have almost the same impact on the pavement structural strength. In this case study, granular base properties have slightly higher impacts.

Table 3.4 (b) below and Figure 1A (appendix A) summarize the impact of changes in the pavement carrying capacity per 10 percent change increment in the design base values. These results show that the load carrying capability of the pavement drops drastically to almost half its designed value for 20 percent decrease in any pavement material base characteristics. For 40 percent material deficiency, the pavement loses 75 percent of its designed strength. At 50 percent deficiency, the remaining pavement load carrying capability is only of the order of 15 percent of its designed value.

Table 3.4 (b)

Percentage change in load carrying capacity per 10 percent change

Increment in design variables base values for a two-way AADT

of 2 000/4 000 vpd and a CBR of 10 percent

Design variables base values	Percent change in design variables base values										
	-50	-40	-30	-20	-10	0	+10	+20	+30	+40	+50
	Percent change in load carrying capacity $(\Delta W_{18}/W_{18})$ *100										
h_{ac} = 4 in.	-83	-75	-64	-48	-27	0	34	82	144	225	329
a_{ac} = 0.42	-83	-75	-64	-48	-27	0	34	82	144	225	329
h_{gb} = 15.89 in.	-85	-77	-66	-50	-29	0	37	89	157	243	364
a_{gb} = 0.14	-85	-77	-66	-50	-29	0	37	89	157	243	364
m_{gb} = 0.8	-85	-77	-66	-50	-29	0	37	89	157	243	364

3.7. Analysis of results

From this sensitivity study it can be concluded that :

1. for any traffic loading and pavement type, the AASHTO flexible design equation is very sensitive to the changes in the material properties and construction methods through the structural number that summarizes the complex interaction between material properties and stiffness, layer thickness, and subgrade stiffness.

2. the structural capability of full-depth granular pavements (DBST pavements) designed to serve low traffic volumes is more sensitive to changes in construction material properties than AC pavements designed for higher traffic volumes.

3. for any traffic loading and pavement type, the impact on pavement service life of using higher material properties and layer thicknesses than the design base values is more significant than poorer material properties and layer thicknesses.

4. material properties such as aggregate's type, gradation and shape and binder content can be screened for acceptance or rejection at the pavement construction phase and a rejectable quality limit (RQL) for layer stiffnesses and thicknesses can be assessed for removal and replacement or acceptance with minimum payment to the contractor.

CHAPTER 4

PAVEMENT PERFORMANCE SIMULATION

4.1 Background

Paved roads deterioration is defined by the trend of its surface condition with time. Surface defects are classified in three major modes of distress: cracking, disintegration and permanent deformations. These major modes of distress are classified by distress type, extent and severity and are namely: cracking, raveling, potholing, rutting, and polishing of the aggregates. Cracking, raveling and potholing develop in two phases. The first phase is the initiation of the distress beginning immediately after the construction of the pavement and ending just before their appearance at the pavement surface. The second phase is characterized by the progression of the distress and during which it develops progressively in extent and severity. These types of distress develop through a number of different mechanisms. Traffic induced stresses and strains in the pavement layers are dependent on the stiffness of materials and layer thickness. These induced stresses and strains give rise to the initiation of cracking through surface fatigue and deformations in all layers to a certain degree dependent on material properties and characteristics.

Cracking and disintegration of pavement surfacing are susceptible to environment and climate effects. Once initiated, cracks develop and spread with increasing severity to a point where spalling and ultimately potholes develop. The intrusion of excess water into the pavement structure through surface cracking and the lack of adequate drainage systems will accelerate the disintegration process, reducing the shearing strength of the unbound granular materials and thus increasing the rate of deformation induced by the traffic loading. Deformations drawn in all layers of the pavement will appear in the form of rutting in the wheelpaths and more generally in the form of distortion at the pavement surface commonly called roughness. The environment, the climate and the drainage have great impacts on the pavement structural capacity. In the same regard, material properties and behavior under the traffic also can lead to distortions and volume changes that contribute directly to the pavement roughness.

4.2 Pavement performance

Pavement performance is usually defined as the time dependent trend in serviceability and is often expressed as a function of the equivalent single axle load applications that the pavement can carry before failure. The serviceability at any point in time is measured by pavement condition indicators and hence performance can be defined as the change in pavement condition indicators over time or with increasing axle load applications.

Pavement performance can be categorized into functional and structural performance. Functional performance relates to the ability of a particular road network to fulfill satisfactorily its intended function of serving traffic over its design period. It is usually defined in terms of pavement condition indicators such as roughness and skid resistance. Structural performance on the other hand, relates to the deterioration in pavement structural condition with time or cumulated traffic. Cracking and rutting are examples of pavement condition indicators used to quantify pavement structural adequacy. Cracking of bituminous surfacing can be categorized as fatigue cracking, thermal cracking and reflection cracking. Fatigue cracking is associated to the load repetition and the environment and is dependent on asphalt type and content, aggregate gradation, percent air void content, density of the compacted asphalt concrete layer and its thickness. Thermal and shrinkage cracking on the other hand, are function of temperature only and dependent on the same deterioration variables as fatigue cracking. Rutting can be regarded as permanent or unrecovered deformations in pavement layers. The causes of permanent deformation can be classified into traffic associated and non traffic associated causes. Traffic loading causes deformation when the stresses induced in the pavement materials are sufficient to cause shear displacements within the materials. Thus single loads or a few excessive loads or tire pressures, causing stresses that exceed the shear strengths of the materials, can cause plastic flow, resulting in depressions under the load. Repeated loading at lesser load and tire pressure levels cause smaller deformations which accumulate over time and become manifest as a rut in the wheelpaths. The extent of these small deformations are dependent on material properties and construction methods, asphalt type and grade, asphalt content, aggregate gradation, percent air void content in the mix, field density and thickness of the pavement layers.

4.3 Factors affecting pavement performance

The generally accepted factors that affect quality in the construction projects are well defined and thoroughly described in the literature. According to Abdun-Nur (1976), the quality assurance system is made up of both technical and social factors (44). Achieving high quality requires hands-on, top level involvement, is enhanced with the long term support of top management and needs the leadership of higher management to generate innovative activities that can lead to process improvement. A team structure and coordination system are necessary for desired results. The purpose of teamwork is to have all people involved in a process working to achieve a common goal. The technical factors involved are those related to material properties and construction practices (M&C). The most commonly known M&C are: the bituminous mix properties, asphalt content, aggregate type and gradation, compaction, and initial roughness.

4.3.1 Bituminous mix properties

Modulus is a crucial factor in the design and construction of pavement structures. According to Golam et al. (1985), one of the major properties of bituminous materials in pavement performance is the dynamic or resilient modulus of the asphalt concrete mix, which is a function of many variables. These include aggregate type, aggregate size and gradation, aggregate shape, asphalt content, asphalt viscosity, void ratio, temperature, and frequency of loading (45).

4.3.2 Asphalt content

Binder content is believed to be one of the critical factors that regulate all mix properties. It controls directly the stiffness of the asphaltic concrete. As the binder content in the mix fluctuates, the amount of void space filled by the binder in the aggregate gradation is also altered. This modification in void space filled affects aggregate interparticule friction, which in tern affects the stability, durability, strength and fatigue of the mix. According to a research study conducted by Prachuab et al. (1983), for the prediction of pavement fatigue life based on asphalt concrete mix properties, as asphalt content increases, fatigue life increases to the point where the optimum asphalt content corresponds to the maximum fatigue life obtainable. From his studies, he concluded that as the voids become filled, the binder cements aggregate particles together, which causes an increase in the strength of bonding. As the voids become overfilled, the aggregate friction decreases and the binder takes more of the load. In this situation, the stiffness decreases as more and more binder is added (46).

4.3.3 Aggregate type and gradation

In an asphalt concrete mix, the aggregate provides the primary load-carrying mechanism and thus is expected to have some effect on pavement fatigue life. Shape, surface texture, durability, and chemical properties are of interest in investigating aggregate types. According to the literature (46), probably the most important of all is durability since past experience has shown that pavement performance has been drastically reduced by aggregate breakdown. The gradation of an aggregate also determines the amount of void space available to be filled with asphalt binder. The degree to which the voids are filled with binder greatly influences the stiffness and fatigue life of a mix. In the same regard, the amount of void space provided by the aggregate also controls the stiffness and fatigue life. Prachuab et al., (1983) in their works related to aggregate gradation, showed that fatigue is related to the percentage passing the No. 200 sieve at each microstrain level, and that the amount of this material that is present increases fatigue life until an optimum is reached (46).

4.3.4 Compaction

The literature (47) reported that compaction is one of the most important factors that affect the ultimate performance of pavements. It is also believed that compaction contributes to the development of material internal strength and improves its properties. According to Linden et al., (1989) for each one percent increase in air voids above seven percent, there is a ten percent decrease in pavement service life (48). Pavement strength is also believed to be affected by compaction. Finn indicates that 'as the density of the mixture is increased, particularly the degree of packing of the aggregate, the fracture strength is also increased'. Deacon, Epps et Monismith (1965), from independent research studies reported that material stiffness are dependent on air void. They show from their experiments, that stiffness increases as air voids decreases and conclude that a denser mixture results in greater load carrying capabilities of the material (49). Pell and Taylor (1969), investigating the effect of compaction on pavement fatigue life, concluded that voids have a significant effect on fatigue properties of pavement materials. They show that an increase in void content leads to a decrease in fatigue life (50). Referring to the works undertaken by Deacon, Epps et Monismith, Finn et al.,

(1980) concluded that pavement fatigue properties can be reduced by 30 to 40 percent for each one percent increase in air void content. Rutting and shoving resulting from lack of resistance to deformation are also strongly affected by the degree of compaction (51). Rutting develops from the continued compaction in the wheelpaths imposed by the traffic. Hence reduction in air voids during the construction stage to as close as practicable to the air void content that will be attained under traffic will prevent rutting failure.

4.3.5 Initial road roughness

Pavement performance is commonly evaluated using the concept of pavement serviceability, in which pavement failure is defined by terminal serviceability instead of strict structural failure. According to Zaghloul (1996), the present serviceability index (PSI), the measure of pavement serviceability, is a function of pavement roughness, cracking, patching, and rutting. Pavement roughness is the major component of PSI and represents more than 95 percent of its value. Because roughness is such an important consideration, changes in roughness control pavement life-cycles, and therefore, construction quality, which influences initial roughness, influences performance and life-cycle as well (52). In addition, road roughness has a major effect on road users and vehicle operating costs. Yoder et al., (1958) reported that road roughness can be caused by any of the following factors (53):

1. construction techniques that deviate from the target or design profile;
2. plastic deformation in one or more of the pavement layers due to repeated loading and non-conforming materials;
3. non-uniform initial compaction.

Among the numerous research studies conducted on road roughness, Rizenburg, referenced by Khaled Ksaibiti (1995) revealed impacts associated to road roughness such as rider non acceptance, discomfort and road safety. In a recent study conducted by Janoff referenced by Khaled Ksaibiti et al.(1995), it was shown that initial roughness affects the long term pavement performance. He reported that initial pavement roughness measurements are highly correlated with roughness measurements made 8 to 10 years after construction (54).

4.4 Review of pavement performance modelling techniques

The techniques used for the empirical studies of pavement performance can be grouped in two categories : those based on accelerated deterioration which take account of the whole life cycle of the road and those based on in service conditions and consider only a portion of the deterioration cycle. In the first category, two methods can be used to achieve the full deterioration cycle of the pavement. This can be done either by carefully under designing the experimental sections with respect to the anticipated traffic load as in the case of AASHO Road Test, or by applying a supra-normal axle loading of say 40 to 100 kN per wheel. Two major problems are associated with the use of both techniques in the development of predictive performance models. One concern is the virtual elimination of the long-term effects which are mainly environmental and rest periods. The second problem is the likely distortion of the pavement layers material which are stress dependent due to the accelerated unrepresentative loading regime.

The second approach based on full deterioration life cycle monitoring of pavement sections in service has served as bases for the British pavement design methods. Although it offers a valuable knowledge on the changes of pavement condition with time for a specific environment and loading regime, it cannot be easily transferable to other climates and materials unless supported by mechanistic interpretation and extrapolation to other material types and loading conditions. The big disadvantages with this approach and which makes it inappropriate for use in pavement predictive modeling are mainly associated with the problems and errors in measuring small changes in pavement condition with time. It is also a time consuming process, limited by the available pavement designs and standards in the network.

The most interesting and promising approach is the window monitoring technique which considers only a portion of the life-cycle of pavements in service under normal loading regimes. This pavement monitoring technique has been used by north American agencies and states. It is also worth mentioning that it has gained a wide popularity within the Strategic Highway Research Program (SHARP). This technique has been selected for the prestigious five to ten years Long-Term Pavement Performance study (LTPP) which comprises 1560 experimental sections in service for performance modeling purposes and 1630 special experimental sections for individual factors studies. All the sections sampling was in accordance with a partial factorial experiment design.

4.5 Emergence of highway design and maintenance standards model (HDM)

Developing countries usually fund their larger construction projects through domestic savings or financial resources from abroad. The funding for formal or smaller construction market sector comes directly from domestic savings. Larger projects in the formal sector usually are funded through loans and grants from international agencies, foreign aid programs, and investment from private firms based in other countries. The most important sources of multilateral finance for construction projects are the World Bank through the International Bank for Reconstruction and Development (IBRD) and the International Monetary Fund (IMF). These institutions lend funds at interest rates very much lower than those charged by private lenders and sometimes lend for very long periods of time at zero interest rate. Other international lending agencies include the United Nations and its associated affiliates, such as the World Health Organization (WHO) and the International Labor Organization (ILO). These agencies normally fund the construction and operation of health centres and those construction projects employing labor-based methods. A more modest source of funding is provided by regional banks (Inter-American Development Bank, the European Economic community (EEC), and the Organization of Petroleum Exporting Countries (OPEC). Private foreign investment is provided mainly by large multinational companies having an economic interest in the country, especially for those projects involving large volumes of raw materials like copper, bauxite coal and so forth.

In the land transportation field, most of the road network in the developing countries was built during the 1960s and 1970s. Such a large capital investment played a significant role in their economic growth and social life. Unfortunately, as a result of neglect or lack of resources, the capital share allocated to maintenance and rehabilitation (M&R) was either too small or not used in a rational way to keep the existing infrastructure in a serviceable condition. Nowadays these countries, left with a large mileage of deteriorated roads, are faced with even larger investment. Most of those M&R needs are beyond their financial means. Such badly needed expenditure would have been significantly reduced had regular monitoring of the road network and proper maintenance been applied at the right time; i.e. at earlier stages. Faced with this problem, the World Bank has initiated in 1971 a vast program of road data collection, processing and analysis over a period of more than fifteen years located in India, Brazil, Kenya, and the Caribbean (55, 56). This multi-stage research project was aimed to sustain the economic growth of developing countries by providing them a powerful road management tool for decision making at both the network and project levels. With the participation of other institutions and local governments, more than twenty (20) million dollars have been financed to support the project. Rigorous and sound empirical relationships were developed and incorporated into a computer program called HDM (Highway Design and Maintenance Standards) which with the Expenditure Budgeting Model (EBM) is capable to simulate and compare multiple maintenance and rehabilitation alternatives and strategies according to economic criteria with or without budget constrains.

4.5.1 HDM development and evolution

In its original version, HDM program uses the pavement performance models derived from the empirical research studies conducted by the American Association of State Highway and Officials (AASHO). Because these early models were based on accelerated experimental data on high performance road sections located in cold regions, their validity was unjustified for conditions prevailing in most developing countries. Hodges et al., (1975) reported that window monitoring technique was used for the first time in the Kenya predictive deterioration models development (58) which according to Robinson et al. (1980) have formed the bases of the British Road Transport Investment model (RTIM) and subsequently to the release of the second version of Highway Design and Maintenance Standards Model (HDM 2) in 1981 (59). Paterson (1987) reported that the development of the HDM version 3 model (HDM-III) was based on the long-term pavement monitoring program that has been on going in Brazil, India, Kenya and the Caribbean. These studies have been conducted on as-constructed and rehabilitated pavements. Models developed through multiple-regression techniques, are used to predict pavement roughness, raveling and cracking, as function of significant variables that characterize the pavement structure, the environment, and the traffic loading (56).

The HDM-III model, developed through the World Bank has been used for over two decades to combine technical and economic evaluations of road projects, programs and strategies. These evaluations were used for the planning, budgeting, monitoring and management of road systems either within the framework provided by existing road management systems or as independent analytical tools to assist institutional management of road systems throughout the world.

While most HDM-III applications have been in developing countries, in recent years many industrialized countries have started adopting the economic approach and principles built into the model. In 1993 an international research and development project was undertaken to produce modern tools for highway development and management with funding from the Overseas Development Administration (ODA) in the United Kingdom, the Asian Development Bank (ADB), the Swedish National Road Administration (SNRA) and the World Bank. This US $ 2.75 million study has been built upon the widely used Highway Design and Maintenance Standards model (HDM-III), extending its scope and updating it's technical and economic relationships. This international HDM-IV study has been conducted over a 3 year period at four locations; (i) the University of Birmingham in the United Kingdom funded by the ODA and was responsible for the logical design and software development, (ii) the research in Sweden was managed by the SNRA drawing on technical experts from the road and traffic research institute (VTI) and other Swedish experts, (iii) at the road research institute of the Ministry of Public Works (IKRAM) in Malaysia funded by the ADB, and managed by a team of researchers from IKRAM and other international experts, and (iv) at the World Bank in the form of inputs by Bank staff and contracted consultants.

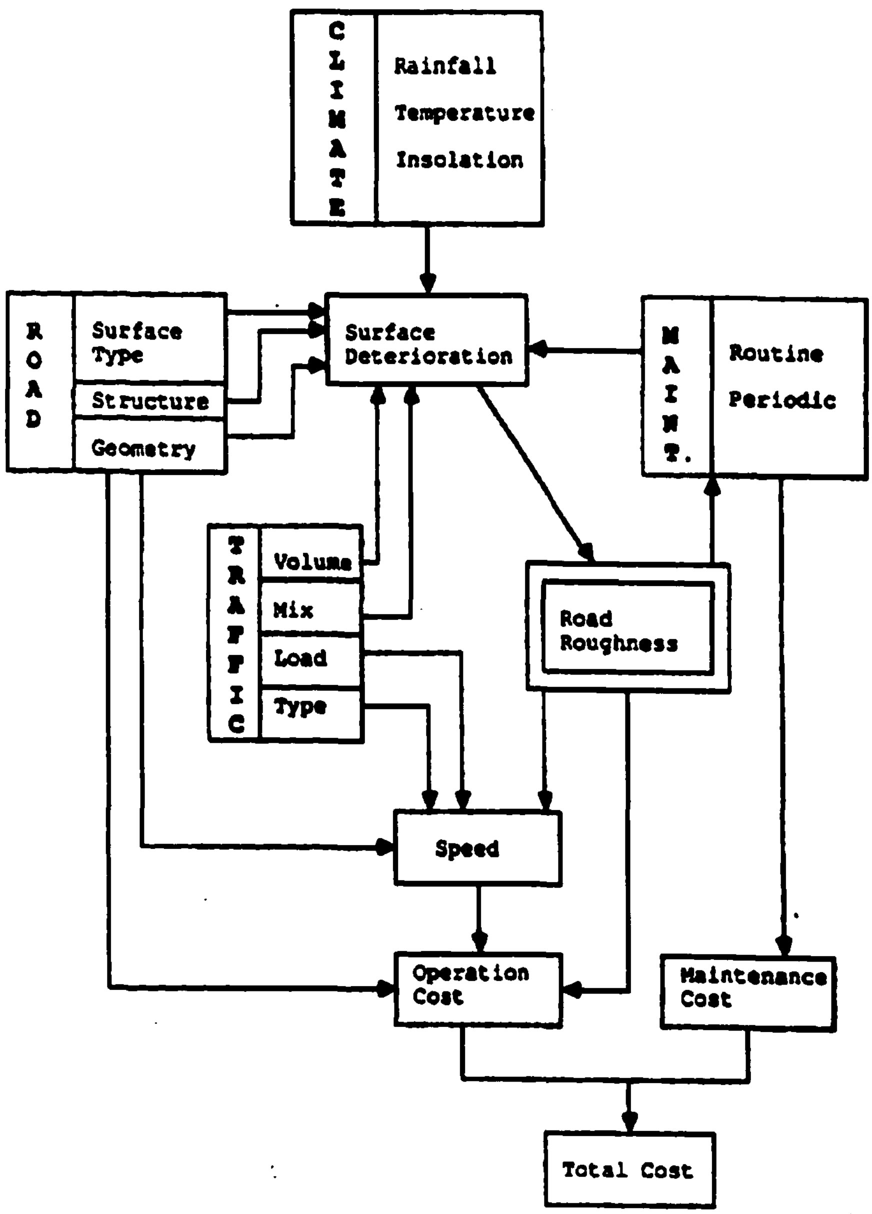
4.5.2 Structure of HDM-III model

The key issues of the research studies conducted from 1971 to 1974 in Kenya followed by a much wider studies from 1976 to 1982 in Brazil, were to collect data on the changes of roughness, cracking and rutting of flexible pavements located in nonfreezing climates, subjected to mixed traffic loading and under different maintenance and rehabilitation strategies.

The fundamental structure of HDM study is based on three major interaction components of cost namely: construction costs (materials and labor), maintenance and rehabilitation (M&R) costs and vehicle operating costs. Figure 4.1 below shows the interaction between the different components (57).

Figure 4.1

Interaction between surface deterioration, maintenance and vehicle operation costs



Construction costs are dependent on local geographical conditions, on the nature of the terrain, on the geometric and the structural design of the road, the rainfall and the unit cost of materials and labor. Maintenance and rehabilitation costs are directly related to road deterioration which is dependent on pavement strength, environment, pavement age and also on the selection of maintenance and rehabilitation strategies and unit material costs. User costs are dependent on the geometric design, road surface condition, vehicle type, and speed. Through the benefit resulting from reduced vehicle operating costs, travel time, and other external benefits, the model provides an economic evaluation for each construction and/or maintenance alternative, expressed in the following economic criteria : net present value (NPV), internal rate of return (IRR), first year benefit (FYB), and equivalent uniform annual cost (EAC) for the whole analysis period. The HDM-III model includes a series of sub-models that are namely : traffic, maintenance level, vehicle operating cost, exogenous factors, and economic evaluation.

4.5.3 HDM-III pavement performance prediction models

According to Paterson (1987), the Kenya studies comprises 49 experimental site sections over a four year monitoring period. The models formulation was based on experimental design techniques and comprises the most influential parameters on the response variables. A partial factorial design of 2 rain fall x 3 pavement types x 3 gradients x 3 curvatures was considered. Using the same approach, the Brazilian research program comprises a wider study which formed the bases of HDM-III release. One hundred and sixteen experimental sections have been studied over a period of two to five years in a partial factorial design of one climate x 2 rehabilitation states x 2 traffic levels x 2 gradients x 2 ages (56).

The deterioration models incorporated in HDM-III allow the prediction of pavement condition over time in terms of roughness, cracking, rutting, raveling and potholes formation. Each condition variable is associated with several models which provide the ability to predict initiation and progression of cracking and raveling and progression of all condition variables for the pavement both before and after a rehabilitation treatment.

4.5.3.1 Cracking models

Cracking is a critical surface indication of distress within the pavement structure. It represents a loss of pavement integrity that can lead to accelerated deterioration of pavement strength and condition. Models that predict cracking behavior therefore also make an essential contribution to the prediction of life-cycle performance and the cost of alternative rehabilitation strategies.

In HDM-III, pavement surface cracking is modeled by two different equations: one for predicting the time until cracking initiation and the other for its progression. The seven probabilistic relationships estimated for the expected age and traffic at initiation of narrow and all cracking are given by Paterson (1987). Narrow cracking initiation is given by different equations for different pavement types and is function of the pavement structural number, traffic characteristics, binder content, construction quality, subgrade strength, deflection and surfacing thickness. For Asphalt concrete on granular base pavements, the model incorporated in HDM-III is given below (56).

$$TY_{cr2} = 4.21 * \exp(0.139\ SNC - 17.1\ YE_4 / SNC^2) \tag{4.1}$$

where

TY_{cr2} = expected (mean) age of surfacing at initiation of narrow cracking, in years;
SNC = modified structural number;
YE_4 = annual traffic loading, million ESALs/lane/year.

The other probabilistic predictive relationships developed for the expected age and traffic at initiation of narrow and all cracking for original asphalt concrete surfacing on granular base pavements are given by Paterson (56). The third model contains the binder content variable (BNO) and seem to be more interesting for use in pavement performance modeling because it includes a measure of durability. The fourth model comprising a quadratic form of surfacing thickness produces a minimum pavement life at a surfacing thickness of 60.5 mm. The model selected for use in HDM-III for narrow cracking for surface treatment on granular base pavements is :

$$TYCRA = K_{ci}\,(F_c * RELIA + CRT) \tag{4.2}$$

where

TYCRA = the predicted number of years to the initiation of narrow cracks since last surfacing or resurfacing;

CRT = the cracking retardation time due to maintenance, in years;

K_{ci} = calibration factor;

F_c = the occurrence distribution factor for cracking initiation for the subsection (i.e., weak = 0.55, medium = 0.98, strong = 1.48);

RELIA = $13.2 * \exp[-20.7\,(1+CQ)\,YE_4 / SNC^2]$;

CQ = construction quality indicator for surfacing and is equal to 1 if the surfacing has construction faults and 0 otherwise.

The initiation of wide cracking for asphalt concrete surfacing and for surface treatment on granular base pavements is given respectively by the following equations :

$$TYCRW = K_{ci}\,(2.46 + 0.93\ TYCRA) \tag{4.3}$$

$$TYCRW = K_{ci} * \max(2.66 + 0.88 * TYCRA,\ 1.16 * TYCRA) \tag{4.4}$$

where K_{ci} and TYCRA are as defined before.

Narrow and wide cracking progression for bituminous, surface treatment surfacing on granular base pavements and asphalt concrete overlays are given respectively by the following equations :

$$\Delta ACRA_d = K_{cp}{*}CRP{*}Z_a\{[Z_a\, a_i\, b_i\, \Delta TCRA + SCRA_a^{bi}]^{1/bi} - SCRA_a\} \quad (4.5)$$

$$\Delta ACRW_d = K_{cp}{*}CRP{*}Z_w\{[Z_w\, c_i\, d_i\, \Delta TCRW + SCRW_a^{di}]^{1/di} - SCRW_a\} \quad (4.6)$$

where

$\Delta ACRA_d$ = predicted change in the area of all cracking during the analysis year due to road deterioration expressed in percent of the total area;

K_{cp} = calibration factor for cracking progression;

CRP = 1 - 0.12 *CRT : retardation of cracking progression due to preventive treatment;

Z_a = 1 if $ACRA_a < 50$; $Z_a = -1$ otherwise;

$\Delta TCRA$ = fraction of the analysis year in which narrow cracking progression applies, expressed in years, and given by :

$\Delta TCRA$ = 0 if AGE2 < TYCRA and $ACRA_a = 0$

$\Delta TCRA$ = (AGE2 - TYCRA) if AGE2 - 1 < TYCRA ≤ AGE2 and $ACRA_a = 0$

$\Delta TCRA$ = 1 if TYCRA ≤ AGE2 - 1 or $ACRA_a > 0$

$SCRA_a$ = min ($ACRA_a$, 100 - $ACRA_a$)

$SCRA_a$ = max ($SCRA_a$, 0.5) if $ACRA_a > 0.5$

Z_w = 1 if $ACRW_a < 50$; $Z_w = -1$ otherwise;

$\Delta TCRW$ = fraction of the analysis year in which wide cracking progression applies, expressed in years, and given by :

$\Delta TCRW$ = 0 if AGE2 < TYCRW and $ACRW_a = 0$

$\Delta TCRW$ = (AGE2 - TYCRW) if AGE2-1 < TYCRW ≤ AGE2 and $ACRW_a = 0$

$\Delta TCRW = 1$ if $TYCRW \leq AGE2 - 1$ or $ACRW_a > 0$

$SCRW_a = \min(ACRW_a, 100 - ACRW_a)$

$SCRW_a = \max(SCRW_a, 0.5)$ if $ACRW_a > 0.5$

and a, b, c, and d are coefficient given in Table 4.1 below :

Table 4.1

Cracking progression prediction coefficients

Surfacing Type	a	b	c	d
Asphalt concrete	1.84	0.45	2.94	0.56
Surface treatment	1.76	0.32	2.50	0.25
Asphalt overlays	1.07	0.28	2.58	0.45

The empirical relationships developed and incorporated in HDM-III for the initiation of both types of cracking show the influence of traffic loading and pavement strength. On the other hand, the rates of cracking progression represented by the above equations are S-shaped functions. Both regressions are only dependent on the area of cracking and the time since cracking initiation. According to Paterson (56), the empirical evidence indicated that the rate of progression was a process related to the variability of the materials. Traffic loading and pavement strength have no significant effects on narrow and wide cracking progression.

4.5.3.2 Rutting models

Rutting is the result of accumulation of the small permanent deformations associated with repetitive traffic loading in presence of expansive subgrade soils and compressive material underlying the pavement structure. Rutting, like other permanent deformation mechanisms is generally controlled in pavement design. That is, most design methods require higher strength materials and indicate combination of layer thickness and stiffness sufficient to attenuate the applied stresses to levels at which the accumulated shear deformations through the full pavement depth are within a specified rut depth criterion. However, for performance modeling, these limiting criterion approaches are not useful because of the need to predict not a limit but the trend of accumulated deformation during the life of the pavement, identifying the response to the actions of traffic, environment and maintenance. Hence for performance modeling purposes, the need of combining material characteristics such as shear strengths on the one hand, and the induced stresses which are dependent on loads, layer thicknesses, and material stiffness on the other hand is relevant. Since rutting is considered as a primary criterion of structural performance for many pavement design methods, only rut depth progression and rut depth standard deviation were considered in the distress modeling efforts. The key variables involved in the rutting equation which is common to all pavement types are the annual traffic loading expressed in equivalent single axle load (ESAL), pavement strength evaluated in term of modified structural number (MSN) or Benkelman deflection, base layer strength, current level of cracking, annual precipitation, and a calibration factor K_{rp}. Mean rut depth equation is not used as a maintenance intervention criterion in HDM-III, but rather as a mean to estimate the variation of rut depth which contributes directly to roughness. The regression equations developed from the Brazilian studies are as follows:

$$RDM = 1.0 * AGER^{0.166} * SNC^{-0.502} * COMP^{-2.30} * NE_4^{ERM} \qquad (4.7)$$

where

RDM = mean rut depth of both wheelpaths, in mm;

AGER = age of the pavement since latest overlay or construction, in years;

SNC = modified structural number of the pavement;

COMP = $\Sigma RC_i \, (H_i/\Sigma H_i)$, i = 2, n : compaction index of pavement;

RC_i = relative compaction, i.e., the ratio of the compaction measured in the field to the nominal compaction, as a fraction ;

H_i = thickness of layer, in mm;

NE_4 = cumulative number of ESALs;

ERM = 0.0902 + 0.0384*DEF - 0.009*RH + 0.00158*MMP*CRX

DEF = mean peak Benkelman beam deflection under 80 kN standard axle load of both wheelpaths, in mm;

RH = rehabilitation state of the pavement and is equal to null for original pavements and to one for overlaid pavements;

MMP = mean monthly precipitation, in m/month;

CRX = area of indexed cracking, in percent.

The rut depth standard deviation is given by a separate equation. Since it plays a significant role in the roughness prediction equations, the adjustments for calibration should be more concerned with the rate of change of the standard deviation than its actual value. The regression equation developed from the Brazilian data is as follow:

$$RDS = 2.063 * RDM^{0.532} * SNC^{-0.422} * COMP^{-1.664} * NE_4^{ERS} \qquad (4.8)$$

where

RDS = rut depth standard deviation of both wheelpaths, in mm;

RDM = mean rut depth of both wheelpaths, in mm;

COMP = compaction index of pavement relative to a standard fraction;

NE_4 = cumulative number of ESALs;

ERS = -0.009 *RH + 0.00116 *MMP *CRX;

MMP, CRX are as defined before.

4.5.3.3 Raveling models

Raveling is defined as the loss of surfacing material from pavements, which in thin surfacings may eventually develop into potholes. For raveling prediction, HDM-III program considers different equations for three different pavement surface types. These prediction equations have been established separately for surface treatment, slurry seal, and cold mix asphalt concrete surfacing and overlays. Raveling of hot mix asphalt concrete was not considered. For surface treatment on granular base, HDM-III predictive equations for raveling initiation and progression are respectively as follows :

$$TYRAV = K_{vi} \{F_r [10.5\exp(-0.655CQ - 0.156YAX)] RRF \} \qquad (4.9)$$

$$\Delta ARAV_d = (K_{vi} * RRF)^{-1} Z_r \{[Z_r 1.560 \Delta TRAV + SRAV_a^{0.352}]^{2.84} - SRAV_a\} \qquad (4.10)$$

where

TYRAV	=	predicted number of years to raveling initiation;		
K_{vi}	=	calibration factor for raveling initiation;		
F_r	=	occurrence distribution factor for raveling initiation for the subsection (0.54, 0.97 and 1.49 for weak, medium and strong subsections used in HDM-III);		
YAX	=	total number of axles of all vehicle classes for the analysis year expressed in millions/lane;		
RRF	=	raveling retardation factor due to maintenance;		
$\Delta ARAV_d$	=	predicted change in the raveling area, expressed in percent;		
Z_r	=	1 if $ARAV_a < 50$; $Z_r = -1$ otherwise;		
$\Delta TRAV$	=	fraction of the analysis year during which raveling progression applies, in years given by :		
$\Delta TRAV$	=	0	if AGE2 < TYRAV	and $ARAV_a = 0$
$\Delta TRAV$	=	(AGE2 - TYRAV	if AGE2 - 1 < TYRAV ≤ AGE2	and $ARAV_a = 0$

$\Delta TRAV = 1 \quad$ if $TYRAV \leq AGE2 - 1 \quad$ or $ARAV_a > 0$

$SRAV_a = \min(ARAV_a, 100 - ARAV_a)$

$ARAV_a = 0.5 \quad$ if $0 < \Delta TRAV < 1$ and $ARAV_a \leq 0.5$

$ARAV_a = ARAV_a$ otherwise

These empirical relationships have probabilistic and incremental forms similar to the cracking models. In raveling initiation, pavement structural properties are not significant, and the only explanatory variables are the surfacing type and construction quality. In raveling progression, no variables except time and area of raveling were significant in the empirical study.

4.5.3.4 Potholing models

Potholing usually develops from the spalling of wide cracking or the raveling of thin surface treatment. It results from the disintegration and loss of surfacing material and, subsequently, base material. In surface treatment, potholes may develop either from raveling which has exposed the base, or from wide cracking which has spalled or reached such intensity that fragments are easily removed. In asphalt surfacing, potholes develop from when wide cracking becomes intense or shows severe spalling. Potholing is therefore the ultimate form of distress consequential upon deferred maintenance of severe cracking or raveling. Paterson (1987) reported that potholing is likely to be most severe in wet climates and low standard base materials. In undistressed pavements, however, potholing also occurs occasionally and can usually be attributed to a random defect in surfacing and / or base materials. In both cases, the initiation and progression of potholes are highly dependent on the mechanical disintegration properties of the base materials and construction practices (56). In HDM-III, the rate of progression as modeled increases linearly with the total area of distress, and with the traffic volume. Base course and surfacing quality are important factors but difficult to quantify, thus the construction quality factor (CQ) and modified structural number (MSN) are incorporated as simple surrogates of quality. The algorithm incorporated in HDM-III for granular base is of the form :

$$TMIN = \max(2 + 0.04 * HS - 0.5 * YAX; 2) \quad (4.11)$$

where

TMIN = predicted time between the initiation of either wide cracking or raveling, whichever occurs earliest, and the probable initiation of potholing, in years;

HS = thickness of the bituminous surfacing, in mm;

YAX = annual number of vehicle axles, in million axles/lane/year.

Modeling of the progression of potholing area is formed of a group of algorithms derived from one of three sources, namely wide cracking, raveling, and the enlargement of existing potholes. The studies from which these regressions were obtained reported potholing progression rates that ranged from 0.1 to 9 percent area per year. Data available were related to mechanistic parameters such as traffic flow, surfacing thickness, and base quality as follows :

$$\Delta APOT = \min(\Delta APOT_{cr} + \Delta APOT_{rv} + \Delta APOT_{pe}; 10) \quad (4.12)$$

where

$\Delta APOT$ = predicted change in the total area of potholes during the analysis year due to road deterioration, limited to 10 percent;

$\Delta APOT_{cr}$ = min (2 *CR_4 *U; 6), predicted change in the area of potholes during the analysis year due to cracking;

$\Delta APOT_{rv}$ = min (0.4 *ARAV *U; 6), predicted change in the area of potholes during the analysis year due to raveling;

$\Delta APOT_{pe}$	=	min { $\Delta APOT$ [K_{base} *YAX *(MMP + 0.1)]; 10}, predicted change in the equivalent area of potholes during the analysis year due to pothole enlargement;
U	=	[(1 + CQ) *(YAX/SNC)]/2.7 *HS;
CR_4	=	area of wide cracking, expressed in percent of pavement area;
ARAV	=	area of raveling, expressed in percent of pavement area;
K_{base}	=	max (2 - 0.02 *HS; 0.3) for granular base;
HS	=	thickness of bituminous surfacing;
YAX	=	annual number of vehicle axles, expressed in million axles/lane/year;
MMP	=	mean monthly precipitation, expressed in meters.

4.5.3.5 Roughness models

Road roughness is highly correlated with serviceability, and is the principal measure of pavement condition that directly relates to vehicle operating cost. Roughness is therefore undoubtedly the most critical of all the various pavement performance predictions. The incremental roughness model developed and incorporated in HDM-III is an additive combination of three major components resulting from structural deformation, superficial defects and environmental factors. Structural deformations are due to plastic flow in the pavement materials under the shear stresses imposed by traffic loading. These deformations, commonly appear as rutting in the wheelpaths and usually considered to appear primarily in the subgrade but can extend to other layers such as the base and the subbase when the material shear strength is inadequate. It can be also the result of saturation or increased axle loading and tire pressure. Surface distress such as potholes, patching, raveling, cracking and localized depressions are generally associated to shallow seated distress originating in either the surfacing or base of the pavement. Environmental and climatic factors exclude structural effects but include temperature and moisture fluctuation and foundation movements causing volume changes in the pavement structure. The incremental form of roughness prediction model formulated from the Brazilian studies and incorporated in HDM-III is given below.

$$\Delta QI_d = 13 * K_{gp} [134 * EMT * (SNCK + 1)^{-5} * YE_4 + 0.114 * (RDS_b - RDS_a) + 0.0066 * \Delta CRX_d + 0.42 * \Delta APOT_d] + K_{ge} * 0.023 * QI_a \quad (4.13)$$

where

ΔQI_d = increase in roughness over time period t, expressed in QI;

K_{gp} = calibration factor for roughness progression;

K_{ge} = calibration factor for the environment related annual fractional increase in roughness;

EMT = exp (0.023 K_{ge} AGE3)

SNCK = max (1.5; SNC - ΔSNK), modified structural number adjusted for the effect of cracking;

ΔSNK = 0.0000758 *(CRX'$_a$ *HSNEW + ECR *HSOLD), predicted reduction in the structural number due to cracking since the last pavement reseal, overlay or reconstruction;

CRX'$_a$ = min (63; CRX_a)

ECR = max [min(CRX'$_a$ - PCRX; 40); 0], predicted excess cracking beyond the amount that existed in the old surfacing layers at the time of the last pavement reseal, overlay or reconstruction;

PCRX = 0.62 *PCRA + 0.39 * PCRW, area of previous indexed cracking in the old surfacing and base layers;

4.5.4 Vehicle operating cost prediction models

Watanatada et al., referenced by Bhandari (1987) reported that, before the initiation of this research program supported by the World Bank, empirical data relating vehicle operating cost to the pavement condition did not exist. Relative decisions concerning road investments relied merely on vague and fragmentary estimations of user costs which were mainly dependent on pavement type (paved, gravel or earth). Based on improved estimations of vehicle free speed and on vehicle operation costs as related to road characteristics, the HDM research program suggests that vehicle operating costs are somewhat less sensitive to changes in road condition than previously estimated (60).

Four different sets of relationships for estimating vehicle operating cost based on four separate empirical studies by different agencies are available for application in the HDM-III model. With the participation of the World Bank, these empirical relationships are derived from:

1. the Kenya and Caribbean studies conducted by the British Transport and Research Laboratory, (TRRL),
2. the Indian relationships developed by the Central Road Research Institute of New Delhi, (CRRI),
3. studies conducted in Brazil by GEIPOT, and the Texas Research and Development Foundation.

4.5.4.1 The Kenyan studies

The Kenyan road user cost studies conducted by the British Transport and Road Research Laboratory (TRRL) was aimed to assist investment decisions related to non-urban road networks in developing countries. These studies initiated from 1971 to 1975 embodied three major components namely: observation of vehicle speeds under normal usage conditions in a sample of the road network, establishment of fuel consumption speed relationships as a function of road characteristics through controlled experiments, and a user cost survey record collection. The predictive relationships developed were simple linear statistical relationships comprising most operating cost components. These cost components are fuel consumption, tires wear, vehicle maintenance costs, driver's time delay and vehicle depreciation. Road characteristics considered in the survey were surface type, roughness, and vertical and horizontal alignments.

4.5.4.2 The Caribbean studies

These studies conducted in the Eastern Caribbean region was aimed to check the validity and the applicability of the Kenyan relationships to the Caribbean local conditions. As a part of the former research, the purpose of the Caribbean studies was to supplement the Kenyan studies with other significant factors associated with road geometry and rough bituminous surfacing. The studies were conducted over a two years period from 1977 to 1982 during which data on various roads and relevant parameters to vehicle operating cost were collected and analyzed. This complementary experiment studies designed and developed by the TRRL have been conducted on six road sections located in four different islands. The topography of the terrain ranges from flat rolling to rolling mountainous and the road roughness from low to high so that a full range of vertical and horizontal curvature, surface condition and road width were included in the design matrix.

4.5.4.3 The Indian studies

The Indian road user cost study was jointly financed by the government of India and the World Bank. It started in 1978 and was completed in 1982. The aim of this project is to build sound relationships between vehicle operating cost components and road, traffic, vehicle, and environmental factors. The research was carried out by collecting real data on the cost of operation of vehicles of different types that operate in the different parts of the country. Vehicle operating cost components studied are fuel, lubricants, tires, spare parts, labor for maintenance and repairs, depreciation, and fixed costs. For determining fuel consumption relationships, two different approaches were used and then compared. The first approach which is the most accurate, was performed through controlled experiments on the common makes of vehicle specially procured for the project. The second approach was based on data collection from a number of operators of vehicles of different types. For the other vehicle operating cost such as tire wear, spare parts consumption, maintenance labor, lubricants, depreciation and fixed costs, a more expedient procedure known as the user cost survey (UCS) was used. This procedure consists of undertaking a detailed survey of the cost of operation in real-life from a large number of vehicle operators.

4.5.4.4 The Brazilian studies

The model forms derived from the Kenyan and the Caribbean studies were found to be not suitable for applications in other regions. The weakness of this early simple linear models lays in the determination of vehicle speeds due to the complex physical and behavioral mechanisms involved and the measurement methods of road roughness. The Brazilian studies were designed to overcome these problems by providing more suitable model forms which can be easily transferable to other regions. These field studies were conducted between 1975 and 1984. More advanced theoretical and statistical techniques and a wider range of road characteristics and vehicle types were used. The improved non-linear models developed through data collection and analysis for vehicle speeds and fuel consumption were based on mechanistic and behavioral concept. By explicitly incorporating the physical mechanisms and governing behavioral constraints omitted in both the Kenyan and the Caribbean studies through a probabilistic limiting velocity approach over the wide range of road characteristics, they are more reliable for use after calibration in any other locations. The vehicles observed in this study were classified in ten groups, namely : small, medium and large passenger cars, utility pickup, buses and light, medium, heavy, and articulated trucks. The relationships developed in this study are as follows:

4.5.4.4.1 Vehicle speed

According to Watanatada et al., (1987) referred by Paterson, the prediction of vehicle speeds in the Brazilian studies is based on an aggregate probabilistic limiting velocity approach to steady-state speed prediction. The approach is termed probabilistic limiting velocity approach because the predicted speed is a probabilistic minimum of five constraining speeds namely:

1. the limiting speed based on vertical gradient and engine power,
2. the limiting speed based on vertical gradient and braking capacity,
3. the limiting speed determined by road curvature,
4. the limiting speed based on road roughness and associated ride severity,
5. the desired speed in the absence of other constraints based on psychological, economic, safety, and other considerations.

The steady-state speed for each road segment is predicted using the respective values of the above limiting speeds. The theoretical concept behind these computations involves treating each of these constraining speeds as a random variable and the resulting steady-state speed prediction as the average value of the minimum of these random variables. The probability model used is the Weibull distribution. The predicted formula for the average speed for a round trip derived from the uphill and downhill speeds is given below.

$$S = [(3.6) * (2) * LGTH] / [LGTH/V_u + LGTH/V_d] = 7.2 / (1/V_u + 1/V_d) \quad (4.14)$$

where

LGTH = the length of the roadway in km;
V_u = the predicted vehicle speed for the uphill segment in m/s;

V_d = the predicted vehicle speed for the downhill segment in m/s;

3.6 = conversion factor from m/s to km/h.

4.5.4.4.2 Fuel consumption

In the Brazilian experimental study, as in predicting vehicle speed, the idealized homogeneous uphill and downhill road segments for predicting fuel consumption were used and a quadratic form with separate coefficients for positive and negative power regimes was adopted. The engine power output was computed using the respective predicted speeds and the characteristics of the vehicles and the homogeneous segments. The prediction of fuel consumption was based on a constant nominal engine speed instead of actual used engine speed. The values of nominal engine speed were calibrated using the validation data from the Brazilian study and used as default values in the fuel model that may be changed by the user. To take account of the changes in vehicle technology in terms of fuel-efficiency for newer makes and models, an energy-efficiency factor denoted by a_1 was introduced in the prediction formula. A fuel adjustment factor denoted by a_2 was also introduced to account for the differences between experimental conditions and real life driving conditions. The basic round trip fuel consumption (FL) prediction equation developed in the Brazilian studies for a vehicle operating on any road section of specified geometric alignment is given by:

$$FL = 500 * a_1 * a_2 * [(UFC_u / V_u) + UFC_d / V_d)] \quad (4.15)$$

where

UFC_u = the predicted unit fuel consumption for the uphill segment, in ml / s;

UFC_u = $(UFC_o + a_3 . HP_u + a_4 . HP_u . CRPM + a_5 . HP_u^2) . 10^{-5}$

UFC_d = the predicted unit fuel consumption for the downhill segment, in ml / s;

UFC_d = $(UFC_o + a_3 . HP_d + a_4 . HP_d . CRPM + a_5 . HP_d^2) . 10^{-5}$ if $HP_d \geq 0$

$UFC_d = (UFC_o + a_6 .HP_d + a_7 .HP_d^2) .10^{-5}$ if $NH_o \leq HP_d < 0$

$UFC_d = (UFC_o + a_6 .NH_o + a_7 .NH_o^2) .10^{-5}$ if $HP_d < NH_o$

$UFC_o = a_o + a_1 .CRPM + a_2 .CRPM_2$

CRPM = the calibrated engine speed, in revolution per minute;

HP_d = the vehicle power on the downhill road segment, in metric hp, given by:

$HP_d = [(1\,000 .CR - RF) .GVW .g .V_d + 0.5 .RH_0 .CD .AR .V_d^3] / 736$

HP_u = the vehicle power on the uphill road segment, in metric hp, given by:

$HP_u = [(1\,000 .CR + RF) .GVW .g .V_u + 0.5 .RH_0 .CD .AR .V_u^3] / 736$

a_o through a_7 and NH_o are the parameters of the mechanistic fuel prediction models estimated using the data from controlled experiments. Their values are given in Watanatada et al., (1987).

4.5.4.4.3 Tire wear

Two relationships for predicting tire wear are used in the HDM-III model: one for cars and utilities, and the other for buses and trucks. Among the variables appearing in the prediction relationships, NT, NR_o and VOL are parameters specific to vehicle types. The other parameters, C_{otc} and C_{tcte} are specific not to the vehicle class, but mainly to the material properties of the tire. The relationships for computing the number of equivalent new tires consumed per a thousand vehicle-kilometers (TC), for the various vehicle types are as follows:

1. For all categories of passenger cars (small, medium and large) and utilities

$$TC = NT *(0.0144 + 0.000137\, QI) \text{ for } 0 < QI \leq 200 \quad (4.16)$$
$$= 0.0388 \quad \text{for } QI > 200$$

2. For light (gasoline and diesel), medium, heavy and articulated trucks and large buses

$$TC = NT\ [[(1+0.01*RREC*NR)[C_{otc} + C_{tctc}*CF2/L]/(1+NR)*VOL\ ]+0.0075] \quad (4.17)$$

where

NT	=	the number of tires per vehicle;
RREC	=	the ratio of the cost one retreading to the cost of one new tire, in percent;
NR	=	the number of retreadings per tire carcass predicted as:
NR	=	NR_o *exp (-0.00248 QI - 0.00118) - 1;
NR_o	=	the base number of recaps;
C_{otc}	=	the constant term of the tread wear model;
C_{tctc}	=	the wear coefficient;
CF_2	=	the average squared circumferential force per tire, given as:
CF_2	=	$(CF_u^2 + CF_d^2)/2$;
Cf_u	=	the average circumferential force per tire (in the direction tangential to the road surface) on the uphill road segment, in newtons, computed as:
Cf_u	=	$[(1\ 000\ .CR + RF)\ .GVW\ .g + 0.5\ .\ RH_0\ .CD\ .AR\ .V_u^2]$;
CF_d	=	the average circumferential force per tire (in the direction tangential to the road surface) on the downhill road segment, in newtons, computed as:
CF_d	=	$[(1\ 000\ .CR - RF)\ .GVW\ .g + 0.5\ .\ RH_0\ .CD\ .AR\ .V_d^2]$;
L	=	the average force per tire in the direction perpendicular to the road surface, in newtons given as:
L	=	$(1\ 000\ .GVW\ .g)\ /\ NT$;
VOL	=	the average wearable rubber volume per tire for a given vehicle axle-wheel configuration and nominal tire size, in dm^3;
0.0075	=	the correction term for the prediction bias due to model non-linearity.

4.5.4.4.4 Maintenance parts

In the Brazilian study, parts consumption were modeled in terms of the ratio of the monetary cost of parts consumed per thousand vehicle-kilometer to the price of a new vehicle in the same period. This modeling procedure was based on the assumption that the prices of spare parts and of a new vehicle vary together by the same proportions. Chesher and Harrison, (1987) referred by Paterson, reported that for a given vehicle class, the parts cost per thousand vehicle-kilometer (PC) was found to be related to roughness and vehicle age. At constant age, the relationship between PC and roughness was exponential especially at low roughness values but at higher roughness values, the exponential relationship tends to overestimate the PC. The prediction equation adopted in HDM-III was a composite of exponential up to a transitional value of roughness and linear at higher values. Mathematically, these relationships are expressed as follows.

$$\begin{aligned} PC &= CKM^{k} * C_{osp} .\exp(C_{spqi} * QI) \quad \text{for } QI < Qiosp \\ &= KM^{k} *(a_o + a_1 * QI) \quad \text{for } QI > Qiosp \end{aligned} \tag{4.18}$$

where

CKM = the average age of the vehicle group in km, defined as the average number of kilometers the vehicles have been driven since they were built, expressed as:

CKM = min (1/2 $LIFE_o$, CKM^1)

$LIFE_o$ = the average vehicle service life in years;

AKM_o = the average number of kilometers driven per year per vehicle type;

CKM^1 = the ceiling on average cumulative kilometerage;

k = the age exponent, a fixed model parameter;

C_{osp} = the constant coefficient in the exponential relationship between spare parts consumption and roughness;

C_{spqi} = the roughness coefficient in the exponential relationship between spare parts consumption and roughness;

Q_{iosp} = the transitional value of roughness, in QI, beyond which the relationship between spare parts consumption and roughness is linear;

a_o = $C_{osp} .\exp(C_{spqi} .QI_{osp}) .(1 - C_{spqi} .QI_{osp})$;

a_l = $C_{osp} .C_{spqi} . \exp .(C_{spqi} .QI_{osp})$;

4.5.4.4.5 Maintenance labor

Maintenance labor hours were found to be related primarily to maintenance parts requirements, and in some cases, to roughness. When significant, the latter was found to be exponential and the two effects are multiplicative. The relationship used in HDM-III is given below.

$$LH = C_{olh} * PCC_{lhpc} * \exp(C_{lhqi} * QI) \qquad (4.19)$$

where

LH = the predicted number of maintenance labor-hours per 1 000 vehicle-km;

PC = the standardized parts cost per 1 000 vehicle-km, expressed as a fraction of new vehicle price;

C_{olh} = the constant coefficient in the relationship between labor hours and parts costs;

C_{lhpc} = the exponent of parts cost in the relationship between labor hours and parts costs;

C_{lhqi} = the roughness coefficient in the exponential relationship between labor hours and roughness.

The other common predictive relationships used with all options derived from the Brazilian studies and other countries such as lubricants consumption, crew requirements, vehicle depreciation and interest and so forth are given by Paterson.

4.5.5 Calibration of HDM-III prediction models

As stated earlier, HDM-III predictive models were based on empirical relationships developed from studies on experimental sections under normal service conditions conducted in Brazil, Kenya, Caribbean, and India. Calibration of the parameters is necessary to predict the most representative values in environments other that those in which the models were developed. HDM-III models calibration require historical data on pavement performance under known maintenance and rehabilitation strategies. The validation of the calibration results would necessitate a representative sample of a range of pavement types and ages, a range of traffic volumes, and a range of pavement strengths within each traffic class. The vehicle operating cost models necessitates a knowledge on the factors attributed to vehicles. Factors involved in HDM calibration are the deterioration predictive models and the vehicle operating cost models.

4.5.5.1 HDM deterioration prediction models

Initiation and progression of pavement deterioration have an important impact on the prediction of the pavement life cycle cost. To ensure that HDM-III simulations reflect reality, the models have to be calibrated to local conditions. This can be done by comparing predicted HDM-III output results to the locally observed pavement performance. The literature reported various calculation procedures for HDM-III models calibration. Such procedures range from simple theoretical calculations to more rigorous methods (55,56,61,62). Simple methods are generally based on the fact that HDM-III prediction models are robust enough and do not need to be changed. A calibration factor is simply defined as the ratio of the predicted distress to the observed distress. One performance models calibration methodology proposed by Watanatada et al., (1987) is to take into account in the HDM-III input data of a set of calibration factors. These factors are selected so that they account for local conditions such that material properties, environment, construction practices and quality, and so on (55). Seven factors are involved in the calibration of HDM-III deterioration models, namely:

1. Cracking initiation and progression factors (K_{ci}, K_{cp}),
2. Potholes progression factor (K_{pp}),
3. Rutting progression factor (K_{rp}),
4. Raveling progression factor (K_{vi}) ,
5. Roughness progression and age-environment factors (K_{gp}, K_{ge}).

Watanatada (1987) suggests that cracking, raveling and potholing are likely to require local adaptation, while rutting and roughness are relatively less sensitive to local changes (55). However, Hass et al., (1996), based on sensitivity studies recommend a different priority for the calibration of these parameters. They suggest that the adjustment of calibration factors for raveling, cracking and rutting has less effect on the net present value (NVP). Based on the above finding, the authors suggested that calibration of these three factors should not be a priority (61).

4.5.5.1.1 Cracking prediction models

Pavement surface cracking is modeled by two different equations: one for predicting the distress initiation and the other for its progression. Cracking initiation is given by different equations for different pavement types and is function of structural number, traffic characteristics, binder content, construction quality, subgrade strength, deflection and the surfacing thickness. The regression equations developed also include variables for cracking retardation due to maintenance, and an adjustment factor (K_{ci}) for local calibration.

According to Hass et al., (1996), because of the lack of existing reliable database to take rigorous adjustments, which is the case in most developing countries, an estimate of the time it takes a local pavement of a given standard to start cracking can be obtained from field personnel. He also suggested a more general procedure for determining the value of a calibration factor. This procedure, which involves an iterative process, is based on the comparison of HDM-III predicted distress to local observations.

Three key variables form the bases of HDM-III cracking progression equation. These variables are: current pavement condition, pavement age, and a calibration factor, K_{cp}. Based on the relative sensitivity of the NPV to HDM-III input parameters, Hass et al., reported that the calibration factor for cracking progression, K_{cp} is at least 10 times more significant than the cracking initiation factor, K_{ci}, (61)

4.5.5.1.2 Roughness prediction models

Pavement roughness is one of the most sensitive variables of the default HDM-III deterioration equations. Road roughness is used as the primary predictor or level of trigger for scheduling maintenance and rehabilitation (M&R) activities. According to Watanatada et al., (1987), it is the primary parameter that significantly affects road user costs through vehicle operating cost (VOC) resource consumption equations (55). Other studies have indicated that the choice of maintenance and rehabilitation options is very sensitive to roughness; however, a great attention should be given to its calibration.

Roughness progression model for paved roads has two calibration factors: roughness calibration factor, K_{gp}, and the age-environment factor, K_{ge}. According to Hass and al., the value of the roughness progression factor K_{gp} is nearly four times more significant than that of the age-environment factor K_{ge}.

4.5.5.1.3 Raveling prediction models

The only one factor involved in the raveling initiation and progression prediction models is the raveling initiation factor K_{vi}. According to the sensitivity studies undertaken by Hass et al., (1996) among all the other calibration factors, K_{vi} is the less sensitive, however, they recommended that K_{vi} should be given a less priority for calibration (61).

4.5.5.1.4 Pothole prediction models

Pothole progression equations are a function of base type, modified structural number (MSN), current condition, and a calibration parameter, K_{pp},. To obtain a closer value to observed local conditions, the pothole prediction equation should be calibrated by adjusting K_{pp}.

4.5.5.1.5 Rutting prediction models

The sensitivity studies conducted by Hass et al., (1996) under Tanzanian conditions showed that K_{rp} has a relative impact of less than 5 percent than the roughness progression factor K_{gp} on the NPV (61).

4.5.5.2 Vehicle operating cost models

HDM-III provides four sets of predictive equations for vehicle resource consumption. These predictive equations were derived from extensive research studies undertaken primarily in Kenya, the Caribbean, and later in India and Brazil. According to Watanatada et al., (55,56) the Brazilian models are far the most appropriate for local adaptation. Hass et al., (1996) suggested a comprehensive methodology for the selection and calibration of HDM-III VOC existing relationships for local applications. They recognize the following three steps (61):

1. The choice of the appropriate VOC sub-model for local application is an important initial step.
2. Once the appropriate relationships have been selected, types of vehicle are identified from standard HDM-III vehicle types that can best represent the local spectrum of traffic composition.
3. Finally, specific VOC model parameters may require to be adjusted to account for local conditions.

Concerning weak pavements (SN below 3.5) which is the case in most developing countries, Hass et al. reported that the most sensitive equations in HDM-III are the prediction of vehicle repair costs with respect to change in riding quality and prediction of change in riding quality with a given maintenance treatment. These statements mean that spare parts component in the VOC models is substantial and its calibration should be fully considered for local adaptation.

4.5.6 Sensitivity of HDM-III pavement performance models

Road deterioration is dependent on the selection of initial pavement strength, traffic characteristics, and subsequent M&R policies to be applied. In this study, pavement performance simulation procedures are based on HDM-III deterioration prediction models under Brazilian conditions. DARWin outputs (SN) calculated previously for a two-way average daily traffic of 500/1 000 vpd and 2 000/4 000 vpd and a CBR of 10 percent together with the respectively simulated diminished strength serve as direct inputs to HDM-III.

4.5.6.1 HDM-III models calibration

Because of the lack of existing data on West African roads for models calibration, throughout this research study, road deterioration and economic simulations are conducted on HDM-III performance and economic models under the Brazilian conditions.

4.5.6.2 HDM-III input parameters

HDM-III input parameters include all the alternative pavement characteristics and strength, environment, traffic loading in terms of volume and growth, and maintenance standards.

4.5.6.2.1 Pavement characteristics and strength

Both double bitumen surface treatment (DBST) and asphalt concrete (AC) roads are considered in this study. The roads designed are located in a sub-tropical, or sub-humid climates found in West Africa. The precipitation rate is estimated at 0.06 m/month and the altitude above the sea level is taken as 35 m. The geometrical characteristics adopted are as follows:

Road length	=	1.0 km
Road width	=	7.4 m
One shoulder width	=	1.8 m
Effective number of lanes	=	2.0
Rise and fall	=	4.7 m/km
Curvature	=	19.0 deg / km

4.5.6.2.2 Initial road roughness

IRI acceptance levels for new roads are very scarce. Paterson (1987) suggested that the initial roughness of new road construction, which depends on the construction method and quality, ranges from 1m/km IRI (4.2 PSI) for high quality paver laid asphalt to 4m/km IRI (2.5 PSI) for poor quality paved construction (56). Queiroz (1991), during his works related to road rehabilitation studies in Guinea-Bissau, observed a value of 2.0 IRI for recently paved roads in excellent condition (63). Zaghloul (1996) reported that the Italian Autostrade (Italy's national highways administration) specifications set the acceptance level for new roads at 1.5m/km IRI over any 400m section (52). In this study, an IRI of 1.5 for new pavements was assumed to be reasonable for road construction in developing countries. This value is derived from the approximate (± 25 percent) correlation suggested by Paterson (1987).

$$IRI \cong 5.5 * \log_e (5.0/PSI) \qquad (4.20)$$

where

IRI = International Roughness Index;
PSI = AASHO Pavement Serviceability Index.

4.5.6.2.3 Traffic characteristics and growth

Traffic composition and annual growth are given in Chapter 2. Table 4.2 below provides the reference values for vehicle characteristics to be used in the analysis.

Table 4.2

Vehicle basic characteristics

Basic Characteristics	Car	Pickup	Bus	Light truck	Medium Truck	Heavy Truck	Articulated Truck
Gross vehicle weight (t)	1.200	1.800	10.900	5.600	11.300	20.800	27.000
ESAL Factor/Vehicle	0.000	0.010	0.500	0.100	1.000	3.000	5.000
Number of axles	2	2	2	2	2	3	5
Number of tires	4	4	6	6	6	18	18
Number of passengers	5	14	26	3	3	3	3

4.5.6.2.4 Maintenance and rehabilitation strategies

Maintenance and rehabilitation strategies used in this study are basic routine maintenance. It is applicable for target and as-constructed pavements, and consists of all the necessary basic maintenance activities to maintain roads usable. It is mainly a regular routine maintenance and includes care for the road side vegetation, shoulder maintenance and repair, and drainage.

4.5.6.3 HDM-III output results

HDM-III output results are expressed in terms of wide and all cracking initiation and progression, rutting progression, potholing initiation and progression, raveling initiation and progression, and roughness progression.

4.5.6.3.1 Cracking initiation and progression

The model predicting time of cracking initiation and incorporated in HDM-III requires only the modified structural number (SNC) and the annual traffic loading as inputs. A sensitivity analysis conducted by Paterson (1987) for the asphalt surfacing model showed that doubling the traffic from 0.15 to 0.30 million ESALs/lane/year on an

SNC of 2 reduces pavement life by about 50 to 65 percent, whereas doubling the traffic from 0.15 to 0.30 million ESALs/lane/year on an SNC of 8 reduces the life by only about 8 to 12 percent (56).

In this sensitivity study, HDM-III output in terms of percent all cracking and percent wide cracking area for target and as-constructed pavements for both pavement types and loading are given in Table 4.3 and 4.4 below and in Figures 1B through 4B (Appendix B).

Table 4.3

Percent all and wide cracking progression for

for a two way-AADT of 500/1 000 vpd

and a CBR of 10 percent

Year	Percent All Cracking			Percent Wide Cracking		
	(ΔSN/SN) *100					
	100%	90%	80%	100%	90%	80%
1	0,0	0,0	0,0	0,0	0,0	0,0
2	0,0	0,0	0,0	0,0	0,0	0,0
3	0,0	0,0	0,0	0,0	0,0	0,0
4	0,0	0,0	0,0	0,0	0,0	0,0
5	0,0	0,0	0,0	0,0	0,0	0,0
6	0,0	0,0	0,0	0,0	0,0	0,0
7	0,0	0,0	0,0	0,0	0,0	0,0
8	0,0	0,0	0,0	0,0	0,0	0,0
9	0,0	0,0	0,0	0,0	0,0	0,0
10	0,0	0,2	0,9	0,0	0,0	0,0
11	2	2,4	3,7	0,0	0,0	0,0
12	6,4	7,2	9,9	0,0	0,0	0,0
13	14,9	16,3	20,9	4,1	4,1	4,1
14	29	31,2	38,3	17,6	17,6	17,6
15	50,2	53,3	61,3	50,2	51,1	51,1
16	71	73,1	78,5	71	73,1	78,5
17	84,8	86,1	89,4	84,8	86,1	89,4
18	93	93,7	95,4	93	93,7	95,4
19	97,3	97,6	97,7	97,3	97,6	97,7
20	97,4	100	100	97,4	100	100

Table 4.4

Percent all and wide cracking progression
for a two way-AADT of 2 000/4 000 vpd
and a CBR of 10 percent

Year	Percent All Cracking			Percent Wide Cracking		
	(ΔSN/SN) *100					
	100%	90%	80%	100%	90%	80%
1	0,0	0,0	0,0	0,0	0,0	0,0
2	0,0	0,0	0,0	0,0	0,0	0,0
3	0,0	0,0	0,0	0,0	0,0	0,0
4	0,0	0,0	0,7	0,0	0,0	0,0
5	0,5	1,8	3,1	0,0	0,0	0,0
6	2,6	5,4	7,6	0,0	0,0	0,0
7	6,8	11,2	14,4	0,0	3,6	4,0
8	13,3	19,4	23,6	4,0	10,3	11,0
9	22,2	30,1	35,3	11,0	19,8	20,8
10	33,6	43,4	49,7	20,8	32,1	33,3
11	47,6	58,5	64,2	33,3	46,8	48,1
12	62,4	71,3	75,9	48,2	62,4	63,5
13	74,5	81,6	85,1	63,5	75,5	76,4
14	84,0	89,3	91,8	76,5	86,0	86,7
15	91,1	94,7	96,3	86,8	93,8	94,2
16	95,9	98,0	98,8	94,3	98,0	98,7
17	98,6	98,8	98,5	98,6	98,8	100
18	98,7	98,4	98,1	98,7	100	100
19	100	100	100	100	100	100
20	100	100	100	100	100	100

From this simulation, for pavements designed to sustain low traffic volumes, (surface treatment on granular base pavements), it can be concluded that :

1. for all alternative pavement strength, both types of cracking initiate at almost the same pavement age (i.e., 10 years for all cracking and 13 years for wide cracking after construction);

2. the incremental increase in all and wide cracking progression is not significant for as-constructed pavements compared to as-designed pavements;

3. For higher traffic asphalt concrete pavement, all and wide cracking initiate at very much lower pavement age and progress at relatively higher rates.

4.5.6.3.2 Rutting progression

It is well known that traffic loading causes deformations when the stresses induced in the pavement materials are sufficient to cause shear displacements within the materials. Thus, a single load or few excessive loads or tire pressures, causing stresses that exceed the shear strength of the materials, can cause plastic flow. Also, repeated loadings at lesser load and tire pressures cause smaller deformations which accumulate over time and become manifest as a rut if the loadings are channelized into the wheelpaths. Since rutting is not a maintenance intervention criterion in HDM-III, the model developed from data collected on typical road networks constructed to modern design codes, do not take account of shear failure. Thus most design methods such as AASHTO design algorithms require higher strength materials in the upper regime of the pavement, and indicate combinations of layer thickness and stiffness sufficient to attenuate the applied stresses to levels at which the accumulated shear deformations through the pavement depth are within a specified rut depth criterion. The results show that, rut depth progression for as constructed pavement are lower than expected for both traffic loading and pavement types. While the trend and the average rate of progression of rut depth in relation to mixed traffic is generally well validated, the variance of the absolute rut depth about the predicted mean may be high. Paterson (1987) reported that, for situations where significant amounts of plastic flow within the pavement materials are evident such as in the case of as-constructed pavements, the model should be enhanced to accommodate the plastic flow phase more fully which requires further research studies

(56). In this simulation, HDM-III output in terms of rutting progression (mm) is given in Table 4.5 below and in Figure 5B and 6B (Appendix B).

Table 4.5

Rutting progression for two-way AADT
of 500/1000 vpd and 2000/4000 vpd
and a CBR of 10 percent

Year	T1 - CBR2			T3 - CBR2		
	(ΔSN/SN) *100					
	100%	90%	80%	100%	90%	80%
1	1,9	2,1	2,2	1,9	2,1	2,3
2	2,1	2,3	2,5	2,1	2,3	2,5
3	2,2	2,4	2,6	2,3	2,4	2,7
4	2,4	2,6	2,8	2,4	2,6	2,8
5	2,5	2,7	2,9	2,5	2,7	2,9
6	2,6	2,8	3	2,6	2,8	3,1
7	2,7	2,9	3,2	2,7	2,9	3,2
8	2,8	3	3,3	2,8	3,1	3,4
9	2,8	3,1	3,3	2,9	3,2	3,5
10	2,9	3,1	3,4	3,1	3,4	3,8
11	3,0	3,2	3,5	3,3	3,6	4,0
12	3,0	3,3	3,6	3,5	3,8	4,2
13	3,1	3,4	3,7	3,7	4,0	4,4
14	3,2	3,5	3,9	3,8	4,2	4,6
15	3,4	3,7	4,1	4,0	4,4	4,8
16	3,5	3,9	4,2	4,2	4,5	4,9
17	3,7	4	4,4	4,3	4,6	5,0
18	3,8	4,1	4,5	4,3	4,6	5,1
19	3,9	4,2	4,6	4,4	4,7	5,1
20	3,9	4,2	4,6	4,4	4,8	5,2

Paterson (1987) reported that, modeling rut depth progression and standard deviation in the Brazilian studies was based on data collected on typical road networks constructed to modern design codes. Thus, while, the values of rut depth used for modeling were generally low on account of modern design standards of the existing pavements, any deviations from this main range of application, (i.e., severely under-designed pavements) may somewhat restrict the validity of these models for cases of extreme behavior (56).

4.5.6.3.3 Pothole initiation and progression

HDM-III pothole initiation and progression modeling was carried out at network level. Although, it would have been most appropriate to include the surface condition, base type, traffic and climate in the statistical estimation of the likelihood of occurrence, only simple models were developed based on data collected in the Caribbean, Ghana, Kenya, and Brazil. Pothole initiation is expressed as a function of the time since the initiation of triggering distress and traffic flow and occurs typically two to six years after wide cracking and three to six years after raveling of thin surface treatments. As with initiation, thick surfacing are modeled to be less sensitive to pothole progression than thin surfacing as shown in HDM-III output results summarized in Table 4.6 below.

Table 4.6 below and Figure 7B and 8B (Appendix B) show that at the end of the analysis year, the incremental increase in the rate of potholing progression for the two-way AADT of 500/1 000 vpd is in the order of 10 percent and 16 percent for 10 percent and 20 percent reduction in pavement strength, respectively. For the two-way AADT of 2 000/4 000 vpd, the incremental increase in the rate of potholing progression is more pronounced and is in the order of 16 percent and 47 percent for 10 percent and 20 percent reduction in pavement strength, respectively.

Table 4.6

Percent area of potholes for two-way AADT
of 500/1000 vpd and 2000/4000 vpd
and a CBR of 10 percent

Year	T1 - CBR2			T3 - CBR2		
	(ΔSN/SN) *100					
	100%	90%	80%	100%	90%	80%
1	0,0	0,0	0,0	0,0	0,0	0,0
2	0,0	0,0	0,0	0,0	0,0	0,0
3	0,0	0,0	0,0	0,0	0,0	0,0
4	0,0	0,0	0,0	0,0	0,0	0,0
5	0,0	0,0	0,0	0,0	0,0	0,0
6	0,0	0,0	0,0	0,0	0,0	0,0
7	0,0	0,0	0,0	0,0	0,0	0,0
8	0,0	0,0	0,0	0,0	0,0	0,0
9	0,0	0,0	0,0	0,0	0,0	0,0
10	0,0	0,0	0,0	0,0	0,0	0,0
11	0,0	0,0	0,0	0,0	0,0	0,1
12	0,0	0,0	0,0	0,1	0,1	0,2
13	0,1	0,1	0,1	0,2	0,2	0,3
14	0,1	0,1	0,1	0,3	0,4	0,6
15	0,2	0,2	0,2	0,5	0,6	0,8
16	0,5	0,5	0,5	0,7	0,9	1,1
17	0,8	0,9	1	1,0	1,2	1,5
18	1,3	1,4	1,6	1,3	1,6	1,9
19	1,9	2,1	2,3	1,6	2,0	2,3
20	2,6	2,9	3,1	2,0	2,4	2,8

4.5.6.3.4 Raveling initiation and progression

HDM-III output results in terms of percent raveled area for as-designed and as-constructed pavements for both traffic loading and a CBR of 10 percent are given in Table 4.7 below and in Figure 9B and 10B (Appendix B).

Table 4.7

Percent raveled area for two-way
AADT of 500/1 000 vpd and
a CBR of 10 percent

Year	(ΔSN/SN) *100		
	100%	90%	80%
1	0,0	0,0	0,0
2	0,0	0,0	0,0
3	0,0	0,0	0,0
4	0,0	0,0	0,0
5	0,0	0,0	0,0
6	0,0	0,0	0,0
7	0,0	0,0	0,0
8	0,0	0,0	0,0
9	0,0	0,0	0,0
10	3,5	3,5	3,5
11	25,0	25,0	25,0
12	71,3	71,3	71,3
13	85,0	83,6	79,0
14	100	100	100
15	100	100	100
16	100	100	100
17	100	100	100
18	100	100	100
19	100	100	100
20	100	100	100

Up to twelve years old, the percentage of raveled area for target and as-constructed pavements are identical. That is because raveling models incorporated in HDM-III use only three explanatory parameters as defined in equations 4.9 and 4.10 and are independent of the pavement strength parameters such as the structural number and the deflection. For all alternative pavements, the predicted time for a level of 50 percent raveling is about 11 to 12 years which is consistent with reseal frequencies adopted in many countries.

4.5.6.3.5 Roughness progression

The roughness model incorporated in HDM-III was developed from a statistical analysis of data collected on in-service pavements. The model estimates the change in roughness over a certain time interval as a function of the initial roughness, equivalent axle loading, and the incremental increase in rut depth, cracking, surface patching, volume of potholes and the rut depth standard deviation. NCHRP Report 332 (1990) reported that because information on other forms of pavement distress are needed to predict changes in roughness, the model is difficult to use for long term pavement performance predictions and is applicable only for scheduling maintenance needs (33).

HDM-III deterioration models used in this simulation are derived from high standard road networks designed to modern codes. Among these deterioration algorithms, the roughness model is the major road attribute for vehicle operating cost and counts for up to 95 percent of pavement serviceability. Because of the lack of considering other forms of failure such as shear deformation in the pavement layers due to improper design or a faulty construction, HDM-III models should be used with caution. Watanadata et al., (55) also, suggested that the model need further improvements with respect to better establishing the effect of small changes in roughness on very smooth paved roads (less than 2.7 m/km IRI).

HDM-III output results in terms of roughness progression (m/km IRI) for as-designed and as-constructed pavements for both traffic loading and a CBR of 10 percent are given in Table 4.8 below and Figure 11B (Appendix B).

Table 4.8

Roughness progression for two-way AADT
of 500/1 000 vpd and 2 000/4 000 vpd
and a CBR of 10 percent

Year	T1 - CBR2			T3 - CBR2		
	(ΔSN/SN) *100					
	100%	90%	80%	100%	90%	80%
1	1,6	1,6	1,7	1,6	1,6	1,7
2	1,7	1,7	1,7	1,7	1,7	1,7
3	1,7	1,7	1,8	1,7	1,8	1,8
4	1,8	1,8	1,8	1,8	1,8	1,9
5	1,8	1,8	1,9	1,8	1,9	1,9
6	1,9	1,9	1,9	1,9	2,0	2,0
7	1,9	1,9	2,0	2,0	2,1	2,1
8	2,0	2,0	2,0	2,1	2,2	2,3
9	2,0	2,1	2,1	2,2	2,3	2,4
10	2,1	2,1	2,2	2,4	2,5	2,6
11	2,2	2,2	2,2	2,5	2,7	2,9
12	2,2	2,3	2,3	2,7	2,9	3,1
13	2,4	2,4	2,5	3,0	3,2	3,4
14	2,6	2,6	2,7	3,2	3,4	3,7
15	2,8	2,9	3,0	3,5	3,7	4,0
16	3,2	3,3	3,4	3,7	4,0	4,3
17	3,5	3,6	3,7	4,0	4,3	4,6
18	3,9	4,0	4,2	4,2	4,6	5,0
19	4,2	4,4	4,6	4,5	4,9	5,4
20	4,7	4,8	5,1	4,8	5,2	5,8

CHAPTER 5

PAVEMENT ECONOMICS AND PAY SCHEDULES

5.1 Background

Considerations of pavement economics from the planning and design stages up to construction, maintenance and rehabilitation are important tasks for a road network management system. This is particularly true for low volume roads where small changes in the thicknesses of the structural layers due to improper design or a faulty construction can lead to a substantial increase in the total transport cost. Therefore determining the most optimum practical design and enforcing contractor's reliability for proper implementation are a big challenge for local and national road agencies.

5.2 Review of pay factor development methodologies

The development of adjusted pay schedules includes the determination of appropriate pay levels for various levels of quality. Several methods were proposed in the literature, and where there was little or no information relating quality measures to performance, the methods have necessarily been quite arbitrary. In cases for which quality-performance relationships have been established, one of the more rational methods of establishing pay schedules is based on the legal principle of liquidated damages: the pay schedule is designed to withhold sufficient payment at the time of construction to cover the cost of future repairs made necessary by defective work (40).

For the development of truly performance related specifications (PRS), Irick (1988) suggested that the algorithms used to relate the material and construction (M&C) variables to performance and to derive the M&C specifications should be the same ones used to design the pavement in the first instance, or derivatives therefrom. They should be also the algorithms from which the effect on performance of deviations from specified quality levels can be measured and, thereby, the economic impact of such deviations assessed (31).

5.3 Existing forms of pay schedules

Normally, a new pavement is designed for a particular environment to sustain (within a specified probability) a specified number of load repetition before major repair or rehabilitation is required. If, because of deficient construction and or material, the pavement is not capable of withstanding the anticipated design traffic loading, it will fail prematurely. The necessity of rehabilitating this pavement at an earlier date results in additional future expenses to both the agency and the users. These expenses usually occur long after any contractual obligations have expired and thus, they should be quantified by the transportation agency at earlier stages of pavement construction. It is the purpose of the adjusted pay schedule to withhold sufficient, fair and equitable payment for both contracting parties at the end of construction to cover the extra cost anticipated in the future as a result of lack of compliance to the specifications.

According to Richard M. Weed (1984), two basic types of pay schedules are in use today : stepped and continuous. Stepped pay schedules define discrete intervals of the quality measure and assign a specific pay factor (PF) for each interval. Continuous pay schedules express the pay factor in equation form as a function of the quality measure. He added that, although stepped pay schedules are in more common use, continuous pay schedules are rapidly gaining favor (40). This mainly because the difference in payments between two adjacent intervals in a stepped pay schedule can be quite substantial and, when the quality measure happens to fall close to an interval boundary, this can lead to disputes over measurement precision, round-off rules, and so forth. With a continuous pay schedule there is a smooth progression of adjusted payment as the quality varies and, consequently, the potential harshness of having just missed the next higher pay level is completely avoided.

NCHRP syntheses 212 reported that recent advances in PRS development have led to a consensus that adjustments to contractor's bid price in response to the work that deviates from the quality level anticipated should correspond to the present worth of the cost differential resulting from such deviations (21). According to this approach, the pay schedule is designed to withhold sufficient payments at the time of construction to cover such costs. It is also designed to award a positive price adjustment in consideration of enhanced performance or service life when the work exceeds the design quality level prescribed. This approach involves incorporating estimates of the percentage loss or enhancement in performance or service life with certain basic concepts of engineering economics. At present, NCHRP syntheses 212 identified three methods for doing this depending on how the quality differential is measured and on which costs are included in the computations. These three methods in current use are (21) :

1. Methods suggested by Weed and based on the difference in estimated pavement life to measure the quality differential. Costs include neither maintenance nor user operating costs.

2. An approach developed by Irick et al., through research sponsored by FHWA that uses the difference in estimated life-cycle costs (LCC), and includes maintenance costs but not user operating costs in the computations.

3. An approach presented by Anderson et al., in NCHRP Report 332, uses estimated economic life, defined as the age at which the minimum equivalent uniform annual total cost occurs. Both maintenance and user operating costs are included in the computations.

The first approach suggested by Weed (1989) has been adopted by New Jersey Department of Transportation (DOT) and other highway agencies (64). It consists of using the quality differential measure and the cost of materials and labor. The philosophy supporting this approach is based on the concepts that for highway pavements, layer thicknesses and material characteristics are selected and characterized to carry the estimated loading for the desired service life, at the end of which the pavement will commence receiving a series of overlays. If the pavement is incapable of carrying the estimated loading for its design life, because of construction deficiencies, it will fail prematurely and the overlays will be moved forward in time, resulting in added expenses to the transportation agency. Similarly, if the pavement is able to carry the estimated loading beyond its design life because of construction quality that exceeded the design, the overlays will be delayed in time, resulting in savings to the transportation agency. Using simple engineering economic principles, Weed suggested the following equation to be used to calculate the appropriate pay factors for various levels of expected pavement life :

$$PF = 100 *[1 + C_o(R^{Ld} - R^{Le})/C_p(1-R^{Lo})] \quad (5.1)$$

where

PF = pay factor (percent of C_p);

C_p = present unit cost of pavement;

C_o = present unit cost of overlay;

Ld = design life of pavement;

Le	=	expected life of as constructed pavement;
Lo	=	expected life of overlay;
R	=	$(1 + R_{inf}/100)/(1 + R_{int}/100)$;
R_{inf}	=	annual inflation rate in percent;
R_{int}	=	annual interest rate in percent.

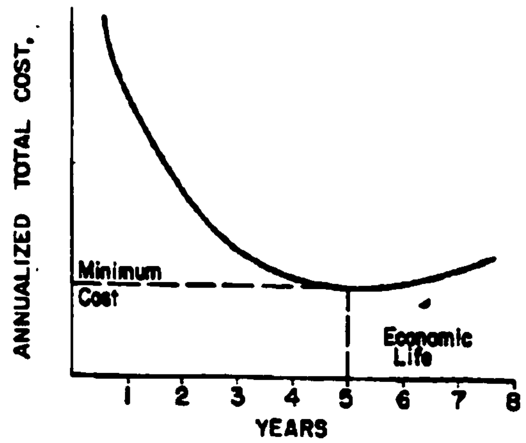
The second approach was sponsored by the FHWA and developed by ARE Inc. (32). It consists of the development of performance related specifications for Portland cement concrete construction. A full development of the methodology can be found in a publication presented by Irick (1990).

The third approach suggested by Anderson et al., (1990) and described in NCHRP Report 332 is based on the principles of pavement economic life which is defined as the age at which the EUATTC is minimum (33). This concept, illustrated in Figure 5.1 below, is frequently used in replacement analysis in industrial engineering applications

Figure 5.1

Concept of economic life in

replacement analysis



According to NCHRP Research team, to develop a rational penalty function for work deviating from specifications, the performance of target (as-designed) and as-constructed pavements should be evaluated and compared using a common denominator approach based on the total life-cycle costs (LCC).

This procedure takes account of all costs incurred including construction, maintenance and rehabilitation costs as well as user operating costs. It allows the agency to minimize the costs incurred to the user who pays user costs directly and indirectly through taxes for construction and maintenance.

The equivalent uniform annual total transport cost at any year n within the analysis period is calculated using the following formula :

$$A_n = [\text{Total cumulative cost at year n}] * [(1-r)/(1-r^n)] \qquad (5.2)$$

where

A_n = equivalent uniform annual total cost at year n expressed in $/m^2/year ;
r = $1/(1+i)$;
i = discount rate.

5.4 Cost responsibility assessment

Total life-cycle cost of pavements can naturally span for several performance periods. However, the contractor's responsibility over a specified performance period can only reasonably be assessed for his conformance to the specifications specified for that particular performance period. However, he cannot be responsible of the succeeding performance periods which are functions of how well other contractors will conform to specifications in each period. NCHRP research team proposed four methods of evaluating cost responsibilities which can be summarized as follows:

The first method, illustrated in Figures 5.2 (a) and (b) below, considers the total cost for only the first performance period and does not take account of the rehabilitation cost at the end of the performance period. The contractor's cost responsibility is then viewed as the difference between the as-designed costs and as-constructed costs. This can be expressed mathematically as:

$$(X_c - X_o) \tag{5.3}$$

where

X_c = costs associated with as-constructed pavement;

X_o = costs associated with as-designed pavement.

Figure 5.2 (a)

Pavement optimum design

and rehabilitation strategy

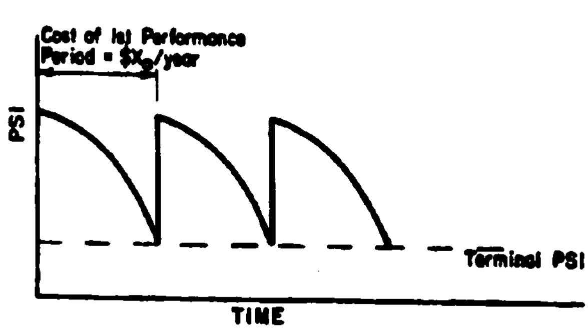
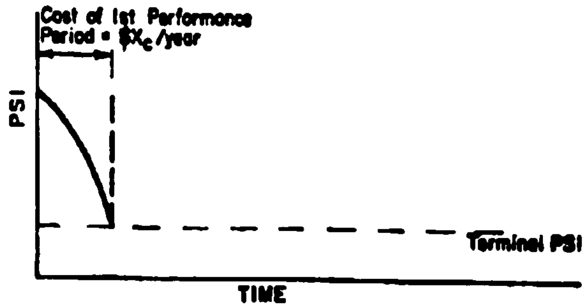


Figure 5.2 (b)

Predicted pavement performance

based on contractor's work



The weakness of this method is that it does not consider the contractor's influence on the subsequent future rehabilitation as a result of lack of conformance to target specifications.

The second method illustrated in Figures 5.3 (a) and (b) below, takes account of the total cost for the first performance period and the subsequent rehabilitation. In this case, the contractor's responsibility is considered to be the difference between the costs of as-designed pavement and the sum of the costs of as-constructed pavement and the subsequent rehabilitation. Mathematically this can be expressed as :

$$(X_c + R_c) - (X_o + R_o) \tag{5.4}$$

where

R_o = rehabilitation for as-designed pavement;

R_c = rehabilitation for as-constructed pavement;

Figure 5.3 (a)

Pavement optimum design

and rehabilitation strategy

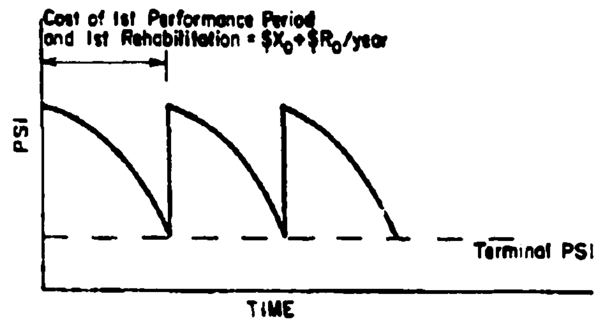
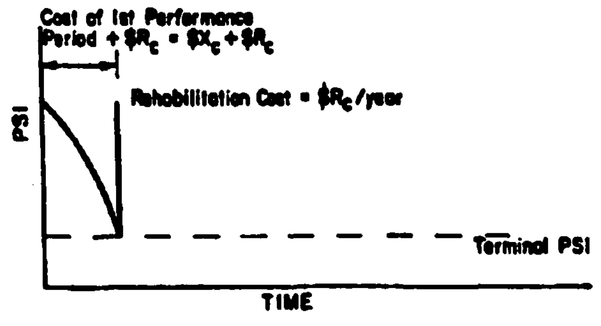


Figure 5.3 (b)

Predicted pavement performance

based on contractor's work and

subsequent rehabilitation



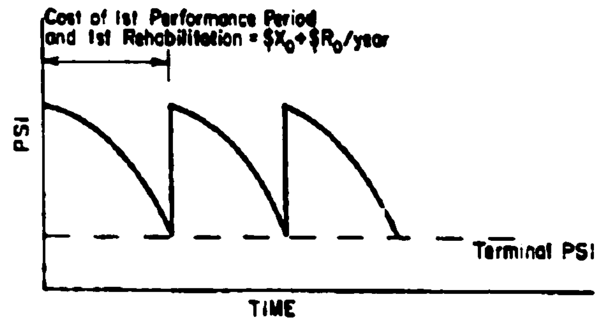
In this method the contractor would be penalized for causing the rehabilitation to occur earlier and for any increase in rehabilitation cost over the planned rehabilitation. The difficulties associated with this approach is that it does not consider the effect of the anticipated rehabilitation on future agency and road user costs. Because of significant reductions in road user costs, this method seems to over-penalize the contractor.

The third approach, illustrated in Figure 5.4 (a) and (b) below, is based on the optimization strategy resulting from the earlier failure of the pavement because of the contractor's non conformance to the specifications. Thus, a new optimum strategy which considers the imposed performance period must be developed. In this case the contractor's cost responsibility would be the difference between the target and as-constructed pavements total costs expressed mathematically as:

$$\Sigma X_c - \Sigma X_o \tag{5.5}$$

Figure 5.4 (a)

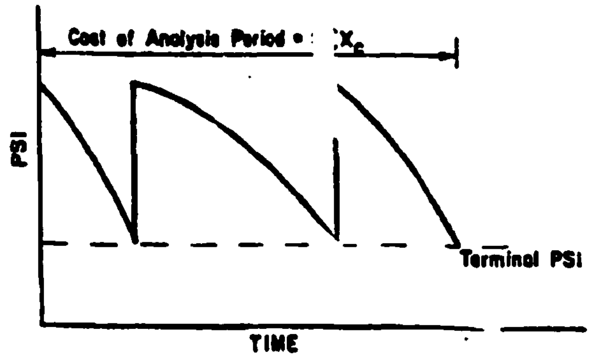
Pavement optimum design
and rehabilitation strategy



The fourth method considers the total cost in two parts: the first performance period and the capitalized cost of all future performance periods. Total cost for the first period being the same as discussed in method one. The difference with the first method for this procedure is that the total cost for the first period is combined with the capitalized cost of all future rehabilitation.

Figure 5.4 (b)

Predicted pavement performance based
on contractor's work and resulting
optimum rehabilitation strategy



In this study, to evaluate the performance of pavements designed for a 20 years analysis period to AASHTO algorithm, concepts of equivalent uniform annual total transport cost (EUATTC) developed by The National Cooperative Highway Research Program (NCHRP) are selected. The aim of this study is to compare the simulated pavement economic life to the analysis period used in the design. According to NCHRP research team, the use of the equivalent uniform annual total cost concept in pavement economics for the development of pay functions has several advantages that can be summarized as follows :

1. The pay function is based on the total transport cost, i.e., construction, agency, and user, estimated from the predicted pavement performance.

2. Failure levels of roughness, rutting, and cracking do not have to be arbitrarily defined. Instead, the economic life determines which combinations of different distress modes will cause failure. However, the economic life will indicate failure when the annual rise in user and maintenance costs overshadows the decline in amortized construction cost.

3. A design period (e.g., 20 years) does not have to be arbitrarily established. Instead, the economic life will indicate when rehabilitation should occur.

5.5 HDM-III performance and economic evaluation

HDM-III performance and economic evaluation models are capable of predicting pavement performance over time and assigning costs to the various pavement design alternatives. These costs include the initial road construction and future maintenance and rehabilitation. Furthermore, user costs associated with the various alternatives (i.e., vehicle operating costs) and user delay costs during maintenance and overlay are also included. HDM-III simulates the conditions and the costs of road's life-cycle as a function of its characteristics, traffic volume and weight, environment conditions, and maintenance operations.

Traffic characteristics and composition and pavement strengths and characteristics are given in Chapter 2. To calculate vehicle operating costs and user delay costs, Table 5.1 below provides the reference values used in the analysis.

Table 5.1

Vehicle basic characteristics

Basic Characteristics	Car	Pickup	Bus	Light truck	Medium Truck	Heavy Truck	Articulated Truck
Service life (years)	16.0	16.0	17.0	21.0	22.0	22.0	24.0
Hours driven/year	1 000	1 500	2 500	2 500	2 500	2 500	2 000
Kilometers driven/year	35 000	50 000	85 000	85 000	85 000	85 000	70 000
Depreciation code	2	2	2	2	2	2	2
Utilization code	1	3	3	3	3	3	3
Annual interest rate (%)	12.00	12.00	12.00	12.00	12.00	12.00	12.00
New vehicle price	3 500	4 500	7 000	15 000	24 000	26 000	28 850
New tire price	17.0	24.7	25.9	73.8	95.4	99.0	104.0
Maintenance labor (/hr)	0.50	0.50	0.50	0.50	0.50	0.50	0.50
Crew cost	0.00	0.30	0.35	0.50	0.50	0.50	0.50
Passenger time	0.10	0.10	0.10	0.10	0.10	0.10	0.10
Cargo time	0.00	0.00	0.00	0.10	0.10	0.10	0.10

Gas / Petrol Price (1000F CFA/liter) = 0.12

Diesel Price (1000F CFA/liter) = 0.13

Lubricants Price (1000F CFA)/liter = 0.07

Maintenance and rehabilitation strategies used in this study are basic routine maintenance. Routine maintenance consists of all the necessary basic maintenance activities to maintain roads usable. It is mainly a regular routine maintenance and includes care for the road side vegetation, shoulder maintenance and repair, and drainage. Routine Maintenance economic unit cost is fixed at 568 000 F CFA per year.

The operations unit costs and material strengths considered are in accordance with those applicable in West Africa in 1996. For as-designed pavements, Tables 5.2 and 5.3 below summarize these inputs. The discount rate is fixed at 12 percent.

Table 5.2

Construction material strengths and costs

Cost & Strength	AC (40mm)	DBST (15mm)	Granular Base (1 000F CFA/m^3)
Cost (1 000 F CFA/m^2)	4.76	2.04	11.20
Strength coefficient	0.42	0	0.14

Table 5.3

Pavement material and construction costs

TRAFFIC	CBR1 = 6%		CBR2 = 10%		CBR3 = 14%	
	STRUCT. LAYERS (in)	COST/m2 (F FCFA)	STRUCT. LAYERS (in)	COST/m2 (F FCFA	STRUCT. LAYERS (in)	COST/m2 (F FCFA
T1 =	DBST 0.59	2 038.09	DBST 0.59	2 038.09	DBST 0.59	2 038.09
2,792,500	GB 29.91	11 518.90	GB 24.91	9 593.31	GB 21.96	8 457.21
(ESALs)/ Lane	Total 30.50	13 557.00	Total 25.50	11 631.41	Total 22.55	10 495.31
T2 =	AC 3.50	10 579.10	AC 3.50	10 579.10	AC 3.50	10 579.10
6, 981,050	GB 21.34	8 218.43	GB 15.63	6 019.41	GB 12.32	4 744.66
(ESALs)/ Lane	Total 24.84	18 797.53	Total 19.13	16 598.51	Total 15.82	15 323.76
T3 =	AC 4.00	12 090.40	AC 4.00	12 090.40	AC 4.00	12 090.40
11,169,600	GB 21.88	8 426.40	GB 15.89	6 119.54	GB 12.32	4 744.66
(ESALs)/ Lane	Total 25.88	20 516.80	Total 19.89	18 209.94	Total 16.32	16 835.06

In this study, HDM-III output results in term of economic total transport cost (TTC) streams is selected. This economic criteria is suitable for the application of pavement economic life principles described earlier. HDM-III economic evaluations expressed in millions of F CFA are summarized in Table 5.4 below.

Table 5.4

Economic T.T.C streams for all combinations

of traffic loading and subgrade CBR

(*1 000 000 F CFA)

Year	TRAFFIC 1			TRAFFIC 2			TRAFFIC 3		
	CBR1	CBR2	CBR3	CBR1	CBR2	CBR3	CBR1	CBR2	CBR3
Initial Construction	100.3222	86.072	77.665	139.102	122.829	113.396	151.825	134.754	124.580
1	20.294	20.294	20.294	49.882	49.883	49.883	79.471	79.472	79.473
2	21.006	21.006	21.007	51.662	51.664	51.665	82.319	82.322	82.324
3	21.739	21.739	21.740	53.495	53.497	53.499	85.253	85.257	85.260
4	22.498	22.499	22.499	55.395	55.397	55.399	88.293	88.298	88.302
5	23.285	23.286	23.287	57.363	57.366	57.369	91.443	91.450	91.456
6	24.101	24.102	24.102	59.404	59.408	59.411	94.710	94.720	94.728
7	24.946	24.947	24.948	61.520	61.527	61.531	98.101	98.113	98.125
8	25.822	25.823	25.824	63.717	63.728	63.734	101.624	101.639	101.655
9	26.729	26.731	26.732	66.001	66.018	66.025	105.289	105.307	105.328
10	27.670	27.672	27.673	68.376	68.399	68.408	109.101	109.124	109.151
11	28.645	28.647	28.649	70.847	70.878	70.890	113.070	113.099	113.132
12	29.658	29.660	29.662	73.420	73.463	73.477	117.210	117.246	117.285
13	30.713	30.716	30.718	76.100	76.158	76.174	121.529	121.572	121.616
14	31.816	31.821	31.824	78.890	78.958	79.141	126.280	126.933	127.574
15	32.974	32.981	32.985	82.140	83.095	83.373	133.207	133.985	134.713
16	34.193	34.202	34.259	86.445	87.500	87.824	140.555	141.485	142.320
17	36.222	36.341	36.423	90.965	92.120	92.500	148.349	149.466	150.431
18	38.512	38.637	38.738	95.709	96.952	97.394	156.589	157.922	159.048
19	40.989	41.142	41.269	100.668	102.041	102.557	165.378	166.968	168.303
20	43.701	43.890	44.049	105.901	107.473	108.083	174.910	176.817	178.412

HDM-III output results are expressed in terms of economic and financial total society cost streams comprising agency and road user costs. Agency cost streams are mainly agency capital investment for road construction and rehabilitation and recurrent expenditure to cover routine maintenance costs. Road user cost streams comprise vehicle operation costs (VOC) and road user delay. The equivalent uniform annual cost components presented in Tables 1E through 6E (Appendix E) are calculated using HDM computations for agency and road user cost streams and conventional engineering economics as follows:

$$NVP = \text{Initial Construction Cost} + \sum \text{Rehab. Cost}_k \left[1/(1+i)^n{}_k \right] \tag{5.6}$$

$$EUAC = NPV \left[i\,(1+i)^n / (1+i)^n - 1 \right] \tag{5.7}$$

where

NPV = net present value of costs;

EUAC = equivalent uniform annual cost;

i = discount rate (fixed at 12 percent);

n = analysis period (fixed at 20 years).

Vehicle operation cost components such as fuel consumption, tire wear, maintenance parts, and maintenance labor can be calculated using equation 4.15 through 4.19 given in Chapter 4.

As shown in Tables 1E, 2E and 3E, (Appendix E), road user costs constitute a large share (64 to 85 percent) of the total road transport cost. Thus, except when traffic volume is extremely low, the effect on total transport cost of even a small percentage change in road user costs is large relative to the effect of changes in construction and maintenance costs.

5.6 Pavement economic life

Using the concepts of pavement economic life described earlier and equation 5.2, HDM-III output results for all combinations of traffic loading and soil CBR were converted into equivalent uniform annual total transport cost (E.U.A.T.T.C). These costs assigned to target pavements are given in tables 5.5 (a), 5.5 (b), and 5.5 (c) below.

Table 5.5 (a)

E.U.A.T.T.C for two-way AADT of
500/1 000 vpd and all combinations
of subgrade strength
(F CFA)

Year	T1-CBR1	T1-CBR2	T1-CBR3
		E.U.A.T.T.C	
8	42 695 291	39 827 363	38 135 571
10	40 820 082	38 298 860	36 811 568
11	40 230 621	37 831 561	36 416 374
12	39 792 525	37 492 958	36 136 494
13	39 468 593	37 251 175	35 943 178
14	39 232 348	37 083 539	35 816 014
16	38 950 529	36 908 666	<u>35 705 431</u>
17	38 894 712	<u>36 897 053</u>	35 720 111
18	<u>38 887 847</u>	36 928 263	35 774 243
20	38 987 309	37 090 383	35 974 499

Table 5.5 (a) above and Figures 1C, 2C, and 3C (Appendix C) show that, after the initial construction, the economic life of as-designed pavements occurs at the years 18, 17, and 16 dependent on subgrade strength. Table 4E in Appendix E shows only a slight variation in the percentage of the road user cost share (63 to 67 percent of the total road transport cost) as compared to that resulting from the entire analysis period. All distress combinations at pavement economic life are summarized in Table 7E of Appendix E. Pavement rehabilitation should occur at roughness between 3.2 and 3.8 m/km IRI dependent on the subgrade strength.

Table 5.5 (b)

E.U.A.T.T.C for two-way AADT of
1 250/2 500 vpd and all combinations
of subgrade strength
(F CFA)

Year	T2-CBR1	T2-CBR2	T2-CBR3
		TOTAL EUAEC	
8	83 402 403	80 129 964	78 233 258
10	81 435 539	78 560 827	76 894 215
11	80 922 890	78 188 860	76 603 518
12	80 611 994	77 993 036	76 473 965
13	80 451 019	<u>77 927 567</u>	<u>76 463 264</u>
14	<u>80 402 829</u>	77 959 378	76 545 929
16	80 589 663	78 317 070	76 988 570
17	80 801 909	78 599 433	77 305 884
18	81 069 302	78 928 628	77 666 210
20	81 718 583	79 684 054	78 475 267

Table 5.5 (b) above and Figures 4C, 5C, and 6C (Appendix C) show that, after the initial construction, the economic life of as-designed pavements occurs at the year 14 for weak soil CBR and at the year 13 for higher subgrade strength. Table 5E (Appendix E) also, shows only a slight variation in the percentage of the road user cost share (73 to 76 percent of the total road transport cost) as compared to that resulting from the entire analysis period. All distress combinations at pavement economic life are summarized in Table 7E of Appendix E. Pavement rehabilitation should occur at roughness of about 3.0 m/km IRI.

Table 5.5 (c)

E.U.A.T.T.C for two-way AADT of
2 000/4 000 vpd and all combinations
of subgrade strength
(F CFA)

Year	T3-CBR1	T3-CBR2	T3-CBR3
		E.U.A.T.T.C	
8	118 866 987	115 436 596	113 393 864
10	117 443 903	114 430 385	112 637 306
11	117 232 139	114 365 925	112 661 257
12	117 231 221	114 485 266	112 852 850
13	117 384 554	114 738 101	113 165 495
14	117 659 168	115 114 573	113 610 303
16	118 602 005	116 225 723	114 834 647
17	119 210 531	116 905 710	115 562 832
18	119 881 000	117 641 432	116 342 839
20	121 351 952	119 229 461	118 011 964

Table 5.5 (c) above and Figures 7C, 8C, and 9C (Appendix C) show that, after the initial construction, the economic life of as-designed pavements occurs at the years 12, 11, and 10 dependent on the subgrade strength. Table 6E in Appendix E shows only a slight variation in the percentage of the road user cost share (79 to 80 percent of the total road transport cost) as compared to that resulting from the entire analysis period. All distress combinations at pavement economic life are summarized in Table 7E (Appendix E). Pavement rehabilitation should occur at roughness between 2.4 and 2.7 m/km IRI dependent on the subgrade strength.

5.7 Analysis of results

From these simulations, it can be concluded that :

1. except when the traffic volume is extremely low, road user costs constitute a large share of the total road transport cost (up to 85 percent in this study). Thus, the effect on total transport cost of even a small percentage change in road user costs is large relative to the effect of changes in construction and maintenance costs;

2. for any traffic loading, pavements designed to AASHTO algorithm for a defined analysis period require rehabilitation earlier than expected except for pavements designed and constructed on weak subgrade to serve low traffic volumes;

3. for any subgrade strength, pavements designed to AASHTO algorithm should be rehabilitated at roughness between 2.5 and 3.0 m/km IRI for high traffic volumes and between 3.0 and 4m/km IRI for low traffic volumes.

4. for the same subgrade strength and performance period, the economic life of a pavement designed to AASHTO algorithm is dependent on the design traffic loading. Pavements designed to withstand high traffic volumes have shorter economic lives compared to those designed for lower traffic and therefore require rehabilitation at earlier dates.

CHAPTER 6

PENALTY SCALE ESTABLISHMENT

6.1 Background

Liquidated damages are usually associated with the time of completion for a construction contract. Used in that manner, liquidated damages are predetermined costs that the agency assesses the contractor for losses as a result of being denied the use of the facility after the scheduled date of contract completion. However, used in a general sense, liquidated damages are any loss that an owner incurs as a result of contractor noncompliance with the contract requirements. Weed (1996), in his published paper, stated that :

> To be legally defensible, liquidated damages must be reasonably commensurate with the amount of loss actually incurred by the owner (65).

PRS are the only specifications which use this legal principle by relating quality characteristics to performance, and by adjusting payments on the basis of life-cycle costs. In this manner, owners are assessing liquidated damages through the use of pay adjustments that are legally defensible. If a contractor's work fails to meet the acceptable quality requirement but the quality exceeds the rejectable quality level, then the owner is legally entitled to assess liquidated damages through a pay adjustment. This pay adjustment reflects the actual losses the owner expects as a result of the contractor's noncompliance with the contract requirements.

6.2 Penalty scale development methodologies

The literature reported different approaches for penalty scale assessment. The most common in practice are based on pavement performance or serviceability, cost of production, operating characteristic curve, and on the cost of quality control.

6.2.1 Cost of production

The cost of production approach is limited in use to only a few quality measures. In this approach, the payment reduction should be greater than the reduction in the cost that results when lower quality materials are being produced. For instance, if the design thickness of a bituminous concrete surfacing is fixed, and the constructed thickness has an average value lower than the target value, then the cost of production is assumed to be the ratio of the as-constructed to the target thickness. In this case the minimum payment reduction to the contractor should therefore be reduced by the product of this ratio by the cost of the material.

6.2.2 Operating characteristic curve

The operating characteristic (OC) curve approach can be used to develop the entire acceptance plan or the price-adjustment portion of the plan only. If it is used in the development of the entire acceptance plan, two points must be defined on the OC curve graph. In other words, the agency must establish the probability for accepting (or rejecting) two different levels of quality for material. The two quality levels should preferably not be spaced too close to each other. In defining the two points, it is easiest for the agency to think in terms of the probability desired for rejecting material that has been designated as good and the probability desired for accepting material that has been designated as poor. Only one OC curve can pass through the two points: the curve that identifies the sample size and the acceptance limits. Once the sample size and the acceptance limits have been established, the agency can use OC curves and curves of expected payment to develop a reasonable schedule. A trial schedule should be devised. In the first step, the price reductions can either be designed to increase sharply with decreasing quality or they can be designed to increase linearly with decreasing quality. Next OC curves (for the various levels of payment) and curves of expected payment can be drawn. If both the agency and the contractors are pleased with the curves, then the schedule can be incorporated into the construction specifications. If not, then either the price-adjustment schedule or the acceptance plan must be modified by means of the changes to the sample size, loosening or tightening the acceptance or specifications limits, increasing or decreasing payments for a given quality level, and increasing or decreasing the number of payment levels. After the appropriate modification has been made, new OC curves and a new curve of expected payment are drawn and the process is repeated until both contractual parties are satisfied.

6.2.3 Cost of quality control

A price-adjustment system based on the contractor's cost of quality control would logically relate the reduction in payment for inferior material to the contractor's reduced spending on quality control. Inherent in the development of such a system is the assumption that there is a direct relation between the contractor's cost of quality control and the quality of the resulting construction. This approach can be fully implemented only if data necessary to determine the relation that existed between what a contractor spent on quality control of a project and the resulting quality of that project are gathered. This relation seemed to be obscured by variable project conditions such as the weather and the distance from the plant to the project which influence quality but cannot readily be associated with the cost of quality control. Nonetheless, this approach has a certain intuitive appeal and may be found to be workable as more cost data become available.

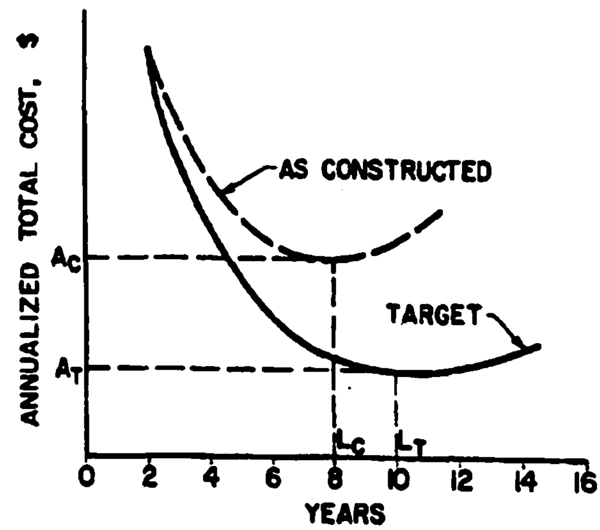
6.2.4 Pavement performance or serviceability approach

The most recent advances in PRS development use the principles of pavement performance. This concept is based on comparing the predicted performance of the as-constructed pavement to the predicted performance of the target pavement. In this concept, pavement performance is quantified in terms of the performance period costs associated with a certain pavement history. In order to develop a rational approach for the economic evaluation, the NCHRP team has developed a procedure based on the economic life of the as-constructed pavement, as compared to the economic life of the target (as-designed) pavement. The term economic life refers to the time in an analysis period when the equivalent uniform annual total transport cost (EUATTC) is minimum. Figure 6.1 below illustrates this concept where the as-constructed cost curve versus the curve of the target pavement are compared on the basis of the economic life of the as-constructed pavement.

Figure 6.1

Economic life of as-constructed

versus target pavements.



In this Figure, the as-constructed pavement was not built to the design standards, and the nonconformance resulted in a higher EUATTC and a shorter economic life compared to the target pavement. To withhold sufficient fund at the time of construction to cover such costs, adjustment to the contractor's bid price in response to the work that deviates from the quality level anticipated should correspond to the present worth of the cost differential resulting from such deviations. Thus, expressed in a mathematical form, the penalty function to be assessed to the contractor can be given by :

$$\text{PENALTY} = (A_C - A_T)\,\{[(1+i)^{LC} - 1] / [i(1+i)^{LC}]\} \qquad (6.1)$$

where

$(A_C - A_T)$ is the equivalent uniform annual total transport cost difference of as-constructed pavement as compared to standards; and $[(1 + i)^{LC} - 1] / [i(1 + i)^{LC}]$ is the present worth factor based on the economic life of as-constructed pavements;

PENALTY	=	amount of fund to be held by the contracting road agency at the time of construction
A_C	=	equivalent uniform annual total transport cost at the end of economic life for as-constructed pavement;
A_T	=	equivalent uniform annual total transport cost at the end of economic life for as-designed pavement;
L_C	=	economic life of as-constructed pavement, in years;
L_T	=	economic life of as-designed pavement.
i	=	discount rate, in percent.

The penalty factor (PF) expressed in percentage of initial contract bid price is derived from the penalty formula (equation 6.1) and the initial contract bid price as follows:

$$PF = [(BID - PENALTY) / BID] * 100 \tag{6.2}$$

where

PF = penalty factor, expressed in percent of initial contract bid price;
BID = contract bid price based on as-designed pavement and;
(BID - PENALTY) is the payment to the contractor.

NCHRP team reported that this penalty function development procedure is based on several basic criteria:

1. The contractor with the lowest bid price is awarded the construction contract. This bid price for the asphalt concrete paving and/or granular materials is then used in the economic life calculations.

2. The target design, which is used for bidding purposes, is specified by the agency; if the contractor meets this target, payment will be at exactly the bid price.

3. If the contractor is either above or below the target, this will affect the predicted pavement performance, and a cost evaluation will be made of the as-constructed pavement using the total of construction bid costs, agency maintenance costs, and user costs.

4. The contractor's bid price payment will be adjusted by the amount of cost difference over the period defined as the economic life of the as-constructed pavement; this adjustment could be either positive (bonus) or negative (penalty).

5. The adjustment to the contractor will likely have some upper limit, so that extreme bonuses and penalties will not occur. Provisions will also be made such that the entire item may be rejected, and, in this instance, no payment will be made.

6.3 Sensitivity of HDM-III performance and economic models

For the development of pay functions for performance related specifications (PRS) and end result specifications (ERS), it is recommended that a sensitivity analysis should be conducted for all the models to be used to evaluate the sensitivity of the performance prediction to various pavement design factors and to evaluate the effects of these pavement design factors and their interactions on the predicted pavement performance. To evaluate HDM-III performance and economic models, the procedure used in this study is based on the optimum economic life of as-constructed pavements, as compared to the economic life of as-designed (target) pavements. This concept is illustrated in Figure 6 (33).

HDM-III program has been widely used in many developing countries as a road management decision making tool to simulate road deterioration under alternative maintenance options. It has also been used to find the best investment under various M&R policies subjected to local constraints. The program predicts pavement condition and users associated costs over a defined analysis period.

Two pavement types are selected for this sensitivity study: DBST and AC on granular base pavements designed to carry respectively, a two-way AADT of 500/1000 vpd and a two-way AADT of 2000/4000 vpd. The subgrade CBR for both pavement types is fixed at 10 percent. All HDM-III input parameters are given in Chapters 2, and 5. HDM-III output results in terms of total transport cost (TTC) streams expressed in millions of F CFA and the converted values in EUATTC (in F CFA) for as-constructed pavements are respectively illustrated in Tables 6.1 (a) and (b) below. Corresponding values for target pavements are given in Chapter 5, Tables 5.4, 5.5 (a), and 5.5 (c).

Table 6.1 (a)

T.T.C streams for a two-way AADT of
500/1 000 vpd and 2 000/4 000 vpd
and a CBR of 10 percent
(*1 000 000)

Year	T1 - CBR2		T3 - CBR2	
	(ΔSN/SN) *100			
	90%	80%	90%	80%
Initial Construction	86.073	86.073	134.754	134.754
1	20.295	20.295	79.474	79.477
2	21.007	21.009	82.328	82.335
3	21.741	21.742	85.265	85.276
4	22.501	22.503	88.309	88.324
5	23.288	23.291	91.465	91.486
6	24.104	24.107	94.741	94.769
7	24.950	24.954	98.145	98.182
8	25.826	25.831	101.686	101.733
9	26.735	26.740	105.371	105.431
10	27.676	27.683	109.207	109.283
11	28.653	28.661	113.203	113.307
12	29.667	29.677	117.375	117.530
13	30.724	30.737	122.277	124.353
14	31.831	31.848	129.207	131.670
15	32.994	33.015	136.594	139.517
16	34.409	34.732	144.466	147.942
17	36.611	37.002	152.834	156.948
18	38.976	39.452	161.794	166.670
19	41.572	42.153	171.533	177.349
20	44.434	45.155	182.214	189.143

Table 6.1 (b)

E.U.A.T.T.C for a two-way AADT of
500/1 000 vpd and 2 000/4 000 vpd
and a CBR of 10 percent

Year	T1 - CBR2		T3 - CBR2	
	(ΔSN/SN) *100			
	90%	80%	90%	80%
8	39 829 194	39 831 334	115 451 062	115 468 664
10	38 300 954	38 303 553	114 451 917	114 475 553
11	37 833 844	37 863 705	<u>114 391 450</u>	<u>114 418 977</u>
12	37 495 436	37 498 592	114 515 079	114 547 888
14	37 086 440	37 090 364	115 237 012	115 415 488
16	36 916 606	<u>36 928 440</u>	116 480 163	116 807 667
17	<u>36 910 354</u>	36 929 945	117 223 844	117 628 807
18	36 947 406	36 975 184	118 023 313	118 508 474
20	37 123 197	37 169 195	119 745 968	120 403 426

Table 6.1 (b) above and Figures 1D and 2D (Appendix D) summarize the results obtained using HDM-III outputs and the engineering economic principles discussed earlier. The respective minimum EUATTC for target pavements for the two-way AADT of 500/1 000 vpd and 2 000/4 000 vpd and a CBR of 10 percent are 36 897 053 F CFA and 114 365 925 F CFA. Their economic lives occur at the years 17, 16, and 11, dependent on the traffic loads and the degree of deficiency. For both traffic volumes, as-constructed pavements need rehabilitation at almost the same year as target pavements except for low volume roads constructed with 20 percent strength deficiency

<u>6.3.1 Lack of QA/QC relative cost increase</u>

The relative cost increase resulting from as-constructed pavements compared to target pavements is based on pavement economic life procedures defined previously. Table 6.2

below shows the results expressed in present worth of any expenses expected to occur in the future as a result of deviations from the specified level of quality prescribed in the specifications. The first column of this Table describes the two traffic levels and the soil CBR selected for the analysis, and column two, the actual structural strength of target and as-constructed pavements. The third column, derived from economic principles, contains the EUATTC at pavement economic life for target and as-constructed pavements, respectively. Column 4 expresses the EUATTC difference resulting from the optimum economic life of the as-constructed pavement as compared to the optimum economic life of the target pavement. Based on the as-constructed pavement economic life, this cost difference is discounted at 12 percent to the present worth (PW). The results are presented in column 5.

Table 6.2

P.W of T.T.C increase of as-constructed pavements as compared to standards

Two-way AADT and Subgrade Strength	Pavement Relative Strength (ΔSN/SN) *100	EUATTC at Pavement Economic Life (F CFA)	EUATTC Difference (F CFA)	PW of TTC difference (F CFA)
AADT of 500/1 000 vpd and a CBR of 10 percent	100% SN	36 897 053	0	0
	90% SN	36 910 354	13 301	4 464 312
	80% SN	36 928 440	31 387	8 226 358
AADT of 2 000/4 000 vpd and a CBR of 10 percent	100% SN	114 365 925	0	0
	90% SN	114 391 450	25 525	1 833 920
	80% SN	114 418 977	53 052	3 811 680

6.3.2 Analysis of results

The present worth of the relative cost increase due to pavement lack of full strength of 10 and 20 percent for both loadings are given in Table 6.2 above. This relative cost increase, if deduced from the original construction contract would not be sufficient to cover future expenses for road repair resulting from work badly performed. The main raison is that HDM-III deterioration models used in these simulations are derived from high standard road networks designed to modern codes. Because of the lack of considering other forms of failure such as shear deformation in the structural layers due to the pavement reduced strength, HDM-III model alone is difficult to use for cases of severe structural deficiencies. Therefore a price adjustment based on the results of this sensitivity study will not persuade the contractor for conformance to the specifications and it would allow him to benefit by producing deficient materials. To be within the main range of HDM-III model applications, as-constructed pavements should be rehabilitated at an appropriate time within the analysis period using any convenient rehabilitation design methodology.

6.4 Lack of QA/QC simulation

Penalties to contractors depend on the quality of material layers and methods of construction. Any deficiencies associated with the selection of prescribed materials or construction methods result in a loss in pavement strength and stiffness. This resulting loss should be offset by adding an additional thickness of material of a higher layer to the as-constructed thickness. Hence, the apparently deficient material or thickness must be strengthen by higher cost material. When compensating for material and construction deficiencies, by using additional materials in the upper layer, the governing criteria is to keep the pavement structural adequacy over its intended performance period.

In the current study, to simulate the lack of QA/QC in pavement design and construction phases, a reduction of 10 percent and 20 percent on the previously calculated target SN obtained from AASHTO design equation were assumed. Reductions in allowable load applications resulting from reduced pavement strength are obtained from the same AASHTO flexible design equation. To withstand the traffic loading for the entire analysis period, as-constructed pavements were rehabilitated using AASHTO overlay design method based on the remaining life approach (37).

6.4.1 AASHTO methods for pavement remaining life assessment

The failure of a pavement can be categorized as structural or functional failure. In the functional failure-based approach, the remaining life is computed on the basis of the performance of the pavement (for example serviceability or rideability) and is expressed in terms of years/80 kN equivalent single axle load applications (ESALs). The reduction of structural capacity, on the other hand, is the primary concern in the structural failure-based approach. Witzcak (1) noted that remaining life estimates based on those two failure criteria will be different. According to AASHTO Guide for Design of Pavement Structures, structural deficiency arises from any conditions that adversely affect the load carrying capability of the pavement structure (37). This include inadequate thickness as well as cracking, distortion and disintegration. It should be noted that several types of distress (example, distresses caused by poor construction techniques) are not initially caused by traffic loads but do become more severe under traffic to the point that they also detract from the load carrying capability of the pavement.

According to AASHTO guide, remaining life can be calculated by a unique curve relating condition factor to remaining life. For flexible pavements, the condition factor (C_x) is defined as the ratio of the actual structural number to the original structural number. To evaluate the condition factor, AASHTO guide recommends three alternative evaluation methods :

1. visual survey and material testing method,
2. nondestructive deflection testing,
3. remaining life method.

The visual survey method is based on the observation of existing pavement conditions. The observation consists of a review of all information available regarding the design, construction and maintenance history of the pavement. This should be followed by a detailed survey to identify the type, amount, severity and location of surface distresses. In addition to surface condition survey, coring, sampling and testing is recommended to verify and identify the causes of the observed surface distress.

By the nondestructive deflection testing (NDT) approach, the pavements in service are evaluated by using falling weight deflectometer (FWD) sensor deflections. For flexible pavements, by making use of the last sensor deflection, the subgrade resilient modulus is calculated first. The subgrade resilient modulus in conjunction with the first sensor deflection is used to estimate the effective modulus (E_p) of the pavement layers above the subgrade. E_p can be obtained from Figure 5.5 of the AASHTO guide or the corresponding equation. Effective structural number (SN_{eff}) can be directly calculated from E_p, using the following equation :

$$SN_{eff} = 0.0045 * D * (E_p)^{1/3} \tag{6.3}$$

where D is the total thickness of the pavement structure above the subgrade.

The remaining life approach to the structural evaluation relies directly to the concepts illustrated in Figure 5.1 of the AASHTO guide. This follows a fatigue damage concept that repeated loads gradually damage the pavement and reduce the number of additional loads the pavement can carry to failure. At any given time, there may no directly observable indication of damage, but there is a reduction in structural capacity in terms of the future load-carrying capacity.

6.4.2 Pavement rehabilitation design methodology

To determine the remaining life of as-constructed pavements, the actual amount of traffic the pavements have carried to date and the total amount of traffic the pavements could be expected to carry to failure (PSI equal to 1.5 to be consistent with AASHO Road Test equations) should be determined. Both traffic volumes are expressed in terms of cumulative 80 kN equivalent single axial load applications (ESAL).

Table 6.3

Trend of cumulated one-way traffic loading

Year	TRAFFIC 1	TRAFFIC 2	TRAFFIC 3
1	99. 3	248. 2	397. 1
2	202	504. 95	807. 9
3	308. 25	770. 55	1 232. 85
4	418. 15	1 045. 3	1 672. 45
5	531. 85	1 329. 55	2 127. 2
6	649. 45	1 623. 6	2 597. 65
7	771. 1	1 927. 75	3 084. 3
8	896. 95	2 242. 4	3 587. 75
9	1 027. 15	2 567. 9	4 108. 55
10	1 161. 85	2 904. 6	4 647. 3
11	1 301. 2	3 252. 95	5 204. 65
12	1 445. 35	3 613. 3	5 781. 2
13	1 594. 45	3 986. 05	6 377. 6
14	1 748. 7	4 371. 65	6 994. 55
15	1 908. 25	4 770. 55	7 632. 8
16	2 073. 3	5 183. 2	8 293. 05
17	2 244. 05	5 610. 1	8 976. 05
18	2 420. 7	6 051. 7	9 682. 6
19	2 603. 45	6 508. 5	10 413. 5
20	2 792. 5	6 981. 05	11 169. 6

The traffic values are converted to time to failure (i.e., time at which an overlay is required). For each traffic volume, Table 6.3 above contains the yearly cumulative one-way traffic loading (expressed in thousands of ESALs) for twenty years analysis period. Pavement remaining life (expressed as a percentage of total traffic to failure) is calculated using the following formula (37):

$$RL = 100 * [1-(N_p/N_{1.5})] \tag{6.4}$$

where :

RL = Pavement remaining life, in percent ;

N_P = total traffic to date, in ESALs ;

$N_{1.5}$ = total traffic to pavement failure ($PSI_f = 1.5$).

Using the RL calculated by the formula above, a condition factor (CF) can be obtained either from Figure 5.2 of the AASHTO guide or using the following formula.

$$CF = 1 - 0.7 * e^{-(RL+0.85)2} \tag{6.5}$$

For flexible pavements the effective structural number is calculated using the following relationship :

$$SN_{eff} = CF * SN_0 \tag{6.6}$$

where

SN_{eff} = as-constructed effective structural number ;

CF = as-constructed pavement condition factor ;

SN_0 = target or as-designed pavement structural number.

Given the effective subgrade resilient modulus, the initial and terminal pavement serviceability index immediately after overlay and at time of the next rehabilitation, the overlay design reliability (R) and the overall standard deviation (S_o) for flexible pavements, the required structural number for future traffic (SN_f) can be computed using AASHTO design equation. In this study the following data were used :

1. subgrade CBR of 6, 10 and 14 percent;
2. initial and terminal PSI fixed at 4 and 2 respectively;
3. overall standard deviation taken as 0.44.

According to AASHTO guide, in order to achieve a certain overall design reliability ($R_{overall}$) in a particular design strategy, the following equation should be applied to establish the individual reliability (R_{stage}) required to design each stage :

$$R_{stage} = (R_{overall})^{1/n} \tag{6.7}$$

In the current study, $R_{overall}$ is fixed at 75 percent for low volume roads and hence R_{stage} is equal to 86.6 percent using the above formula.

The required overly thickness for structural improvement is a function of the structural capacity required to meet future traffic loads and the structural capacity of the existing pavement. The required thickness to increase the structural capacity to carry the future traffic is given by the following equation.

$$D_{ol} = (SN_f - SN_{eff}) / a_{ol} \tag{6.8}$$

where :

D_{ol} = required overlay thickness in inches;

SN_{eff} = as-constructed effective structural number;

a_{ol} = structural coefficient for the asphalt concrete overlay;

SN_f = required pavement structural number to carry the future traffic.

6.4.3 Pavement strengthening and scheduled time of application

Using the above methodology, Table 6.4 below summarizes the required AC overlay thickness and the scheduled time of application for all combinations of traffic loading and subgrade strength. The asphalt concrete overlay strength coefficient (a_{AC}) is taken as 0.40.

Table 6.4

Required overlay thickness and scheduled time of application for all combinations of traffic loading and subgrade strength

(ΔSN/SN)*100	Actual Pavement SN	Required Overlay Thick.(in)	Rehab. Year	Actual Pavement SN	Required Overlay Thick.(in)	Rehab. Year	Actual Pavement SN	Required Overlay Thick.(in)	Rehab. Year
		T1 - CBR1			T1 - CBR2			T1 - CBR3	
100%	3.35	0.00	None	2.79	0.00	None	2.46	0.00	None
90%	3.01	1.37	8	2.51	1.10	8	2.21	1.02	8
80%	2.68	2.45	4	2.23	2.02	4	1.96	1.82	4
		T2 - CBR1			T2 - CBR2			T2 - CBR3	
100%	3.86	0.00	None	3.22	0.00	None	2.85	0.00	None
90%	3.47	1.55	8	2.89	1.27	8	2.56	1.15	8
80%	3.08	2.80	4	2.57	2.32	4	2.28	2.12	4
		T3 - CBR1			T3 - CBR2			T3 - CBR3	
100%	4.13	0.00	None	3.46	0.00	None	3.06	0.00	None
90%	3.71	1.62	8	3.11	1.40	8	2.75	1.25	8
80%	3.30	3.02	4	2.76	2.52	4	2.44	2.32	4

From these calculations, it can be concluded that within the specified range of traffic loading and subgrade strength, the respective scheduled time for overlay applications to restore the specified structural deficiencies resulting from improper initial construction, are the end of the eighth and fourth year of pavement service life.

6.5 Experimental design

To assess penalties to contractors for lack of conformance to design specifications, a full three level factorial design is selected so that all the independent variables and their two-level interactions could be investigated and considered in the analysis. The independent variables considered in the current study are: (1) the traffic loading expressed in equivalent single axial load (ESAL), (2) the subgrade CBR and (3) the relative loss in pavement full strength expressed in percent effective structural number ($\Delta SN/SN$). To withstand the anticipated traffic loading for the entire analysis period, the structural capabilities of as-constructed pavements are restored using the optimum rehabilitation strategy that results from the contractor's poor performance. The dependent variable is the resulting penalty to be assessed to the contractor (PF) expressed in percent of target pavement bid price. The experimental matrix is presented in Chapter 2 and all HDM-III input parameters are also given in Chapters 2, and 5.

6.5.1 HDM-III output results

Pavement performance simulations are based on HDM-III performance and economic prediction models. DARWin outputs (SN) calculated previously for each pavement type, subgrade CBR, and traffic loading and the resulting AC overlay thickness and unit cost serve as direct inputs to HDM-III. HDM-III output results expressed in millions of F CFA are summarized in Tables 6.5 (a), (b) and (c) below. These cost streams include agency and user costs. Agency costs are the initial construction cost, the rehabilitation cost, and the yearly routine maintenance cost. User costs include vehicle operation and user delay costs. Overlay cost was fixed at 127 500 F CFA/m^3.

Table 6.5 (a)

T.T.C streams for two-way AADT of 500/1 000
and all combinations of subgrade strength

Year	T1 - CBR1		T1 - CBR2		T1 - CBR3	
	()SN/SN) *100					
	90 %	80 %	90 %	80 %	90 %	80 %
Initial Construction	100.322	100.322	86.072	86.072	77.665	77.665
1	20.294	20.295	20.295	20.295	20.295	20.295
2	21.007	21.009	21.007	21.009	21.008	21.009
3	21.740	21.742	21.741	21.742	21.741	21.743
4	22.500	81.000	22.501	71.565	22.501	66.281
5	23.288	23.251	23.288	23.251	23.289	23.251
6	24.104	24.064	24.104	24.064	24.105	24.064
7	24.949	24.906	24.950	24.906	24.951	24.907
8	58.659	25.779	52.244	25.779	50.358	25.780
9	26.660	26.684	26.660	26.684	26.660	26.684
10	27.596	27.621	27.596	27.621	27.596	27.622
11	28.566	28.592	28.566	28.593	28.566	28.593
12	29.570	29.599	29.571	29.600	29.571	29.601
13	30.612	30.643	30.612	30.644	30.613	30.645
14	31.691	31.726	31.692	31.727	31.692	31.729
15	32.809	32.850	32.811	32.852	32.811	32.854
16	33.969	34.019	33.970	34.022	33.971	34.024
17	35.171	35.235	35.173	35.239	35.174	35.241
18	36.418	36.514	36.421	36.521	36.422	36.526
19	37.713	37.844	37.717	37.855	37.719	37.862
20	39.060	39.225	39.064	39.240	39.067	39.250

As shown in Table 1F of appendix F, for a twenty years analysis period, the relative total transport cost (TTC) increase for as-constructed pavement as compared to standards is approximately equal to 4 percent for 10 percent structural deficiency and to 11 percent for 20 percent structural deficiency. The agency relative cost increase is much higher and ranges between 12 and 27 percent. Only a slight decrease (less than one percent) in road user costs resulted from the road rehabilitation.

Table 6.5 (b)

T.T.C streams for two-way AADT of 1 250/2 500
and all combinations of subgrade strength

Year	T2 - CBR1		T2 - CBR2		T2 - CBR3	
	()SN/SN) *100					
	90 %	80 %	90 %	80 %	90 %	80 %
Initial Construction	139.102	139.102	122.829	122.829	113.396	113.396
1	49.884	49.886	49.884	49.886	49.885	49.886
2	51.666	51.670	51.667	51.671	51.668	51.672
3	53.500	53.506	53.502	53.507	53.503	53.508
4	55.401	122.586	55.403	110.888	55.405	106.361
5	57.370	57.275	57.374	57.275	57.376	57.276
6	59.414	59.308	59.418	59.309	59.421	59.309
7	61.534	61.415	61.540	61.416	61.544	61.417
8	101.477	63.598	93.937	63.600	91.301	63.601
9	65.798	65.860	65.798	65.863	65.798	65.864
10	68.139	68.204	68.140	68.208	68.140	68.210
11	70.564	70.634	70.566	70.639	70.567	70.641
12	73.078	73.153	73.080	73.159	73.082	73.162
13	75.683	75.765	75.686	75.773	75.689	75.778
14	78.383	78.474	78.388	78.484	78.391	78.493
15	81.182	81.287	81.189	81.301	81.193	81.317
16	84.085	84.213	84.094	84.232	84.100	84.256
17	87.096	87.259	87.109	87.284	87.119	87.318
18	90.223	90.433	90.241	90.466	90.258	90.509
19	93.476	93.743	93.499	93.782	93.525	93.869
20	96.862	97.218	96.893	97.272	96.931	97.322

Using the values incorporated in Table 6.5 (b), Table 2F of appendix F summarizes for a twenty years analysis period, the total transport cost components relative percentage deviation from the standard values. The relative T.T.C increase of as-constructed pavements as compared to standards was found to be approximately equal to

2 percent for 10 percent structural deficiency and to 6 percent for 20 percent structural deficiency. Agency cost increase is relatively more significant and ranges between 10 and 23 percent. A slight decrease (less than -1 percent) in road user costs has resulted from the road rehabilitation.

Table 6.5 (c)

T.T.C streams for two-way AADT of 2 000/4 000
and all combinations of subgrade strength

Year	T3 -CBR1		T3 - CBR2		T3 - CBR3	
	()SN/SN) *100					
	90 %	80 %	90 %	80 %	90 %	80 %
Initial Construction	151.825	151.825	134.754	134.754	124.580	124.580
1	79.473	79.476	79.474	79.477	79.475	79.478
2	82.325	82.333	82.328	82.335	82.330	82.337
3	85.261	85.272	85.265	85.276	85.268	85.278
4	88.303	160.780	88.309	148.708	88.313	143.806
5	91.458	91.300	91.465	91.300	91.471	91.301
6	94.731	94.554	94.741	94.555	94.749	94.556
7	98.128	97.925	98.145	97.928	98.156	97.929
8	140.532	101.419	135.274	101.422	131.515	101.425
9	104.936	105.039	104.937	105.044	104.937	105.048
10	108.683	108.791	108.685	108.798	108.687	108.803
11	112.565	112.680	112.569	112.690	112.572	112.696
12	116.588	116.713	116.594	116.725	116.599	116.733
13	120.758	120.894	120.768	120.911	120.776	120.922
14	125.081	125.233	125.096	125.259	125.107	125.272
15	129.564	129.740	129.586	129.780	129.601	129.797
16	134.213	134.428	134.249	134.487	134.269	134.508
17	139.042	139.310	139.099	139.393	139.126	139.421
18	144.063	144.400	144.148	144.508	144.183	144.545
19	149.289	149.707	149.410	149.886	149.457	149.939
20	154.735	155.279	154.955	155.415	155.016	155.966

Using the values incorporated in Table 6.5 (c), Table 3F of appendix F summarizes for a twenty years analysis period, the total transport cost components relative percentage deviation from the standard values. The relative T.T.C increase of as-constructed pavements as compared to standard was found to be approximately equal to 0.7 percent for 10 percent structural deficiency and to 4 percent for 20 percent structural deficiency. Agency cost increase is relatively more significant and ranges between 10 and 23 percent. With increasing traffic volumes, the decrease in road user costs resulting from the road rehabilitation is more pronounced (up to -1.6 percent).

6.5.2 Pavement economic life

Using the concepts of pavement economic life described earlier and equation 5.2, HDM-III output results for all combinations of traffic loading and soil CBR were converted into equivalent uniform annual total transport cost (E.U.A.T.T.C). Tables 6.6 (a), (b) and (c) below and Figures 1E, 2E ,and 3E (Appendix E) show the results obtained for target and as-constructed pavements for all combinations of independent variables.

Table 6.6 (a)

E.U.A.T.T.C. for two-way AADT of 500/1 000 vpd
and all combinations of subgrade strength

Year	T1 – CBR1		T1 - CBR2		T1 - CBR3	
	()SN/SN) *100					
	90 %	80 %	90 %	80 %	90 %	80 %
8	45 366 513	50 165 840	41 977 053	46 090 361	40 131 801	43 722 304
10	43 159 984	47 382 477	40 179 992	43 799 342	38 557 658	41 717 418
12	41 919 563	45 773 535	39 201 393	42 505 252	37 721 575	40 606 259
14	41 212 826	44 816 720	38 672 583	41 762 402	37 289 650	39 987 788
16	40 823 239	44 250 671	38 409 035	41 347 930	37 094 700	39 661 413
18	40 630 669	43 930 773	38 308 364	41 138 620	37 044 047	39 516 367
20	40 563 517	43 770 849	38 309 659	41 061 228	37 082 615	39 486 954
22	40 577 212	43 715 586		41 069 601		39 532 459
24	40 642 925	43 739 129		41 143 099		39 635 247

Table 6.6 (a) above shows that for 10 percent structural deficiency the economic life of as-constructed pavement occurs at the years 20, and 18 after initial construction. For 20 percent structural deficiency, the economic life of as-constructed pavement occurs at higher economic life for weaker subgrades (year 22, and 20 after road initial construction). At pavement economic life, Table 4F in appendix F shows a significant increase in the total transport cost as compared to that associated with the entire analysis period (5 to 13 percent approximately). This is mainly due to the substantial increase in road user cost (1.65 to 3.31 percent for strong subgrade). Distress combinations at pavement economic life are summarized in Table 7E of appendix E. Pavement rehabilitation should occur at roughness between 3.2 and 3.8 m/km IRI dependent on subgrade strength.

Table 6.6 (b)

E.U.A.T.T.C. for two-way AADT of 1 250/2 500 vpd
and all combinations of subgrade strength

Year	T2 - CBR1		T2 - CBR2		T2 - CBR3	
	()SN/SN) *100					
	90%	80%	90%	80%	90%	80%
8	86 477 977	91 962 754	82 591 304	87 191 106	80 479 718	84 713 640
10	84 113 103	88 942 945	80 696 022	84 748 162	78 839 529	82 570 167
12	83 027 040	87 438 956	79 910 312	83 613 149	78 217 034	81 626 699
14	82 629 747	86 758 560	79 717 257	83 183 726	78 134 995	81 327 735
16	82 625 859	86 555 682	79 858 185	83 158 924	78 354 627	81 395 946
18	82 851 936	86 639 360	80 190 092	83 372 865	78 744 216	81 678 382
20	83 211 521	86 896 600	80 628 785	83 727 570	79 226 382	82 084 990

Table 6.6 (b) above and Figures 4E, 5E and 6E (Appendix E) show that for 10 and 20 percent structural deficiency, the economic lives of as-constructed pavements occur at the same year (year 16 after road initial construction for weak subgrade and year 14 for

strong subgrades). At intermediate subgrade strength, as-constructed pavement economic life occurs at the year 14 and 16 for 10 and 20 percent deficiency respectively. At pavement economic life, Table 5.6 in appendix F shows a significant increase in the total transport cost as compared to that associated with the entire analysis period (3 to 8 percent approximately). This is mainly due to the substantial increase in road user cost (1.83 to 1.82 percent for weak subgrades). Distress combinations at pavement economic life are summarized in Table 7E of appendix E. Pavement rehabilitation should occur at roughness of about 3.0 m/km IRI approximately.

Table 6.6 (c)

E.U.A.T.T.C. for two-way AADT of 2 000/4 000 vpd
and all combinations of subgrade strength

Year	T3 - CBR1		T3 - CBR2		T3 - CBR3	
	()SN/SN) *100					
	90%	80%	90%	80%	90%	80%
8	122 040 380	128 081 295	118 181 862	123 102 201	115 832 121	120 428 350
10	120 187 581	125 511 442	116 795 374	121 134 573	114 729 611	118 784 280
12	119 684 667	124 551 366	116 590 887	120 559 948	114 706 939	118 416 729
14	119 888 370	124 445 952	116 997 878	120 717 152	115 237 845	118 714 982
16	120 476 894	124 818 120	117 731 151	121 276 647	116 059 253	119 374 694
18	121 272 935	125 460 505	118 634 274	122 057 308	117 027 123	120 228 908
20	122 172 836	126 251 243	119 616 746	122 952 848	118 058 460	121 186 716

Table 6.6 (c) above and Figures 7E, 8E and 9E (Appendix E) show that for 10 and 20 percent structural deficiency, the economic lives of as-constructed pavements occur at the same year (year 12 after road initial construction for all subgrade strength). At pavement economic life, Table 6.6 in appendix F shows a significant increase in the total transport cost as compared to that associated with the entire analysis period (2.1 to 6.2 percent approximately). This is mainly due to the substantial increase in road user cost

(up to 2.3 percent for strong subgrades). Distress combinations at pavement economic life are summarized in Table 7E of appendix E. Pavement rehabilitation should occur at roughness between 2.4 and 2.7 m/km IRI dependent on subgrade strength.

6.6 Penalty scale establishment

The penalty factor (to be assessed to the contractor) expressed in percent of initial contract bid price is obtained by using conventional principles of engineering economics. It is the present worth of any expenses expected to occur in the future as a result of deviations from the specified level of quality prescribed in the specifications. The results obtained are summarized in Table 6.7 below. In this Table, Column 3 contains the EUATTC for target (A_T) and as-constructed pavements (A_C) at the end of the economic life. Column 4 is the difference in the EUATTC between target and as-constructed pavements at the end of the economic life (A_T - A_C). Column 5 (penalty) is calculated using equation 6.1 and column 6, which is the penalty factor to be assessed to the contractor, is obtained from equation 6.2.

Table 6.7

Penalty factor assessment

Traffic & CBR Combinations	(ΔSN/SN)*100 (Percent of Target SN)	E.U.A.T.T.C at pavement Economic Life (F CFA)	E.U.A.T.T.C Difference (F CFA)	Penalty (F CFA/Km)	Penalty Factor (PF) %
	100	38 887 847	0	0	100.00
T1-CBR1	90	40 562 275	1 674 428	12 662 030	87.37
	80	43 715 586	4 827 739	36 906 355	63.21
	100	36 897 053	0	0	100.00
T1-CBR2	90	38 299 042	1 401 989	10 326 739	88.00
	80	41 056 428	4 159 375	31 453 208	63.45

Table 6.7 (continued)

Traffic & CBR Combinations	(ΔSN/SN)*100 (Percent of Target SN)	E.U.A.T.T.C at pavement Economic Life (F CFA)	E.U.A.T.T.C Difference (F CFA)	Penalty (F CFA/Km)	Penalty Factor (PF) %
	100	35 705 431	0	0	100.00
T1-CBR3	90	37 044 047	1 338 616	9 704 525	87.50
	80	39 486 954	3 781 523	28 245 873	63.63
	100	80 402 829	0	0	100.00
T2-CBR1	90	82 590 912	2 188 083	14 902 737	89.28
	80	86 555 682	6 152 853	42 909 912	69.15
	100	77 927 567	0	0	100.00
T2-CBR2	90	79 717 257	1 789 690	11 862 367	90.34
	80	83 133 223	5 205 656	35 455 018	71.13
	100	76 463 264	0	0	100.00
T2-CBR3	90	78 126 840	1 663 576	10 686 061	90.57
	80	81 327 447	4 864 183	33 129 292	70.78
	100	117 231 221	0	0	100.00
T3-CBR1	90	119 684 667	2 453 446	15 197 563	89.99
	80	124 420 881	7 189 660	46 183 130	69.58
	100	114 365 925	0	0	100.00
T3-CBR2	90	116 590 753	2 224 828	13 210 360	90.19
	80	120 559 948	6 194 023	38 368 097	71.52
	100	112 637 306	0	0	100.00
T3-CBR3	90	114 625 149	1 987 843	11 803 214	90.52
	80	118 416 729	5 779 423	35 799 909	71.26

6.7 Analysis of variance

The values obtained for the dependent variable (PF) for all combinations of independent factors are shown in Table 6.7 above. These penalties to be assessed to the contractor because of deviations from the target design are expressed in percent of initial contract bid price. Using these values, the analysis of variance (ANOVA) was conducted to investigate the main effects of the traffic loading, the subgrade CBR, and the lack of pavement full strength (ΔSN/SN), together with their two level interaction effects on the penalty factor (PF). The computer program (STATGRAPHICS) ANOVA output including the calculated F ratio is shown in Table 6.8 below.

For this analysis we selected the 5 percent confidence level for testing the significance of the main effects and the two level interactions effects. The results show that the main effects that are significant at the 5 per-cent level are the traffic loading and the relative loss in pavement full strength (ΔSN/SN). Furthermore, all the interactions between these two variables are significant since they have a p value less than 0.05. On the other hand, the subgrade CBR and all its interactions with the other factors were found to be not significant at the 5 percent confidence level and are excluded from the analysis.

Table 6.8

ANOVA table adjusted for significant factors

Source	Sum of Squares	Df	Mean Square	F-Ratio	P-Value
A: Traffic	49.6606	1	49.6606	58.42	0.0000
C: ΔSN/SN	4 552.46	1	4 552.46	5 355.16	0.0000
AA	12.7915	1	12.7915	15.05	0.0009
AC	40.5941	1	40.5941	47.75	0.0000
CC	163.132	1	163.132	191.90	0.0000
Total Error	17.8522	21	0.850106		
Total (corr.)	4 836.49	26			

R^2 = 99.63 percent

R^2 adjusted for DF = 99.54 percent

Standard error of estimate = 0.92

Mean absolute error = 0.64

The R-squared statistic indicates that the model as fitted explains 99.63 percent of the variability in the pay function. The adjusted R-squared statistic, which is more appropriate for comparing models with different numbers of independent variables, is 99.54 percent. The standard error of the estimate shows the standard deviation of the residuals to be 0.92. The mean absolute error of 0.64 is the average value of the residuals. The regression equation that can be fitted to the data to calculate the PF is :

$$PF = -509.615 + 5.51059 * 10^{-6} * TRAFFIC + 11.2826 * (\Delta SN/SN) - 8.32257 * 10^{-14} * TRAFFIC^2 - 0.0439114 * 10^{-6} * TRAFFIC * (\Delta SN/SN) - 0.0521428 * (\Delta SN/SN)^2 \quad (6.10)$$

This Penalty Factor regression equation to be assessed to contractors because of the lack of conformance to target specifications is expressed in percent of as-designed pavement bid price (initial contract bid price). However, the same approach can be used to derive composite or separate equations for each quality characteristic for which data that relate quality to performance is available such as asphalt concrete, base and subbase thicknesses and material properties. Table 8E in appendix E includes:

1. the calculated value of PF,
2. the predicted value of PF using the fitted model, and,
3. the 95 percent confidence limits for the mean response.

Table 6.9 below and Figure 10E in appendix E contain information about values of PF generated using the fitted model.

Table 6.9

Estimation results for PF

(ΔSN/SN) *100 (%)	Penalty Factor Percent Initial Contract Bid Price		
	Traffic 1	Traffic 2	Traffic 3
80	64	69	71
81	66	71	73
82	69	74	75
83	72	76	78
84	74	78	80
85	76	81	82
86	79	83	84
87	81	85	85
88	83	86	87
89	85	88	89
90	87	90	90
91	88	91	91
92	90	93	93
93	92	94	94
94	93	95	95
95	94	96	96
96	95	97	97
97	97	98	97
98	98	99	98
99	98	100	99
100	100	100	100

6.8 Analysis of results

From these results, it can be concluded that:

1. For full-depth granular pavements (DBST surfacing) designed to carry low traffic volumes, one percent deviation from the target pavement full strength results in a higher percentage reduction in payments to the contractor. For 10 percent structural deficiency the contractor will be penalized by 13 percent reduction on the initial contract bid price. For 20 percent deviation, the percentage reduction in payment increases drastically to a value of 37 percent.

2. For asphalt concrete pavements designed to carry high traffic volumes, for up to 10 percent reduction in pavement full strength, the penalties to be assessed to the contractor are directly proportional to the percentage reduction in pavement full strength. For 10 percent structural deficiency, the contractor will be penalized by 10 percent reduction on the initial contract bid price. For 20 percent deviation from target values, the percentage reduction in payments increases at a lower rate than for low volume roads and attains a value of 30 percent.

3. For almost any pavement type and strength, the payment to the contractor drops drastically to 50 percent of the initial contract bid price for about 25 percent decrease in pavement full strength. For 40 percent strength deficiency, no payments should be made.

CONCLUSIONS AND RECOMMANDATIONS

Based on pavement performance and economic models incorporated in World Bank's HDM-III and AASHTO design algorithm, a comprehensive decision making system for assessing penalties to road contractors resulting from work that is inferior to contract specifications was developed. These penalties are intended to prevent wilful or conscious deviations from contract specifications, neglect or lack of QA/QC in pavement construction and rehabilitation. Specifically, the following conclusions are made :

1. The results of these investigations clearly point to the many benefits that might be gained through the use of the World Bank's Highway Design and Maintenance Standards (HDM-III) within the pavement management system, provided that the regional specificity has been evaluated realistically. It also provides a robust methodology for life-cycle cost evaluations of road design, construction and maintenance alternatives, and hence the most promising potential choice as an appropriate decision-making tool in the road transportation sector.

2. The performed HDM-III model-related investigations show clearly that criteria for deciding at which point in time pavement rehabilitation activities should begin can be determined using HDM-III total transport cost stream and pavement economic life concepts. This procedure defines the time, within the analysis period, at which rehabilitation should occur. Distress combination (roughness, cracking, rutting and potholing) at pavement failure can also be obtained.

3. Based on pavement performance and economic models incorporated in World Bank's HDM-III and AASHTO design and rehabilitation algorithms, an appropriate penalty scale establishment procedure was developed. This procedure is developed on the basis that it would be justifiable to deduce a sufficient amount of funds from the contract price at the time of construction to cover future road repairs resulting from a reduced road life.

4. Because the procedure allows the determination of the economic consequences resulting from the lack of QA/QC in pavement construction as compared to standards, agencies may use it to enforce contractor's reliability through price reduction or enhancement. The results derived from this study indicate that the developed procedure is rational and more consistent than the subjective conventional approach based on the skills and the judgement of the engineer.

5. The developed penalty scale procedure is applicable to all variables that describe pavement strength (structural number, asphalt concrete, base and subbase resilient modulus and thicknesses and subgrade California Bearing Ratio) and amenable to control during construction. As developed, this procedure is applicable to any type of pavement construction (flexible, semi-flexible or rigid pavements) provided that the appropriate pavement life-material property relationships are known. It addresses the lack of QA/QC in both the design stage and the construction stage and provides an application of a sound decision-making methodology to establish penalties for contractors.

The development of this procedure has suggested additional areas of research that would improve the flexibility of HDM-III model and the range and accuracy of the predictions obtained. The following recommendations are :

1. HDM-III empirical algorithms were derived from data collected on typical road networks constructed to modern design codes. These prediction equations cannot be used for pavement structural design such as AASHTO design equation and therefore, the hybrid approach (HDM-III / any appropriate design and rehabilitation methodologies) is necessary to conduct realistic pavement performance and economic analysis.

2. Another limitation of HDM-III model is the inflexibility of defining trigger levels for different maintenance intervention. For instance, hot mix asphalt concrete strengthening can be triggered by roughness level or by fixed time intervals only. It would be desirable for purposes of stage construction to schedule overlays by cumulative traffic loading. The model does not provide such an option.

3. The EUATTC represent the net present value of all discounted costs and benefits of an alternative pavement design or construction. It is a particularly useful indicator in pavement economics. HDM-III model provides this output option only for the entire analysis period. It would be desirable for purposes of pavement design optimization and penalty functions development to improve its capability to provide the EUATTC at any year of the analysis period.

4. Each individual parameter is to some extent unreliable. All input data revolve more or less around a single average value, which means that the accuracy of the registered parameter values relies on a single statistical probability. As it is recognized that the overall performance of a pavement is a function, not of just the average value of the

material property, but of the entire distribution of the property, it would be desirable to improve HDM-III models to take account of field construction variability data.

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APPENDIX A

Sensitivity of AASHTO design algorithm

Table 1A

Percentage change in pavement strength per 10 percent change increment in AC strength coefficient for a two-way AADT of 2 000/4 000 vpd and a CBR of 10 percent

m_{ac}	a_{ac}	h_{ac} (in.)	m_{gb}	a_{gb}	h_{gb} (in.)	SN	(ΔSN/SN)*100 (%)
1	0,21	4	0,8	0,14	15,89	2,61	-24,28
1	0,25	4	0,8	0,14	15,89	2,78	-19,43
1	0,29	4	0,8	0,14	15,89	2,95	-14,57
1	0.33	4	0,8	0,14	15,89	3,12	-9,72
1	0,37	4	0,8	0,14	15,89	3,29	-4,86
1	0,42	4	0,8	0,14	15,89	3,46	0
1	0,46	4	0,8	0,14	15,89	3,62	4,84
1	0,50	4	0,8	0,14	15,89	3,79	9,70
1	0,54	4	0,8	0,14	15,89	3,96	14,55
1	0,58	4	0,8	0,14	15,89	4,13	19,41
1	0,63	4	0,8	0,14	15,89	4,29	24,26

Table 2A

Percentage change in pavement strength per 10 percent change increment in AC thickness for a two-way AADT of 2 000/4 000 vpd and a CBR of 10 percent

m_{ac}	a_{ac}	h_{ac} (in.)	m_{gb}	a_{gb}	h_{gb} (in.)	SN	(ΔSN/SN)*100 (%)
1	0,42	2	0,8	0,14	15,89	2,61	-24,28
1	0,42	2,4	0,8	0,14	15,89	2,78	-19.43
1	0,42	2,8	0,8	0,14	15,89	2,95	-14,57
1	0,42	3,2	0,8	0,14	15,89	3,12	-9,72
1	0,42	3,6	0,8	0,14	15,89	3,29	-4,86
1	0,42	4	0,8	0,14	15,89	3,46	0
1	0,42	4,4	0,8	0,14	15,89	3,62	4,84
1	0,42	4,8	0,8	0,14	15,89	3,79	9,70
1	0,42	5,2	0,8	0,14	15,89	3,96	14,55
1	0,42	5,6	0,8	0,14	15,89	4,13	19,41
1	0,42	6	0,8	0,14	15,89	4,29	24,26

Table 3A

Percentage change in pavement strength per 10 percent change increment in GB drainage coefficient for a two-way AADT of 2 000/4 000 vpd and a CBR of 10 percent

m_{ac}	a_{ac}	h_{ac} (in.)	m_{gb}	a_{gb}	h_{gb} (in.)	SN	(ΔSN/SN)*100 (%)
1	0,42	4	0,4	0,14	15,89	2,56	-25,72
1	0,42	4	0,48	0,14	15,89	2,74	-20,58
1	0,42	4	0,56	0,14	15,89	2,92	-15,44
1	0,42	4	0,64	0,14	15,89	3,10	-10,29
1	0,42	4	0,72	0,14	15,89	3,28	-5,15
1	0,42	4	0,8	0,14	15,89	3,46	0
1	0,42	4	0,88	0,14	15,89	3,63	5,13
1	0,42	4	0,96	0,14	15,89	3,81	10,27
1	0,42	4	1,04	0,14	15,89	3,99	15,42
1	0,42	4	1,12	0,14	15,89	4,17	20,56
1	0,42	4	1,2	0,14	15,89	4,34	25,70

Table 4A

Percentage change in pavement strength per 10 percent change increment in GB strength coefficient for a two-way AADT of 2 000/4 000 vpd and a CBR of 10 percent

m_{ac}	a_{ac}	h_{ac} (in.)	m_{gb}	a_{gb}	h_{gb} (in.)	SN	(ΔSN/SN)*100 (%)
1	0,42	4	0,8	0,07	15,89	2,56	-25,72
1	0,42	4	0,8	0,08	15,89	2,74	-20,58
1	0,42	4	0,8	0,09	15,89	2,92	-15,44
1	0,42	4	0,8	0,11	15,89	3,10	-10,29
1	0,42	4	0,8	0,12	15,89	3,28	-5,15
1	0,42	4	0,8	0,14	15,89	3,46	0
1	0,42	4	0,8	0,15	15,89	3,63	5,13
1	0,42	4	0,8	0,16	15,89	3,81	10,27
1	0,42	4	0,8	0,18	15,89	3,99	15,42
1	0,42	4	0,8	0,19	15,89	4,17	20,56
1	0,42	4	0,8	0,21	15,89	4,34	25,70

Table 5A

Percentage change in pavement strength per 10 percent change increment in GB thickness for a two-way AADT of 2 000/4 000 vpd and a CBR of 10 percent

m_{ac}	a_{ac}	h_{ac} (in.)	m_{gb}	a_{gb}	h_{gb} (in.)	SN	(ΔSN/SN)*100 (%)
1	0,42	4	0,8	0,14	7,94	2,56	-25,72
1	0,42	4	0,8	0,14	9,53	2,74	-20,58
1	0,42	4	0,8	0,14	11,12	2,92	-15,44
1	0,42	4	0,8	0,14	12,71	3,10	-10,29
1	0,42	4	0,8	0,14	14,30	3,28	-5,15
1	0,42	4	0,8	0,14	15,89	3,46	0
1	0,42	4	0,8	0,14	17,47	3,63	5,13
1	0,42	4	0,8	0,14	19,06	3,81	10,27
1	0,42	4	0,8	0,14	20,65	3,99	15,42
1	0,42	4	0,8	0,14	22,24	4,17	20,56
1	0,42	4	0,8	0,14	23,83	4,34	25,70

Table 6A

Percentage change in pavement strength per 10 percent change increment in all independent variables for a two-way AADT of 2 000/4 000 vpd and a CBR of 10 percent

m_{ac}	a_{ac}	h_{ac} (in.)	m_{gb}	a_{gb}	h_{gb} (in.)	SN	(ΔSN/SN)*100 (%)
1	0,21	2	0,4	0,070	7,94	0,64	-81,43
1	0,25	2,4	0,48	0,084	9,53	0,98	-71,41
1	0,29	2,8	0,56	0,098	11,12	1,43	-58,56
1	0,33	3,2	0,64	0,112	12,71	1,98	-42,58
1	0,37	3,6	0,72	0,126	14,30	2,65	-23,17
1	0,42	4	0,8	0,14	15,89	3,46	0
1	0,46	4,4	0,88	0,154	17,48	4,40	27,21
1	0,50	4,8	0,96	0,168	19,06	5,49	58,80
1	0,54	5,2	1,04	0,182	20,65	6,74	95,06
1	0,58	5,6	1,12	0,196	22,24	8,17	136,30
1	0,63	6	1,2	0,210	23,83	9,78	182,84

Table 7A

Percentage change in pavement strength per 10 percent change increment in GB drainage coefficient for a two-way AADT of 500/1 000 vpd and a CBR of 10 percent

m_{DBST}	a_{DBST}	h_{DBST} (in.)	m_{gb}	a_{gb}	h_{gb} (in.)	SN	$(\Delta SN/SN)*100$
0	0	0	0,4	0,14	24,91	1,39	-50
0	0	0	0,48	0,14	24,91	1,67	-40
0	0	0	0,56	0,14	24,91	1,95	-30
0	0	0	0,64	0,14	24,91	2,23	-20
0	0	0	0,72	0,14	24,91	2,51	-10
0	0	0	0,8	0,14	24,91	2,79	0
0	0	0	0,88	0,14	24,91	3,07	10
0	0	0	0,96	0,14	24,91	3,34	20
0	0	0	1,04	0,14	24,91	3,62	30
0	0	0	1,12	0,14	24,91	3,90	40
0	0	0	1,2	0,14	24,91	4,18	50

Table 8A

Percentage change in pavement strength per 10 percent change increment in GB strength coefficient for a two-way AADT of 500/1 000 vpd and a CBR of 10 percent

m_{DBST}	a_{DBST}	h_{DBST} (in.)	m_{gb}	a_{gb}	h_{gb} (in.)	SN	$(\Delta SN/SN)*100$
0	0	0	0,8	0,07	24,91	1,39	-50
0	0	0	0,8	0,084	24,91	1,67	-40
0	0	0	0,8	0,098	24,91	1,95	-30
0	0	0	0,8	0,112	24,91	2,23	-20
0	0	0	0,8	0,126	24,91	2,51	-10
0	0	0	0,8	0,14	24,91	2,79	0
0	0	0	0,8	0,154	24,91	3,07	10
0	0	0	0,8	0,168	24,91	3,34	20
0	0	0	0,8	0,182	24,91	3,62	30
0	0	0	0,8	0,196	24,91	3,90	40
0	0	0	0,8	0,21	24,91	4,18	50

Table 9A

Percentage change in pavement strength per 10 percent change increment in GB thickness for a two-way AADT of 500/1 000 vpd and a CBR of 10 percent

m_{DBST}	a_{DBST}	h_{DBST} (in.)	m_{gb}	a_{gb}	h_{gb} (in.)	SN	(ΔSN/SN)*100
0	0	0	0,8	0,14	12,455	1,39	-50
0	0	0	0,8	0,14	14,946	1,67	-40
0	0	0	0,8	0,14	17,437	1,95	-30
0	0	0	0,8	0,14	19,928	2,23	-20
0	0	0	0,8	0,14	22,419	2,51	-10
0	0	0	0,8	0,14	24,91	2,79	0
0	0	0	0,8	0,14	27,401	3,07	10
0	0	0	0,8	0,14	29,892	3,34	20
0	0	0	0,8	0,14	32,383	3,62	30
0	0	0	0,8	0,14	34,874	3,90	40
0	0	0	0,8	0,14	37,365	4,18	50

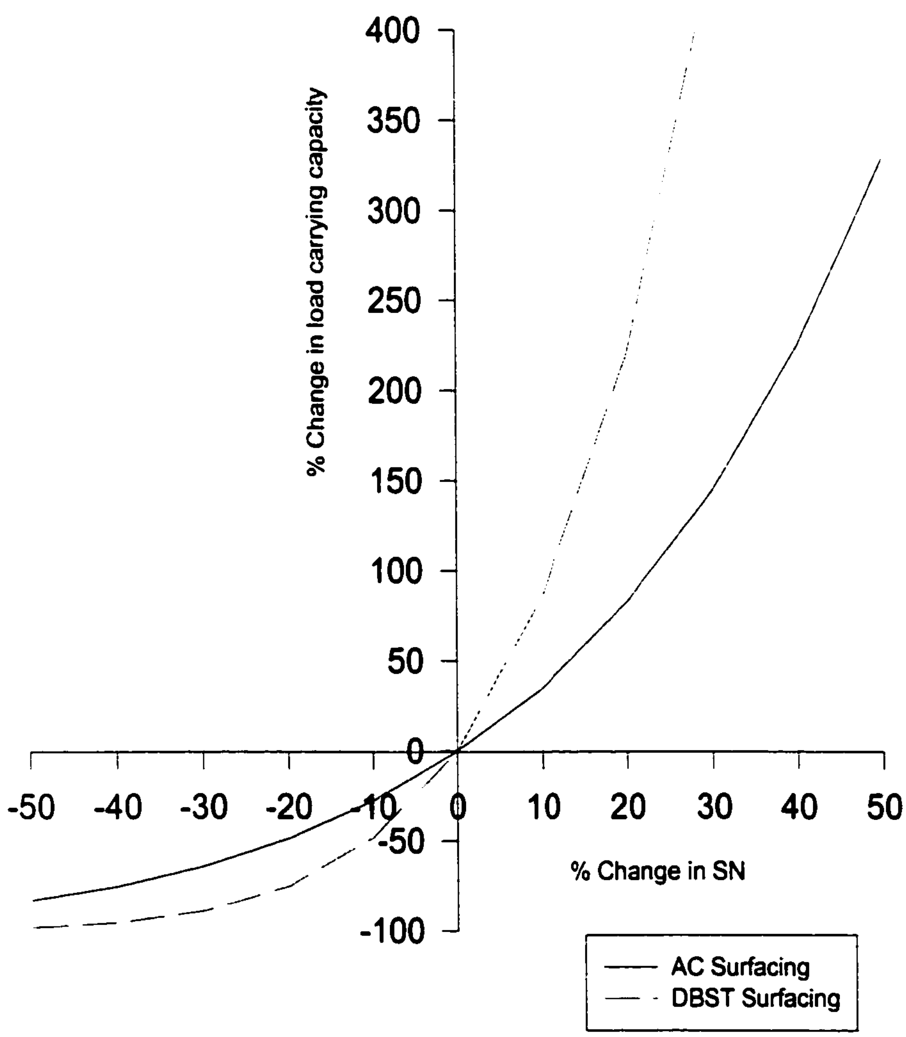
Table 10A

Percentage change in load carrying capacity per 10 percent change increment in pavement strength for two-way AADT of 500/1 000 vpd and 2 000/4 000 vpd and a CBR of 10 percent

ΔSN/SN	AC Surfacing		DBST Surfacing	
SN	W	(ΔW/W)*100	W	(ΔW/W)*100
50%	1897171	-83,01	44987	-98,38
60%	2756687	-75,31	124638	-95,53
70%	4017196	-64,03	308826	-88,94
80%	5745035	-48,56	696516	-75,05
90%	8080706	-27,65	1449990	-48,07
100%	11169600	0	2792500	0
110%	15054603	34,78	5180669	85,52
120%	20386939	82,52	8908037	218,99
130%	27335488	144,73	15054603	439,10
140%	36331411	225,27	24673470	783,56
150%	47908702	328,92	39441648	1312,41

Figure 1A

Sensitivity analysis of pavement load carrying capacity to changes in pavement strength for two-way AADT of 500/1 000 vpd and 2 000/4 000 vpd and a CBR of 10 percent



APPENDIX B

HDM-III deterioration prediction models

Figure 1B

Percent all cracking for target versus as-constructed pavements for 500/1000vpd and a CBR of 10 percent

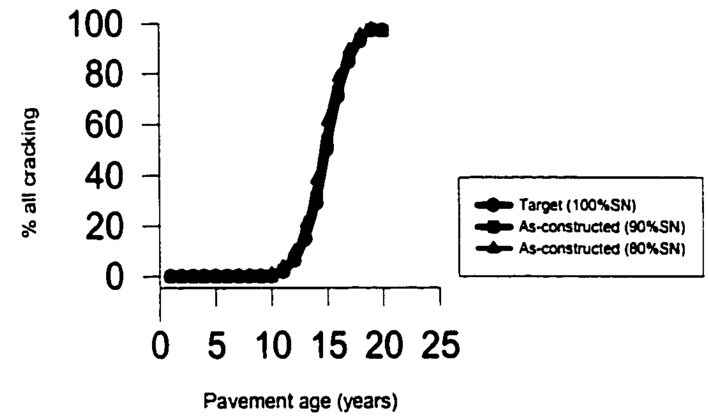


Figure 2B

Percent wide cracking for target versus as-constructed pavements for 500/1000vpd and a CBR of 10 percent

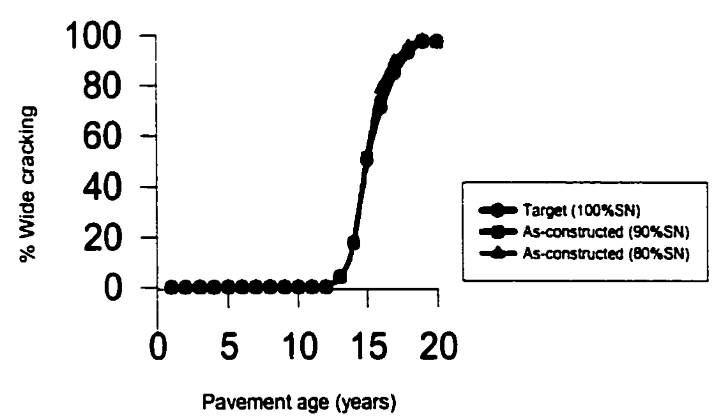


Figure 3B

Percent all cracking progression for target versus as-constructed pavements for 2000/4000vpd and a CBR of 10 percent

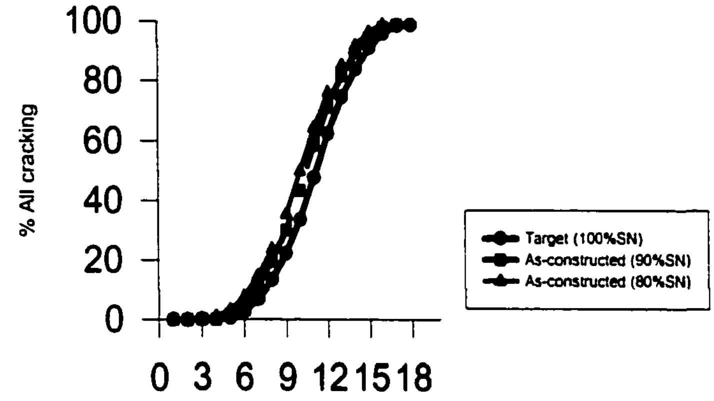


Figure 4B

Percent wide cracking for target versus as-constructed pavements for 2000/4000vpd and a CBR of 10 percent

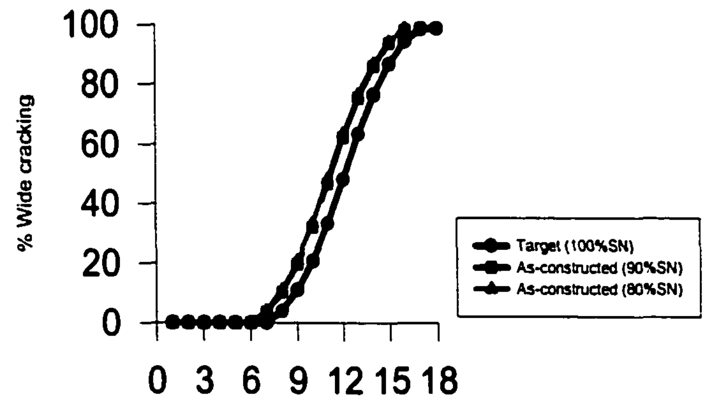


Figure 5B

Rutting progression for target versus as-constructed pavements for 500/1000vpd and a CBR of 10 percent

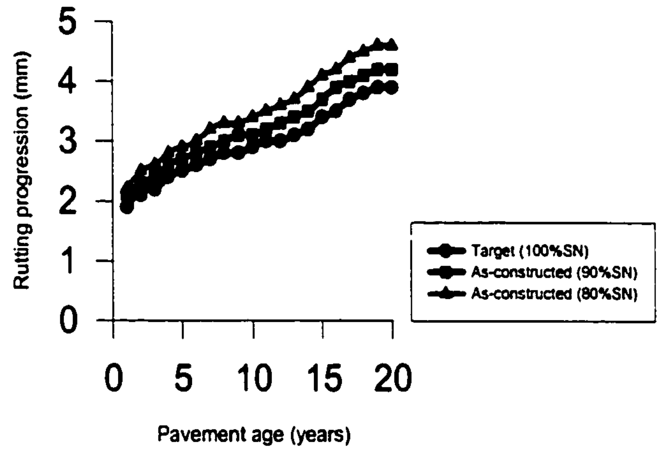


Figure 6B

Rutting progression for target versus as-constructed pavements for 2000/4000vpd and a CBR of 10 percent

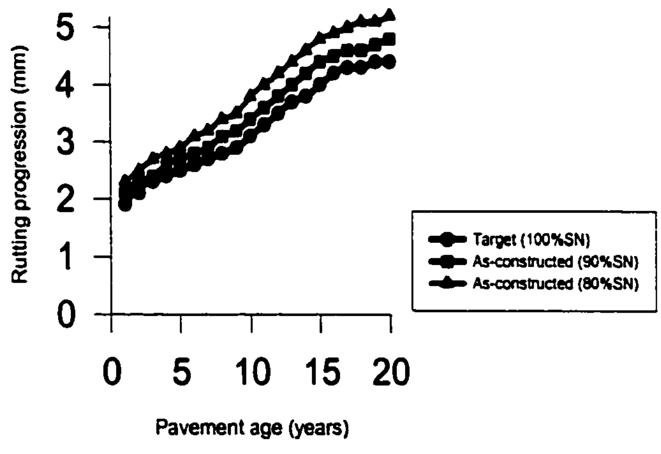


Figure 7B

Percent area of potholes for target versus as-constructed pavements for 500/1000vpd and a CBR of 10 percent

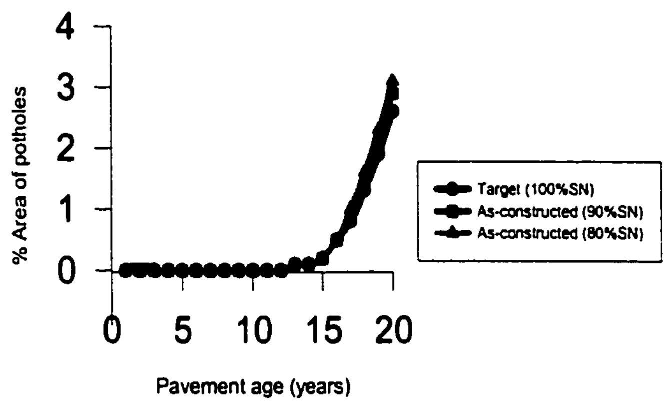


Figure 8B

Percent area of potholes for target versus as-constracted pavements for 2000/4000vpd and a CBR of 10 percent

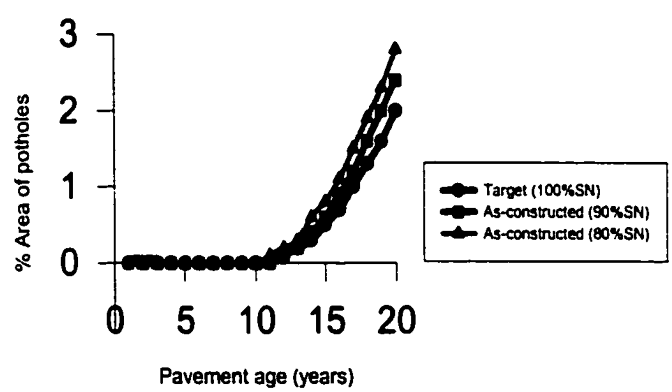


Figure 9B

Percent ravelling area for target versus as-constructed pavements for 500/1000vpd and a CBR of 10 percent

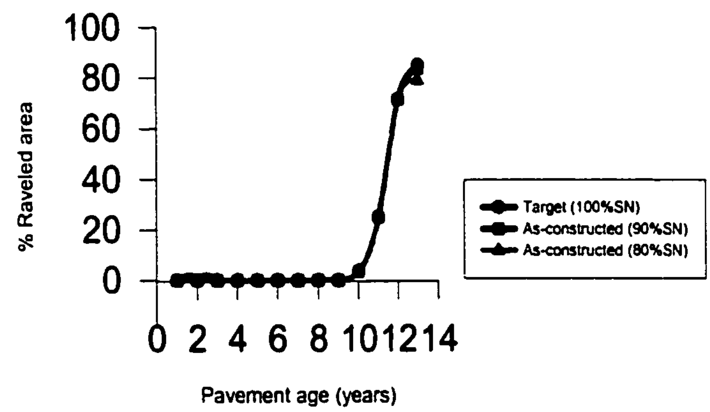


Figure 10B

Roughness progression for target versus as constructed pavements for 500/1000vpd and a CBR of 10 percent

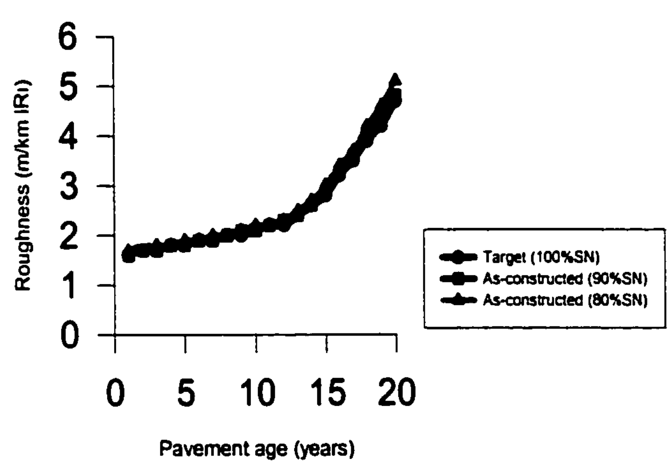
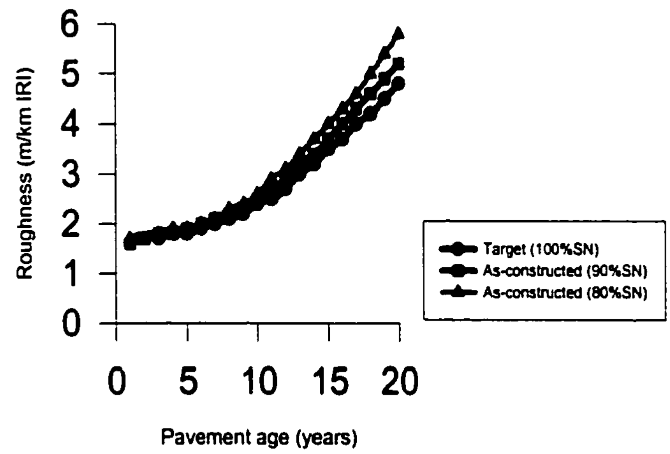


Figure 11B

Roughness progression for target versus as-constructed pavements for 2000/4000vpd and a CBR of 10 percent



APPENDIX C

Target pavements economic lives

Figure 1C
Pavement economic life for
500/1000vpd and CBR of 6 percent

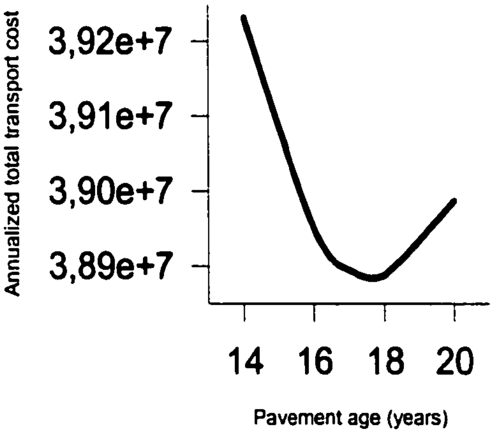


Figure 2C
Pavement economic life for
500/1000vpd and a CBR of 10 percent

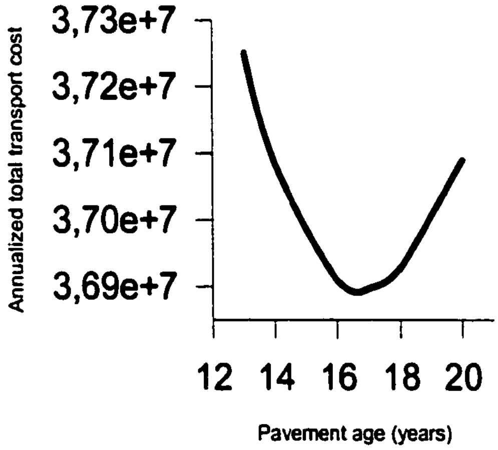


Figure 3C
Pavement economic life for
500/1000vpd and a CBR of 14 percent

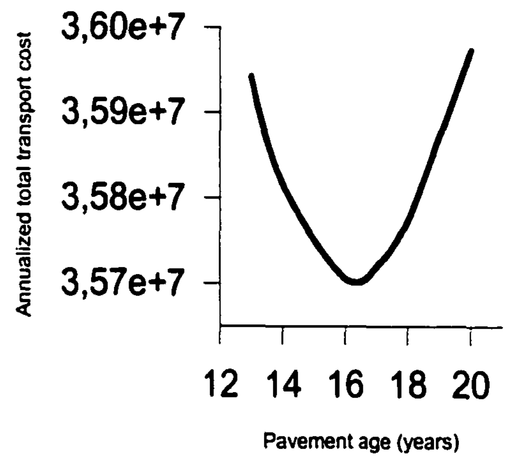


Figure 4C
Pavement economic life for
1250/2500vpd and a CBR of 6%

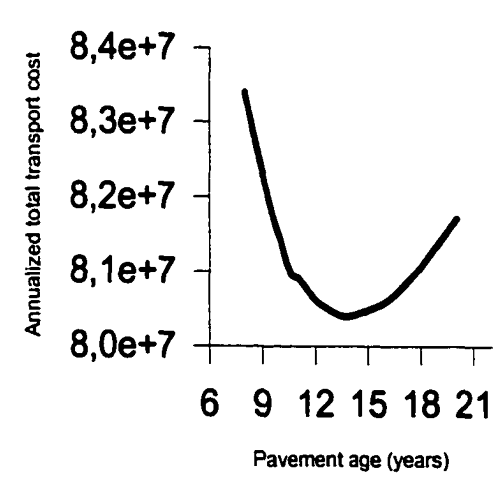


Figure 5C
Pavement economic life for
1250/2500vpd and a CBR of 10 percent

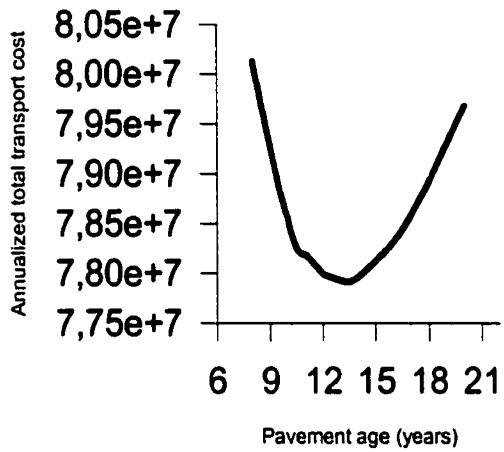


Figure 6C
Pavement economic life for
1250/2500vpd and a CBR of 14 percent

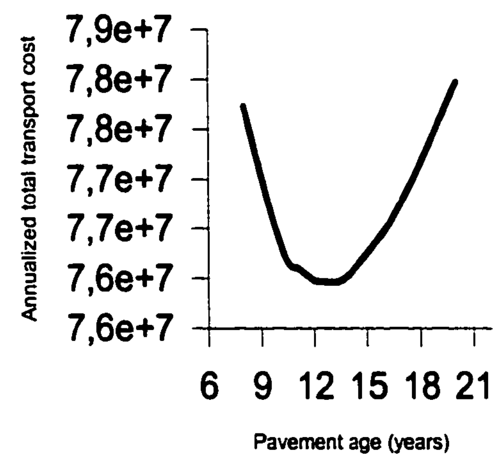


Figure 7C
Pavement economic life for
2000/4000vpd and a CBR of 6 percent

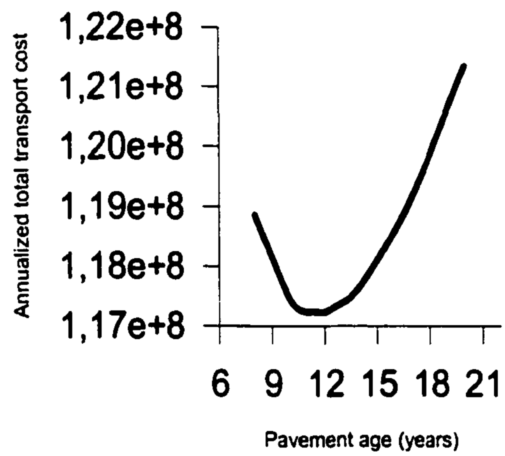


Figure 8C
Pavement economic life for
2000/4000vpd and a CBR of 10 percent

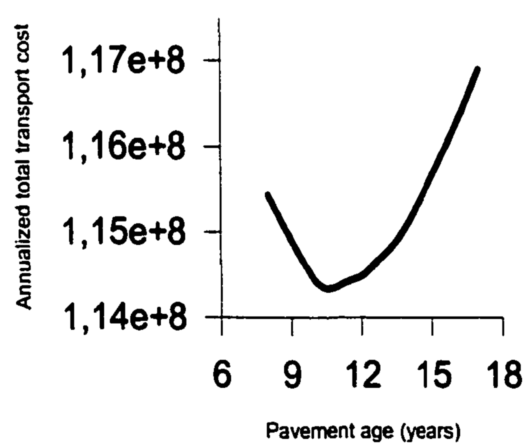


Figure 9C
Pavement economic life for
2000/4000vpd and a CBR of 14 percent

APPENDIX D

Sensitivity of HDM-III performance models

Figure 1D

Economic life of target versus as-constructed pavements for 500/1000vpd and a CBR of 10 percent

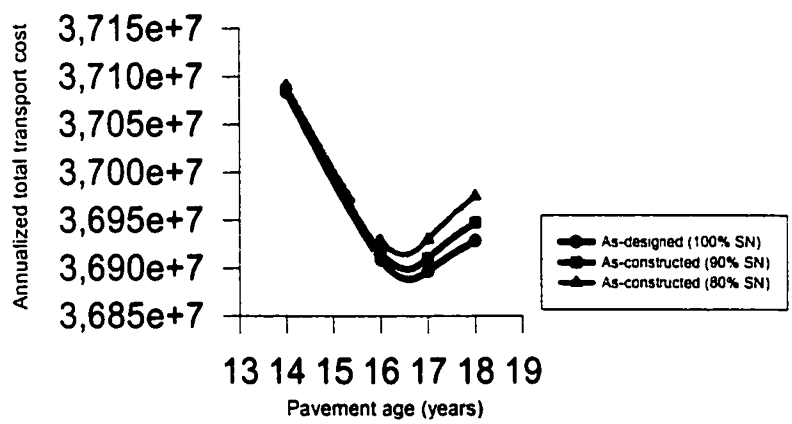
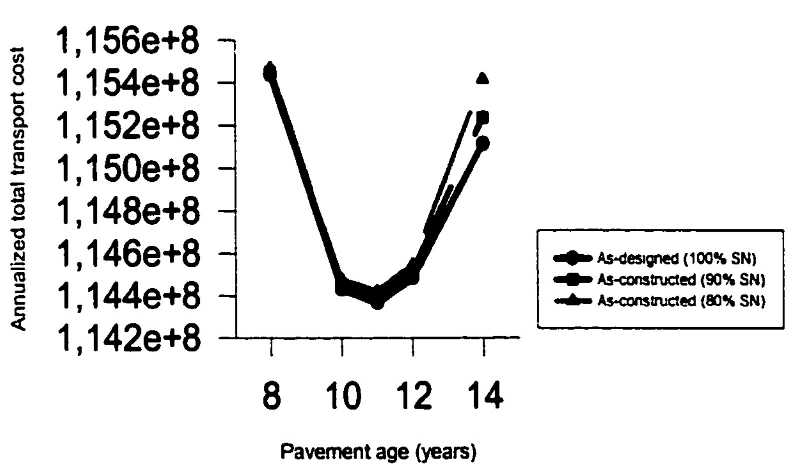


Figure 2D

Economic life of target versus as-constructed pavements for 2000/4000vpd and a CBR of 10 percent



APPENDIX E

HDM-III simulation results and penalty scale establishment

Table 1E

Target pavements

Agency and road users costs expressed in percent of total society cost

for two-way AADT of 500/1 000vpd and CBR of 6, 10 and 14 percent for

20 years analysis period

EQUIVALENT UNIFORM ANNUAL COSTS (EUAC)	T1 - CBR1		T1 - CBR2		T1 - CBR3	
	EAUVC	% of TSC	EAUVC	% of TSC	EAUVC	% of TSC
TOTAL SOCIETY COST (TSC)	38,987	100	37,09	100	35,975	100
AGENCY	13,999	35,90	12,091	32,60	10,966	30,48
1. Capital	13,431	34,45	11,523	31,06	10,398	28,90
2. Recurrent	0,568	1,45	0,568	1,53	0,568	1,58
ROAD USERS	24,988	64,09	24,999	67,40	25,009	69,52
1. Vehicle operation	22,547	57,83	22,557	60,81	22,567	62,73
2. Travel time	2,441	6,26	2,442	6,58	2,442	6,79

Table 2E

Target pavements

Agency and road users costs expressed in percent of total society cost

for two-way AADT of 1 250/2 500vpd and CBR of 6, 10 and 14 percent for

20 years analysis period

EQUIVALENT UNIFORM ANNUAL COSTS (EUAC)	T2 - CBR1		T2 - CBR2		T2 - CBR3	
	EAUVC	% of TSC	EAUVC	% of TSC	EAUVC	% of TSC
TOTAL SOCIETY COST (TSC)	81,719	100	79,684	100	78,475	100
AGENCY	19,191	23,48	17,012	21,35	15,749	20,06
1. Capital	18,623	22,79	16,444	20,63	15,181	19,34
2. Recurrent	0,568	0,69	0,568	0,71	0,568	0,72
ROAD USERS	62,528	76,51	62,672	78,65	62,726	79,93
1. Vehicle operation	56,421	69,04	56,561	70,98	56,614	72,14
2. Travel time	6,107	7,47	6,111	7,67	6,112	7,79

Table 3E

Target pavements

Agency and road users costs expressed in percent of total society cost for two-way AADT of 2 000/4 000vpd and CBR of 6, 10 and 14 percent for 20 years analysis period

EQUIVALENT UNIFORM ANNUAL COSTS (EUAC)	T3 - CBR1		T3 - CBR2		T3 - CBR3	
	EAUVC	% of TSC	EAUVC	% of TSC	EAUVC	% of TSC
TOTAL SOCIETY COST (TSC)	121,352	100	119,23	100	118,013	100
AGENCY	20,894	17,21	18,609	15,61	17,247	14,61
1. Capital	20,326	16,75	18,041	15,13	16,679	14,13
2. Recurrent	0,568	0,47	0,568	0,47	0,568	0,48
ROAD USERS	100,458	82,78	100,621	84,39	100,766	85,38
1. Vehicle operation	90,676	74,72	90,835	76,18	90,976	77,09
2. Travel time	9,782	8,06	9,786	8,21	9,79	8,29

Table 4E

Target pavement

Agency and road users costs expressed in percent of total society cost for two-way AADT of 500/1 000vpd and CBR of 6, 10 and 14 percent at pavement economic life

EQUIVALENT UNIFORM ANNUAL COSTS (EUAC)	T1-CBR1		T1-CBR2		T1-CBR3	
	EAUVC	% TSC	EAUVC	% TSC	EAUVC	% TSC
TOTAL SOCIETY COST (TSC)	38,887	100	36,897	100	35,705	100
AGENCY	14,406	37,04	12,657	34,30	11,704	32,78
1. Capital	13,838	35,58	12,089	32,76	11,136	31,18
2. Recurrent	0,568	1,46	0,568	1,53	0,568	1,59
ROAD USERS	24,481	62,95	24,24	65,69	24,001	67,22
1. Vehicle operation	22,083	56,78	21,864	59,25	21,648	60,63
2. Travel time	2,398	6,16	2,376	6,43	2,353	6,59

Table 5E

Target pavements

Agency and road users costs expressed in percent of total society cost for two-way AADT of 1 250/2 500vpd and CBR of 6, 10 and 14 percent at pavement economic life

EQUIVALENT UNIFORM ANNUAL COSTS (EUAC)	T2-CBR1		T2-CBR2		T2-CBR3	
	EAUVC	% TSC	EAUVC	% TSC	EAUVC	% TSC
TOTAL SOCIETY COST (TSC)	80,402	100	77,928	100	76,463	100
AGENCY	21,554	26,80	19,69	25,26	18,221	23,83
1. Capital	20,986	26,10	19,122	24,53	17,653	23,08
2. Recurrent	0,568	0,70	0,568	0,72	0,568	0,74
ROAD USERS	58,848	73,19	58,238	74,73	58,242	76,17
1. Vehicle operation	53,084	66,02	52,536	67,41	52,54	68,71
2. Travel time	5,764	7,16	5,702	7,31	5,702	7,45

Table 6E

Target pavements

Agency and road users costs expressed in percent of total society cost for two-way AADT of 2 000/4 000vpd and CBR of 6, 10 and 14 percent at pavement economic life

EQUIVALENT UNIFORM ANNUAL COSTS (EUAC)	T3-CBR1		T3-CBR2		T3-CBR3	
	EAUVC	% TSC	EAUVC	% TSC	EAUVC	% TSC
TOTAL SOCIETY COST (TSC)	117,231	100	114,367	100	112,637	100
AGENCY	25,078	21,39	23,263	20,34	22,617	20,08
1. Capital	24,51	20,90	22,695	19,84	22,049	19,57
2. Recurrent	0,568	0,48	0,568	0,49	0,568	0,50
ROAD USERS	92,153	78,61	91,104	79,66	90,02	79,92
1. Vehicle operation	83,136	70,91	82,194	71,87	81,221	72,10
2. Travel time	9,017	7,69	8,91	7,79	8,799	7,81

Table 7E

Distress combination at pavement economic life

PAVEMENT DISTRESS	T1 - CBR1	T1 - CBR2	T1 - CBR3	T2 - CBR1	T2 - CBR2	T2 - CBR3	T3 - CBR1	T3 - CBR2	T3 - CBR3
	Pavement Distress Combination at Pavement Economic Life								
Percent all cracking	92.2	84.8	71.9	78.6	71.7	73.1	60.2	47.6	37.8
Percent wide cracking	92.2	84.8	71.9	63.6	63.2	63.6	48.2	33.3	20.8
Rutting (mm)	3.7	3.7	3.6	3.5	3.5	3.6	3.3	3.3	3.2
Percent area of Potholes	1.3	0.8	0.5	0.1	0.1	0.1	0.1	0.0	0.0
Percent raveled area	100	100	100	NA	NA	NA	NA	NA	NA
Roughness (m/km. IRI)	3.8	3.5	3.2	3.0	2.9	2.9	2.7	2.5	2.4

Table 8E

Estimation results for PF

Run	Traffic (ESALs)	CBR (%)	ΔSN/SN (%)	Calculated Value of PF (%)	Fitted Value of PF (%)	Lower 95% CL for Mean	Upper 95% CL for Mean
1	2 792 500	6	80	63.21	64.20	63.21	65.20
2	6 981 050	6	80	69.15	69.16	68.34	69.99
3	11 169 600	6	80	69.58	71.20	70.21	72.20
4	2 792 500	10	80	63.45	64.20	63.21	65.20
5	6 981 050	10	80	71.13	69.16	68.34	69.99
6	11 169 600	10	80	71.52	71.20	70.21	72.20
7	2 792 500	14	80	63.63	64.20	86.33	65.20
8	6 981 050	14	80	70.78	69.16	89.45	69.99
9	11 169 600	14	80	71.26	71.20	89.66	72.20
10	2 792 500	6	90	87.37	87.16	86.33	87.98
11	6 981 050	6	90	89.28	90..28	89.45	91.10
12	11 169 600	6	90	89.99	90.48	89.66	91.31
13	2 792 500	10	90	88.00	87.16	86.33	87.98
14	6 981 050	10	90	90.34	90.28	89.45	91.10
15	11 169 600	10	90	90.19	90.48	89.66	91.31
16	2 792 500	14	90	87.50	87.16	86.33	87.98
17	6 981 050	14	90	90.57	90.28	89.45	91.10
18	11 169 600	14	90	90.52	90.48	89.66	91.31
19	2 792 500	6	100	100.0	99.69	98.69	100.68
20	6 981 050	6	100	100.0	100.97	100.14	101.79
21	11 169 600	6	100	100.0	99.33	98.34	100.32
22	2 792 500	10	100	100.0	99.69	98.69	100.68
23	6 981 050	10	100	100.0	100.97	100.14	101.79
24	11 169 600	10	100	100.0	99.33	98.34	100.32
25	2 792 500	14	100	100.0	99.69	98.69	100.68
26	6 981 050	14	100	100.0	100.97	100.14	101.79
27	11 169 600	14	100	100.0	99.33	98.34	100.32

Figure 1E

Economic life of target versus as-constructed
pavements for 500/1000vpd and a CBR of 6 percent

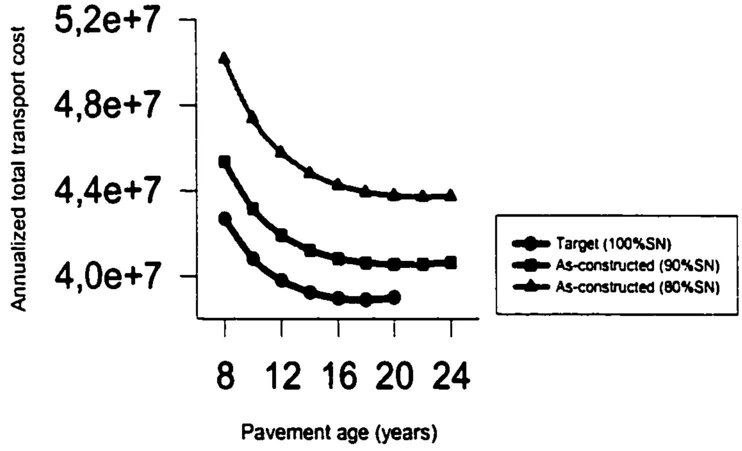


Figure 2E

Economic life of target versus as-constructed
pavements for 500/1000vpd and a CBR of 10 percent

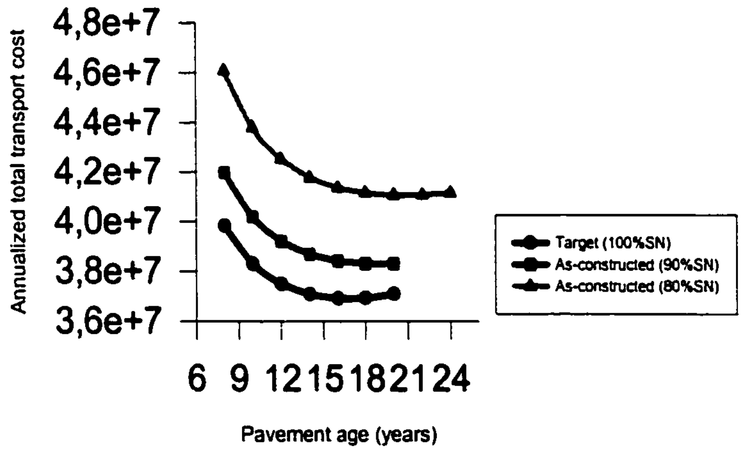
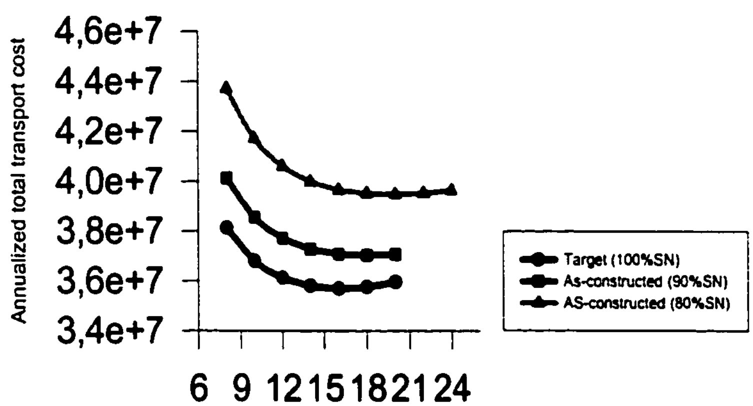


Figure 3E

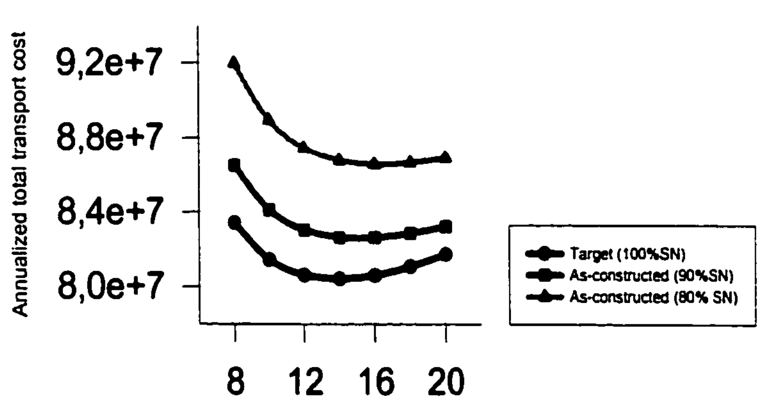
Economic life of target versus as-constructed pavements for 500/1000vpd and a CBR of 14 percent



Pavement age (years)

Figure 4E

Economic life of target versus as-contructed pavements for 1250/2500vpd and a CBR of 6 percent



Pavement age (years)

Figure 5E

Economic life of target versus as-constructed pavements for 1250/2500vpd and a CBR of 10 percent

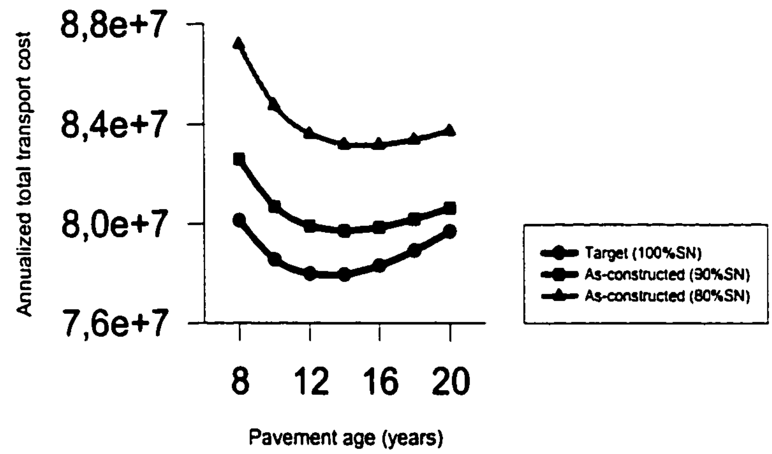


Figure 6E

Economic life of target versusas-constructed pavements for 1250/2500vpd and a CBR of 14 percent

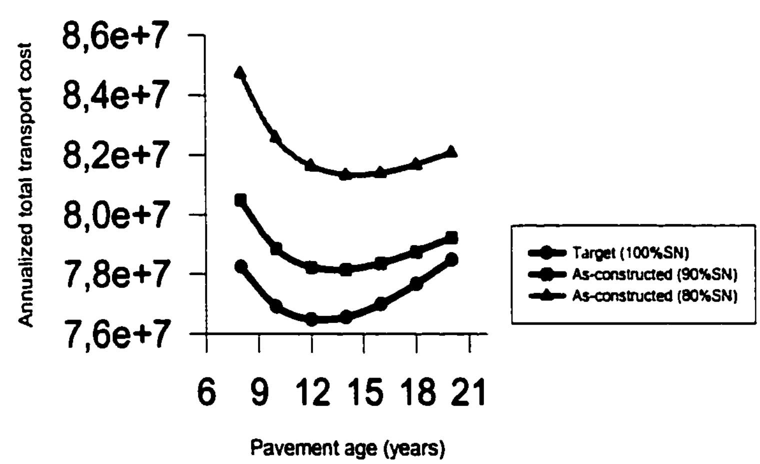


Figure 7E

Economic life of target versus as-constructed pavements for 2000/4000vpd and a CBR of 6 percent

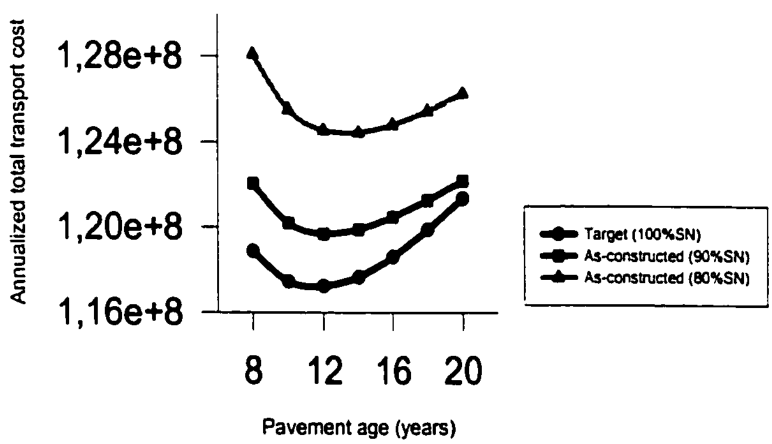


Figure 8E

Economic life of target versus as-constructed pavements for 2000/4000vpd and a CBR of 10 percent

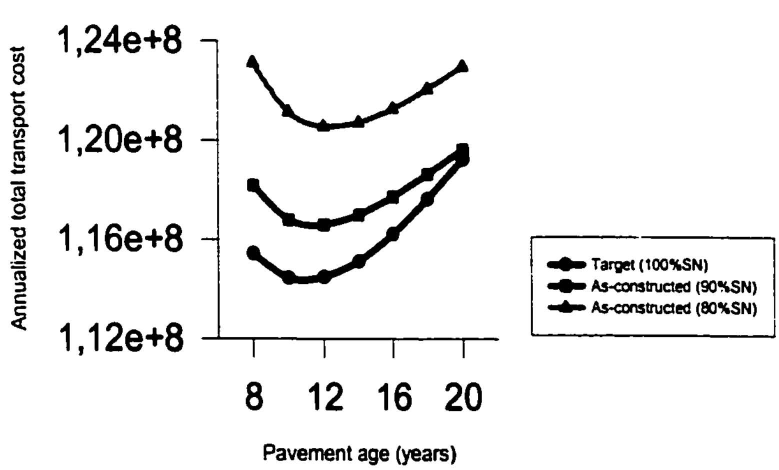
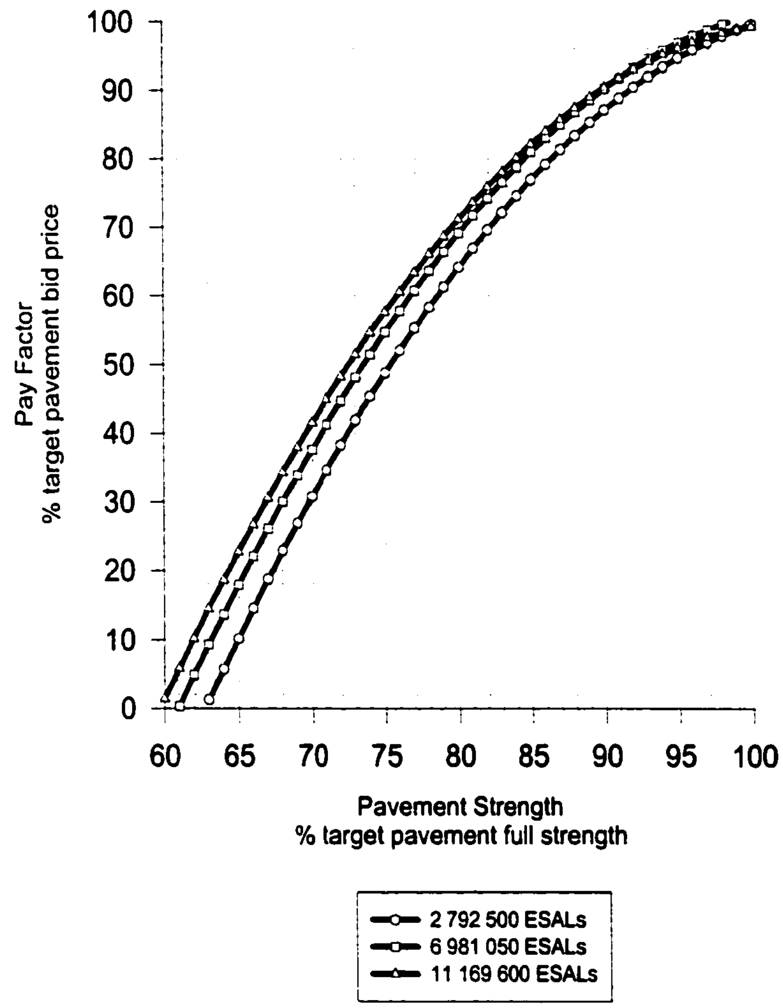


Figure 9E

Economic life of target versus as-constructed pavements for 2000/4000vpd and a CBR of 14 percent

Figure 10E

Pay adjustment charts



APPENDIX F

EUATTC components (agency and user costs)

Table 1F

Rehabilitated as-constructed pavements

Percent change in agency and road users costs as compared to standard

for two-way AADT of 500/1 000vpd and CBR of 6, 10 and 14 percent

for 20 years analysis period

EQUIVALENT ANNUAL VALUE OF COSTS (EUAC)	T1 - CBR1					T1 - CBR2					T1 - CBR3				
	EUAC (ΔSN/SN) *100			% CHANGE		EUAC (ΔSN/SN) *100			% CHANGE		EUAC (ΔSN/SN) *100			% CHANGE	
	100%	90%	80%			100%	90%	80%			100%	90%	80%		
TOTAL SOCIETY COST	38,987	40,563	43,77	3,88	10,92	37,09	38,31	41,061	3,18	9,67	35,975	37,082	39,485	2,98	8,88
AGENCY	13,999	15,774	18,976	11,25	26,22	12,091	13,52	16,266	10,56	25,66	10,966	12,292	14,69	10,78	25,35
1. Capital	13,431	15,206	18,408	11,67	27,03	11,523	12,952	15,698	11,03	26,59	10,398	11,724	14,122	11,31	26,37
2. Recurrent	0,568	0,568	0,568	0	0	0,568	0,568	0,568	0	0	0,568	0,568	0,568	0	0
ROAD USERS	24,988	24,789	24,794	-0,80	-0,78	24,999	24,79	24,795	-0,84	-0,82	25,009	24,79	24,795	-0,88	-0,86
1. Vehicle operation	22,547	22,355	22,359	-0,85	-0,84	22,557	22,356	22,36	-0,89	-0,88	22,567	22,356	22,36	-0,94	-0,92
2. Travel time	2,441	2,434	2,435	-0,28	-0,24	2,442	2,434	2,435	-0,32	-0,28	2,442	2,434	2,435	-0,32	-0,28

Table 2F

Rehabilitated as-constructed pavements

Percent change in agency and road users costs as compared to standard

for two-way AADT of 1 250/2 500vpd and CBR of 6, 10 and 14 percent

for 20 years analysis period

EQUIVALENT ANNUAL VALUE OF COSTS (EUAC)	T2 - CBR1					T2 - CBR2					T2 - CBR3				
	EUAC			%CHANGE		EUAC			%CHANGE		EUAC			%CHANGE	
	(ΔSN/SN) *100					(ΔSN/SN) *100					(ΔSN/SN) *100				
	100%	90%	80%			100%	90%	80%			100%	90%	80%		
TOTAL SOCIETY COST	81,719	83,211	86,895	1,79	5,95	79,684	80,629	83,727	1,17	4,82	78,475	79,227	82,084	0,94	4,39
AGENCY	19,191	21,231	24,906	9,60	22,94	17,012	18,645	21,732	8,75	21,71	15,749	17,239	20,084	8,64	21,58
1. Capital	18,623	20,663	24,338	9,87	23,48	16,444	18,077	21,164	9,03	22,30	15,181	16,671	19,516	8,93	22,21
2. Recurrent	0,568	0,568	0,568	0	0	0,568	0,568	0,568	0	0	0,568	0,568	0,568	0	0
ROAD USERS	62,528	61,98	61,989	-0,88	-0,86	62,672	61,984	61,995	-1,10	-1,09	62,726	61,988	62	-1,19	-1,17
1. Vehicle operation	56,421	55,894	55,902	-0,94	-0,92	56,561	55,898	55,907	-1,18	-1,16	56,614	55,901	55,911	-1,27	-1,25
2. Travel time	6,107	6,086	6,087	-0,34	-0,32	6,111	6,086	6,088	-0,41	-0,37	6,112	6,087	6,089	-0,41	-0,37

Table 3F

Rehabilitated as-constructed pavements

Percent change in agency and road users costs as compared to standard

for two-way AADT of 2 000/4 000vpd and CBR of 6, 10 and 14 percent

for 20 years analysis period

EQUIVALENT ANNUAL VALUE OF COSTS (EUAC)	T3 - CBR1					T3 - CBR2					T3 - CBR3				
	EUAC (ΔSN/SN) *100			%CHANGE		EUAC (ΔSN/SN) *100			%CHANGE		EUAC (ΔSN/SN) *100			%CHANGE	
	100%	90%SN	80%SN			100%	90%SN	80%SN			100%	90%SN	80%SN		
TOTAL SOCIETY COST	121,352	122,173	126,251	0,67	3,88	119,23	119,616	122,951	0,32	3,02	118,013	118,058	121,185	0,03	2,61
AGENCY	20,894	22,996	27,059	9,14	22,78	18,609	20,425	23,746	8,89	21,63	17,247	18,859	21,967	8,54	21,48
1. Capital	20,326	22,428	26,491	9,37	23,27	18,041	19,857	23,178	9,14	22,16	16,679	18,291	21,399	8,81	22,05
2. Recurrent	0,568	0,568	0,568	0	0	0,568	0,568	0,568	0	0	0,568	0,568	0,568	0	0
ROAD USERS	100,458	99,177	99,192	-1,29	-1,27	100,621	99,191	99,205	-1,44	-1,42	100,766	99,199	99,218	-1,57	-1,56
1. Vehicle operation	90,676	89,438	89,451	-1,38	-1,36	90,835	89,451	89,463	-1,54	-1,53	90,976	89,458	89,475	-1,69	-1,67
2. Travel time	9,782	9,739	9,741	-0,44	-0,42	9,786	9,74	9,742	-0,47	-0,45	9,79	9,741	9,743	-0,50	-0,48

Table 4F

Rehabilitated as-constructed pavements

Percent change in agency and road users costs as compared to standard

for two-way AADT of 500/1 000vpd and CBR of 6, 10 and 14 percent

at pavement economic life

EQUIVALENT ANNUAL VALUE OF COSTS (EUAC)	T 1 - CBR 1					T 1 - CBR 2					T 1 - CBR 3				
	EUAC (ΔSN/SN) *100			%CHANGE		EUAC (ΔSN/SN) *100			%CHANGE		EUAC (ΔSN/SN) *100			%CHANGE	
	100%	90%	80%			100%	90%	80%			100%	90%	80%		
TOTAL SOCIETY COST	38,887	40,563	43,715	4,31	12,41	36,897	38,308	41,061	3,82	11,28	35,705	37,045	39,486	3,75	10,59
AGENCY	14,406	15,774	18,555	9,49	28,80	12,657	13,912	16,266	9,91	28,51	11,704	12,648	14,691	8,06	25,52
1. Capital	13,838	15,206	17,986	9,88	29,97	12,089	13,344	15,698	10,38	29,85	11,136	12,08	14,122	8,47	26,81
2. Recurrent	0,568	0,568	0,569	0	0,17	0,568	0,568	0,568	0	0	0,568	0,568	0,569	0	0,17
ROAD USERS	24,481	24,789	25,16	1,25	2,77	24,24	24,396	24,795	0,64	2,28	24,001	24,397	24,795	1,65	3,31
1. Vehicle operation	22,083	22,355	22,687	1,23	2,73	21,864	22,002	22,36	0,63	2,26	21,648	22,003	22,36	1,64	3,29
2. Travel time	2,398	2,434	2,472	1,50	3,08	2,376	2,394	2,435	0,75	2,48	2,353	2,394	2,435	1,74	3,48

Table 5F

Rehabilitated as-constructed pavements

Percent change in agency and road users costs as compared to standard

for two-way AADT of 1 250/2 500vpd and CBR of 6, 10 and 14 percent

at pavement economic life

EQUIVALENT ANNUAL VALUE OF COSTS (EUAC)	T 2 - CBR 1					T 2 - CBR 2					T 2 - CBR 3				
	EUAC			%CHANGE (ΔSN/SN) •100		EUAC			%CHANGE (ΔSN/SN) •100		EUAC			%CHANGE (ΔSN/SN) •100	
	100%	90%	80%			100%	90%	80%			100%	90%	80%		
TOTAL SOCIETY COST	80,402	82,625	86,555	2,76	7,65	77,928	79,717	83,159	2,29	6,71	76,463	78,135	81,327	2,18	6,36
AGENCY	21,554	22,699	26,635	5,31	23,57	19,69	20,939	23,236	6,34	18,00	18,221	19,355	22,561	6,22	23,82
1. Capital	20,986	22,131	26,067	5,45	24,21	19,122	20,371	22,668	6,53	18,54	17,653	18,787	21,993	6,42	24,58
2. Recurrent	0,568	0,568	0,568	0	0	0,568	0,568	0,568	0	0	0,568	0,568	0,568	0	0
ROAD USERS	58,848	59,926	59,92	1,83	1,82	58,238	58,778	59,923	0,92	2,89	58,242	58,78	58,766	0,92	0,90
1. Vehicle operation	53,084	54,051	54,046	1,82	1,81	52,536	53,021	54,048	0,92	2,87	52,54	53,023	53,01	0,92	0,89
2. Travel time	5,764	5,875	5,874	1,92	1,90	5,702	5,757	5,875	0,96	3,03	5,702	5,757	5,756	0,96	0,94

Table 6F

Rehabilitated as-constructed pavements

Percent change in agency and road users costs as compared to standard for two-way AADT of 2 000/4 000vpd and CBR of 6, 10 and 14 percent at pavement economic life

EQUIVALENT ANNUAL VALUE OF COSTS (EUAC)	T3 - CBR1					T3 - CBR2					T3 - CBR3				
	EUAC (ΔSN/SN) *100			%CHANGE		EUAC (ΔSN/SN) *100			%CHANGE		EUAC (ΔSN/SN) *100			%CHANGE	
	100%	90%	80%			100%	90%	80%			100%	90%	80%		
TOTAL SOCIETY COST	117,231	119,684	124,447	2,09	6,15	114,367	116,591	120,56	1,94	5,41	112,637	114,707	118,417	1,83	5,13
AGENCY	25,078	27,612	30,422	10,10	21,31	23,263	24,512	28,517	5,36	22,58	22,617	22,624	26,372	0,03	16,60
1. Capital	24,51	27,044	29,854	10,33	21,80	22,695	23,944	27,949	5,50	23,15	22,049	22,056	25,804	0,03	17,03
2. Recurrent	0,568	0,568	0,568	0	0	0,568	0,568	0,568	0	0	0,568	0,568	0,568	0	0
ROAD USERS	92,153	92,072	94,025	-0,08	2,03	91,104	92,079	92,043	1,07	1,03	90,02	92,083	92,045	2,29	2,25
1. Vehicle operation	83,136	83,062	84,815	-0,08	2,02	82,194	83,069	83,036	1,06	1,02	81,221	83,073	83,038	2,28	2,23
2. Travel time	9,017	9,01	9,21	-0,07	2,14	8,91	9,01	9,007	1,12	1,08	8,799	9,01	9,007	2,39	2,36