AN ECONOMIC CONSIDERATION OF PAVEMENT OVERLAY DESIGN STRATEGIES: A CASE STUDY OF BAUCHI – JOS ROAD

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DEPARTMENT OF CIVIL ENGINEERING

AHMADU BELLO UNIVERSITY, ZARIA

DECEMBER, 2006

DECLARATION

I hereby declare that this work entitled “An Economic Consideration of Pavement Overlay Design Strategies : a case stydy of Bauchi- Jos road” has been performed by me and it is a record of my own work. It has not been presented before in any previous application for a higher degree. Reference made to published literature has been duly acknowledged.

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YARO, Tijjani Ahmad **Date**

CERTIFICATION

This project entitled "**AN ECONOMIC CONSIDERATION OF PAVEMENT OVERLAY DESIGN STRATEGIES: A CASE STUDY OF BAUCHI –JOS ROAD**" by Tijjani Yaro Ahmad meets the regulations governing the award of Master of science degree in Civil (Transportation and Highway) Engineering of Ahmadu Bello University, Zaria and is approved for its contribution to knowledge and literary presentation.

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DEDICATION

This work is dedicated to Yaro Ahmad's family for their wonderful support, understanding and encouragement throughout the duration of the programme.

ACKNOWLEDGEMENTS

To Almighty Allah be the glory for giving me the health, strength and wisdom to undertake the course and finally put together this research work.

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ABSTRACT

Pavement deterioration is revealed through many indications, among which is roughness of its surface which is a collective consequence of the various deterioration factors of the pavement. The economic impact of road deterioration is considerable, and usually outweighs the consideration of riding comfort, and thus providing the strongest objective for evaluating roads improvement policies and corresponding cost implications. This work attempts to address the effect of applying different strategies of overlay as remedies to arrest the escalation of costs incurred by both road agencies and users, and finally, determines the economic optimum strategy that minimize the total system cost. In order to effectively achieve this, six different strategies (the do nothing, 200mm,150mm 100mm,75mm, and 50mm) were investigated to reflect both service performance and economic evaluation. Relevant data for evaluating performance were collected from the pavement evaluation unit (PEU) of the Federal Ministry of Works, Kaduna and data concerning unit costs of road works were determined from the World Bank's "ROCKS" (Road cost knowledge system). The Highway Design and Maintenance Model (HDM III) was used in the analysis of the strategies. The model predicts the pavement deterioration and maintenance effects, and calculates annual costs of road construction, vehicle and operation, needed to perform the economic evaluation of the alteration.

Results obtained show that, for all the strategies considered, road user cost exceeded all other costs and the strategy with the thickest overlay (i.e. 200mm) has the lowest road user cost of ₦33.4b. Also, the rate of deterioration was lowest for the 200mm strategy, and is also associated with the highest user

benefit ₦10.4b. On the other hand, the 75mm thick overlay strategy had the lowest Total Systems Cost (TSC) of ₦35.5b and the highest net present value (NPV) of ₦7.3b. The 75mm design strategy is therefore selected as the optimum strategy.

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CHAPTER ONE

INTRODUCTION

1.1 Background

The failure of a pavement structure is a gradual process since its design

Procedure incorporates, as a major parameter of design, the cumulative axle load applications. As traffic continue to use the facility it's life is being consumed in proportion to the cumulative axle load application during which distress features start to appear and continues to increase in extent and severity until, eventually, failure of the pavement occurs. Two distinct failure types have been identified; functional and structural failures. Structural failure implies collapse of one or more of the pavement layers while functional failure refers to the situation where the pavement can no longer perform its function without causing adverse effects on convenience and losses on its users.

As the road deteriorates, various maintenance measures of moderate cost are employed to enhance its serviceability level and prolong it service life. Such procedures as patching retard structural deformations while non-structural treatments such as surface treatment improve skid resistance, riding quality and impermeability to water. However, despite these measures, a stage is reached when the pavement will require strengthening. This stage, retrospectively defines the service life of the pavement.

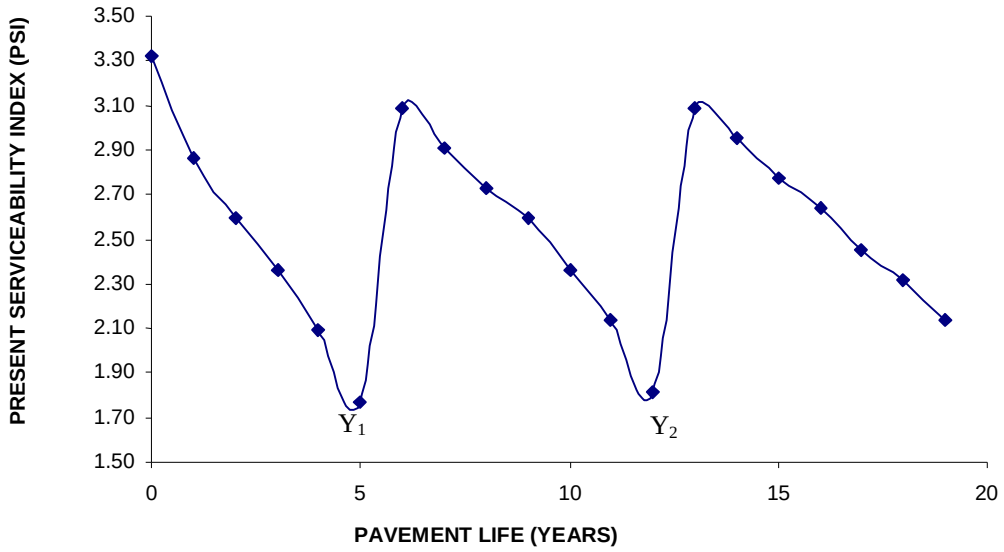


Fig 1.1: DETERIORATION POFILE OF A PAVEMENT

A generalized relationship between serviceability and age of a pavement is illustrated as shown in Fig. 1.1. Starting at year 0, the pavement has initial high serviceability. As traffic is applied to the pavement the serviceability decreases; the rate of decrease depends on the amount of routine maintenance applied to the pavement. At year y_1, the road may have major maintenance applied to it, such as resurfacing in the form of overlay, and the serviceability is again raised to its initial value. As traffic progress the serviceability again drops to year y_2, and the process is continued throughout the life of the pavement.

An overlay is a form of periodic rehabilitation that involves the application of a relatively thick layer of bituminous bound aggregate over the existing wearing course of the weakened pavement (Oglesby and Hicks, 1982). It serves the purposes of; leveling out minor surface distortions and a means of strengthening as well as restoring structural integrity of the pavement. When applied to serve purposes including the latter one, it needs to be properly designed.

Since a major objective of engineering design is to evolve a system that will serve the purpose of its design at minimum cost possible, the designer is confronted with the problem of selecting among various design alternatives, a particular one that will serve its purpose most economically. Such a selection is made only after a detailed economic evaluation of the various alternatives must have been carried out. The purpose is to compare the probable benefits from an investment to its cost plus annual operating and maintenance expenses. This type of system cost-benefit analysis provides a comparison framework for determining whether the investment is worthwhile, optimum or otherwise.

1.2 Statement of the problem

In the last two decades, the Federal Government of Nigeria has made tremendous efforts to improve and extend her road network. As a result, considerable achievement was recorded in highway development particularly during the oil boom of the seventies (Adebisi, 1991). However,

the emphasis on expansion and extension of the road network apparently relegated the issue of maintenance to the background. Unfortunately, because of the attendant increase of traffic during the same period, the roads began to deteriorate at a rapid rate.

In Nigeria, according to a Petroleum Trust Fund (PTF) consultant-ECOPLAN, the usual practice with regard to pavement overlay employed for strengthening purpose has been, application of a fifty millimeters (50mm) of regulation course and a uniform overlay thickness of one hundred millimeters (100mm) regardless of the strengthening requirements of the road in question and the economic implications of the thickness applied. This practice may not be advantageous economically or structurally or both.

At the outset, the design decision relative to the life that might be expected from the pavement is a trade-off decision, where the engineer balances increased maintenance costs with increased initial cost. A very thick overlay has the advantage of keeping maintenance costs low and extending the pavement's service life relatively far into the future thereby eliminating the need for frequent overlay and consequent disruption of traffic. However, the very high initial cost and constructional difficulties associated with very thick overlays, and in some instances the possible need for realignment in future makes such choice highly uneconomical. On the other hand, a very thin overlay would result in lower initial cost and minimum delay to traffic during construction, but the associated frequent

maintenance and resurfacing needs within a given period of time resulting in very high maintenance costs may detract from its competitiveness with other alternatives. It is expected that between these two opposing extremes lies a certain (optimum) thickness that will maximize the stream of benefits and minimize the cost stream. This optimum thickness is what this work hopes to determine.

In order to demonstrate the achievement of the above mentioned objective, the Jos-Bauchi highway was chosen as a case study for analytical purposes only. The road link was surveyed in 1994 by the Pavement Evaluation Unit of the Federal Ministry of Works and Housing, Kaduna (PEU) for the purpose of rehabilitation design. It is the information collected that time that was used in analyzing the overlay requirement of the road and in running the Highway Design and Maintenance (HDM III model). The approximate length of the road is one hundred and twelve kilometers (112km). It was initially constructed as a surface dressed road but it was later reconstructed with a stone base and asphalt concrete surfacing in 1976. The road has been rehabilitated by the Petroleum (Special) Trust Fund. The rehabilitation carried out was not based on any functional design. For sections in fairly good condition, only overlay was applied while in very bad sections (sections that have failed completely up to subgrade level) the layers of the pavement were removed and new base constructed and surface dressed before a new surfacing was applied. Numerous problems were encountered as the work progressed, ranging

from very poor subgrade, poor quality material in the sub-base course, subsurface and surface drainages and a host of others including pavement at some sections being at the same level with the surrounding topography.

1.3 Aim and Objectives of Research

The aim of the work was to economically consider pavement overlay strategies using the Bauchi – Jos road as a case study. Against the background of the problems stated earlier, the work was designed to achieve the following objectives:

(i) Determine the overlay requirement using the rehabilitation design procedure for roads developed by the Federal Ministry of Works and Housing in conjunction with the Texas Research and Development Foundation (FMW&H – TRDF).

(ii) Identify the most suitable method of economic evaluation for overlay designs among all the available methods of economic evaluation are suitable for evaluating public investments.

(iii) Assess the influence of variation of overlay thickness on construction, maintenance management and user costs.

(iv) Determine the overlay thickness that minimizes the total Pavement system cost.

1.4 Scope and Limitations

This work is an aspect of the pavement management system (PMS) as described by Oguara and Irikuma (1987). It consists of two parts brought together to achieve the two main objectives of arriving at an optimum overlay thickness:

(a) Investigation of the overlay requirement in terms of size, deserving maintenance and replacement timing in accordance with prevailing traffic.

(b) Economic appraisal of the different overlay thickness and their economic comparison.

The overlay requirement investigation, data (information) was obtained from the Pavement Evaluation Unit (PEU) of the Federal Ministry of Works and Housing Katabu, Kaduna was for the case study (i.e. Jos-Bauchi road link), are comprised of the following:

(i) Pavement deflection

(ii) Condition survey

(iii) Traffic count and axle load

(iv) Roughness and

(v) Material strength characteristics (CBR)

These data have been combined and used together with the various other design inputs to design different overlay thicknesses. The designs were made by the use of computer facility at the Pavement Evaluation Unit.

The economic appraisal of the various thicknesses and in particular, their comparison was given more emphasis. The discounted cash flow

(DCF) technique was used in the appraisal and comparisons. The DCF relies on the concept of present value in order to reduce to manageable proportions, the distortions involved in investment project analysis with long life spans. The technique requires the calculation of the streams of costs and benefits attributed to the project over an extended period of time. Cost components such as, construction, maintenance and vehicle operation were estimated by the HDM III Model.

This study did not involve taking field measurement directly for the purpose of the overlay design; rather it used the information, which is adequate for the design, obtained from field measurements conducted by the Pavement Evaluation Unit (PEU) in 1994, traffic volume count was taken in 2002 to adjust the predicted values based on the 1994 count. Only overlays on flexible pavements were considered in the work.

In the economic appraisal and comparison aspect of the work, no attempt was made to analyze or determine the appropriate interest rate, rather, the rate at which government sells bond (considered to be the social opportunity cost rate) was used in all the computations. This is because all the design alternatives were compared on the same basis. In addition to that, the work did not include verification or recalibration of the HDM III equations for the prediction of the deterioration profiles.

CHAPTER TWO

LITERATURE REVIEW

2.1 Current Overlay Design Practice in Nigeria

Past experience suggests that the use of asphalt overlays that are not based on detailed field investigations have resulted in premature failure. A case study is the Ilorin-Bokani section of trunk road A1 which was overlaid in 1977/78 with a 58mm asphalt concrete, but was in need of additional strengthening in 1981 because of premature distress resulting from heavy axle load(Highway sector study, 198..). The practice has been to apply a selected thickness based on the experience of the engineer and on the amount of resources available for the road maintenance and rehabilitation. Adegbesan (1989) reported that in some cases the Federal Ministry of Works and Housing relies on empirical methods of overlay design, such as the Transport and Road Research Laboratory (TRRL) method. Smith (1973) noted that the measurement of surface deflection of a pavement under a standard wheel load does not of course measure any absolute properties of the pavement system and in order to be useful, it is necessary to know in broad terms at least, the composition of the pavement layers and the properties of the subgrade. The limitations of the deflection method of overlay design in Nigeria have been highlighted by Hudson and Elkins (1984) as follows:

1. **Surface deflection is the only measure of pavement strength used because in situ material properties are not used.**

2. **No deflection-based method has been specially developed for the Nigerian environment.**
3. **There is no modeling of the pavement distress such as fatigue cracking and rutting.**
4. **The individual layers of the pavement are not modeled separately as in the case with elastic layer theory.**

In addition to these, it has been observed during the course of the study that traffic counts and axle load surveys are not conducted prior to the applications of the overlay.

In summary, the deflection-based procedures fail to consider the structural strength of the existing pavement, its remaining life and an evaluation of the layers as they will function in an overlain pavement system. Hudson and Elkin (1984) observed that the procedure have built-in factors that give them regional applicability. Therefore, the deflection reduction (or deflection based) design methods have since been considered unsatisfactory.

2.2 Theories of Pavement Overlay Design

According to Fwa (1991), majority of the pavement overlay design methods in use today can be broadly classified into two distinct categories: (1) The thickness deficiency base approach and (2) the analytically based approach. The former is an empirical procedure that derives the required overlay thickness by comparing the effective

structural thickness of the existing pavement under consideration with the required structural thickness if a new pavement were to be constructed. The layer is a trial and error design procedure in which each trial overlay is analyzed using fundamental material properties and relationships to determine the various pavement responses under design loads.

Gichaga and Parker (1988), noted that empirical and semi-empirical design methods have been developed on the basis of long term pavements performance for specific traffic loading and environmental conditions. This therefore, means that as long as condition for which these methods prevail the performance of the pavement should be satisfactory. Hence as pointed out by Hudson and Elkins (1984), these empirical and semi-empirical methods have built-in factors that give them regional applicability. Gichaga and Parker (1988), advised that these design methods should be seen as models which have been used by various authorities to design pavements and thus economic criteria as a model selection should be applied.

Analytical approach to pavement overlay design involves the assumption of a pavement structure system. The strength characteristics in terms of deformation (or elastic) modulus and Poisson's ratio for each layer may be established or can be assumed. Traffic loading is then introduced and structural analysis carried out to determine the stresses and strains at critical points in the structure. The values of stresses and strains obtained from the analysis are compared with the maximum allowable values, to determine whether the design is satisfactory.

Otherwise, another system is tried or appropriate material thicknesses are increased.

According to Gichaga and Parker (1987); analytical design should involve the following steps:

(i) Development of mathematical models to represent pavement behavior.

(ii) Selection of appropriate solution technique to the equation(s) developed and thus computation of stresses, strains and deformations at critical points in the pavement structure.

(iii) Material characterization of the various layers of pavement structure for a given environment.

(iv) Establishing design criteria, such as in terms of allowable stresses or strains in certain critical points in the structure.

Although analytical design approach is theoretically sound and attractive it has up to now, not gained wide acceptance as a practical design tool by pavement engineers. Gichaga and Parker (1988), has identified two reasons for this, firstly, the complexity of the mathematical models involved and secondly, inadequate material characterization which prevents a designers from carrying out a theoretical analysis with reasonable level of confidence.

2.3 Mechanistic Modeling of Pavement Structure

The behavior of pavement subjected to wheel load applications has been studied using full-scale experiments as well as theoretically based. The main advantage of theoretical analysis is the flexibility of being able to vary different parameters without having to carry out full-scale experiments. However, as pointed out by Gichaga and Parker (1988), full scale experiments are necessary to ensure that the selected model is applicable to the problem being considered. It is also important that model selected be tested to ensure that they are relevant and capable of predicting the behavior of the pavement structure under repetitive loading.

The development of visco-elastic theories has led to its adoption by pavement engineers as the best alternative theoretical basis for analyzing flexible pavements. Boussinesq and Burmister's fundamental theories of an elastic behaviour of multilayered flexible pavements form the basis of such analytical approaches to pavement design. According to Chua et al (1993), mechanistic approach model the pavement as a structure consisting of various components acted upon by wheel loads. The stresses and strains produced determine the rate of deterioration of the pavement. These responses may be adequately determined using the multilayered model.

The performance of flexible pavements is related to certain failure mechanisms. From structural mechanics point of view, flexible pavements may experience two kinds of failures: fatigue, which shows as excessive alligator cracking; and rutting, which shows as pavement deformation. For

the study of pavement deformation, several failure criteria have been recommended (Chua, 1992; Collop and Cebon, 1996; Fin et al 1986). Other rutting criteria correlate the rate of permanent strain to elastic vertical strain and the number of load repetitions. Of these, the Asphalt Institute rutting criteria is among the most widely used. The criteria predict the allowable number of ESAL repetitions for various levels of compressive strains at subgrade. Collop and Cebon (1994), developed a linear viscoelastic rutting model to idealize the asphaltic layers of the pavement where permanent deformation is primarily due to viscous flow. The model predicts the increase in net depth per vehicle pass to be proportional to the static vehicle load and inversely proportional to vehicle speed.

$$Z = \int_{-\infty}^{\infty} I(x - v\tau)\, f(\tau)\, d\tau$$

Where

Z = permanent deformation in the longitudinal direction

I = influence function (rate of permanent deformation due to a uniform force)

v **= poisson's ratio**

Another approach known as the layer strain approach has been proposed by Brown and Bell (1977). This type of model accounts for all the pavements layers by assuming either linear or non linear relationship between the elastic stress field and pavement deformation in each layer. The relationships were usually determined from triaxial tests. Although the

strain layer approach could account for pavement deflection in each pavement layer, it could not model the characteristic ridge formed adjacent to the wheel paths.

For fatigue cracking, the maximum tensile strain at the bottom of the asphalt-bound layers is the controlling structural response. Cracking of the asphalt bound layer will result after a number of repeated traffic loads depending on the magnitude of the tensile strain and the stiffness of the asphalt concrete. The National Cooperative Highway Research Program (NCHRP) presented a set of equations to predict the number of load repetitions needed to reach fatigue failure and to develop 10 percent and 45 percent fatigue cracking of wheel tracks. Thus:

$$Log\, N_f\,(10\%) = 15.747 - 3.291 \log\left(\frac{\varepsilon}{10^{-6}}\right) - 0.854 Log\left(\frac{E}{10^{-3}}\right) \qquad \text{.....2.1a}$$

And

$$Log\, N_f\,(45\%) = 16.086 - 3.291 \log\left(\frac{\varepsilon}{10^{-6}}\right) - 0.854 Log\left(\frac{E}{10^{-3}}\right) \qquad \text{.....2.1b}$$

Where

N_f **= number of load applications required to cause 10% or 45% class II cracking of the wheel tracks;**

ε = tensile strain at the bottom of asphalt concrete layer (in/in), and

E = resilient modulus of asphalt concrete layer (ksi)

The 10% and 45% are based on the wheel path area, which corresponds to 5% and 22.5% of the total pavement area, respectively.

2.4 Methods of Economic Evaluation of Transportation Improvement

Schemes

According to Riggs (1977). The relationship of engineering to economics (in terms of importance) can be linked to that of engineering in physics: The sciences are devoted to the discoveries and explanation of nature's laws. Engineers work with the scientist and extend the relation to practical applications. The "Laws" of economics are not as precise as those of physics, but their obvious application to production and utilization of scarce resources assures increasing attention from engineers.

However, economics does not dictate keeping all roads in the best condition considering the various demands by government agencies on limited government financial resources. It must be noted that roads that are heavily used should not be allowed to deteriorate and then be reconstructed, when a much smaller expenditure at the right time would make reconstruction unnecessary (Olugbekan, 1991). In the process of managing highway pavements, there always arises the need to compare different pavement designs, maintenance and rehabilitation strategies in order to arrive at the most desirable economic solution. It is now mandatory and common practice to apply the principles of engineering economy in the evaluation of relative merits of pavement design or rehabilitation strategies. Thus, Herendeen (1978) reported the various techniques for evaluating pavement improvement plans as follows:

(i) The net present worth method

(ii) Equivalent uniform annual worth method

(iii) Benefit cost ratio method, and

(iv) Rate of return method, and

(v) Cost effectiveness method

An understanding of the capabilities and limitations of these techniques in the context of the overall planning process will, no doubt assist the planner in the selection of the appropriate technique to identify specific planning objective.

2.4.1 Economic concepts

In determining the economic implications of a transportation improvement, a clear understanding of three basic concepts is essential (also known as economic values). These are:

(1) Period of analysis (2) inflation rate; and (3) the rate of interest or discount rate. The choices of each of these elements can significantly affect the result of the economic analysis and their meanings

2.4.2 Period of Analysis

Period of analysis is the time over which costs and benefits associated with a particular plan are assumed to accrue. The selection of an appropriate period of analysis depends on the nature and purpose of

analysis. It is convenient to limit evaluations period to between 20 and 30 years for road and 50 and upward for rail lines and major port facilities. Heggie (1972), gave justifications for the conventions that; discounted cash flow (DCF) analysis is not sensitive to benefits incurred towards the end of a reasonably long period of evaluation, and that uncertainties must be allowed for, especially for projects in developing countries. Shorter periods of analysis could only be used where salvage values of the capital projects would be easily computed. Herendeen (1978), further observed that using analysis period of 30 to 50 years would render the salvage value insignificant and can therefore be ignored.

2.4.3 Inflation rate

Inflation rate is an assumed rate of change in value of monetary unit with time. The rate of inflation could be used to determine actual cash outlays or revenues or the monetary value at some time in the future. Khisty (1990), noted that, inflation could be ignored in performing most aspects of basic economic evaluation if the evaluation is prepared in present value terms, since inflation has no effect on present value. Herendeen (1978), observed that, unless the engineer has reason to assume that inflation rate will affect the different cost components, revenues or benefits in different ways, where separate inflation rate will be used for each of the altenatives.

2.4.4 Rate of interest

The rate of interest (Discount rate) at which benefits and costs are discounted is one of the most important accounting prices the evaluator has. It is a measure of time value of money. Because discount rate is the market time value of money and inflation is in the market, effects of inflation are automatically included in it. There has been, and continue to be, debates about the appropriate discount rate to use in evaluating public projects. John et al (1977), classified the various arguments, over the correct philosophy to use in selecting appropriate discount rate for public projects as follows:

(1) A zero interest rate is appropriate when tax monies are used for financing

(2) The interest value needs only to reflect the society's time preference rate.

(3) The rate of interest should match that paid by government for borrowed money.

(4) The appropriate interest rate is dictated by the opportunity cost of those investments forgone by private investors who pay taxes or purchase bonds.

(5) The appropriate interest rate is dictated by the opportunity cost of those investments forgone by government agencies due to budget constraints.

Because of the nature of transportation projects, usually long range, the last augment seem to be more favored by transport economists; Adler (1987), Heggie (1972) and Herendeen (1978). And so, this philosophy is adopted as a guide in selecting the discount rate used.

2.5. Analysis Techniques

The purpose of carrying out cost benefit analysis is primarily to ensure that an adequate return in terms of benefits results from making a capital investment. An additional purpose is to ensure that the investment option adopted gives the highest return or lowest total systems cost of all the available options considered.

2.5.1 Net present worth technique

The net present worth method calculate the total amount flowing into an organization as a result of a project, after paying all the capital use at an acceptable interest rate. It is therefore a measure of the discounted value of all benefits less the costs associated with a plan throughout the period of analysis. Implementation of the technique begin with a time period accounting (usually year by year) of all expenditures required to implement, operate and maintain the facility through the period of analysis. Revenues are subtracted from costs to obtain net cost by time period. The sum of expenditures for each time period is then discounted, using present worth factors appropriate for the selected discounted and the number of

time periods before the expenditure is expected. The sum of the present and future discounted expenditures is the Net Present Worth. The present worth equivalent for a cash flow series is given by the formula; John et al (1977):

$$P = \sum_{k=1}^{n} A_k (1+i)^{-k} \qquad \text{.....2.2}$$

Where;

P = the equivalent value of cost at time zero.

***i* = the interest rate per interest period**

n = number of interest periods.

According to Herendeen (1978), the net present worth technique is helpful when two or more alternatives for achieving the same goal involving trade-off (between high initial cost – low running expenditure and low initial costs-high recurring expenditure) options are considered.

2.5.2 Equivalent Annual Value

The Equivalent Annual Value (EAV) is the equivalent constant annual sum that a project will contribute to a firm's finances, after which the capital has been paid for. This is more reasonable when viewed from the financial side of a business, because company finances are carried out on a yearly basis. It is calculated as the Net Present worth times the capital recovery factor for the appropriate period of analysis. A mathematical relationship was given by John et al (1977), thus:

$$EAV = NPW \Big/ \left[\sum_{n=1}^{l} (1 + r)^{-n} \right] \qquad \text{.....2.3}$$

It has been noted that the Net present worth is more meaningful analysis if the decision maker is accustomed to considering the amount of bond issue that would have to be floated to pay all project cost for the period of analysis and that the uniform annual cost is more meaningful if the decision maker is accustomed to thinking in terms of budget.

2.5.3 The rate of return technique

The technique was developed as a guide to investment options where a revenue is expected from the implementation of a plan. As with other techniques discussed above, expenditures are accounted for by time period and either discounted to the present cost or converted to the annual cost. Revenues are similarly enumerated by time period and reduced to the same base. The difference between revenue and cost is the return on the investment, and the return divided by the cost is the rate of return. It therefore measures achievement in terms of the revenue generated. Heggie (1972), argued that the rate of return is not particularly efficient, either in concept or in application in evaluating public projects. It often leads to premature selection of projects. Herendeen (1978), has reported an extended version of the rate of return technique in which benefits are substituted for revenues, so that the rate of return technique in which benefits are substituted for revenues, so that the rate of return can be considered as a measure of the return received by public for an investment of public funds.

Calculation of rate of return is not straight forward and is found by solving the equation for *i* in the formula given by John et al (1977):

$$\sum_{k=1}^{n} A_k (1+i)^{-k} = 0 \quad \text{……..2.4}$$

Solutions are normally found graphically or by iteration.

2.5.4 Benefit - Cost technique

This is similar in concept to the rate of return, except that the numerator of the factor is the total benefit (or revenue) and not the difference between benefits and costs. Benefit-cost ratio was used as the standard form of economic analysis for public works project and the results provided a simple, understandable estimate of the economic value of proposed plan. However, Cartens (1976), noted that conventional benefit-cost analysis that only compares mutually exclusive plans do not indicate to the decision maker those combination of project and alternatives that may provide an optimum improvement program. Oglesby and Hicks (1982), observed that because the interest charges appear only on the cost portion of the equation for the Benefit-Cost ratio formula, an increase in the interest rate, increase the denominator and result in less favorable benefit-cost ratio. Despite these arguments, John et al (1977), argued that the other three methods of economics analysis discussed above are seldomly used in government investments, because government investments do not necessarily result in any monetary income to the government to the citizenry.

2.5.5 The cost effectiveness technique

This method of analysis was developed to permit the examination of the economic performance of alternative with respect to objectives that do not lend themselves to conversion into equivalent monetary benefit. The process requires that all costs be reduced to a net-present cost or equivalent uniform annual cost. The costs are then divided by the measure of effectiveness or the degree to which an objective is achieved. Cost effectiveness assist the decision maker by providing information regarding the relative economic efficiency of alternative plans for achieving some objective, but it cannot, as argued by Herendeen (1978), give information relative to economic justification of any of the alternatives nor provide for the analysis of alternative designs to accomplish different objectives.

Herendeen (1978), gave a general criteria that will help in selecting suitable economic measure of merit in plan selection as follows:

The net worth techniques (Net present worth and equivalent form of annual worth) are applicable to the evaluation of the economic comparison of two or more plans for achieving the same objectives. The rate of return and benefit-cost methods are useful for examining justifications of a group of alternatives with respect to measurable public benefit.

2.5.6 The project balance method

While conventional bases for comparison consolidate certain economic facts about a cash flow in to a single index, the Project Balance describes the

equivalent loss or profit to a cash flow as a function of time. The result is a time profile that measures the net equivalent amount of Naira tied up or committed to the project at each point in time over the life of the project. If the cash flow is unexpectedly terminated at the end of time, t, the Project Balance $PB_t(i)$, identifies the equivalent loss or profit associated with the cash flow at that time.

The Project Balance is defined mathematically, according to Thusesen and Fabrycky (1984) :

$$PB(i)_t = (1+i)PB(i)_{t-1} + F_t \qquad \text{.....2.5}$$

Where $PB(i)_0 = F_0$

$PB(i)_t$ = Project balance at time t

$PB(i)_t$ = Project balance at time t-1

F_t = cash receipt (+) or disbursement (-) at time t and

N = duration of cash flow.

Thuessen and Fabrycky (1984), highlighted four desirable features of the Project Balance that makes it a superior to the conventional methods of economic appraisal of engineering projects. These elements are :

1. The net future worth, $PB(i)_n$: $PB(i)_n$ measures the equivalent future worth of the investments receipts less equivalent future worth of disbursement evaluated at an interest rate of *i.* This element of information can be used just as present value or annual equivalent is used in conventional analysis

$$PV(i) = \frac{PB(i)_n}{(1+i)^n} \qquad \text{.....2.6}$$

2. The discounted payback period: A valuable piece of information associated with any investment concerns the time at which the investment is judged to have no risk of loss. This condition occurs when the Project Balance becomes non-negative. Generally, if the project is terminated after the Discounted Payback Period, no economic loss will be incurred.
3. The exposure to risk of loss: the Project Balance describes period-by-period how much committed capital is exposed to risk of loss if the project is terminated before n' (the discounted payback period).
4. The profit potential: Once the investment becomes profitable, the project balance diagram indicates the magnitude of profits expected and the rate at which they will be accumulated.

2.5.7 Capital Recovery Factor

This is use to determine the amount of each future annuity payment required to accumulate to a given present value when the interest rate and number of payments are Known.

Formulae: $A = P\dfrac{i(1+i)^n}{(1+i)^n - 1}$ = P (A/P, i, n) ……3.98

In other words capital recovery cost is the amount that goes into paying back the original capital invested + the interest that would accrue from it.

The capital recovery factor accounts for both the repayment of capital and the interest charge on the unrecovered portion of the investment in one annual payment.

Using the capital-recovery factor

Equivalent annual cost (P-S) (A/P,I,n) + Si

Where,

P = Interested amount (Present value)

S = Salvage or scrap value

i = Interest rate.

2.6 Pavement Performance Consideration in Life Cycle Cost Analysis

Current practices of economic evaluation of pavement projects do not consider explicitly the different pavement performances levels associated with the projects. Fwa and Sinha (1991), demonstrated that incorporating quantified monetary values of pavement performance into the computation could have significant impacts on the outcome of engineering economic analysis. They advanced their argument in support as follows:

1. **Although pavement condition and serviceability data are commonly used in deriving the values of individual cost component, no consideration is explicitly given to the overall pavement performance in the analysis.**
2. **While the time variations of individual cost component can always be related to pavement performance in some non-linear form, they do not represent performance for the following reasons: (a) the relationship cannot be expressed as a monotonically increasing or**

decreasing function and the rate of change of costs differ from that of pavement performance.

3. **Each of the agency and road user cost items have a different physical meaning from pavement performance.**

4. **Most highway agencies formulate strategies that include a minimum serviceability level as intervention level so as to keep the condition of pavements above this level. However this is not equivalent to pavement performance consideration, as many strategies with different overall pavement performance can be formulated to satisfy this requirement.**

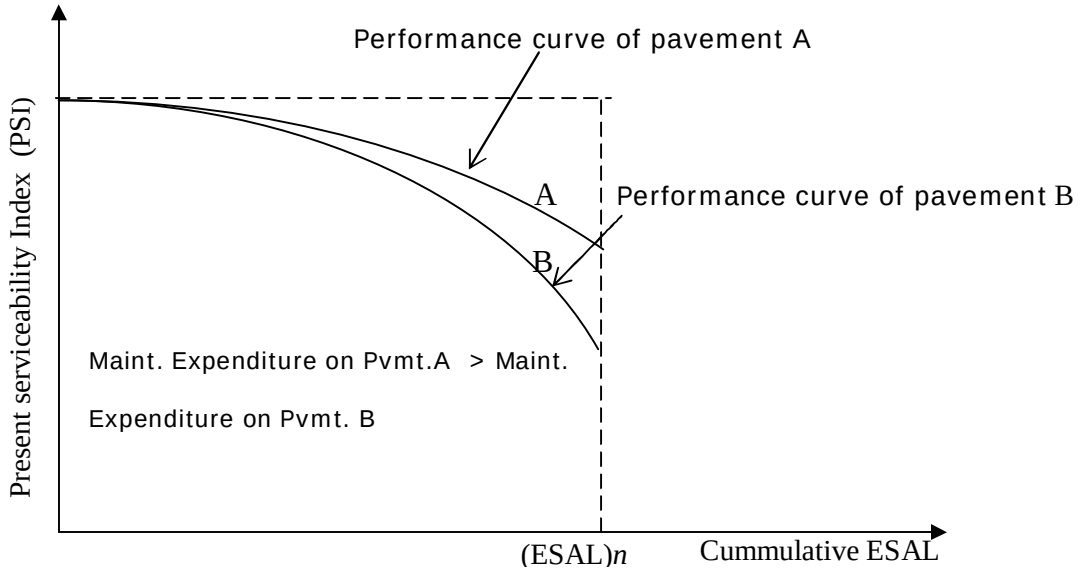


Fig. 2.1 Performance Curves of two Pavements with Different Maintenance Levels

(After Fwa and Sinha, 1991)

In quantifying the performance of pavement, Fwa and Sinha (1991) suggested a quantitative measure known as the PSI-equivalent single axle load (ESAL) Loss. The definition of PSI-ESAL loss is

PSI-ESAL loss at stage n = $\int_{o}^{Esal} [(PSI)_o - (PSI)_r] d(ESAL)$ **…….2.7**

Where

$(PSI)_o$ and $(PSI)_r$ = PSI at time zero and time t respectively; and

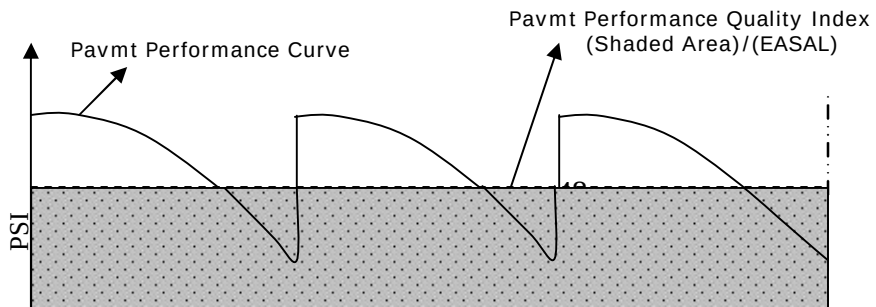
ESAL = cumulative ESAL at stage n.

In 1991 a more convenient measure was introduced, called the PSI-ESAL capacity which was meant to overcome problems posed by pavements with different initial PSI, or where beginning PSI levels of subsequent performance cycles of rehabilitation are different. The complement of PSI-ESAL loss known as PSI-ESAL capacity has the following mathematical definition: $\int_{o}^{Esal} [PSI)_o \, d(ESAL)$

PSI-ESAL capacity at stage n = **…….2.8**

According to this definition, higher PSI-ESAL capacity values mean better overall performance. However for the purpose of benefit computation, another quantity performance index known as the Pavement Performance Quality Index (PPQI) was proposed by Fwa and Sinha (1991). Hence;

$$(PPQI)_n = \frac{(PSI - ESAL\ \ Capacity)_n}{(ESAL)_n}$$ **…….2.9**



PPQI is a weighted average of PSI computed with respect to ESAL for the analysis period considered and is an aggregate representation of the overall performance of a pavement for the analysis period considered.

In order to quantify both agency's and user values of pavement performance, Fwa and Sinha (1992), demonstrated the use of expressed preference approach to access the willingness to pay for the service provided.

2.7 Improvement Prioritization Techniques

According to Robinson et al (1996), decision support systems that evolved over time have become sophisticated in the way they:

1. **Subdivide the road network in to sections**
2. **Use intervention levels to determine treatments**
3. **Enable the analysis of different numbers of treatment options per section**
4. **Carry out economic analysis and**
5. **Prioritize decisions when there are budget constraints.**

Consequently, decision support systems have been grouped in to 'first', 'second', or 'third' generation, according to their approach to addressing the above issues.

First generation decision support systems are characterized by prioritization methods where ranking is based upon the present condition of the road. The methods available under this category are:

1. Those based on defectiveness indices that are used to characterize the condition of the sections of the road. The use of these 'defect indices', provide a mechanism that enables defects of different types to be combined to determine a 'priority index' (e.g. present serviceability index (PSI) and are used both for treatment selection and for prioritization.

2. The second group methods are those based on degree of defectiveness in which priorities depend on the amount by which a measured condition or defect rating exceeds an intervention level: the more the intervention is exceeded the higher the priority score. It is worthy to note that both the defectiveness index and the degree of defectiveness methods set priorities purely on the basis of the severity of condition, and no account of the treatment to be applied or traffic is considered.

3. The third group is that known as the treatment method. Methods in this group determine priorities on the choice of the treatment to be applied, rather than simply the condition of the road. These methods assume that there is a hierarchy among maintenance activities in terms of

their impact on the condition of the road, and is combined in some manner with the traffic hierarchy to provide bases for setting priorities. Such methods have been designed to ensure that every road in the network receives some minimum maintenance needed to keep it operational.

The second generation methods base priorities on consideration not only present cost but also the future implications in terms of other costs and condition of the road. Future consequences are considered in terms of costs and benefits of each alternative, and therefore use criteria of cost-effectiveness or cost performance surrogates to determine priorities.

Sometimes these second generation cost effectiveness methods make use of conceptual performance curves to determine a particular treatment. Effectiveness represents the area under the pavement condition curve multiplied by the volume of the traffic. This concept is illustrated below: (Robinson et al, 1996)

$$E = (ADT)\, t \sum_{i=1}^{t} P_i$$

Or **…….2.9**

$$E = (ESAL)\, t \sum_{i=1}^{t} P_i$$

Where

P*i*= road condition at time t

t = treatment time in years.

The third generation methods involve formal optimization methods, and full life cycle analysis of the road agency and road user costs. The methods aim at a set of treatment options that meet certain predetermined

criteria. These prioritization criteria are expressed in mathematical terms by an objective function that is maximized or minimized. The aim of the prioritization that are defined by the objective function, is to find the solution in terms of treatment options , for each year of the analysis that maximizes the NPV, or minimizes the total systems costs subject to constraints of minimum road standard and maximum annual budget.

Robinson (1996), highlighted two methods under this category:

1. **Total enumeration method in which all possible solutions are evaluated and choice is made of that which optimizes the objective function. However, because of the quantity of computation required, this method is only applied where the size of the prioritization problem is relatively small.**
2. **Dynamic programming method: this uses the assumption that any optimal solution can consist only of optimal sub sections. Thus dynamic programming divides the optimization problem in to a number of smaller problems that are easier to solve. It, in essence carefully chose a subset, rejecting those combinations that cannot possibly lead to optimal solution. Once a problem is solved for a long period, results are applicable to shorter periods (Shahin, 1994).**

Other methods in this generation are the 'effective gradient' method (Ahmed,1983), the incremental NPV/Cost method (Philips,1994) and the 'economic boundary' or efficiency frontier' methods (Robinson et al,1996).

CHAPTER THREE

COLLECTION OF DATA AND STUDY METHODOLOGY

3.1 The HDM III Model Structure and Basic parameter

The broad concept of the HDM model, as shown in fig. 3.1 below is quite simple. These interacting set of cost relationship are added together over time in discounted present value where the cost are determined by first predicting physical quantities of resources consumption which are then multiplied by unit cost or price.

Construction cost = f_1 [terrain, soils, rainfall, geometric design, pavement design, unit cost].

Maintenance = f_2 [road deterioration, (pavement design, climate, time traffic), maintenance standard]

Road user = f_3 [geometric design, road surface condition, vehicle speed, vehicle type; unit cost]

Vehicle speed which is a major determinant of vehicle operating cost, itself related through a complex set of probabilistic function to road geometric design, surface condition, vehicle type and driver behavior.

The HDM model is used to make comparative cost estimate and economic evaluations of different policy option, including different time staging strategies, either for a given road (as the case of this project) on a specific alignment or group of links on an entire network. It can quickly estimate the total cost for large number of alternative project designs and policies year by year for up to 25 years, discounting the future costs if desired at

different postulated interest rates so that the user can search for comparison in terms of rate of return, net present value or first year benefit. Another capability of the model, using the expenditure budgeting model is finding the set of design and maintenance option that would minimize total discounted transport costs or maximizes net present value of an entire highway system under multi-year budget constraint. In-addition, for alternatives, the model can analyze sensitivity of the result to changes in assumption about key variable such as unit cost, traffic growth rates, discount rate, and the value of passengers time. A general summary of the scope of the model in terms of input requirements, their limit and the outputs that can be generated are provided in table 3.1.

As seen from the table in a single computer run, the model can evaluate up to twenty different links, each having up to ten sections with different design standard and environmental conditions. Since there is always the possibility of error in loading the inputs enshrined in the table 3.1, the model includes an extensive checking program which examine the input for formal errors and internal inconsistencies. Warning messages are automatically produced when such error or inconsistencies are found.

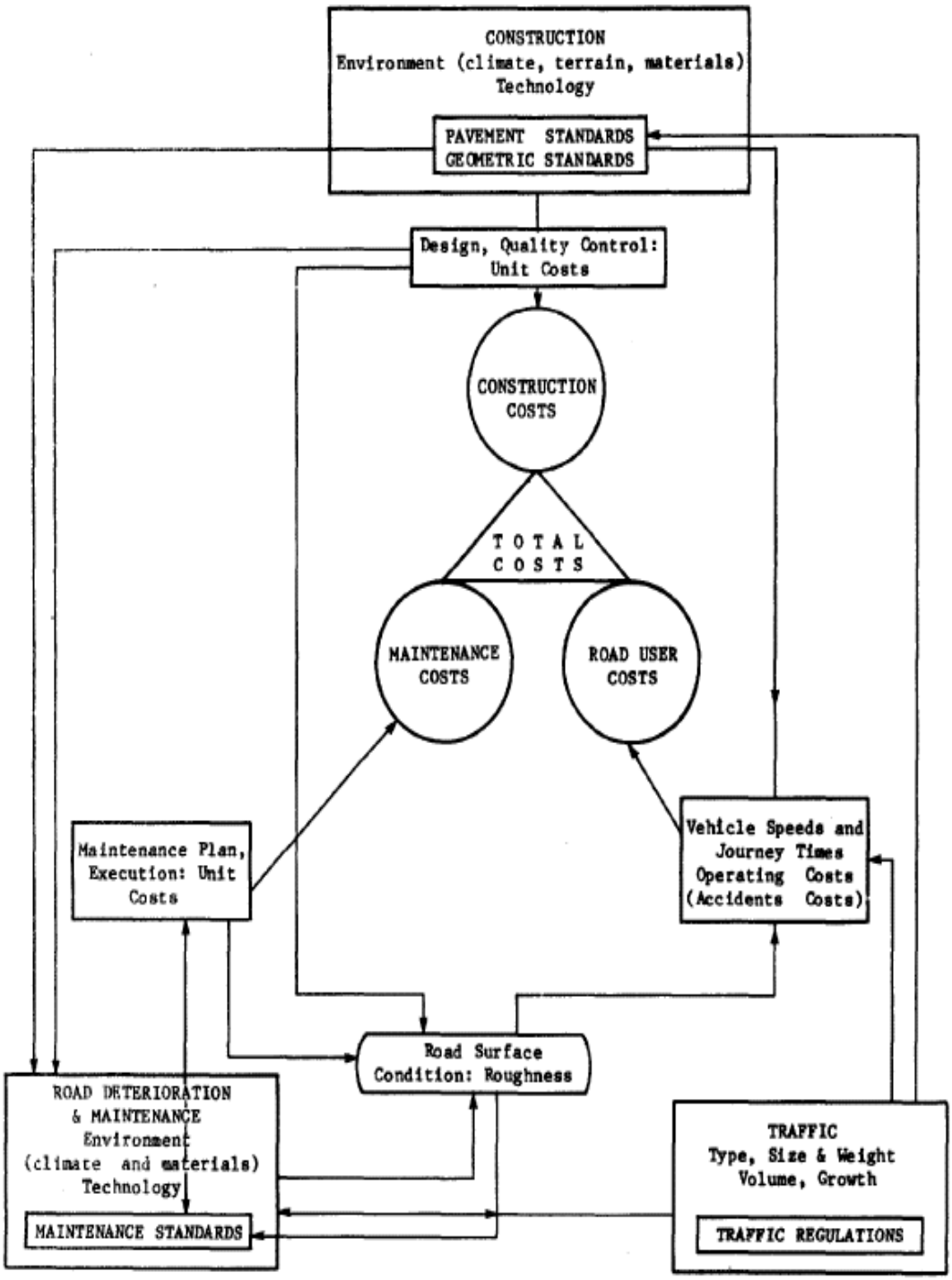


Fig. 3.1: The HDM Model: Interaction of cost of Road Construction Maintenance and use (After Watanatada et al, 1987)

Once input error have been corrected, the model estimates, speeds and resources consumption of the vehicle as well as road deterioration and resource for maintenance for all combinations. The resource requirement for road construction for each design option may be endogenously estimated by the model or may be directly specified by the user if he has more specific information or local data. After physical quantities involve in construction maintenance and vehicle operation are estimated, user-defined prices and unit cost are applied to determine financial and economic costs. Comparisons in terms of relative benefits, present value, and rate of return calculation then follows. Because some of the model relationship have highly complex non-linear forms, simulation rather than any formal optimization technique is employed in the model and the optimization which takes place is merely the selection of the group of alternatives with the highest discounted net benefit among those specified by the user. However , when the user consider the impart of expenditure constraint on the composition of the best feasible group among the alternatives, formal optimization technique in the form of dynamic program or heuristic approximation there to, are available in the expenditure budgeting model.

Table 3.1: HDM III Model Inputs and Outputs (Watanatada et al, 1987)

Inputs	Input Limits
• Link characteristics (existing road and environmental factors)	20 links
• Construction projects and costs (widening or reconstruction standards for assigning to links)	50 projects with maximum duration of 5 years for any one project.
• Maintenance standards and unit costs (intervention criteria, properties and costs for assigning to links)	30 standards
• Vehicle fleet characteristics and unit costs (common to all link-groups)	9 vehicle types
• Traffic volumes, distribution and growth (sets for assigning to links)	20 traffic sets
• Exogenous costs and benefits (sets for assigning by link)	20 sets
• Link alternatives (assign to links the above construction and maintenance standards, traffic and exogenous C-B sets)	100
• Group alternatives (assign link alternatives to link-groups)	100 group alternatives involving not more than 20 groups or 100 link-alternatives
• Number of studies, economic comparisons and sensitivity analysis (defines groups to be compared and type of analysis)	Up to 5 studies with maximum group comparisons of 50 with the number of alternatives compared not to exceed 200; 5 discount rates (in addition to zero) per study.
• Report requests (uniform per run)	Maximum of 500 reports
• Analysis period (uniform per run)	Up to 30 years with the product of link alternatives and number of analysis years not to exceed 800

Outputs
• Road maintenance summary (by link or group)
• Annual road maintenance costs and physical quantities (by link or group)
• Annual traffic (link only)
• Annual road conditions (link only)
• Annual road user costs and physical resources consumption (link only)
• Financial costs of alternative (link or group)
• Economic and foreign exchange costs of alternative (link and group)
• Comparison of costs of alternatives (link and group)
• Summary of comparison of alternatives by discount rate (link and group)
• Summary of costs and comparisons by discount rate (link alternatives only).

3.1.1 Limitations of the HDM III model

According to Pearson (1987), the HDM III model has the following limitations:

1. Vehicle operation cost sub model has not been validated for congestion traffic conditions.
2. Road deterioration sub model has not been validated for freezing climate.
3. Does not encompass rigid pavement.
4. Does not endogenously address the issue of road accidents.
5. Does not address endogenously broader environmental impact such as air pollution and noise.

It is rather a tool for economic analysis of alternative standards.

3.1.2 Applicability of the HDM III model to diverse physical and economic environments

The development of the HDM III has been guided by the objective to develop a general model which can be transferred with limited local calibration, to diverse country around the world, while extensive work to quantify the various mode relationship empirically under relatively wide range of condition have been done. In the case of vehicle operating costs the study in Kenya, Caribbean, Brazil and India constitutes the main source, and in the case of road deterioration relationships are through studies in Kenya, USA and Brazil – the question still remains, to what extent can the models be

trusted to give accurate answer over the world wide diversity of physical and economic environments?

In addressing this issue it is essential to distinguish between model forms and the model parameters (coefficients). While most form are based on well established theories of physical and behavioral phenomena and in corporate all (or almost) of the major determining factors and mechanism in sufficient details, such model forms (as distinct from model parameters) should in principle be transferable across diverse environments. The fact that the various studies in Kenya, the Caribbean, Brazil and India have separately established essentially the factor playing similar roles and generally with the same relative order of magnitude effects support the view that most of the major factor have been incorporated.

In the HDM economic models, cost component are with few exception predicted as the product of a predicted physical quantity and exogenously supplied resource price or unit cost. This separation of physical quantities and prices is obviously essential to permit transferability of the model across different economies or time in a given economy. However, because some of the models coefficients have been determined based on the economic circumstances, market condition and governmental policies prevailing at that time and place of the original studies they are expected to vary across different environments. The manner in which these various issue are dealt with is through the user's exogenous determination and specification.

3.1.3 Model operation

The HDM III operation takes place in three phases. The first is data input and diagnostic phase in which input data are first examined for possible format and numerical errors and internal inconsistencies. Any serious input errors detected in this phase will stop the execution of the remaining phases. The second phase is the stimulation of the traffic flows and of the changes in the roads as they go from initial construction through cycle of deterioration and maintenance with possible constructing project to upgrade them. This phase generates information from which at the user option report may be printed out for specific period or annually, giving road conditions as well as physical quantities and cost for road construction, maintenance and vehicle operation.

The third phase encompasses the economic analyses and comparison of alternative construction and maintenance policies for selected group to road links. Reports are generated to give differences between financial and economic cost of repair of alternatives and compare them in terms of net present value, at various discount rates internal rate of return etc.

The other model with which the HDM III may be interfaced with the Expenditure Budgeting Model (EBM) which select the optimal combination of the projects and maintenance policies under budget constraints.

3.1.4 The stimulation phase

The sequence of operation of the simulation phase is shown in Fig. 3.2. For each year of the analysis period, the sub models are applied in succession to

each link with various alternative construction programs and maintenance policies that have been specific for it.

3.1.5 The traffic sub model

The traffic sub model simply takes data that have been specified by the user and uses them to calculate the each type of vehicle in each year on road link. It also calculates, for each link-alternative the number of vehicle axles and the number of equivalent single axles

This it does for each vehicle group in the form of equivalent standard axle load. These values are used in determining the deterioration of the road surface. The equivalent standard axle-load factors ($SAL4_K$) are computed as follows, for the different load-spreading (and damage-reducing) effects of group axle, single wheels and these are incorporated by varying the standard load. SAXLj, used in determining the load ratio.

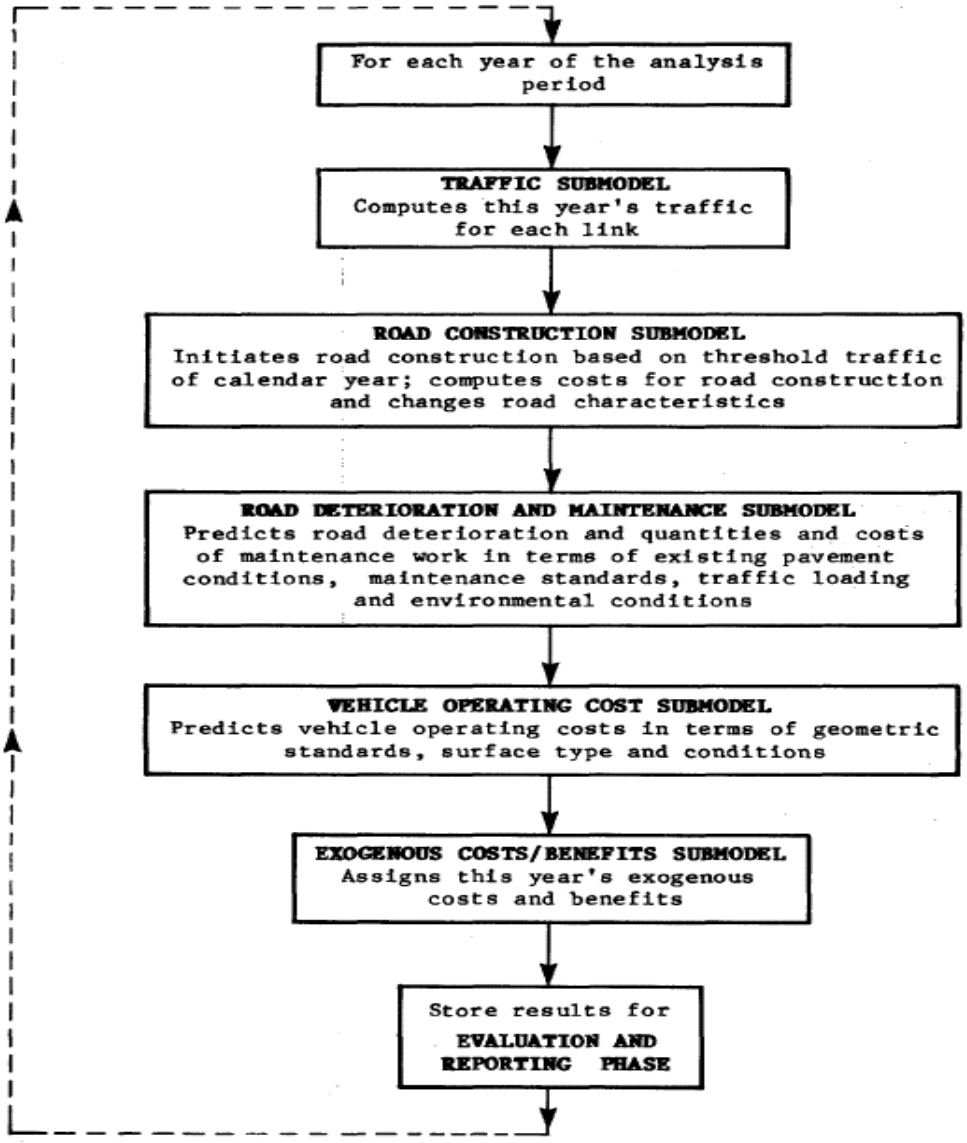


Fig. 3.2: Simulation of a link alternative (Watanatada et al, 1987)

$$AF\ 4_k = \sum_{i=1}^{I_k} \frac{P_{ki}}{100} \sum_{j=1}^{J_{ki}} \left[\frac{AXL_{kij}}{SAXL_j} \right]^{4.0}$$

……..3.3

$$AF\ 2_k = \sum_{i=1}^{I_k} \frac{P_{ki}}{100} \sum_{j=1}^{J_k} \left[\frac{AXL_{kij}}{SAXL_j} \right]^{2.0}$$

Where:

$AF4_K$ = the equivalent standard axle load factor based on the equivalency exponent of LE = 4.0 for the vehicle group K_1 in ESA per vehicle.

$AF2_K$ = the equivalent standard axle load factor based on the equivalency exponent of 2.0 for vehicle group k, in ESA per vehicle.

Le = axle load equivalency exponent

Pki= the percentage of vehicle in sub group of the vehicle group k.

Ik= the number of subgroups in vehicle group.

Jki= the number of single axles per vehicle in subgroup I of vehicle k(a tandem axle is treated as two separate single axle).

$AXLk_{ij}$ = average single axle load j of load range I of vehicle group k (tons).

$SAXL_j$ = standard single axle load of axle group type j:

= 6.60 ton for single-wheel axle

= 8.16 ton for dual-wheel single axle

= 7.63 ton for dual-wheel triple axle

$SAXL4_k$ = standard single axle load factor with exponent 4

3.1.6 Pavement Performance Sub Model

This model performs the important functions of linking constant standards (and costs), road maintenance standards (and costs), and road cost user cost through the road deterioration relationship. The HDM model considers the relationships in detail because the deterioration of road condition, ultimately manifest in the roughness of the road surface, causes significant increases in road user costs.

The sub model estimates the combined effects of traffic environment and age on the condition of the road; given data on its construction and materials and proceeds year by year to predict the change of surface condition under specified maintenance and rehabilitation policies throughout the course of analysis period. Condition is predicted by distress mode, cracking, potholes, rutting and roughness. The relationships incorporated in the sub model are structured on mechanistic concept of pavement behavior and are empirical models statistically qualified (Peterson, 1987).

In line with the structure of HDMIII, road deterioration is computed as the incremental change in pavement condition during each year of analysis due to traffic, environment and maintenance and the current pavement condition is updated each successive year during the analysis period.

3.2 Collection of Data

Data pertaining to the design of overlay have been collected from the Pavement Evaluation Unit (PEU) Katabu, Kaduna. The data was collected in

order to analyze the existing situation, i.e. overlay requirement. The data collected include those mentioned in section 1.4. The sample data for the overlay investigation and design for a typical section is give below:

Visual Condition Survey**:**

AASHO Class 1 Cracking 22.7%

AASHO Class 2 Cracking 27.9%

AASHO Class 3 Cracking 17.2%

Rut depth 0-19cm

Traffic Data**:**

Past ESAL N_p : 2,487,664

Five year Design ESAL 1,623,130

Ten year Design ESAL 3,504,782

Roughness**:**

Quarter car Index 91.14QI = 7.01 IRI (m/km)

Standard Deviation 1.88

Coef. of variation 3.56

Deflection**:**

Sensors	**1**	**2**	**3**	**4**	**5**
Deflection Basin (mils):	**2.03**	**1.32**	**0.53**	**0.31**	**0.18**

Rehabilitation Checks**:**

Patch and Seal No

Overlay Yes

Reconstruction No

Pavement deflection measurements are used to measure existing pavement response to load application, and in conjunction with other measurements such as roughness, to select locations for material sampling and destructive testing. Areas with similar deflection patterns are grouped for overlay analysis.

Visual condition survey includes evaluation of; fatigue cracking summarized by AASHO severity classes; wheel path rutting depth; patched areas, seals, and areas where pavement is completely distressed; and drainage conditions.

Fatigue cracking provides decision criteria relative to the elastic modulus and type of analysis to be performed. Rut depth measurement gives the amount of leveling up required on the existing surface prior to overlaying. Sealed areas and deep surface patches give an estimate of the amount of patching before overlaying. Also the amount of deteriorated areas helps in determining whether to patch and overlay or reconstruct the pavement.

Drainage condition evaluation gives data as input to the consideration of trade-off between overlaying and drainage improvement. While construction data such as quality control records, construction company records give additional information that assist in selection of sections of similar conditions.

Traffic estimates are converted to 8200kg equivalent standard axle load (ESAL) using a program TRUKWT. Two types of estimates were made;

The ESAL carried by the existing pavement since construction, and the ESAL projected for the design life of the new overlay.

Construction cost data for all types of road works were obtained from the World Bank's Road Construction Knowledge System (ROCKS). This is a data base program that gives the construction costs of road works for different (almost all) countries of the world.

3.3 Pavement Design Philosophy and Study Methodology

The primary objective of both pavement design and maintenance is to ensure that the pavement gives adequate quality of service to road users.

Initially the primary concern structural design of pavement was to control rutting, the development of excessive channelised deformation in the wheel paths. Acceptable rut depths of 20mm - 25mm for secondary roads and 10mm -12mm for primary roads. The early structural design methods concentrated on estimating the cover thickness of high quality materials that would be sufficient to protect the natural subgrade from excessive deformation within these limits. While the primary criterion of structural performance was rut depth, a major criterion of service was the riding quality assessed quantitatively but with physical measures of roughness (Paterson, 1987).

3.3.1 Concept of Pavement Serviceability

The concept of serviceability of a pavement then came up in order to measure the level of service provided by a road to the road using public and to

provide a basis for relating the various measures of pavement condition, not only rutting. This concept was first defined by Iric in 1960, and pavement serviceability was subjectively measured by a team of expert (panel rating). The performance of a pavement was then defined as the trend of serviceability with time, commonly depicted as concave curve with accelerating negative slope (Paterson, 1987).

However, panel ratings are difficult to control so as to ensure repeatability and their reproducibility, especially across regional and national boundaries. A physical interpretation of serviceability was estimated by AASHTO Road test by a statistical correlation between PSR and various measures of pavement distress. The correlation equations evolved define the present Serviceability Index' as follows:

$$PSI = 5.03 - 1.91Log_{10}(1+SV) - 1.38RD^2 - 0.01(C+P)^{0.5} \quad \text{.....3.1}$$

SV = slope variance, a measure of longitudinal roughness

RD = average rut depth, in inches and

C + P = area of class 2 and class 3 cracking plus patching in *ft/1,000 ft²*

Paterson (1986) pointed out three major problems with using single summary statistics such as PSI as a performance parameter:

1. **Different types of maintenance are appropriate for different levels of intervention levels and relative levels of each type of distress. For example, resurfacing is considered when cracking reaches about 30**

percent of the area, even at 100 percent cracking loss of serviceability by PSI formula is only about 0.33.

2. **The relative seriousness of different defects varies with the pavement type, the environment, the rate of deterioration and the maintenance standard. Thus, the relative weightings developed for asphalt pavements in the wet-freezing climate of Ilinois is not necessarily applicable for pavements with thin surfacing, or semi arid pavements, nor for pavements in a dry non-freezing climate.**
3. **Each type of distress evolves at different rates in different types of pavements and under different conditions of traffic and environments. Thus modeling performance by the PSI (or PSR) alone must average over many different combinations of these types, with a consequently high variance that may mask or suppress the effects which are of interest.**

The philosophy behind the analysis for overlay requirement adopted in this work is that entrenched in the pavement rehabilitation design model developed by the Texas Research and Development (TRDF) for the Pavement Evaluation Unit, Federal Ministry of Works and Housing. The approach employs purely mechanistic principles. It uses comprehensive field data from in service roads in Nigeria and advanced statistical techniques to estimate models for the analysis. However, for the deterioration prediction, the HDM model is used because the PEU model does not predict road deterioration, rather, it predict just the magnitude of

the tensile stresses and strains at the bottom of the asphalt layer as a result of traffic loading. Moreover, because it is based purely on mechanistic principles, it cannot predict roughness, or take account of the aging effects and variability (Paterson, 1987).

3.4 Pavement deterioration concepts

Road deterioration is predicted through five separate distress modes i.e. cracking, potholes. Raveling, rutting and roughness. Surfacing distress namely, cracking, raveling, potholing is characterized by two phases, i.e.

(1) Initiation phase- the period before surfacing distress of a given mode of severity develops.

(2) Progression phase- period during which the area and severity of distress increases.

Separate sets of equation are developed for the two phases, cracking is characterized by two degrees of severity “all cracking” (narrow and wide) and “wide cracking” which are subsequently combined in a composite index. Deformation distress namely in-depth and roughness are continuous and represented by only progression equations.

As an aid to keeping track of the many variables used in this section, the following convection have been adopted;

Subscripts

a = (condition) at the beginning of analysis year (after maintenance) of the previous year.

b = (condition) at the end of analysis year (before maintenance)

d = (change) due to deterioration

m = (change of condition) due to maintenance.

Prefixes

P = previous value, before maintenance

Δ = change of condition variable during analysis year.

Also the primary variables used here from one analysis year to the next to define pavement condition, history and strength are defined in table 4 and maybe classified into group as follows;

CONDITION = (ACRA, ACRW, ARAV, RDM, RDS, QI)

HISTORY = (AGE1, AGE2, AGE3)

TRAFFIC = (YE4, YAX)

STRUCTURE = (SNC, DEF, HS….. PCRA, PCRW, CRT, RRF)

The computational logic has the following sequences,

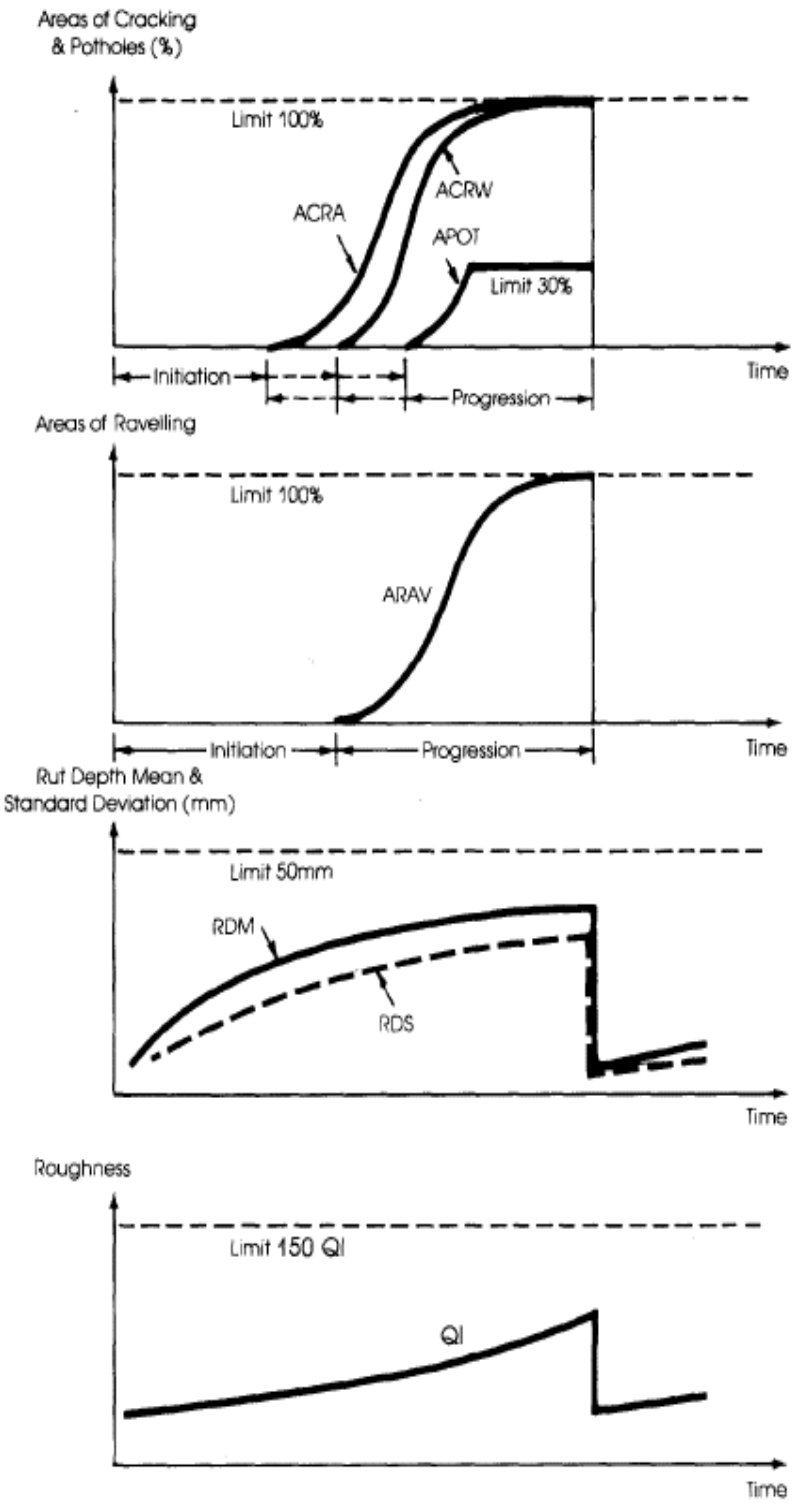


Fig. 3.3: Primary modes of pavement distress estimated in deterioration and maintenance model (After, Paterson, 1987)

1. The pavement condition at the beginning of the analysis year is initialized either from the input data if it is the first year of analysis or the first year of construction or otherwise the result of the previous year's condition after maintenance.
2. The surface condition before maintenance at the end of the year are predicted (refer to sections 4) (ACRA, ACRW, ARAV, APOT)$_b$=(ACRA. ACRW,ACAV,APOT)a + Δ(ACRA, ACRW, ARAV,APOT)
3. The rut depth and roughness before maintenance at the end of the year are predicted (see sections 4…..) [RDM,RDS,QI]$_b$= [RDM,RDS,QI] $_a$+ Δ[RDM, RDS, QI]$_d$
4. Maintenance intervention criteria are applied to determine the nature of maintenance to be applied, if any

   condition is responsive:

   If [ACRW, ARAV, APOT, QI$_b$] ≥ [ACRW, ARAV, APOT, QI] $_{intervention}$

   or condition responsive:

   if [AGE1, AGE2, AGE3]$_b$ ≥ [AGE1, AGE2, AGE3] $_{intervention}$

5. Highest ranking applicable maintenance is applied and the effects on pavement condition computed.

   [Condition]$_{a\ (next\ year)}$ = [Condition]$_b$ + Δ[Condition]$_m$

3.4.1 Pavement structural characteristics

In performance prediction models it is necessary to use measures of pavement strength which summarized the complex interactions between material

types and stiffness, layer thickness and depths, subgrade stiffness and surface conditions. The most popular measures include:-

Modified structure number; and

Surface depletion and curvature

In empirical research by Peterson (1987), the modified structural number (SNC) was found to be the most statistically significant measure of pavement strength affecting the deterioration of pavements.

The modified structural number, SNC, is defined as a linear combination of the layer strength coefficients a_i and thickness h_i of the individual layers above the subgrade, and a contribution from the subgrade denoted by SNSG i.e.

$$SNC = 0.0394 \sum_{i=1}^{n} a_i h_i + SNSG \qquad \text{.....3.4}$$

Where

a_i = The strength coefficient of the ith layer

h_i = Thickness, of the ith layer in mm

n = The number of pavement layers

$SNSG = 3.51 \log_{10} CBR - 0.85 (\log_{10} CBR)^2 - 1.43$ 3.5

and CBR = California bearing ratio of the subgrade at in situ conductions of moisture and density, in percent.

The strength variables are updated following maintenance to take account of changes in pavement strength due to overstaying, resealing and cracking. The user is required to specify only one of the primary variables, *SNC* or *DEF* and the model automatically compute the other.

3.4.2 Pavement classification

For prediction of surfacing distress (Cracking, raveling and potholing), neither SNC nor DEF are sufficient predictors for all pavement types. This pavement are classified by seven surface types and three base types thus:

1. Surface Treatment (ST)
2. Asphalt Concrete (AC)
3. Slurry on surface treatment (SSST)
4. Reseal on surface treatment (RSST)
5. Reseal on Asphalt Concrete (RSAC)
6. Open graded Cold mix on surface treatment (OCMS); and
7. Asphalt overlay or slurry seal on Asphalt Overlay on surface treatment (OVSA).

The three base types are; granular, cement stabilized and bituminous.

3.4.3 Traffic characteristics

For paved roads, traffic is defined by two variables which present both the volume and loading aspects of mixed vehicular traffic. These are flow of all vehicles axle (YAX) and flow of equivalent 80KN standard axle load (YE4), both expressed in millions per lane, and on annual basis.

To compute YAX and YE4, the total number of vehicle axles and equivalent standard axles applicable during the analysis year are divided by the effective number of lanes, denoted by ELANES

$$ELANES = \begin{cases} 1.0 & \text{if} \quad W < 4.5 \\ 1.5 & \text{if} \quad 4.5 < W < 6.0 \\ 2.0 & \text{if} \quad 6.0 < W < 8.0 \\ 3.0 & \text{if} \quad 8.0 < W, 11.0 \\ 4.0 & \text{if} \quad 11.0 \leq W \end{cases} \qquad \ldots\ldots\ldots\ldots\ldots\ldots 3.6$$

W = Carriage way width, meters

3.4.4 Environment and geometry

Environment factors are incorporated in the deterioration models through two variables, the modifies structural number (SNC) and the Mean Monthly Precipitation (MMP). The effect of ageing and weathering are represented through time variables incorporated in the prediction models.

SNC includes the contribution of the subgrade (SNSG) and the strength coefficient of the material properties ($\mathbf{a}_i$) under in situ condition of the local environment of the point. The combined effects of drainage and rainfall on the behavior of the pavement are thus represented through the effects of moisture on the strength properties and stiffness (e.g. resilient modulus) of the materials in each pavement layer. The equilibrium moisture content (EMC) in the pavement layers is estimated using the following relationship (Paterson, 1987).

$$EMC = 0.26\ P\ 075 + 0.019\ IM + 2.10 \qquad \ldots\ldots\ldots\ldots\ldots\ldots 3.7$$

EMC = the equilibrium moisture content, in % by mass.

P 075 = the amount of material passing 0.075 mm sieve in % by mass.

IM = Thornthwaites' moisture index,

Table 3.2: Recommended values of environmental coefficient 'm' for the various classes of climate (Watanatada,1987)

Moisture classification	Moisture index[1]	Temperature classification[2]		
		Tropical nonfreezing	Subtropical nonfreezing	Temperate freezing
Arid	-100 to 61	0.005	0.010	0.025
Semiarid	-60 to -21	0.010	0.016	0.035
Subhumid	-20 to +19	0.023	0.030	0.050
Humid	20 to 100	0.030	0.040	0.070+

Note: In the HDM-III model, m = 0.023 K_{ge}.
[1] After Thornthwaite (1955).

3.4.5 Consideration of variability and uncertainty

Road segment which have identical mean values of pavement strength indicators such as the modified structural number, surface type etc are often observed to deteriorate at different rates. Structural properties, drainage characteristics and construction qualities vary along the road length and between areas of the same type. Uncertainty in the prediction of a statistical model arises through the limitations of the variables that can be practically included in and the inference space of the model's derivation.

In view of this the HDM III model automatically divides the road section (part of a link) into three subsection of equal part, identified as (relatively) "weak",

"medium" and "strong" before starting simulation. The model assumes that the weak subsection deteriorates fastest and the strong deteriorates slowest. This assumption is experienced in the model in term of the occurrence of distribution factors, F, appearing in each relationship to predict the time at which cracking, or raveling starts.

3.5 Deterioration Prediction and Modeling

At the beginning of the analysis year the traffic loading variables (YAX and YE4) are computed for the year based on the user specified traffic data and vehicle load characteristics. The values of the variables that describe the environment (MMP), road geometry, (ELANES and W), pavement characteristics (SN, SNC, SNSG,DEF, HSNEW, HSOLD, HBASE, CQ,RH, CMOD, COMP) are predicted.

The variables that describe pavement history (AGE1, AGE2, and AGE3) and pavement condition ($AVRA_a$, $ACRW_a$, $ACRAV_a$,$APOT_a$, RDM_a, RDS_a, QI_a, CRT, RRF, PCRA and PCRW) are initialized to reflect a new pavement as follows:

[AGE1, AGE2, AGE3] = 1

[$ACRA_a$, $ACRW_a$, $ARAV_a$, $APOT_a$] = 0

[RAD_a, RDS_a, PCRA, PCRW] = 0

$QI_a = QII_0$

CRT = 0, RRF = 1

Where:

QII_a = the initial road roughness after pavement construction or reconstruction specified by the user or provided as a default value according to surface types shown below:

25 QI for AC or OVSA;

35 QI for ST, SSST, and RSST; and

45 QI for OCMS.

3.5.1 Cracking initiation and progression

Initiation is defined by distress extending over five percent (5%) of the sub section area. The relationships for predicting the time to initiation (which is the surfacing age) for all cracking (TYCRA) and wide cracking (TYCRW) are :

For Asphalt concrete surface with granular base

$TYCRA = K_{ci} \{F_c\ RELIC + CRT\}$ ……..3.7

$TYCRW = K_{ci} \{2.46 + 0.93\ TYCRA\}$ ……..3.8

Where

$RELIC = 4.21 \exp (0.14\ SNC – 17.1\ YE4/SNC)$

TYCRA = The predicted number of years to initiation of narrow cracks since last resurfacing (when AGE2 = 0)

TYCRW = The predicted number of years to years to initiation of wide cracks since last resurfacing (when surfacing age AGE2 = 0)

K_{ci} = The user specified deterioration factor for cracking initiation

F_c = The occurrence of distribution factor for cracking initiation.

2. Asphalt overlay on asphalt concrete, granular or bituminous base

$TYCRA = K_{ci}\{F_c\ RELIC + CRT\}$ ……3.9

$TYCRW = K_{ci}\{2.46 + 0.98\ TYCRA\}$ ……3.10

For cracking progression, the propagation area was found to be a non-linear, S-shaped function, the rate of progression depending primarily on the area of cracking and the time since cracking initiation, without significant effects of either traffic loading or pavement strength (Paterson,1987).

In the HDM III Model the relationship have a general fisrt-order relationship.

All cracking area

$$\Delta ACRA_d = K_{cp}\ CRP\ Z_a\{(Z_a\ a_i\ b_i\ \Delta TCRA + SCRA^{bi}_a)^{1/di} - SCRA_a\} \quad ....3.11$$

Wide cracking area

$$\Delta ACRW_d = K_{cp}\ CRP\ Z_w\{(Z_w\ C_i\ d_i\ \Delta TCRW + SCRW^{di}_a)^{1/di} - SCRW^a\} ...3.12$$

Where

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

$\Delta TCRA$ = the fraction of the analysis year in which all cracking progression applies, given by :

$$\Delta TRCA = \begin{vmatrix} 0 & if\ AGE2 < \mathrm{TYCRA} = 0 & \text{and ACRA} = 0 \\ (AGE3 - TYCRA) & if\ AGE2 - 1 < \mathrm{TYCRA} \le \mathrm{AGE2} & \text{and ACRA} = 0 \\ 1 & if\ TYCRA \le AGE2 - 1 & and\ ACRA = 0 \end{vmatrix} \quad .....3.13$$

$SCRA_a = min\ (ACRA_a,\ 100 - ACRA_a)$ …… ……..3.14

$SCRA_a = max\ (SCRA_a,\ 0.5)$ *if* $ACRA_a > 0.5$ ……3.15

$$ACRA_a = \begin{vmatrix} 0 & if\ \ \Delta TCRA = 0 \\ 0.5 & if\ \ 0 < \Delta TCRA < 1 \\ ACRA_a & otherwise \end{vmatrix} \quad \text{……..3.16}$$

$\Delta ACRW_d$ = the predicted change in the area of wide cracking during the analysis year due to road deterioration, in percent of total area.

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

$$\Delta TCRW = \begin{vmatrix} 0 & if\ \ AGE2 < TYCRW & and\ \ ACRW_a = 0 \\ (AGE2 - TYCRW) & if\ \ AGE2 - 1 < TYCRW \le AGE2 & and\ \ ACRW_a = 0 \\ 1 & if\ \ TYCRW \le AGE2 - 1 & and\ \ ACRW_a > 0 \end{vmatrix} \quad \text{..3.17}$$

$SCRW_a = min\ (ACRW_a,\ 100 - ACRW_a)$ ……3.18

$SCRW_a = max\ (SCRW_a,\ 0.5)$ if $ACRW_a > 0.5$ ……3.19

$$ACRW_a = \begin{vmatrix} 0 & if\ \ \Delta TCRW = 0 \\ 0.5 & if\ \ 0 < \Delta TCRW < 1\ \ and\ \ ACRW \le 0.5 \\ ACRW_a & Otherwise. \end{vmatrix} \quad \text{…… ……..3.20}$$

Table 3.3: Coefficients for prediction of Cracking Progression

Relationship	Pavement type [1]	a	b	c	d
C	Asphalt concrete	1.84	0.45	2.94	0.56
A	Surface treatment	1.76	0.32	2.50	0.25
B, F	ST on cemented base	2.13	0.35	3.67	0.38
G	Asphalt overlays	1.07	0.28	2.58	0.45
E, H	Reseals	2.41	0.34	3.4	0.35
D	Slurry seals	2.41	0.34	3.4	0.35

[1] Surfacing as indicated on flexible (granular or bituminous) base unless otherwise stated.

Source: Paterson (1987).

a_i, b_i, c_i, d_i = coefficient for type of pavement given table 4.10

K_{cp} = the user specified deterioration factor for cracking.

CRP = the retardation of cracking progression due to preventive treatment, given by $CRP = 1 - O.12\ CRT$ ……3.21

$Z_a = 1$ if $ACRA_a < 50$; $Z_a = -1$ otherwise ……3.22

$Z_a = 1$ if $ACRW_a < 50$; $Z_a = -1$ otherwise. ……3.23

3.5.2 Potholing initiation and progression

Potholing usually develop from the spalling of wide cracking or raveling of thin surface treatments, although in new surface treatment construction it can develop from random local defects in the surfacing or base. In both cases the initiation and progression of potholes are highly dependent on the mechanical disintegration properties of the base.

Potholing initiation is expressed as a function of the time (TMIN) since the initiation of triggering stress and traffic flow, and occurs typically 2 – 6 years after wide cracking and 3 to 6 years after raveling of thin surface treatments.

$$TMIN = \begin{cases} \max(6 - YAX\ ;\ 2) & \text{if base is cemented} \\ \max(2 + 0.04\ HS - 0.5YAX\ ;\ 2) & \text{otherwise} \end{cases} \quad \text{……3.24}$$

$$HS = \begin{cases} (HSNEW + HSOLD & \text{if base is granular} \\ (HSNEW + HSOLD + HBASE) & \text{if base is bituminous} \end{cases} \quad \text{……3.25}$$

The initiation indicator INPOT is set to 1 either if the age of the oldest relevant distress remaining after maintenance estimated from the rate of progression is greater than TMIN, or if potholing is present in the current condition or if potholing was reset in the first analysis year and there has been no subsequent reseal. Overlay-construction

as:

$$INPOT = \begin{cases} 1 & \text{if AGE2 – TYCRW >= TMIN and } ACRW_a >= 20 \\ \text{or} & \text{AGE2 – TYRAV} \geq \text{TMIN + (HSNEW + HSOLD) /10 and } ARAV_d \geq 30 \\ \text{or} & APOT_d > 0 \\ \text{or} & APOT_a \text{ (first analysis year) > 0 and [AGE1 – AGE1] (= analysis year – first analysis year)} \\ 0 & \text{otherwise.} \end{cases} \quad \ldots\ldots\ldots\ldots\ldots 3.26$$

Potholing is measured or converted to an equivalent percent area. Potholing progression comprises of three components, i.e., new potholes caused by wide cracking ($\Delta APOTCR_d$), new potholes caused by raveling ($\Delta APOTRV_d$) and the enlargement of existing potholes ($\Delta APOTP_d$). They are defined as follows:

$\Delta APOT_d$ = the predicted change in the total area of potholes during the analysis year due to road deterioration in percent.

$$\Delta APOTCR_d = \begin{cases} K_{pp}\ INPOT \min (2\ ACRWa\ u; 6) & \text{If } ACRW_d > 20 \\ 0 & \text{otherwise} \end{cases} \quad \ldots\ldots\ldots 3.28$$

$$u = (1+CR)\ (YAX/SNC) / ((HSNEW + HSOLD)\ 0.8W/ELANES): \quad \ldots\ldots\ldots 3.29$$

$\Delta APOTRV_d$ = the predicted change in the area of potholes during the analysis year due to raveling, given by:

$$\Delta APOTRV_d = \begin{cases} k_{pp}\ INPOT\ \min[0.4\ ARAV_a\ U;\ 6] & \text{If } ARAV_a > 30 \\ 0 & \text{Otherwise.} \end{cases} \quad \ldots\ldots.3.30$$

$\Delta APOTP_d$ = the predicted change in the equivalent area of potholes during the analysis year due to enlargement.

$$\Delta APOTP_d = \min\ [APOT_a\ [KBASE\ YAX\ (MMP+0.1)\ ]\ ;10] \quad \ldots\ldots\ldots\ldots.3.31$$

$$KBASE = \begin{cases} \max\ [2 - 0.02\ (HSNEW + HSOLD);\ 0.3] & \text{if base granular} \\ 0.6 & \text{if base is cement treated} \\ 0.3 & \text{Otherwise.} \end{cases} \quad \ldots\ldots\ldots.3.32$$

Where

K_{PP} = User specified deterioration factor for pothole progression equation for predicting damage areas before maintenance are:

$$APOT_b = \min\ (30,\ APOT_a + \Delta APOT_d) \quad \ldots\ldots\ldots\ldots.3.33$$

$$ACRA_b = \min(100 - APOT_b\ ;\ ACRA_a + ACRA_d - \Delta APOTCR_a) \quad \ldots\ldots\ldots.3.34$$

$$ACRW_b = \min\ (100 - APOT_b\ ;\ \Delta ACRW_d - \Delta APOTCR_d\ ;\ ACRA_b) \quad \ldots\ldots\ldots.3.35$$

$$ARAV_b = \min\ (200 - APOT_b - ACRA_b\ ;\ ARAV_a\ ;\ \Delta ARAV_b - \Delta APOTRV_d) \quad \ldots\ldots\ldots.3.36$$

3.5.3 Rut depth progression

The variation of rut depth which directly influences roughness is expressed by the standard deviation and is a strong function of the mean rut depth as follows (Paterson,1987):

$$RDS = \frac{K_{rp}\, 4390\, RDM_d^{\,0.532}\, (YE\,4\,x10^{6})^{ERS}}{SNC^{\,0.422}\, COMP^{\,1.66}} \quad \text{.....3.37}$$

Where

RDS = standard deviation of rut depth (mm)

K_{rp} = user specified deterioration factor for rut depth

RDM_d = rut mean depth (mm)

YE4 = number of ESAL based on equivalency exponent of 4

SNC = modified structural number of the pavement

COMP = relative compaction in the base, sub base and selected subgrade

The above relation is used in the first year when RDM_a = 0 to estimate ΔRDM_d. However, subsequent incremental change in rut depth standard deviation derived from the above is follows.

$$\Delta RDS_d = K_{rp}\left[\frac{0.532(RDM_d - RDM_a)}{RDM_a} + \frac{ERS}{AGE3} + 0.0159MMP \quad \Delta CRX_d\, \ell n\{\max(1;\, AGE3\, YE4)\} RDS_a\right]$$

.....3.38

Where ΔRDS_d = Predicted change in the standard deviation of rut depth due to road deterioration

Where ERS = exponent which is a function of the road surfaces characteristics and precipitation given as:

ERS = - 0.086 RH + 0.00115 MMP CRX_a ……3.39

ΔRDS_a = predicted change in standard deviation of not depth in mm.

3.5.4 Roughness progression

Roughness progression is predicted as the sum of three components: Structural deformation related to roughness, equivalent standard axle load flow, and structural number; surface condition, related to changes in cracking, potholing and rut depth variation; and an age –environment-related roughness term. The equation is given by Paterson (987):

$$\Delta QI_D = 23K_{gp}\left[134EMT(SNCK+1)^{-0.5}YE4 + 0.114(RDS_b - RDS_a) + 0.0066\Delta CRX_d + 0.42\Delta APOT_d\right]$$

……3.40

Where ΔQI_d = predicted change in road roughness during the analysis year due to road deterioration, in QI;

K_{gp} = user-specified deterioration factor for roughness progression

K_{ge} = user-specified deterioration factor for the environment-related annual fraction increase in roughness

EMT = exp (0.023 K_{ge} *AGE3)* …… ……..3.41

SNCK = modified structural number adjusted for the effect cracking given by:

SNK = max (1.5 ; SNC - ∆SNK) ……3.42

∆SNK = predicted reduction in the structural number due to cracking since the last reseal, overlay or reconstruction (when the surfacing age, AGE2, equals zero), given by :

$$\Delta SNK = 0.000758\left[CRX'_a\ HSNEW + ECR\ HSOLD\right] \quad \text{.....3.43}$$

CRX'$_a$ = min (63 ;CRX) ; ……3.44

ECR = predicted excess cracking beyond the amount that existed in the old surfacing layers at the time of the last pavement overlay or reconstruction, given by :

$$ECR = \max\left[\min\left(CRX_a - PCRX\,;40\right);40\right] \quad \text{.....3.45}$$

PCRX = area of previous indexed cracking in the old surfacing and base layers, given by :

PCRX = 0.62 PCRA + 0.39 PCRW. ……3.46

Roughness at the end of the analysis year befog maintenance and imposing an upper unit of 150 QI_b given by:

QI_b = min (150;QI_a + ∆QI_d) ……3.47

3.6 Maintenance Intervention

The HDM III model classifies maintenance operations into six primary categories based on the impact on pavement condition and strength these are

listed in table 4.12 and the timing of intervention is specified for each type of maintenance individually as either.

1. Scheduled, that is a fixed amount per year (e.g. m^3/kr) at fixed intervals of tone (e.g. every 5 year) or;
2. Condition – responsive that is intervening when the pavement condition is predicted to reach a critical threshold level, which is specified by the users.

Table 3.4: Classification and hierarchy of maintenance for paved Maintenance (Watanatada, 1987)

Class	Hier-archy	Frequency	Intervention Criteria (IC)	Type options	Effects
Routine-miscel-laneous	6	Annual, automatic	Nil	Nil	Nil; absence would indicate negative effects.
Patching	5	Periodic, specified by IC, % area, fixed area, or limit	1. Scheduled if damaged, limited area 2. Condition a) all surface b) potholes only	a. Surface patching	Distress and roughness.
Preven-tive treatment	4	Periodic, specified by IC, type	1. Scheduled fixed interval 2. Condition low cracking, ravelling	a. Fog seal b. Rejuvena- c. Slurry seal	Life, and (distress not applicable at high distress levels) Resealing
Reseal-ing	3	Periodic, specified by IC, type thickness	1. Scheduled fixed interval 2. Condition distress 3. Condition roughness	a. Surface treatment (i.e., chip seal) b. Slurry seal c. Chip seal with shape correction	Surface type, all distress, roughness (minor).
Overlay-ing	2	Periodic, specified by IC, type, thickness	1. Scheduled fixed interval 2. Condition roughness	a. Asphalt concrete b. Open-graded cold-mix asphalt c. Auto-level control asphalt concrete	Surface type, distress, rut depth, roughness, strength
Recon-struc-tion	1	Periodic, specified by IC, new pavement	1. Scheduled fixed age 2. Condition roughness	Any surface and base, strength	All pavement charac-teristics

For the purpose of this work only pothole patching and overlay are considered.

For scheduled patching policy, the area of patching calculated by the mode is given by.

$$\Delta ASP = \min\left[\frac{ASPS_O}{10W};\ ADAMS\right] \qquad \text{.....3.48}$$

Where

ΔASP = the area of patching performed in percent of total carriage way surface area.

$ASPS_0$ = The specified maximum annual patching in m^2 /km; and

$ADMS = ACRW_d + ARAV_b + APOT_b$ ……3.49

For the condition responsive case the area of patching is given by :

$$\Delta ASP = \min\left[ASPS;\ \frac{\Delta ASPMAX}{10W} \right] \quad \text{……3.50}$$

$$ASPS = \begin{cases} \frac{FDAM_O}{100} ADAMS & \text{if responsive to severe damage} \\ \frac{FPOT_O}{100} APOT & \text{if responsive to potholing only} \end{cases} \quad \text{……3.51}$$

where

$FDAM_0$ = The percentage of the damaged area to be patched in a year, specified by the user.

$FPOT_0$ = The percentage of the potholing area to be patched in a year, specified by the user.

$\Delta ASPMAX$ = Maximum applicable area of patching in a year in m^2/km

The resultant effect of pothole patching on the roughness is predicted by the relation.

$$\Delta QI_m = \min\left[0.13\min(\Delta ASP;\ 10) + 0.086\ \Delta CRX_m + 4.91\ \Delta APOT_m\ ;\ 150\ QI_b\right]$$

……3.52

Where

QI_m = The predicted change in road roughness during the analysis year due to maintenance, in QI ;

ΔCRX_m = The predicted change in cracking index (weighed for cracking severity) due to maintenance given by:

$$\Delta CRX_m = 0.62\ \Delta ACRA_m + 0.39\ \Delta ACRW_m \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots 3.53$$

3.6.1 Effect of overlay

The overlay operation in the sub model applies to bituminous overlay placed by mechanical paver – finisher; other overlays are dealt with as follows;

1. Open graded cold mix asphalt, regular manual unveiling
2. Hot mixed asphalt concrete, regular or manual resealing consol.
3. Hot mixed asphalt concrete, with long base automatic leveling control.

The affects of overlay on pavement deterioration are defined through the reclassification of surface type and the associate deteriorations prediction relationships.

Overlay policy comprises of fixed types, thickness and strength coefficients specified by the user and applied as follows:

1. Scheduled policy, in which the overlay affixed specification is applied when ever the construction (rehabilitation) age, AGE3, equals or exceeds a fixed time interval specified.

2. Condition Responsive, in which the overlay of fixed specification is applied when the roughness before maintenance exceeds a maximum allowable roughness specified by the user.

Under either policy, an overlay is not performed if the construction age AGE3, is less than the user – specified minimum applicable overlay interval or if the last applicable year has been exceeded. If performed, the amount of overlay, in m^2/km is equal to 1000W (W is the weight used for averaging the cracking in the old and new surfacing).The cost of the overlay is computed by multiplying with this amount the user specified unit cost (per m^2)

When the overlay is performed the surface is changed to either of the two types; asphalt concrete (OVSA) or cold mix (OCMS), depending on the type of overlay specified by user, regardless of the old surfacing. Thus alters the applicable set of surfacing distress relation ships to represent the different deterioration behavior.

For immediate affect of overlay, the model sets to zero the amount of cracking, raveling, and potholing and the cracking retardation time, deconstruction fault code; and resets the raveling retardation factor, the preventive treatment, surfacing and construction ages. The rut depth is reduced by 85 percent, in recognition of the residual rut remaining after compaction of a single layer. The roughness is reset to the (optional) value specified by the user, QI_0 , or to the default values depending upon the quality of paving, as computed in the formulae for ΔQI_m below:

Option (a): Regular pavers

$$\Delta QI_m = \begin{cases} QII_0 - QI_b & if \quad QII_0 \quad is \ provided \\ \min\left\{0;\ \ 20+10k_{ov}+\left[28\max\left(QI'_b-50;0\right)/\max\left(H'_o;28\right)\right]-QI'_b\right\} & \\ \text{otherwise.} & \end{cases} \quad \ldots\ldots\ldots\ldots\ldots 3.53$$

Option (b) : Automatic-leveling long base pavers

$$\Delta QI_m = \begin{cases} QII_0 - QI_b & if \quad QII_0 \quad is \ provided \\ \min\left\{0;\max\left[\left(19.42-0.78QI'_b-0.068H_O\right);\left(-19.5-0.008QI_b\max\left(H_O-20;0\right)\right)\right]\right\} & \end{cases} \quad \ldots\ldots\ldots 3.54$$

Where

QI I_o = initial road roughness after overlay specified by the user as an option, in QI :

H_o = thickness of overlay, in mm

QI'_b = max (13 ; QI $_b$= 4.91 APOT) : ……3.54

H_o' = min (H_o ; 80) ; and ……3.55

$$H_O = 3-\left[\frac{\min\left(H_O;80\right)+\min\left(H_O;40\right)}{40}\right] \quad \ldots\ldots\ldots 3.56$$

In addition the pavement strength parameters are updated to take account of the change in pavement strength due to the new overlay and the underlying cracks (if any). The procedure for the modification is the same as follows :

$$\Delta\left[ACRA, ACRAW, ARAV, APOT\right]_m = -\left[ACRA, ACRAW, ARAV, APOT\right]_b \quad \ldots\ldots 3.57$$

except for the following redefinitions:

a_0 = the strength coefficient of the overlay; and

H_0 = as defined above: and

$$W = \begin{cases} max\ (HSNEW_{(before)} / HSOLD_{(after)}\ ;\ 0.6 & \text{if base type is not cemented} \\ \max\left[HSNEW_{(before)} / \left(HSOLD_{(after)} + HBASE\right); 0.6\right], & \text{otherwise. ……..3.58} \end{cases}$$

3.7 Vehicle Operating Cost Sub Model

The function of the vehicle operating cost sub model is to simulate the effects of the physical characteristics and condition of a road on the operating speeds of various types of vehicles on their consumption of fuel and lubricants, on their maintenance requirement, and so on, and to determine their total operating cost. The quantities of resource consumed, such as liters of fuel, numbers of tires, man- hours of labor etc are determine together with the vehicle speeds as function of the characteristics of each type of vehicle and geometry, surface type and current condition of the road.

The procedure followed by the vehicle operating cost model in computing speeds, resource use, and costs for the traffic using a given road section is summarized as follows:

1. Compute the average operating speed for each vehicle group.
2. Compute the amount of resources used per vehicle kilometer by each group for the following components:
    a. Fuel

b. Tire wear
c. Maintenance part
d. Maintenance labor
e. Lubricant
f. Crew
g. Depreciation
h. Interest
i. Overhead
j. Passenger time
k. Cargo holding
l. Miscellaneous costs.

3. Apply unit costs to the resource consumption amounts to obtain cost per vehicle – kilometer for each vehicle group.
4. Multiply cost per vehicle- kilometer by the section length and by the year traffic volume of each vehicle group to obtain the total road user cost for the year vehicle group.
5. Sum the group totals to obtain the overall total road user cost for the year.

In the HDMIII model there are four different sets of relationship for estimating vehicle speeds and the consumption of some of the operating resources, based on four separate empirical studies by different agencies. The user must choose and specify which of these sets is to be employed in any given run. The principal set of relationships is those derived from the Brazil study, The Texas Research and Development Foundation

(TRDF) and the World Bank. The alternative relationship are those from the Kenya and Caribbean studies by the British Transport and Road Research Laboratory (TRRL), and those from the Indian Study by Central Road Research Institute New Delhi.

The Brazil portion of the vehicle operating cost (VOC) sub model is made up of relationships based on Brazil- UNDP study. Vehicle speed being the most important variable in the vehicle operating cost relationship was determined based on what may be described as according to Watanatada *et al*., (1987) an aggregate probabilistic limit velocity approach to study state speed prediction.

For the purpose of predicting the average speed for a round trip, any given road way of various vertical and horizontal alignment could be idealized by two homogeneous segments one of positive grade (uphill) and the other of negative grade (downhill), both having the same lengths, roughness, average horizontal curvature, and average rise plus fall.

The steady state speed of each type of vehicle is predicted as a function of the road characteristics and the attributes of the vehicles. Then the average of the round trip is computed to correspond to the space- mean speed over the two segments. The information for the roadway needed for the application of the speed prediction method is; the surface type of the roadway, an average measure of its horizontal curvature, an average value of the super elevation. The average roughness value is computed internally. The gradient is expressed as the average plus fall, RF, which is defined as the sum of the absolute values in meters, of all ascend and descends along a section, divided by the length of the

section in kilometers (equal to 10X mean absolute gradient in percent). The average horizontal curvature, C, is defined as the sum of the absolute values of angular deviation (in degrees) of successive tangent lines of the road alignment. The average super elevation, SP, is defined as the weighted average of super elevations (in percent) of the curvy sections of the roadway.

The condition of a vehicles steady-state speed on a given road segment makes use of a set of limiting (or constraining) velocities, corresponding to several different factors that tend to limit the speed. The constraining velocities are (all in meters/sec):

VDRIVE = The limiting speed based on vertical gradient and engine power.

VBRAKE = The limiting speed based on vertical gradient and braking capacity.

VCURVE = The limiting speed determined by road curvature.

VROUGH = The limiting speed based on roughness and associated ride severity, and

VDESIR = The desired speed in the absence of other constraints based on psychological, economical, safety, and other consideration.

The two gravity related constraining speeds, V12, VDRIVE and VBRAKE, have different values for the uphill and the downhill segments representing a given roadway.

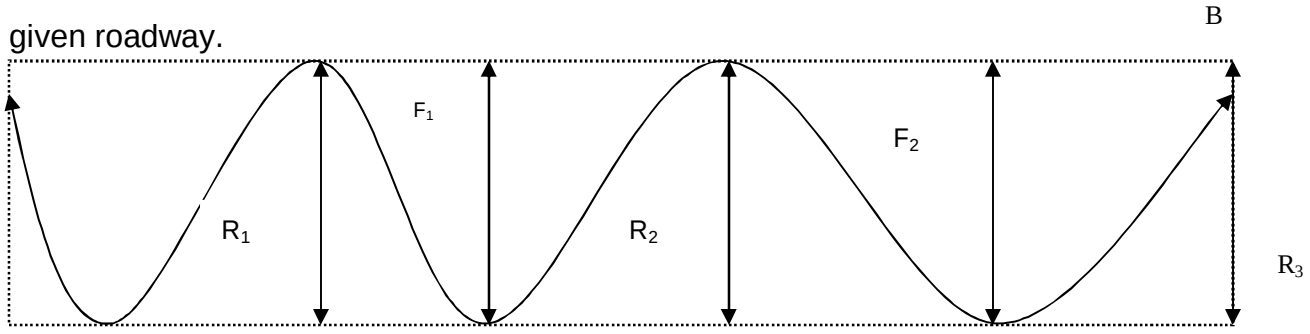


Fig. 3.4: Illustration of road rise plus fall of a road profile

Average rise plus fall, $RF = \frac{R_1 + R_2 + R_3 + F_1 + F_2}{L_{ab}} = \frac{Meters}{Km}$

Average horizontal curvature, $C = \frac{C_1 + C_2 + C_3 + C_4}{L_{ab}} \quad \frac{degree}{Km}$

Using the respective values of the five limiting speeds for each road segment, the predicted steady- state speed for the segment is computed. The theory behind these computations involves treating each of the limiting speeds for a segment as random variable and the resulting steady state speed production as the average value of the minimum of these random variables. The probability model used is Weibull distribution (Watanatada et al, 1987).

$$V_u = \frac{E^O}{[(1/VDRIVE_u)^{1/\beta}+(1/VBRAKE_u)^{1/\beta}+(1/VCURVE)^{1/\beta}+(1/VROUGH)^{1/\beta}+(1/VDESIR)^{1/\beta}]^{\beta}}$$

$$V_d = \frac{E^O}{[(1/VDRIVE_d)^{1/\beta}+(1/VBRAKE_d)^{1/\beta}+(1/VCURVE)^{1/\beta}+(1/VROUGH)^{1/\beta}+(1/VDESIR)^{1/\beta}]^{\beta}}$$

The subscript used stand for the uphill and the downhill segment, the coefficient, determines the shape of the assumed Weibull distribution (Watanatada et al, 1987) and is a member of the set of parameters estimated for each type of vehicle.

E_0 = Bias correction factor. Equal to exp ($0.5\ \sigma^2$)

σ = An estimated standard error of the residuals in the estimation.

The average speed for the round speed is given by;

$$S = \frac{(3.6)(2)\ LGTH}{(LGTH/V_u)+(LGTH/V_d)} = \frac{7.2}{(1/V_u)+(1/V_d)} \qquad \text{……..3.63}$$

Where

LGTH = the length of the roadway in Km

3.6 = Conversion factor from m/s to km/hr

VDRIVE, the speed for a given road segment as limited by power and gradient, is arrived at, from the hypothesis that the vehicle is driven at steady- state speed on a smooth, straight road employing a high level of power called the “used driven power”, HPDRIVE.

VDRIVE is related to HPDRIVE and the gradient through the balance of forces in the absence of acceleration.

[drive force] = [rolling resistance] + [grade resistance] + [air resistance]

Where the various terms, all measured in Newton, are given by the following (Watanada et al):

Drive Force = (736 HDRIVE)/VDRIVE ……..3.64

Rolling resistance = *g 1000 GWW CR* ……..3.65

Grade resistance = *g 1000 GVW GR* ……..3.66

Air resistance = 0.5 *RHO CD AR VDRIVE*^{2} ……..3.67

Where 736 = number of watts in one metric Hp

GVW = Gross vehicle weight, in tone

g = acceleration due to gravity = 9.81 m/s^2

CR = Coefficient of rolling resistance

GR = Vertical gradient expression as a function

RHO = Mass density of air in Kg/ m^3 (a function of alt).

CD = Aerodynamic drag coefficient of vehicle; and

AR = Projected frontal area of the vehicle, in m^2

GR = RF/1000, where RF = Average rise plus fall.

$$CR = \begin{cases} 0.0128 + 0.0000467\ QI & \text{For cars and utilities} \\ 0.0139 + 0.0000198\ QI & \text{For buses and Trucks} \end{cases} \qquad \text{……..3.68}$$

$$RHO = 1.225\ [1-2.26 \times 10^{-5}\ A]^{4.255} \qquad \text{……..3.69}$$

A = the road altitude in meters above sea level

VBRAKE

The speed for a given road segment as limited by braking capacity and gradient is arrived at using the concept of “Use braking power”, which is a positive quantity, denoted by HPBRAKE, in metric horse power units. The underlying concept is that the steady-speed attained on a long, smooth, straight downhill section is limited by the braking capacity, HPBRAKE, which depends on the vehicles type.

When the constraints are applied, VBRAKE and HPBRAKE are related, as before, through the force balance.

$VBRAKE_u \equiv \infty$

$$VBRAKE_d = \begin{cases} \infty & \text{If } CR \geq RF/1000 \\ \dfrac{736\ HPBRAKE}{g\ 1000\ GVW\ (CR - RF/1000)}, & \text{If } CR < RF/1000 \end{cases} \quad \ldots\ldots 3.70$$

VCURVE

The curvature - limited speed, VCURVE, is arrived at, from the postulate that when curvature is significant the speed is limited by the tendency of the wheels to skid. An indicator of the tendency to skid is the ratio of the side force on the vehicle type and the surface. It is called the "perceive function ratio", denoted by *FRATIO*

$$FRATIO = \frac{VCURVE^2}{g\ RC} - \frac{SP}{100} \qquad \ldots\ldots 3.71$$

Whence

$$VCURVE = \sqrt{(FRATIO + SP/100)\ g\ RC} \qquad \ldots\ldots 3.72$$

$$FRATIO = Max\ (0.02,\ FRATIO_o - FRATIO_1\ LOAD) \qquad \ldots\ldots 3.73$$

SP = Super elevation

$FRATIO_o$ and $FRATIO_1$ are parameters depending on vehicles type and surface type and are estimated by the model.

$$RC = \frac{180{,}000}{\pi\ Max(18/\pi, C)} \qquad \ldots\ldots 3.74$$

RC = Radius of curvature

If the user does not apply values for SP, the model computes Sp values from the following formulae

$$SP = \begin{cases} 0.012C, \text{ For paved roads} \\ 0.017C, \text{ For unpaved roads} \end{cases} \qquad \text{……..3.75}$$

VROUGH

VROUGH is the limiting speed corresponding to the “maximum allowable” suspension motion of the vehicles, which governs ride severity. It is related to roads roughness by the following relationships

$$ARV = 0.0882\ VQI \qquad \text{……..3.76}$$

Where

ARV = Average rectified velocity of suspension motion of the standard Maysmeter vehicle traveling at speed V, in mmls;

The constraint speed due to ride severity, VROUGH is govern by the Max allowable ARV, called *ARVMAX*, as follows:

$$VROUGH = ARVMAX / 0.0882QI \qquad \text{……..3.77}$$

ARVMAX is estimated by the model and utilized endogenously

VDESIR

This is the desired speed, i.e., the speed at which a vehicle is assumed to be operated in the absence of the constraints based on the vertical grade, curvature, and ride severity.

$$VDESIR = \begin{cases} VDESIR_o \, B_w / 3.6 & \text{If effective number of lanes = 1} \\ VDESIR_o / 3.6 & \text{If effective number of lanes is more than 1} \end{cases} \quad \text{.....3.78}$$

$VDESIR_o$ = Unmodified *VDESIR* obtained in the Brazil - UNDP study, in km/hr.

B_w = Dimensionless width parameter adjusting desire speed estimated by the model depending on the width of the roadway.

3.7.1 Fuel Consumption

The fuel consumption prediction model makes use of the concept of time-rate fuel consumption, denoted by UFC (in ml/sec). The basic principle of internal combustion engine suggests that, under ideal conditions, the unit fuel consumption is a function of power out put (HP, in metric hp) and engine speed (RPM, in rpm). Thus,

$$UFC = UFC(HP, RPM) \quad \text{.....3.79}$$

In the Brazil study, a quadratic form of function was employed for vehicle operation on any road section of specified geometric alignment, the average round trip fuel consumption, FL is given by

$$FL = 500\alpha_1\alpha_1\left[\left(UFC_u/V_u\right)+\left(UFC_d/V_d\right)\right] \qquad ml/s \quad \text{.....3.80}$$

Where UFC = as defined above, subscript $_u$ and $_d$ represent uphill and downhill.

TIRE WEAR

The HDM model employs two relationships one for cars and utilities, and the other for trucks and buses. Thus:

$$TC = \begin{cases} NT(0.0114 + 0.000137\ QI & \text{For } 0 < QI \leq 200 \\ 0.0388 & \text{For } QI > 200 \end{cases} \quad \text{.....3.81}$$

For trucks the relationship is given by

$$TC = \left[\frac{(1+ 0.0\ 1RREC\ \ NR)\ (C_o tc + C_{tctc}\ CF^2/L)}{(1 + NR)\ VOL} + 0.0075\right] \quad \text{.....3.82}$$

Where

NT = number of tires per vehicles

RREC = the ratio of the cost of one rethreading to the cost of one new tire in percent.

NR = number of rethreading tires carca, predicted by

$NR = NR_o \exp(-0.00248\ QI - 0.0018C) - 1$ 3.83

NR_o = the base number of recaps

$C_o$tc = Constant for the tread wear model

C_{tcte} = Wear coefficient

CF^2 = Average squared circumferential force per tire, given as

$CF^2 = ½ (CF^2_u + CF^2_d)$ ……..*3.84*

VOL = Average wearable rubber volume per tire in dm^3

L = Average force per tire in the direction perpendicular to the road surface, in Newton, given by:

$$L = \frac{1000GVW}{NT} g$$ ……..*3.85*

C = Curvature in degree/Km

$CF_u = 1/NT [1000CR – RF) GVW g + 0.5 RHO CD AR V^2_u]$ ……..*3.86*

$CF_u = 1/NT [1000CR – RF) GVW g + 0.5 RHO CD AR V^2_d]$ ……..*3.87*

3.7.2 Maintenance parts

Parts consumption is modeled in term of plants consumed per 1000 vehicle- Km.

MPC = NVP, PC ……..*3.88*

MPC = Monetary cost of spare parts consumed per 1000 vehicle- Km.

NVP = Average price of new vehicle

PC = the part cost per 1000 vehicles- Km.

3.7.3 Maintenance labour

Maintenance labour hour are related primarily to maintenance parts requirement, and roughness. The relationship is

$$LH = C_o L_h PC^{Clhpc} exp\ (C_{lhqi}\ QI) \qquad \text{.....3.89}$$

C = Lh, C_{Lpc} and C_{Lhqi} are coefficient determined by the model.

Other components of vehicle operating cost are:

1. Lubricant consumption predicted as a function of roughness.
2. Crew requirement predicted as inversely proportional to vehicle speed.
3. Vehicle depreciation and interest.
4. Annual Average utilization
5. Overhead costs: and
6. Miscellaneous costs.

Formulae showing relationships for computing each of the above are given (in the HDM standard series Vol.1) by Paterson (1987).

3.8 Economic Analysis

The HDM III model was designed to facilitate the making of economic comparisons between many alternatives, with difference initial construction standards and maintenance operations, and interrelating road quality and operating cost of vehicles. The submodels thus far described, provide the cost of construction, maintenance, and vehicle operating costs. The differences in these costs are part of the basis for the economic evaluation of one alternative relative

to another or to the “base” case. Also involved are benefit due to increased volumes of traffic and exogenous benefits and costs.

3.8.1 Road benefit and costs

For every road link alternative for every year of the analysis period, the physical quantities involved in construction and in maintenance are calculated and then multiplied by their unit prices. The resulting cost differences are identified as either capital or recurrent. These total component costs in the two categories (construction and maintenance), the cost different between alternatives for any link in a given year are calculated using the relations:

$$\text{Road Capital Cost} = \Delta CAP_{(m-n)} = CAP_m - CAP_n$$

$$\text{Road recurrent cost} = \Delta REC_{(m-n)} = REC_m - REC_n$$

Where;

$\Delta CAP_{(m-n)}$ = difference in road capital cost of alternative m relative to alternative n (for given link for a given year).

CAP; = Total road capital cost incurred by alternatives;

$\Delta REC_{(m-n)}$ = different in road recurrent cost of alternative m relative to n.

REC; = Total road recurrent cost incurred by alternative; . These differences are calculated separately for each year for each pair of link- alternative that are to be compared.

3.8.2 Vehicle benefits and costs

The annual economic benefit in terms of vehicle operating cost and travel time savings are calculated separately for normal and generated traffic. In evaluating road user benefit the model treats each vehicle group as a dem and entity or market segment which is associated with its own unit price of travel. The benefit are first computed for each group and then summed up overall vehicle groups to give total road user benefits. The road user benefit of alternative m relative to alternative n, for a given link in a given year, are calculated using the following formulae

a. Vehicle operating benefit due to normal traffic

$$\Delta VCN_{(m\text{-}n)} = \sum_{l}^{n} TN_i \left(UC_{ni} - UC_{mi}\right) \qquad \text{.....3.90}$$

b. Vehicle operating benefits due to generated traffic

$$\Delta VCG_{(m\text{-}n)} = \sum_{i} \tfrac{1}{2}\left[TG_{mi} + TG_{ni}\right]\left[UC_{ni} - UC_{mi}\right] \qquad \text{.....3.91}$$

c. Vehicle travel time due to normal traffic

$$\Delta TCN_{(m\text{-}n)} = \sum_{i} TN_i \left[UT_{ni} - UT_{mi}\right] \qquad \text{.....3.92}$$

d. Vehicle travel time saving (benefits) due to generated traffic

$$\Delta TCG_{(m\text{-}n)} = \sum_{i} \tfrac{1}{2}\left[TG_{mi} + TG_{ni}\right]\left[UG_{ni} - UG_{mi}\right] \qquad \text{.....3.93}$$

Where

$\Delta VCN_{(m\text{-}n)}$ = Vehicle operating benefits due to normal traffic.

TN_i = Vehicle group I normal traffic in number of vehicles per year in both direction.

UC_{ji} = Average operating cost per vehicle trip over the link for vehicle group i under alternative j.

$VCG_{(m-n)}$ = Vehicles operating benefits due to generated traffic of alternative in relative to m.

TGji = the generated traffic of vehicle group i due to alternative j, relative to the baseline alternative, in numbers of vehicles per year in both direction.

$\Delta TCN_{(m-n)}$ = Travel time benefit due to generated traffic of alternative m in relative to alternative n.

UTji = Average travel time cost per vehicle trip over the link for vehicle group i under alternative j.

$\Delta TCG_{(m-n)}$ = Travel time benefits due to generated traffic of alternative n on the given link in the given year.

3.8.3 Exogenous benefit and cost

The savings in vehicles operation costs and travel time endogenously calculated by the HDM III model constitute the great majority of the total benefits to the society of the road improvement. The principal exceptions are reduction in accidents, environmental impacts (e.g. noise and air pollution), they can be estimated separately and entered as exogenous addition to the effect that the model handles internally.

The exogenous benefit- cost sub model parameters were furnished by the user to computes, for each year the exogenous benefits or costs and makes the

result available for the model's economic analysis. For both benefits and cost the user has option of entering the value of particular years relative to the start of the stream or defining a stream growing rate at particular years relative to the start of a stream, or defining a stream growing from an initial value at either a linear or compound rate.

For each part of alternative being compared in a given year for a given link, the differences in net exogenous benefits are as follows:

$\Delta EXB_{(m-n)} = EXB_m - EXC_m - EXB_n - EXC_n$ ……3.94

Where

$\Delta EXB_{(m-n)}$ =the difference in net exogenous benefit.

EXB_j = Exogenous benefit for alternative m in relative to n (for a given link in a year)

EXC_j = Exogenous cost for alternative j, given by the submodel for that link and year.

3.9 Economic Analysis and Comparisons

For each pair of linked alternatives that are involved in comparing the formulae, above are used to calculate, year by year, the benefits or cost of one relative to the other, combining road, vehicle and exogenous components, both capital and recurrent. For any link group or alternative, K, in year Y, the net economic benefit of alternative m relative to alternative n,

$\Delta NB_{(m-n)}$ is computed as:

$$\Delta NB_{(m-n)} = \Delta VCN_{kg\,(m-n)} + \Delta VCG_{kg\,(m-n)} + \Delta EXB_{kg\,(m-n)} + TCN_{kg\,(m-n)}$$

$+ \Delta TCG_{kg\ (m-n)} + \Delta CAP_{kg\ (m-n)} - \Delta REC_{kg\ (m-n)}$ …….3.95

Where $\Delta NB_{kg\ (m-n)}$ = net economic benefit of alternative m relative to alternative n in year y, the subscripts K and Y indicate link or group and year.

The model in addition computes the net present values and well as the internal rate of return, benefit cause ratios, and first year benefits.

3.9.1 The net present value

The Net Present Value (NPV) of alternative m relative to n, $NPV_{k(m-n)}$ is computed as;

$$NPV_{k(m-n)} = \sum_{y=1}^{Y} \left\{ \frac{\Delta NB_{ky(m-n)}}{\left[1+0.01r\right]^{y-1}} \right\} \qquad \text{…….3.96}$$

Where $\Delta NB_{ky(m-n)}$ = the net economic benefit of alternative m relative to alternative n in year y;

r = the annual discount rate

Y = Analysis period in years

The internal economic rate of return (r) in percent is the discount rate at which the net present value equals to zero, i.e.;

$$\sum_{y=1}^{r} \left\{ \frac{\Delta NB_{ky(m-n)}}{\left[1+0.01r\right]^{y-1}} \right\} = 0 \qquad \text{…….3.97}$$

CHAPTER FOUR

ANALYSIS AND DISCUSSION OF RESULT

4.1 Description of Data

The unit costs used in the economic analysis have been obtained from the World Bank's road works data base known as 'Road Cost Knowledge System' (ROCKS), which has been developed by the World Bank's Transport Unit, primarily based on the experience of Bank staff and the information contained in roads and highways projects in developing countries. The main objective of the system is to develop an international knowledge system on road work costs –to be used primarily in developing countries– to establish an institutional memory, and obtain average and range unit costs based on historical data that could ultimately improve the reliability of new cost estimates and reduce the risks generated by cost overruns. The reason being to avoid the effect of inflation and instability of exchange rates, and unreliable information regarding contract prices.

The use of the HDM III Model in this work has been greatly simplified by the World Bank's HDM Manager - a user friendly shell environment. The HDM Manager stores the input data efficiently, creates all the required HDM input files, runs the HDM program, collect the results and presents the result in a practical way.

4.2 Results

The tables and figures in this section are meant to give the summary of the result of the various strategies considered in this work.

4.2.1 The base (Do nothing) strategy

The results of the first strategy which is the base or do nothing strategy are shown in Tables 4.1 and 4.2 and also in Figures 4.1 and 4.2:

TABLE 4.1: DETERIORATION FACTORS FOR THE DO NOTHING (BASE) STRATEGY

Year	Roughness (IRI) m/km	All Crack Area %	Wide Crack Area %	Pot hole Area %	Rut Depth mm	Modified Struc No	2-Way ADT (vpd)	Annual 2-Way ESA (000)
1	10.8	70.1	44.6	11.1	16.6	2.1	1375.0	572.0
2	11.5	87.4	75.9	12.5	26.2	2.1	1423.0	589.3
3	11.5	85.6	85.6	14.4	27.2	2.1	1474.0	607.0
4	11.5	83.4	83.4	16.5	27.7	2.1	1527.0	625.3
5	11.5	80.9	80.9	19.1	28.2	2.1	1581.0	644.2
6	11.5	78.0	78.0	22.0	28.6	2.1	1637.0	663.6
7	11.5	74.6	74.6	25.4	28.8	2.1	1696.0	683.6
8	11.5	70.6	70.6	29.4	28.8	2.1	1756.0	704.2
9	11.5	70.0	70.0	30.0	28.8	2.1	1819.0	725.5
10	11.5	70.0	70.0	30.0	28.8	2.1	1884.0	747.4
11	11.5	70.0	70.0	30.0	28.8	2.1	1952.0	769.9
12	11.5	70.0	70.0	30.0	28.8	2.1	2022.0	793.1
13	11.5	70.0	70.0	30.0	28.8	2.1	2094.0	817.1
14	11.5	70.0	70.0	30.0	28.8	2.1	2169.0	841.7
15	11.5	70.0	70.0	30.0	28.8	2.1	2247.0	867.1
16	11.5	70.0	70.0	30.0	28.8	2.1	2337.0	901.8
17	11.5	70.0	70.0	30.0	28.8	2.1	2430.0	937.9
18	11.5	70.0	70.0	30.0	28.8	2.1	2528.0	975.4
19	11.5	70.0	70.0	30.0	28.8	2.1	2629.0	1014.4
20	11.5	70.0	70.0	30.0	28.8	2.1	2734.0	1055.0

The table above shows that the all cracking and wide cracking areas reached their threshold values in the fifth year, rut depth also reached the maximum value of (28.8 mm) in the third year . Roughness reached it's threshold (11.5 m/km) in the second year of the analysis period. All these parameters indicate that the road has reached the poorest condition that could be assessed by the model.

TABLE 4.2: ECONOMIC COSTS FOR THE DO NOTHING (BASE) STRATEGY

YEAR		ADT	OPER	IRI	ECONOMIC AGENCY COSTS	ECONOMIC USER COSTS	ECONOMIC TOTAL COSTS
1995	0	1375		10.8	3.2	3844.240	3847.464
1996	1	1423		11.5	3.2	4355.793	4359.017
1997	2	1474		11.5	3.2	4565.382	4568.605
1998	3	1527		11.5	3.2	4715.061	4718.285
1999	4	1581		11.5	3.2	4869.738	4872.962
2000	5	1637		11.5	3.2	5029.585	5032.809
2001	6	1696		11.5	3.2	5194.778	5198.002
2002	7	1756		11.5	3.2	5365.499	5368.723
2003	8	1819		11.5	3.2	5541.937	5545.161
2004	9	1884		11.5	3.2	5724.286	5727.510
2005	10	1952		11.5	3.2	5912.750	5915.974
2006	11	2022		11.5	3.2	6107.537	6110.761
2007	12	2094		11.5	3.2	6308.864	6312.088
2008	13	2169		11.5	3.2	6516.954	6520.178
2009	14	2247		11.5	3.2	6732.041	6735.265
2010	15	2337		11.5	3.2	7001.322	7004.546
2011	16	2430		11.5	3.2	7281.376	7284.599
2012	17	2528		11.5	3.2	7572.630	7575.854
2013	18	2629		11.5	3.2	7875.535	7878.759
2014	19	2734		11.5	3.2	8190.557	8193.781

The economic performance of the do nothing strategy is as bad as the service performance. Both the user and total costs rise steadily at the same rate from about ₦3b at the first year of the analysis period to about ₦8b at the end of twenty years period.

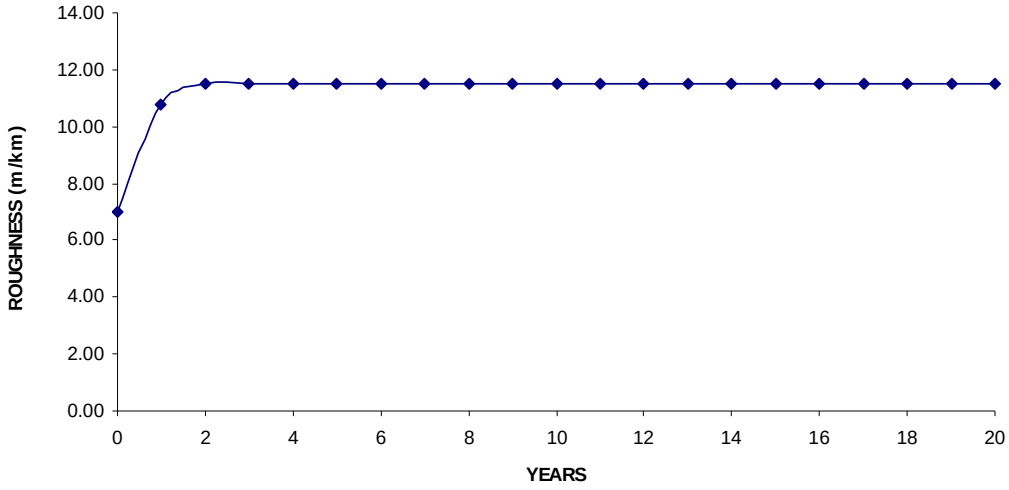


FIGURE 4.1: ROUGHNESS PLOT SHOWING DETERIORATION OF THE DO NOTHING ALTERNATIVE

4.2.2 Second (200mm thickness) strategy

This strategy involves the application of 200mm thick bituminous concrete on the existing surface of the pavement. Tables 4.3 and 4.4 below give the deterioration factors and the economic implication s of adopting this strategy, on a year by year basis; while the Figures 4.2- 4.5 and 4.6, show the roughness progression, IRR determination, the project's pay back period, project balance and the PSI-ESAL plots for this strategy.

Table 4.3: Deterioration factors for the 200mm strategy

Year	Operation Applied	Roughness (IRI) m/km	All Crack Area %	Wide Crack Area %	Pot hole Area %	Rut Depth mm	Modified Struct No.	2-Way ADT (vpd)	Annual 2- Way ESA (000)
1	OVER	2.0	0.0	0.0	0.0	2.5	5.9	1375.0	572.0
2		2.3	0.0	0.0	0.0	3.8	5.9	1423.0	589.3
3		2.6	0.0	0.0	0.0	4.9	5.9	1474.0	607.0
4		2.8	0.0	0.0	0.0	5.8	5.9	1527.0	625.3
5		3.1	0.0	0.0	0.0	6.5	5.9	1581.0	644.2
6		3.4	0.0	0.0	0.0	7.2	5.9	1637.0	663.6
7		3.6	0.0	0.0	0.0	7.9	5.9	1696.0	683.6
8		3.9	0.0	0.0	0.0	8.5	5.9	1756.0	704.2
9		4.2	0.0	0.0	0.0	9.1	5.9	1819.0	725.5
10		4.5	0.0	0.0	0.0	9.6	5.9	1884.0	747.4
11		4.8	0.0	0.0	0.0	10.1	5.9	1952.0	769.9
12		5.2	2.0	0.0	0.0	10.9	5.9	2022.0	793.1
13		5.7	6.9	0.0	0.0	12.1	5.9	2094.0	817.1
14	OVER	2.0	0.0	0.0	0.0	2.2	9.7	2169.0	841.7
15		2.3	0.0	0.0	0.0	3.3	9.7	2247.0	867.1
16		2.6	0.0	0.0	0.0	4.1	9.7	2337.0	901.8
17		2.8	0.0	0.0	0.0	4.9	9.7	2430.0	937.9
18		3.1	0.0	0.0	0.0	5.5	9.7	2528.0	975.4
19		3.3	0.0	0.0	0.0	6.1	9.7	2629.0	1014.4
20		3.6	0.0	0.0	0.0	6.6	9.7	2734.0	1055.0

Table 4.3 shows the deterioration factors on a year-by-year basis. Overlay was applied at the first year bringing down the roughness from 7.0 m/km to 2.0 m/km, and all-cracking and wide cracking to zero percent, the modified structural number to about 6.0 and rut depth to 2.5 mm. Wide cracking and potholing were never allowed to start. The roughness had a steady rise from 2.0 to 5.7 m/km in the thirteenth year, then dropped to 2.0 m/km after the application of the overlay (200 mm) and terminated at 3.6 m/km in the twentieth year.

Table 4.4: Economic Performance for 200mm Strategy

Year	Economic Agency Capital	Economic Agency Recurrent	Economic Vehicle Operation	Economic Travel Time	Economic Society Total	Net Economic Benefits
1	3495.240	2.531	2913.929	381.133	6792.833	-3495.007
2	0.000	2.531	2372.925	357.426	2732.882	1003.418
3	0.000	2.531	2455.067	370.832	2828.430	1087.517
4	0.000	2.531	2539.760	384.790	2927.081	1117.163
5	0.000	2.531	2627.404	399.338	3029.273	1147.552
6	0.000	2.531	2734.973	414.515	3152.019	1161.817
7	0.000	2.531	2849.877	430.369	3282.777	1172.653
8	0.000	2.531	2970.322	446.952	3419.805	1181.958
9	0.000	2.531	3096.838	464.325	3563.694	1189.301
10	0.000	2.531	3229.980	482.560	3715.071	1194.223
11	0.000	2.531	3370.344	501.739	3874.613	1196.221
12	0.000	2.531	3524.206	522.196	4048.932	1188.863
13	0.000	2.531	3704.149	544.750	4251.430	1158.931
14	3495.240	2.531	3938.715	571.785	8008.271	-2419.546
15	0.000	2.531	3577.233	574.117	4153.880	1619.204
16	0.000	2.531	3729.707	597.363	4329.600	1674.297
17	0.000	2.531	3888.087	621.614	4512.231	1731.711
18	0.000	2.531	4053.118	646.935	4702.584	1791.005
19	0.000	2.531	4248.931	673.396	4924.858	1828.364
20	0.000	2.531	4460.172	701.077	5163.780	1859.461

Currency in Million Naira

Table 4.4: shows the various economic costs and benefits associated with this strategy (200 mm). The total society cost dropped sharply from ₦6.8b in the second year due to application of the first overlay to ₦2.7b and then rose gradually as a result of continuous deterioration to ₦8.0Ib in the fourteenth year when the roughness reached about 6.0 m/km. The net benefit started with a negative value due to the heavy investment on the overlay at the beginning and continue to rise positively until the next overlay application when it became negative once more, rise again and ended again with a negative value (-₦2.4b) due to

another application of overlay at that terminal year. This has made the 200 *mm* strategy to have the highest salvage value. The vehicle operating cost and the total society costs both have gradual decrease in between the negative values of the net economic benefit.

Table 4.5 gives a summary of the economic performance summary for the 200mm strategy.

TABLE 4.5: SUMMARY OF ECONOMIC VALUES FOR THE 200mm STRATEGY

ECONOMIC PERFORMANCE VALUE	VALUE
Project NPV at 12.00 % Discount Rate:	4951.58
Capital Recovery Cost	662.91
Project Internal Rate of Return (%):	31.0%
Modified Internal Rate of return (MIRR)	16.7%
Benefit Cost Ratio BCR	4.5
PAVEMENT PERFORMANCE QUALITY INDEX	1.2

The Internal Rate of Return of 31.0% which is greater than the discount rate used in the calculation of NPV indicates that the strategy is worthwhile. With modified internal rate of return of 16.7% means that after the initial investment, the proceeds could be reinvested to yield a return of 16.7%. A Benefit - Cost ratio of 4.5 which is greater than 1.0 means that the project is worthwhile and attractive.

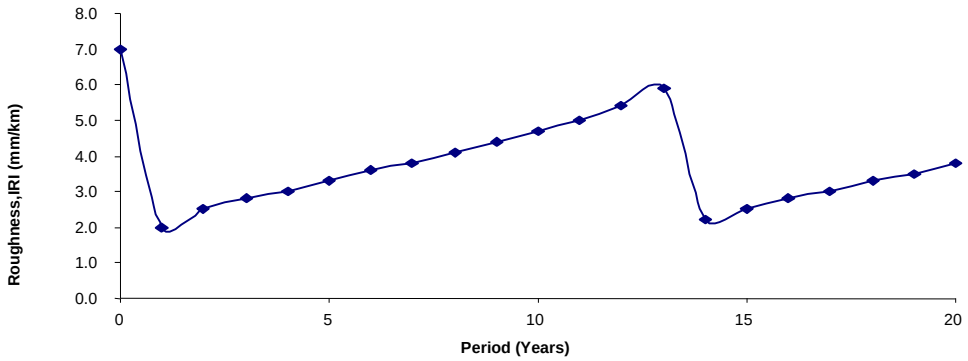


Fig: 4. 2 Deterioration Profile for 200mm Strategy

Figure 4.2 shows the variation of roughness as a measure of deterioration with time period in years. The roughness progression is however restricted to a maximum value of 6.0 m/km for economic reasons. The terminal value of the roughness for this strategy is about 3.0m/km, showing that the option has some residual value.

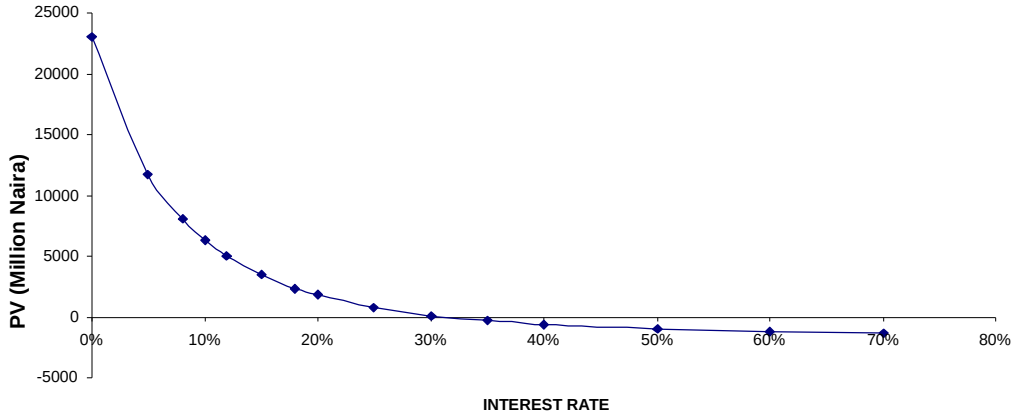


Fig: 4. 3 PLOT OF PRESENT VALUE Versus INTEREST RATE FOR 200mm STRATEGY

Figure 4.3 shows the plots of Net Present Values of the 200 mm design strategy at various interest rates. The point where the curve cuts the interest rate axis gives the Internal rate of return for that strategy, in this case it is at 31.0%.

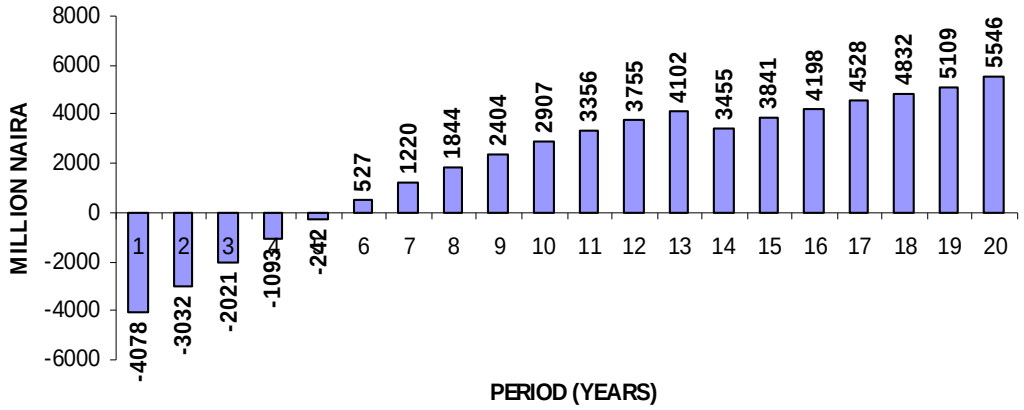


Fig: 4.4: DISCOUNTED PAYBACK PERIOD FOR 200m m STRATEGY

Figure 4.4 shows that the payout period for the 200mm strategy is five and a half (5.5 years).

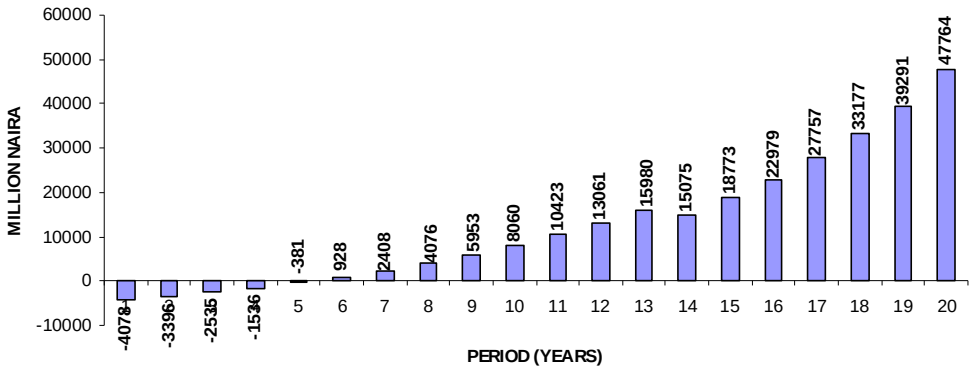


Fig: 4.5: PROJECT BALANCE FOR 200mm STATEGY

Figure 4.5 shows the variation of the initial capital tied-up to the project during the design period.

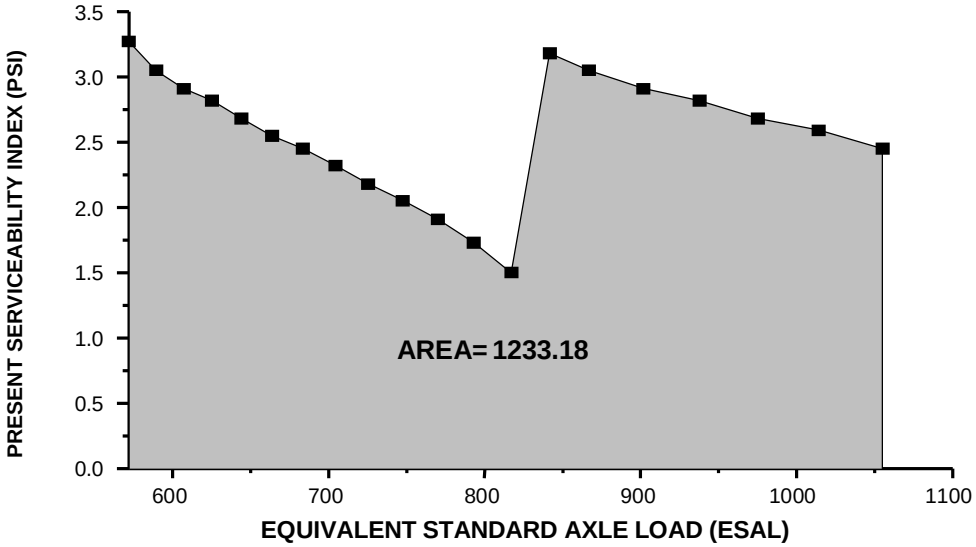


FIGURE 4.6 PSI-ESAL PLOT FOR 200mm STRATEGY

The area under the curve represents the PSI-ESAL capacity for the 200mm strategy and equals to the integral $\int_0^{(ESAL)n} (PSI)_t \, d(ESAL)$ = 1233.18. The pavement performance quality index (PPQI) which is more appropriate measure of economic performance is $\text{PPQI} = \frac{\text{shaded area}}{(\text{ESAL})_n} = \frac{1233.18}{1055} =$ **1.169**

4.2.3 Third (150mm thickness) strategy

This strategy involves the application of 150mm thick bituminous concrete on the existing surface of the pavement. Table4.6 and 4.7 give the deterioration factors and the economic implication s of adopting this strategy, on a year by year basis; while the Figures 4.6 - 4.11 show the roughness progression, IRR determination, the project's pay back period, project balance and the PSI-ESAL plots for this strategy.

Table 4.6: Deterioration factors for the 150 mm strategy

Yr	Operation Applied	Roughness (IRI) m/km	All Crack Area %	Wide Crack Area %	Pot hole Area %	Rut Depth mm	Modified Struct No.	2-Way ADT (vpd)	Annual 2-Way ESA (000)
1	OVER	2.0	0.0	0.0	0.0	2.5	3.5	1375.0	572.0
2		2.4	0.0	0.0	0.0	3.9	3.5	1423.0	589.3
3		2.7	0.0	0.0	0.0	5.1	3.5	1474.0	607.0
4		3.0	0.0	0.0	0.0	6.1	3.5	1527.0	625.3
5		3.4	0.0	0.0	0.0	6.9	3.5	1581.0	644.2
6		3.7	0.0	0.0	0.0	7.7	3.5	1637.0	663.6
7		4.1	0.0	0.0	0.0	8.5	3.5	1696.0	683.6
8		4.5	0.8	0.0	0.0	9.2	3.5	1756.0	704.2
9		4.9	3.6	0.0	0.0	10.1	3.5	1819.0	725.5
10		5.6	10.5	0.0	0.0	11.5	3.5	1884.0	747.4
11	OVER	2.0	0.0	0.0	0.0	2.1	4.9	1952.0	769.9
12		2.3	0.0	0.0	0.0	3.2	4.9	2022.0	793.1
13		2.6	0.0	0.0	0.0	4.1	4.9	2094.0	817.1
14		2.9	0.0	0.0	0.0	4.9	4.9	2169.0	841.7
15		3.2	0.0	0.0	0.0	5.6	4.9	2247.0	867.1
16		3.5	0.0	0.0	0.0	6.2	4.9	2337.0	901.8
17		3.8	0.0	0.0	0.0	6.7	4.9	2430.0	937.9
18		4.1	0.8	0.0	0.0	7.3	4.9	2528.0	975.4
19		4.5	3.6	0.0	0.0	8.0	4.9	2629.0	1014.4
20		5.1	10.5	0.0	0.0	9.2	4.9	2734.0	1055.0

Table 4.6 shows the deterioration factors on a year-by-year basis. Overlay was applied at the first year bringing down the roughness from 7.0 m/km to 2.0 m/km, and all-cracking and wide cracking to zero percent, the modified structural number to about 3.5 and rut depth to 2.5 mm. Wide cracking and potholing were never allowed to start. The roughness had a steady rise from 2.0 to 5.6 m/km in the tenth year, then dropped to 2.0 m/km after the application of the overlay (150 mm) and terminated at 5.1 in the twentieth year.

Table 4.7: Economic Performance for 150mm Strategy

Year	Economic Agency Capital	Economic Agency Recurrent	Economic Vehicle Operation	Economic Travel Time	Economic Society Total	Net Economic Benefits
1	2970.954	2.531	2913.929	381.133	6268.548	-2970.721
2	0.000	2.531	2373.516	357.441	2733.487	1002.812
3	0.000	2.531	2457.029	370.899	2830.459	1085.489
4	0.000	2.531	2543.422	384.952	2930.905	1113.339
5	0.000	2.531	2647.199	399.654	3049.385	1127.440
6	0.000	2.531	2766.248	415.074	3183.852	1129.984
7	0.000	2.531	2891.978	431.290	3325.799	1129.632
8	0.000	2.531	3026.399	448.439	3477.369	1124.394
9	0.000	2.531	3174.487	466.817	3643.835	1109.159
10	0.000	2.531	3348.337	487.220	3838.087	1071.207
11	2970.954	2.531	3577.430	512.125	7063.040	-1992.205
12	0.000	2.531	3253.478	514.502	3770.511	1467.284
13	0.000	2.531	3367.215	533.923	3903.670	1506.691
14	0.000	2.531	3484.583	554.164	4041.278	1547.447
15	0.000	2.531	3610.173	575.284	4187.988	1585.096
16	0.000	2.531	3793.648	598.928	4395.107	1608.790
17	0.000	2.531	3987.291	623.714	4613.536	1630.406
18	0.000	2.531	4194.122	649.794	4846.446	1647.143
19	0.000	2.531	4421.966	677.524	5102.021	1651.202
20	0.000	2.531	4690.308	707.935	5400.773	1622.467

Table 4.7: shows the various economic costs and benefits associated with this strategy (150 mm). The total society cost dropped sharply from ₦6.2b in the second year due to application of the first overlay to ₦2.7b and then rose gradually as a result of continuous deterioration to ₦7.0b in the ninth year when the roughness reached about 5.6 m/km. The net benefit started with a negative value due to the heavy investment on the overlay at the beginning and continued to rise gradually until the next overlay application when it became negative once more, rise again and ended with again a negative value (-₦2.0b) due to another application of overlay in the eleventh year. This has made the 150 *mm* strategy to have some salvage value of . The vehicle operating cost and the total society costs both have gradual decrease in between the negative values of the net economic benefit.

Table 4.8 shows the economic performance summary for the 150mm strategy.

Table 4.8: Economic Values for the 150mm Strategy

ECONOMIC PERFORMANCE VALUE	VALUE
Project NPV at 12.00 % Discount Rate:	5108.61
Project Internal Rate of Return (%):	35.2%
Benefit Cost Ratio (BCR)	2.25
Modified Internal Rate of return (MIRR)	15.9%
Capital Recovery Cost	683.93
Pavement Performance Quality Index (PPQI)	1.128

The Internal Rate of Return of 35.2% which is greater than the discount rate used in the calculation of NPV indicates that the strategy is worthwhile. With modified internal rate of return of 15.9% means that after the initial investment , the proceeds could be reinvested to yield a return of 15.9%. A Benefit - Cost ratio of 2.25 which is greater than 1.0 means that the project is worthwhile and attractive.

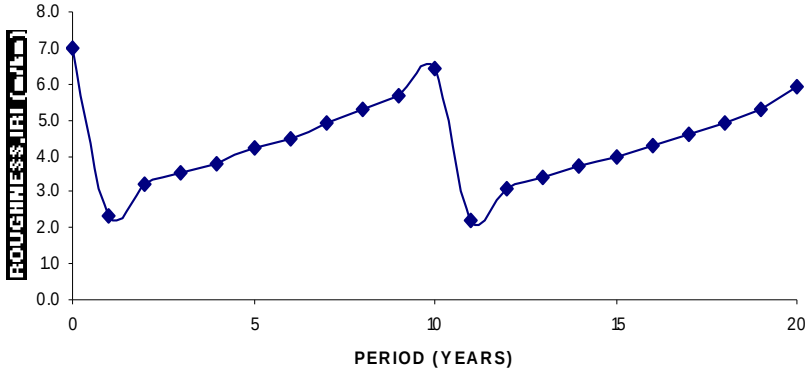


Fig: 4.7 DETERIORATION PROFILE FOR 150mm STRATEGY

Figure 4.7 shows the variation of roughness as a measure of deterioration with time period in years. The roughness progression is however restricted to a maximum value of 6.0 m/km for economic reasons. The terminal value of the roughness for this strategy is precisely 6.0m/km, showing that the option has no residual value.

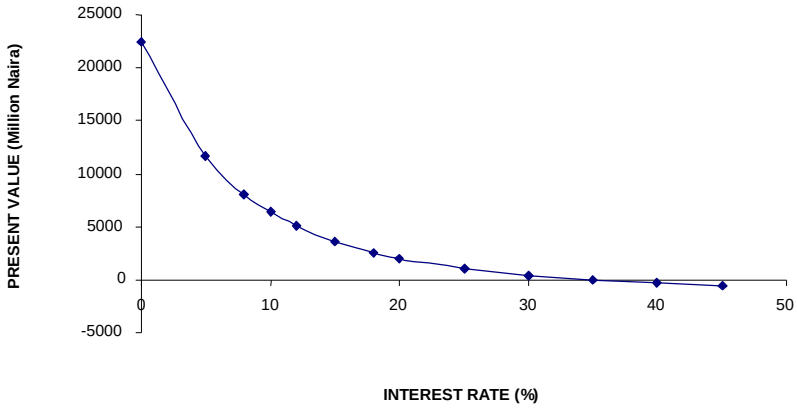


Fig: 4. 8 PLOT OF PRESENT VALUE AND INTEREST RATE FOR 150mm STRATEGY

Figure 4.8 shows the plots of Net Present Values of the 150 mm design strategy at various interest rates. The point where the curve cuts the interest rate axis gives the internal rate of return for that strategy, in this case it is at 36.0%.

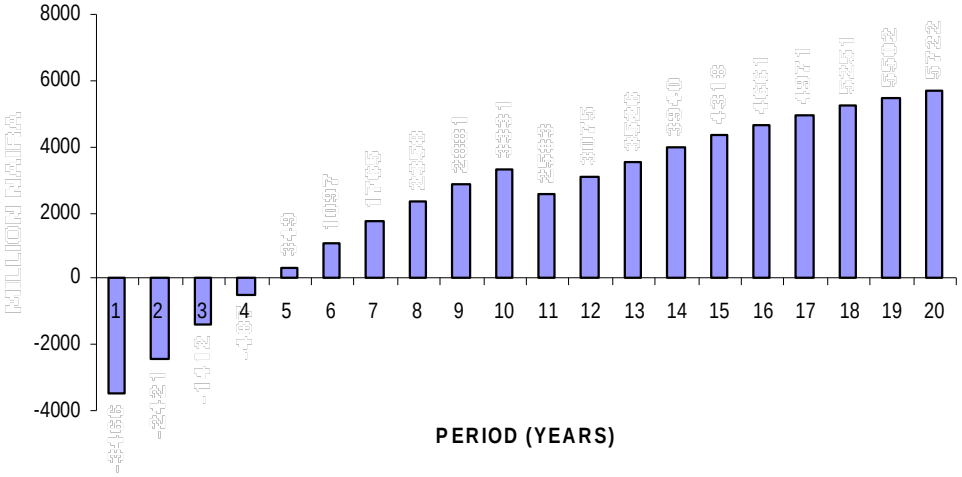


Fig: 4.9 DISCOUNTED PAYBACK PERIOD FOR 150mm

Figure above shows that the payout period for the 150mm strategy is five and a half (4.5 years).

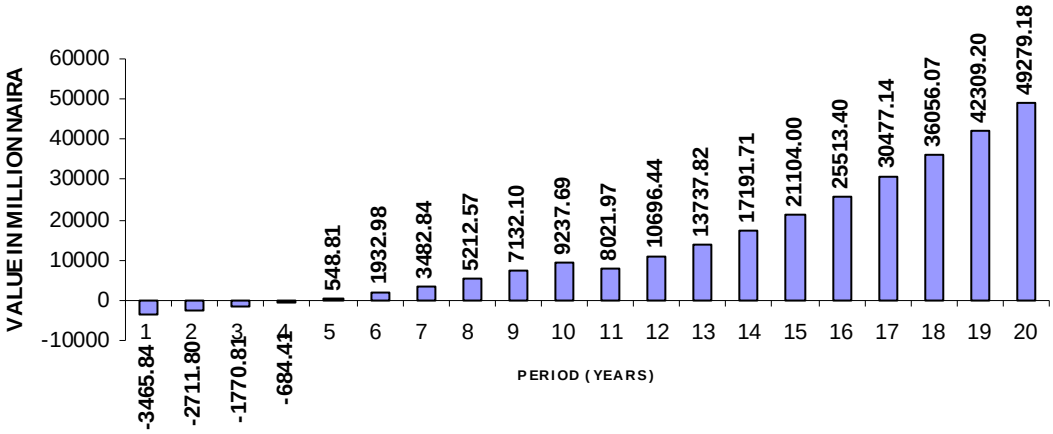


Fig: 4.10 PROJECT BALANCE FOR 150mm STRATEGY

Figure 4.10 shows the variation of the initial capital tied-up to the project during the design period.

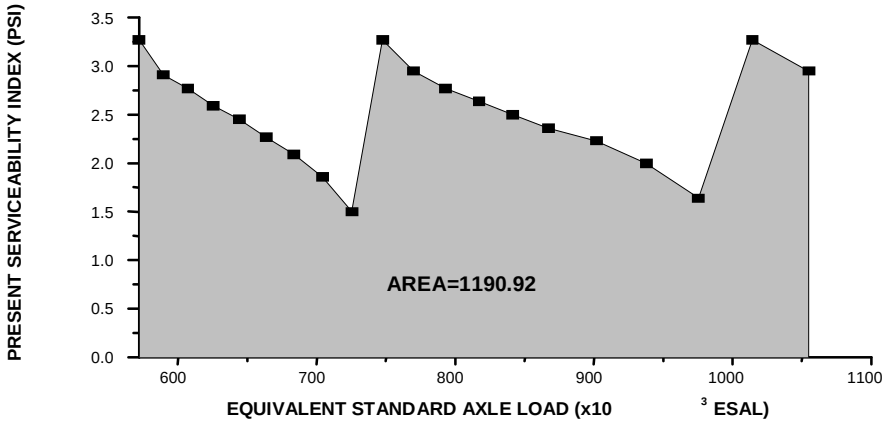


Fig 4.11: PSI-ESAL PLOT FOR 150mm STRATEGY

The area under the curve represents the PSI-ESAL capacity for the 150mm strategy and equals to the integral $\int_0^{(ESAL)n} (PSI)_t \, d(ESAL)$ = 1190.12.

The pavement performance quality index (PPQI) which is more appropriate measure of economic performance is $\text{PPQI} = \frac{\text{shaded area}}{(\text{ESAL})_n} = \frac{1190.12}{1055} = \mathbf{1.128}$

4.2.4 Fourth (100mm thickness) strategy

This strategy involves the application of 150mm thick bituminous concrete on the existing surface of the pavement. Tables 4.9 - 4.11 give the deterioration factors and the economic implication s of adopting this strategy, on a year by year basis; while the Figures 4.13 - 18, show the roughness progression, IRR determination, the project's pay back period, project balance and the PSI-ESAL plots for this strategy.

Table 4.9: Deterioration Factors for 100mm Strategy

Year	Operation Applied	Roughness (IRI) m/km	All Crack Area %	Wide Crack Area %	Pot hole Area %	Rut Depth mm	Modified Struc No	2-Way ADT (vpd)	Annual 2-Way ESA (000)
1	OVER	2.0	0.0	0.0	0.0	2.5	3.3	1375.0	572.0
2		2.4	0.0	0.0	0.0	4.0	3.3	1423.0	589.3
3		2.7	0.0	0.0	0.0	5.1	3.3	1474.0	607.0
4		3.1	0.0	0.0	0.0	6.1	3.3	1527.0	625.3
5		3.4	0.0	0.0	0.0	7.0	3.3	1581.0	644.2
6		3.8	0.0	0.0	0.0	7.8	3.3	1637.0	663.6
7		4.2	2.0	0.0	0.0	8.7	3.3	1696.0	683.6
8		4.7	6.9	0.0	0.0	9.7	3.3	1756.0	704.2
9		5.5	17.5	0.0	0.0	11.3	3.3	1819.0	725.5
10	OVER	2.0	0.0	0.0	0.0	2.1	4.6	1884.0	747.4
11		2.3	0.0	0.0	0.0	3.3	4.6	1952.0	769.9
12		2.7	0.0	0.0	0.0	4.3	4.6	2022.0	793.1
13		3.0	0.0	0.0	0.0	5.0	4.6	2094.0	817.1
14		3.3	0.0	0.0	0.0	5.7	4.6	2169.0	841.7
15		3.6	0.0	0.0	0.0	6.4	4.6	2247.0	867.1
16		3.9	2.0	0.0	0.0	7.1	4.6	2337.0	901.8
17		4.4	6.9	0.0	0.0	8.0	4.6	2430.0	937.9
18		5.2	17.5	0.0	0.0	9.4	4.6	2528.0	975.4
19	OVER	2.0	0.0	0.0	0.0	1.9	5.9	2629.0	1014.4
20		2.3	0.0	0.0	0.0	2.9	5.9	2734.0	1055.0

Table 4.9 shows the deterioration factors on a year-by-year basis. Overlay was applied at the first year bringing down the roughness from 7.0 m/km to 2.0 m/km, All cracking and wide cracking to zero percent, the modified structural number to about 3.3 and rut depth to 2.5 mm. Wide cracking and potholing were never allowed to start. The roughness had a steady rise from 2.0 to 5.5 m/km in the ninth year, then dropped to 2.0 m/km after the application of the overlay (100 mm) and terminated at 3.6 in the twentieth year.

Table 4.10: Economic Performance for 100mm Strategy

Year	Economic Agency Capital	Economic Agency Recurrent	Economic Vehicle Operation	Economic Travel Time	Economic Society Total	Net Economic Benefits
1	1922.382	2.531	2913.929	381.133	5219.976	-1922.149
2	0.000	2.531	2373.634	357.444	2733.609	1002.691
3	0.000	2.531	2457.422	370.913	2830.866	1085.081
4	0.000	2.531	2544.162	384.987	2931.680	1112.564
5	0.000	2.531	2651.670	399.725	3053.926	1122.899
6	0.000	2.531	2772.610	415.202	3190.343	1123.494
7	0.000	2.531	2902.857	431.566	3336.953	1118.477
8	0.000	2.531	3048.742	449.150	3500.422	1101.340
9	0.000	2.531	3224.458	468.861	3695.851	1057.144
10	1922.382	2.531	3466.366	493.785	5885.064	-975.770
11	0.000	2.531	3152.517	496.064	3651.111	1419.723
12	0.000	2.531	3263.309	514.805	3780.644	1457.150
13	0.000	2.531	3377.657	534.351	3914.538	1495.823
14	0.000	2.531	3506.915	554.766	4064.211	1524.513
15	0.000	2.531	3660.130	576.122	4238.782	1534.302
16	0.000	2.531	3853.733	600.174	4456.438	1547.458
17	0.000	2.531	4071.478	625.887	4699.896	1544.046
18	0.000	2.531	4336.825	654.541	4993.896	1499.693
19	1922.382	2.531	4680.581	689.145	7294.639	-541.417
20	0.000	2.531	4352.760	698.514	5053.805	1969.436

Currency: Million Naira

Table 4.10: shows the various economic costs and benefits associated with this strategy (100 mm). The total society cost dropped sharply from ₦6.8b in the second year due to application of the first overlay to ₦2.7b and then rose gradually as a result of continuous deterioration too ₦8.0lb in the fourteenth year when the roughness reached about 6.0 m/km. The net benefit started with a negative value due to the heavy investment on the overlay at the beginning and continue to rise positively until the next

overlay application when it became negative once more, rise again and ended with again a negative value (-₦2.4b) due to another application of overlay at that terminal year. This has made the 100 *mm* strategy to have some salvage value. The vehicle operating cost and the total society costs both have gradual decrease in between the negative values of the net economic benefit.

Table 4.11 shows the economic performance summary for the 150mm strategy.

Table 4.11: Economic Performance Values for the 100mm Strategy

ECONOMIC PERFORMANCE VALUE	VALUE
Project NPV at 12.00 % Discount Rate:	6403.07
Project Internal Rate of Return (%):	54.7%
Modified Internal Rate of Return MIRR(%):	19.8%
Benefit Cost Ratio BCR :	3.30
Capital Recovery Cost	857.23
Pavement Performance Quality Index (PPQI)	1.162

The Internal Rate of Return of 54.7% which is greater than the discount rate used in the calculation of NPV indicates that the strategy is worthwhile. With modified internal rate of return of 19.8% means that after the initial investment , the proceeds could be reinvested to yield a return of 19.8%. A Benefit - Cost ratio of 4.5 which is greater than 1.0 means that the project is worthwhile and attractive.

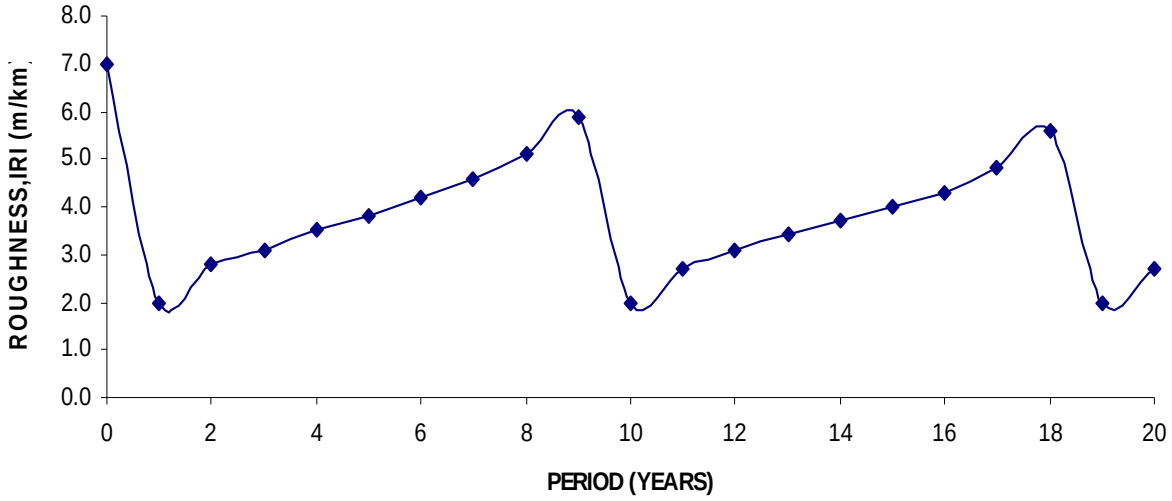


Fig: 4.12 DETERIOATION PROFILE FOR 100mm STRATEGY

Figure 4.16 shows the variation of roughness as a measure of deterioration with time period in years. The roughness progression is however restricted to a maximum value of 6.0 m/km for economic reasons. The terminal value of the roughness for this strategy is about 3.0m/km, showing that the option has some residual value.

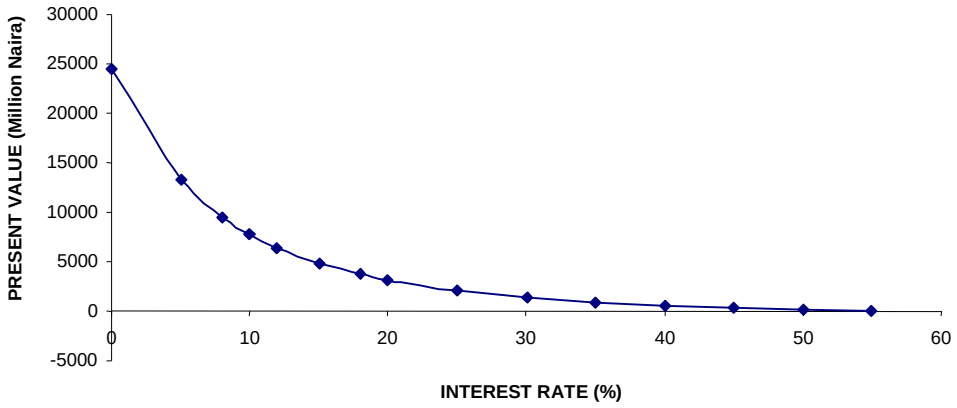


Fig: 4.13 PLOTS OF PRESENT VALUES AND INTEREST RATE FOR 100mm STRATEGY

Figure 4.17 shows the plots of Net Present Values of the 100 mm design strategy at various interest rates. The point where the curve cuts the interest rate axis gives the internal rate of return for that strategy, in this case it is at 55.5%.

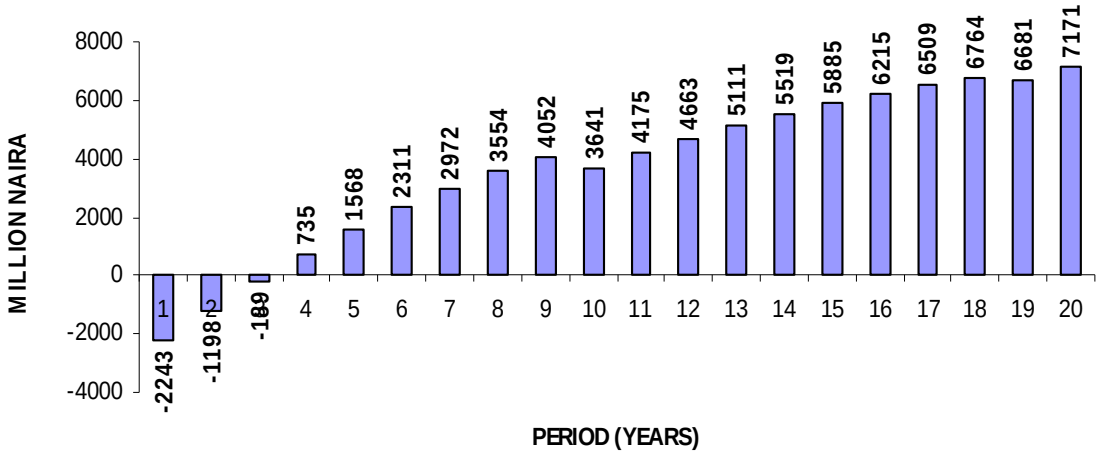


Fig: 4.14 DISCOUNTED PAYBACK PERIOD FOR 100mm STRATEGY

Figure 4.14 shows that the payout period for the 100mm strategy is five and a half (3.5 years).

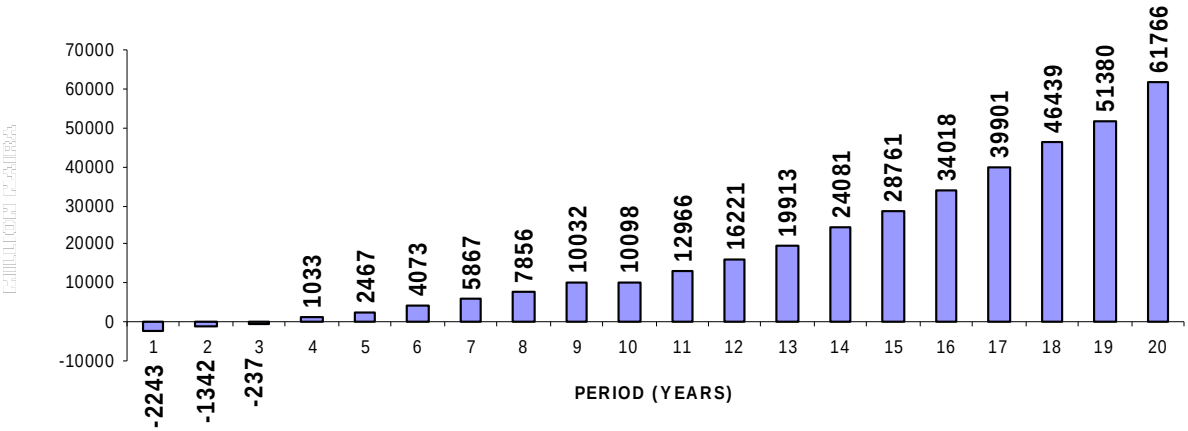


Fig: 4.15 PROJECT BALANCE FOR 100mm STRATEGY

Figure 4.15 shows the variation of the initial capital tied-up to the project during the design period.

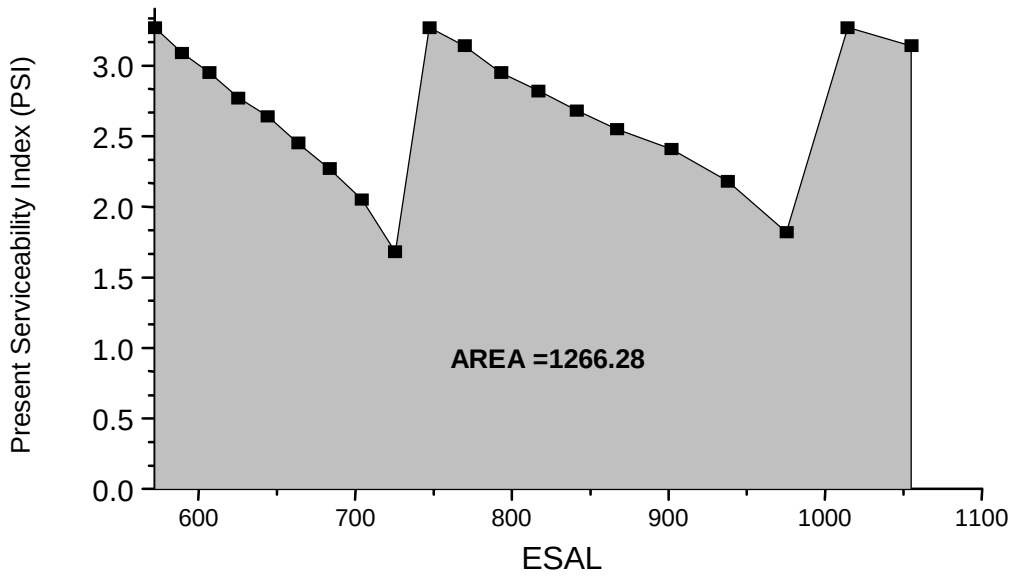


Figure 4.16 PSI-ESAL PLOT FOR 100mm STRATEGY

The area under the curve represents the PSI-ESAL capacity for the 200mm strategy and equals to the integral $\int_0^{(ESAL)n} (PSI)_t \, d(ESAL)$ **= 1266.28. The pavement performance quality index (PPQI) which is more appropriate measure of economic performance is** $PPQI = \frac{\text{shaded area}}{(\text{ESAL})_n} = \frac{1266.28}{1055} =$ **1.200**

4.2.5 Fifth (75mm thickness) strategy

This strategy involves the application of 75mm thick bituminous concrete on the existing surface of the pavement. Tables 4.11 - 4.13 give the deterioration factors and the economic implication s of adopting this strategy, on a year by year basis; while the Figures 4.19 - 4.23, show the roughness progression, IRR

determination, the project's pay back period, project balance and the PSI-ESAL plots for this strategy.

Table 4.12: Deterioration Factors for 75mm Strategy

Year	Operation Applied	Roughness (IRI) m/km	All Crack Area %	Wide Crack Area %	Pot hole Area %	Rut Depth mm	Modified Struc No	2-Way ADT (vpd)	Annual 2-Way ESA (000)
1	OVER	2.5	0.0	0.0	0.0	2.5	3.0	1375.0	572.0
2		2.9	0.0	0.0	0.0	4.0	3.0	1423.0	589.3
3		3.3	0.0	0.0	0.0	5.2	3.0	1474.0	607.0
4		3.8	0.0	0.0	0.0	6.2	3.0	1527.0	625.3
5		4.2	0.4	0.0	0.0	7.2	3.0	1581.0	644.2
6		4.7	2.4	0.0	0.0	8.1	3.0	1637.0	663.6
7		5.2	7.8	0.0	0.0	9.1	3.0	1696.0	683.6
8	OVER	2.5	0.0	0.0	0.0	1.6	4.0	1756.0	704.2
9		2.8	0.0	0.0	0.0	2.5	4.0	1819.0	725.5
10		3.1	0.0	0.0	0.0	3.2	4.0	1884.0	747.4
11		3.4	0.0	0.0	0.0	3.8	4.0	1952.0	769.9
12		3.7	0.2	0.0	0.0	4.4	4.0	2022.0	793.1
13		4.0	2.2	0.0	0.0	4.9	4.0	2094.0	817.1
14		4.4	7.3	0.0	0.0	5.6	4.0	2169.0	841.7
15		5.1	18.5	0.0	0.0	6.6	4.0	2247.0	867.1
16	OVER	2.5	0.0	0.0	0.0	1.3	4.9	2337.0	901.8
17		2.8	0.0	0.0	0.0	2.0	4.9	2430.0	937.9
18		3.1	0.0	0.0	0.0	2.5	4.9	2528.0	975.4
19		3.3	0.0	0.0	0.0	3.0	4.9	2629.0	1014.4
20		3.6	0.0	0.0	0.0	3.4	4.9	2734.0	1055.0

Table 4.12 shows the deterioration factors on a year-by-year basis. Overlay was applied at the first year bringing down the roughness from 7.0 m/km to 2.5 m/km, and all-cracking and wide cracking to zero percent, raised the modified structural number to about 3.0 and rut depth to 2.5 mm. Wide cracking and potholing were never allowed to start. The roughness had a steady rise from 2.5 mm/km to 5.2 m/km in the seventh year, then dropped to 2.5 m/km after the application of the second overlay, rose again to 5.1mm/km and dropped to 2.5mm/km and terminated at 3.6 in the twentieth year.

Table 4.13: Economic Performance for 75mm Strategy

Year	Economic Agency Capital	Economic Agency Recurrent	Economic Vehicle Operation	Economic Travel Time	Economic Society Total	Net Economic Benefits
1	1048.572	2.531	2913.929	381.133	4346.166	-1048.339
2	0.000	2.531	2384.916	357.818	2745.264	991.036
3	0.000	2.531	2479.997	371.534	2854.063	1061.885
4	0.000	2.531	2598.374	385.962	2986.867	1057.376
5	0.000	2.531	2723.561	401.211	3127.302	1049.522
6	0.000	2.531	2858.445	417.464	3278.440	1035.397
7	0.000	2.531	3009.239	435.133	3446.903	1008.527
8	1048.572	2.531	3190.175	455.308	4696.586	-94.823
9	0.000	2.531	2972.161	461.575	3436.267	1316.728
10	0.000	2.531	3076.005	479.110	3557.646	1351.648
11	0.000	2.531	3204.307	497.419	3704.256	1366.579
12	0.000	2.531	3342.876	516.573	3861.979	1375.816
13	0.000	2.531	3490.646	536.703	4029.880	1380.481
14	0.000	2.531	3653.585	558.089	4214.205	1374.520
15	0.000	2.531	3844.785	581.459	4428.774	1344.310
16	1048.572	2.531	4127.281	610.748	5789.132	214.764
17	0.000	2.531	3886.172	621.533	4510.236	1733.706
18	0.000	2.531	4052.099	646.881	4701.511	1792.078
19	0.000	2.531	4247.445	673.372	4923.348	1829.874
20	0.000	2.531	4461.057	701.095	5164.683	1858.558

Currency: Million Naira

Table 4.13: above shows the various economic costs and benefits associated with this strategy (75 mm). The total society cost dropped sharply from ₦4.3b in the second year due to application of the first overlay to ₦2.7b and then rose gradually as a result of continuous deterioration to ₦4.7b in the eighth year when the roughness reached about 6.0 m/km. The net benefit started with a negative value due to the heavy investment on the overlay at the beginning and continues to rise positively until the next overlay application when it became negative once more, rise again and ended with value of ₦2.1b due to another application of overlay in the

sixteenth year. This has made the 75 *mm* strategy to have some salvage value. The vehicle operating cost and the total society costs both have gradual decrease in between the low values of the net economic benefit.

Table 4.14 gives summary the economic performance for the 75mm strategy.

Table 4.14: Economic Performance Values for the 75mm Strategy

ECONOMIC PERFORMANCE VALUES	VALUE
Project NPV at 12.00 % Discount Rate:	7339.91
Project Internal Rate of Return IRR(%):	97.0%
Modified Internal Rate of Return MIRR(%):	24%
Benefit Cost Ratio (BCR)	5.2
Capital Recovery Cost	982.66
Pavement Performance Quality Index (PPQI)	1.119

The Internal Rate of Return of 97.0% which is far greater than the discount rate used in the calculation of NPV indicates that the strategy is worthwhile and attractive. With modified internal rate of return of 24.0% means that after the initial investment, the proceeds could be reinvested to yield a return of 24.0%. A Benefit - Cost ratio of 5.2 which is greater than 1.0 shows that the project is worthwhile and attractive.

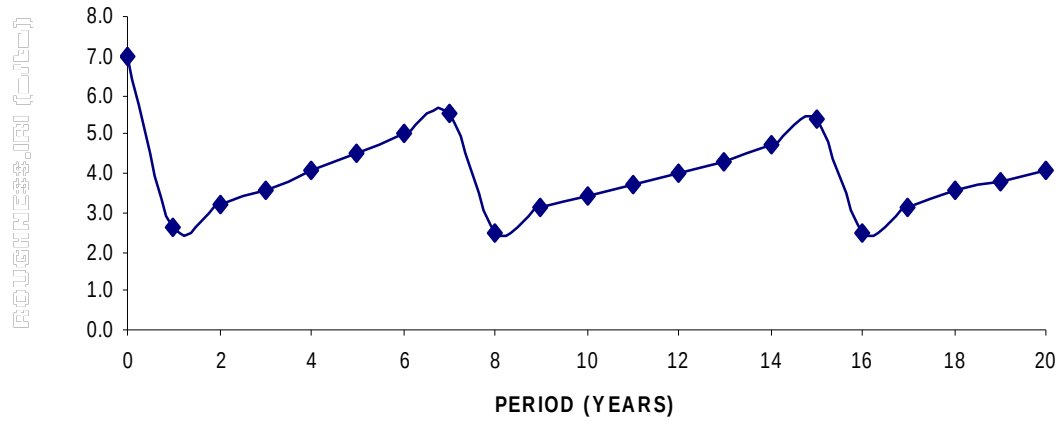


Fig: 4.17 DETERIORATION PROFILE FOR 75mm STRATEGY

Figure 4.17 shows the variation of roughness as a measure of deterioration with time period in years. The roughness progression is however restricted to a maximum value of 6.0 m/km for economic reasons. The terminal value of the roughness for this strategy is about 3.8m/km, showing that the option has some residual value.

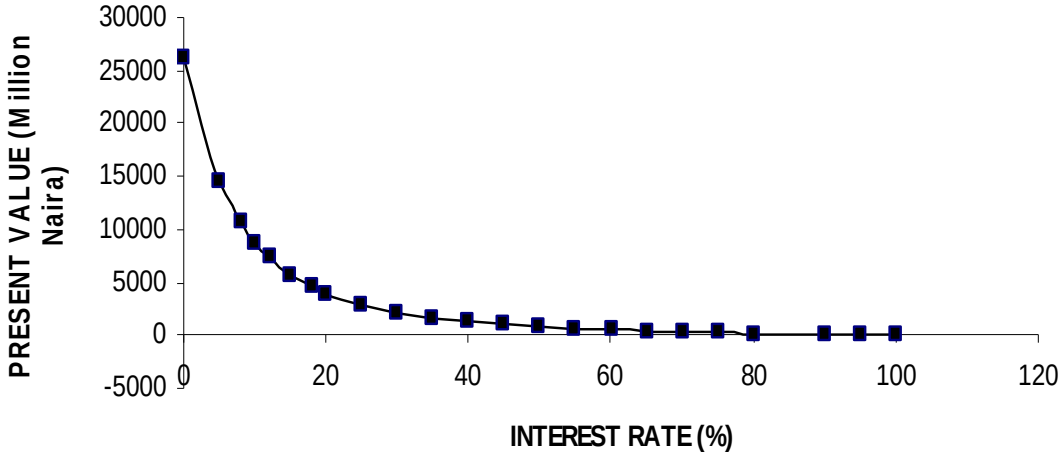


Fig: 4.18 PLOT OF PRESENT VALUE AND INTEREST FOR 75mm STRATEGY

Figure 4.18 shows the plots of Net Present Values for the 75 mm design strategy at various interest rates. The point where the curve cuts the interest rate axis gives the Internal rate of return for that strategy, in this case it is at 97.0%.

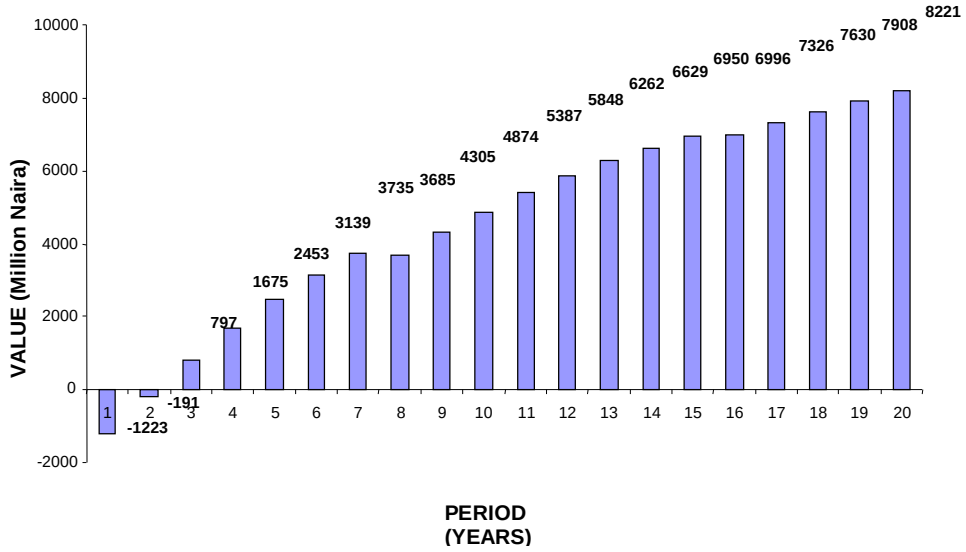


Fig: 4. 19 DISCOUNTED PAYBACK FOR 75mm STRATEGY

Figure 4.19 shows that the payout period for the 75mm strategy is two and a half (2.5 years).

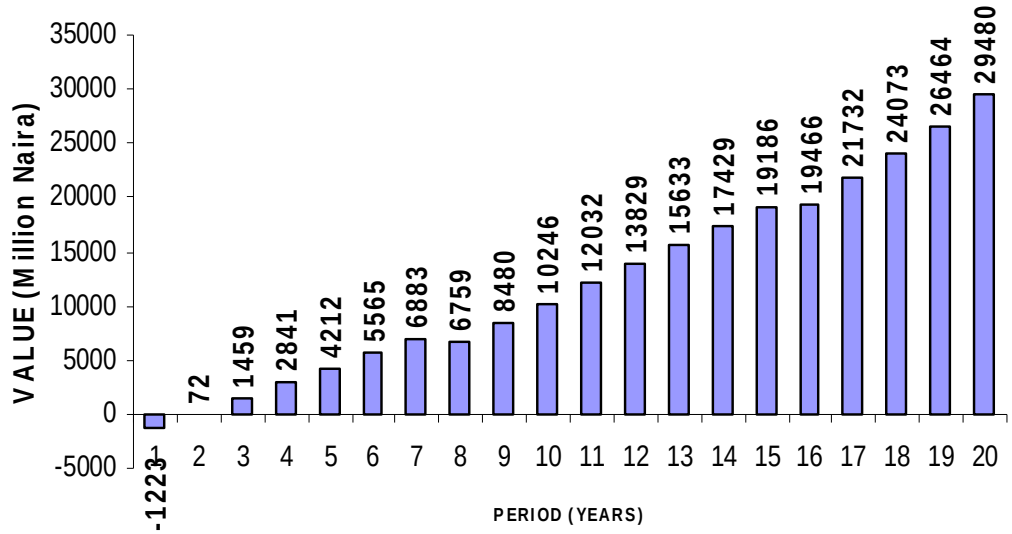


Fig: 4.20 PROJECT BALANCE FOR 75mm STRATEGY

Figure 4.20 shows the variation of the initial capital tied-up to the project during the design period.

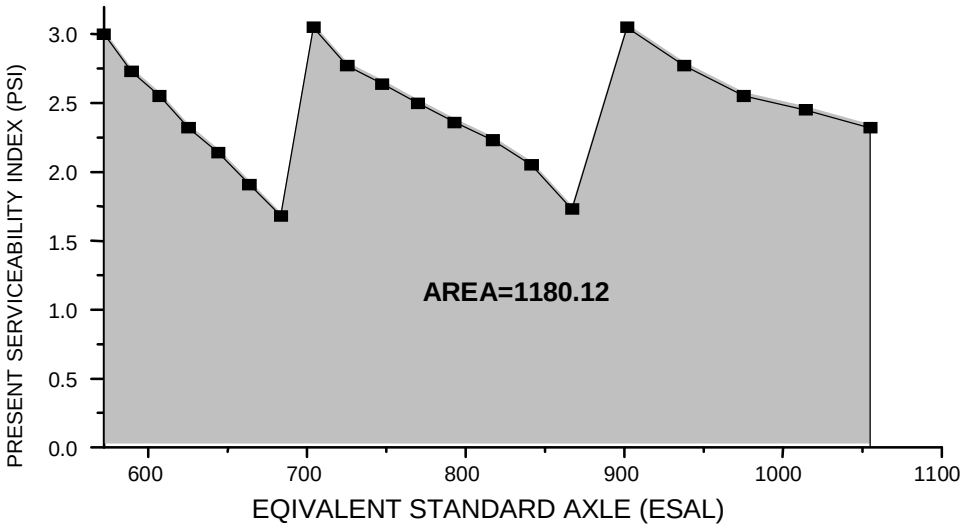


Fig 4.21 PSI-ESAL PLOT FOR 75mm STRATEGY

The area under the curve represents the PSI-ESAL capacity for the 200mm strategy and equals to the integral $\int_0^{(ESAL)n} (PSI)_t \, d(ESAL)$ = 1180.12. The pavement performance quality index (PPQI) which is more appropriate measure of economic performance is $\text{PPQI} = \frac{\text{shaded area}}{(\text{ESAL})_n} = \frac{1180.12}{1055} = 1.119$

4.2.6 Sixth (50mm thickness) strategy

This strategy involves the application of 50mm thick bituminous concrete on the existing surface of the pavement. Tables 4.15 - 4.17 show the deterioration factors and the economic implications of adopting this strategy, on a year by year basis; wile the figures 4.23 - 4.28, show the roughness progression, IRR determination, the project's pay back period, project balance and the PSI-ESAL plots for this strategy.

Table 4.15: Deterioration Factors for 50mm Strategy

Year	Operation Applied	Roughness (IRI) m/km	All Crack Area %	Wide Crack Area %	Pothole Area %	Rut Depth mm	Modified Struc No	2-Way ADT (vpd)	Annual 2-Way ESA (000)
1	OVER	3.0	0.0	0.0	0.0	2.5	2.5	1375.0	572.0
2		3.5	0.0	0.0	0.0	4.1	2.5	1423.0	589.3
3		4.1	0.0	0.0	0.0	5.3	2.5	1474.0	607.0
4		4.6	0.8	0.0	0.0	6.4	2.5	1527.0	625.3
5		5.2	3.6	0.0	0.0	7.5	2.5	1581.0	644.2
6		5.9	10.5	0.0	0.0	8.6	2.5	1637.0	663.6
7	OVER	3.0	0.0	0.0	0.0	1.5	3.1	1696.0	683.6
8		3.4	0.0	0.0	0.0	2.4	3.1	1756.0	704.2
9		3.8	0.0	0.0	0.0	3.2	3.1	1819.0	725.5
10		4.1	0.5	0.0	0.0	3.8	3.1	1884.0	747.4
11		4.6	2.5	0.0	0.0	4.4	3.1	1952.0	769.9
12		5.1	8.1	0.0	0.0	5.0	3.1	2022.0	793.1
13		5.8	19.9	0.0	0.0	6.0	3.1	2094.0	817.1
14	OVER	3.0	0.0	0.0	0.0	1.1	3.6	2169.0	841.7
15		3.3	0.0	0.0	0.0	1.8	3.6	2247.0	867.1
16		3.7	0.0	0.0	0.0	2.3	3.6	2337.0	901.8
17		4.0	0.0	0.0	0.0	2.8	3.6	2430.0	937.9
18		4.4	0.0	0.0	0.0	3.2	3.6	2528.0	975.4
19		4.7	0.0	0.0	0.0	3.5	3.6	2629.0	1014.4
20		5.1	1.1	0.0	0.0	3.9	3.6	2734.0	1055.0

Table 4.15 shows the deterioration factors on a year-by-year basis. Overlay was applied at the first year bringing down the roughness from 7.0 m/km to 3.0 m/km, and all-cracking and wide cracking to zero percent, the modified structural number to about 2.5 and rut depth to 2.5 mm. Wide cracking and potholing were never allowed to start. The roughness had a steady rise from 3.0 to 5.9 m/km in the sixth year, then dropped to 3.0 m/km after the application of the overlay (50 mm), rose to 5.8 mm/km in the thirteenth year, fell back to 3.0 mm/km and terminated at 5.1 in the twentieth year.

Table 4.16: Economic Performance for 50mm Strategy

Year	Economic Agency Capital	Economic Agency Recurrent	Economic Vehicle Operation	Economic Travel Time	Economic Society Total	Net Economic Benefits
1	873.810	2.531	2913.929	381.133	4171.404	-873.577
2	0.000	2.531	2414.600	358.452	2775.583	960.716
3	0.000	2.531	2540.619	372.665	2915.815	1000.132
4	0.000	2.531	2674.413	387.867	3064.811	979.433
5	0.000	2.531	2819.699	404.347	3226.577	950.248
6	0.000	2.531	2983.506	422.706	3408.742	905.094
7	873.810	2.531	3181.613	444.468	4502.422	-46.992
8	0.000	2.531	2909.297	445.779	3357.607	1244.156
9	0.000	2.531	3044.608	463.090	3510.229	1242.766
10	0.000	2.531	3186.881	481.326	3670.738	1238.556
11	0.000	2.531	3339.215	500.672	3842.417	1228.417
12	0.000	2.531	3507.353	521.500	4031.383	1206.412
13	0.000	2.531	3703.713	544.728	4250.972	1159.389
14	873.810	2.531	3956.395	572.961	5405.697	183.027
15	0.000	2.531	3627.136	575.543	4205.210	1567.874
16	0.000	2.531	3818.698	599.406	4420.634	1583.262
17	0.000	2.531	4020.354	624.484	4647.369	1596.573
18	0.000	2.531	4234.277	650.920	4887.728	1605.861
19	0.000	2.531	4462.232	678.885	5143.647	1609.575
20	0.000	2.531	4708.076	708.673	5419.280	1603.961

Currency: Million Naira

Table 4.16: above shows the various economic costs and benefits associated with this strategy (50mm). The total society cost dropped sharply from ₦4.1b in the second year due to application of the first overlay to about ₦2.8b and then rose gradually as a result of continuous deterioration to ₦4.5b in the seventh year when the roughness reached about 6.0 m/km. The net benefit started with a negative value due to the heavy investment on the overlay at the beginning and continue to rise positively until the next overlay application when it became negative once more, rise again and fall (but still positive) when the third overlay was

applied and ended with a value of ₦1.6b) at the terminal year. The vehicle operating cost and the total society costs both have gradual decrease in between the negative values of the net economic benefit.

Table 4.17 shows summary the economic performance for the 75mm strategy.

Table 4.17: Economic Performance Values for the 50mm Strategy

ECONOMIC PERFORMANCE VALUES	VALUE
Project NPV at 12.00 % Discount Rate:	6958.34
Project Internal Rate of Return (%):	110.0%
Modified Internal Rate of Return MIRR(%)	25.3%
Benefit Cost Ratio (BCR)	5.5
Capital Recovery Cost	931.57
Pavement Performance Quality Index (PPQI)	1.033

The Internal Rate of Return of 110.0% which is greater than the discount rate used in the calculation of NPV indicates that the strategy is worthwhile. With modified internal rate of return of 25.3% means that after the initial investment, the proceeds could be reinvested to yield a return of 25.3%. A Benefit - Cost ratio of 5.5 which is greater than 1.0 means that the project is worthwhile and attractive.

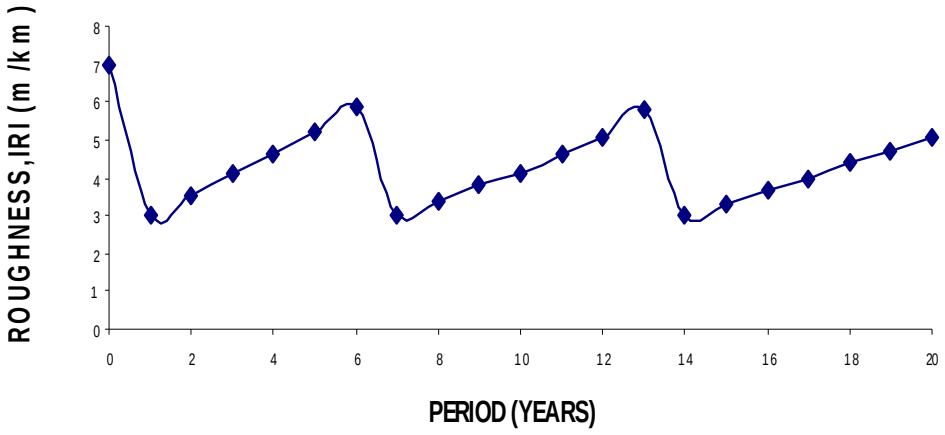


Fig: 4.22 Deterioration Profile For 50mm Strategy

Figure 4.22 shows the variation of roughness as a measure of deterioration with time period in years. The roughness progression is however restricted to a maximum value of 6.0 m/km for economic reasons. The terminal value of the roughness for this strategy is about 5.5 m/km, showing that the option has some residual value.

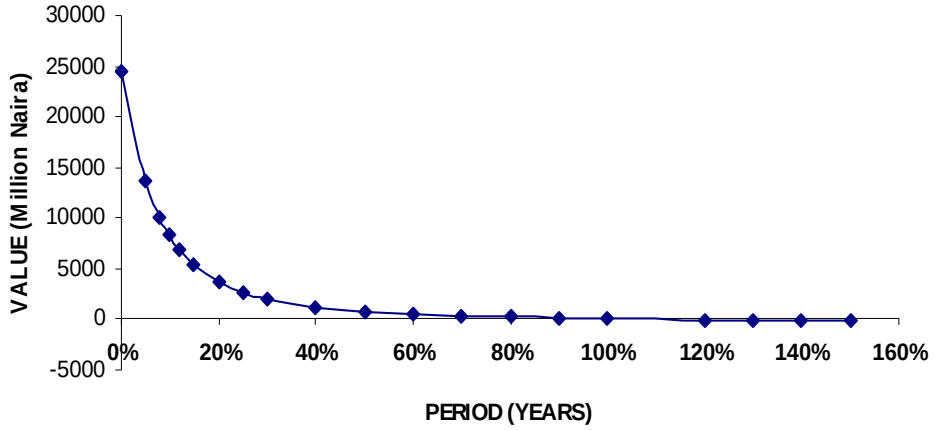


Fig: 4.23 PLOT OF NPV Versus INTEREST RATE FOR 50mm STRATEGY

Figure 4.23 shows the plots of Net Present Values of the 50 mm design strategy at various interest rates. The point where the curve cuts the interest rate axis gives the Internal rate of return for that strategy, in this case it is at 110.0%.

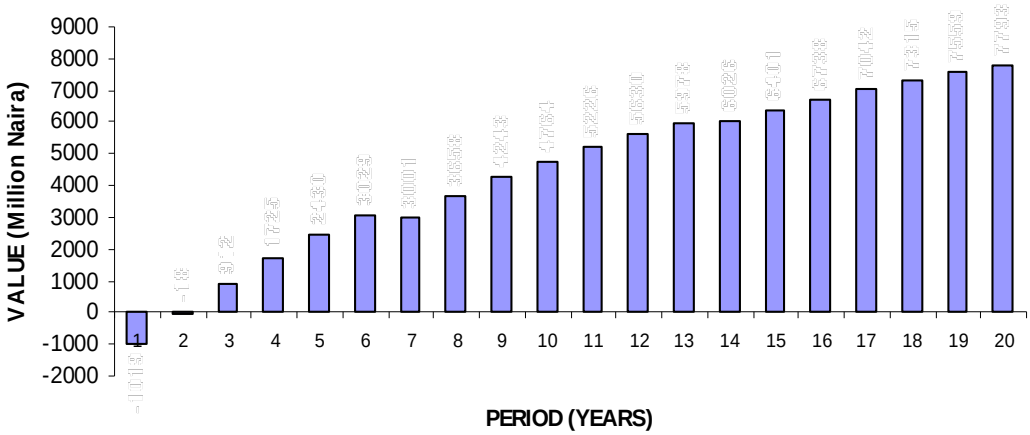


Fig: 4.24 DISCOUNTED PAYBACK FOR 50mm STRATEGY

Figure 4.24 shows that the payout period for the 50mm strategy is two and a half (2.5 years).

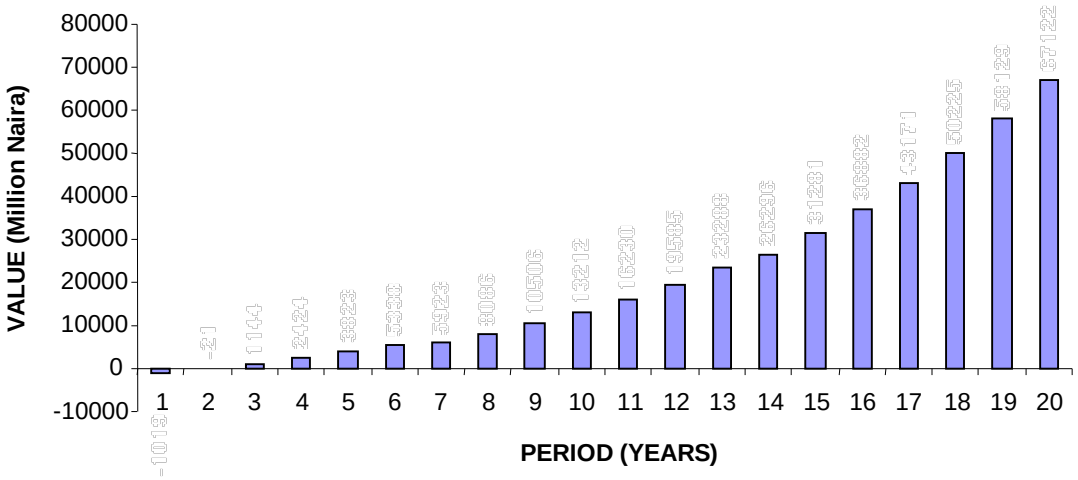


Fig: 4.25 PROJECT BALANCE FOR 50mm STRATEGY

Figure 4.25 shows the variation of the initial capital tied-up to the project during the design period.

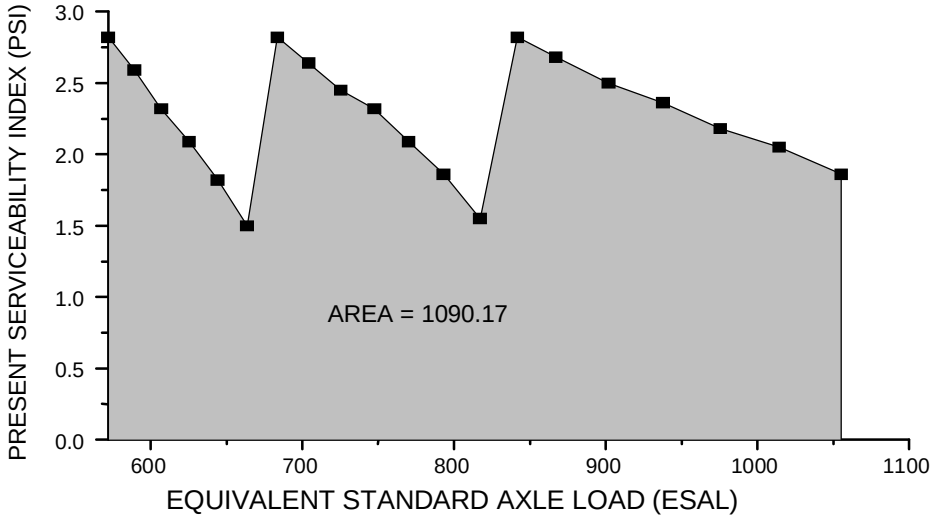


Fig: 4.26 PSI-ESAL PLOT FOR 50mm STRATEGY

The area under the curve represents the PSI-ESAL capacity for the 50mm strategy and equals to the integral $\int_0^{(ESAL)n} (PSI)_t \, d(ESAL)$ = 1090.17. The pavement performance quality index (PPQI) which is more appropriate measure of economic performance is $\text{PPQI} = \frac{\text{shaded area}}{(\text{ESAL})_n} = \frac{1090.17}{1055} = 1.033$

4.3 COMPARISON OF THE ECONOMIC COSTS AND BENEFITS

This section shows the result of the economic comparisons for all the strategies combined in graphical form. It consists of Figures 4.27 to 4.38.

Table 4.18: Comparison of the economic performance values

Economic Indicator	**200mm**	**150mm**	**100mm**	**75mm**	**50mm**
Total System Cost (Million Naira)	38215	38039	36589	35540	35968
Net Present Value (Million Naira)	4952	5109	6403	7340	6958
Equivalent Annual Value (Million Naira)	663	684	857	983	932
Capital Recovery cost(Million Naira)	663	684	857	983	932
Project's TSC/NPV Ratio	7.12	7.45	5.71	4.84	5.17
Project's IRR(%)	31	35	54	97	110
Project's MIRR(%)	17	16	20	24	25
Project's BCR	4.5	2.3	3.3	5.2	5.5
Project's PPQI	1.2	1.1	1.2	1.1	1.0

Table 4.18 shows the values of the economic performance indices for the various strategies. For the TSC/NPV performance index, the lower it's value the better the desirability of the strategy economically.

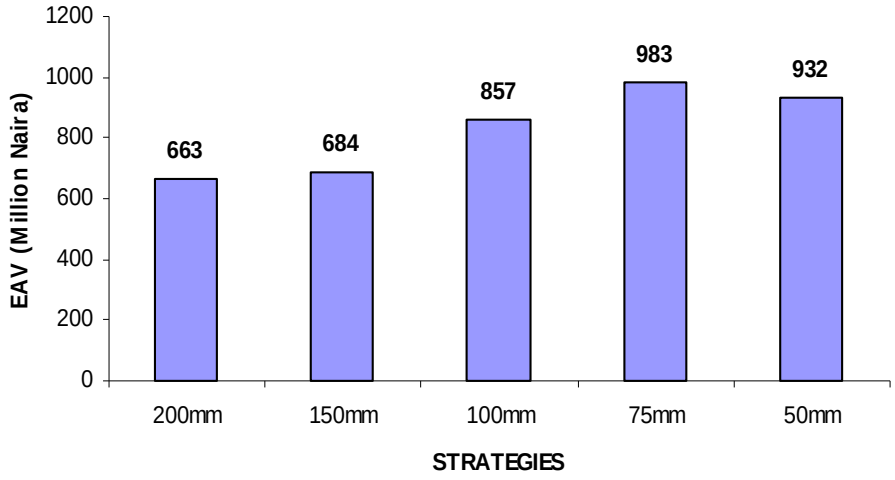


FIG. 4.27 PLOT OF EQUIVALENT ANNUAL VALUE FOR THE VARIOUS STRATEGIES

Figure 4.27 shows that the third strategy (75mm) has the highest value of EAV, hence the most desirable according to this criterion.

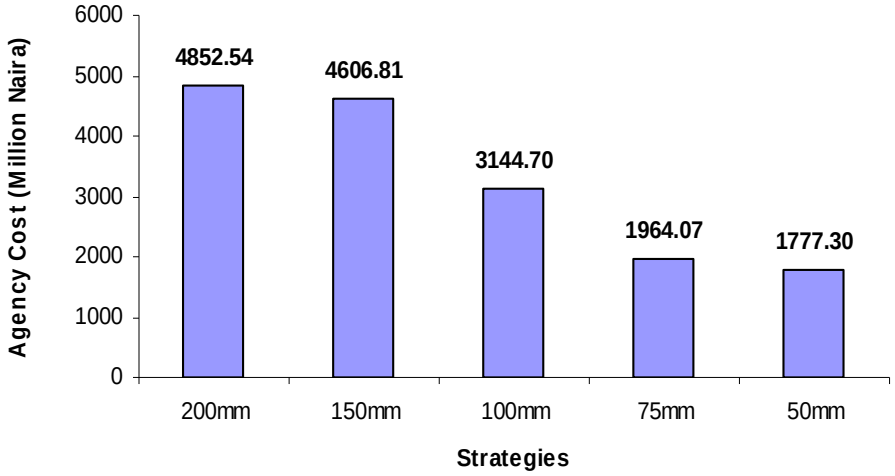


Fig: 4.28 Present Value of Agency Cost for the Various Strategies

Figure 4.28 above shows the values of the agency costs in Million Naira, with the second strategy having the highest followed by third (150mm) in that order, sixth (50mm) having the lowest cost. Low agency cost is desirable.

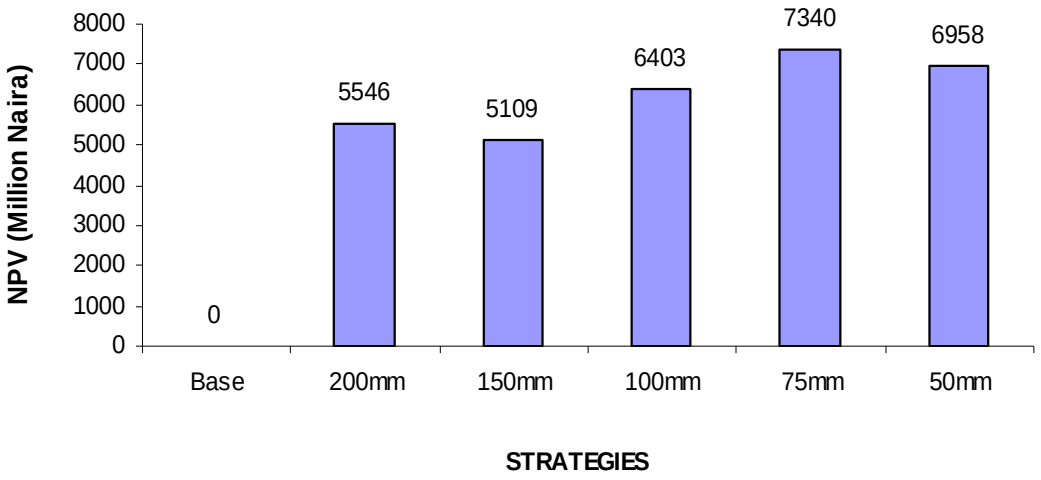


FIG. 4.29 PLOT OF NET PRESENT BENEFIT AND INTERNAL RATE OF RETURN FOR THE VARIOUS STRATEGIES

Figure 4.29 shows the plot of the Net Present Value for the various strategies. It indicates that the best strategy is the fifth (75mm) having an NPV of ₦7.3b followed by the sixth (50mm) with ₦6.9b, then the fourth (100mm), the second and lastly the third strategy, the base strategy having been ruled out .

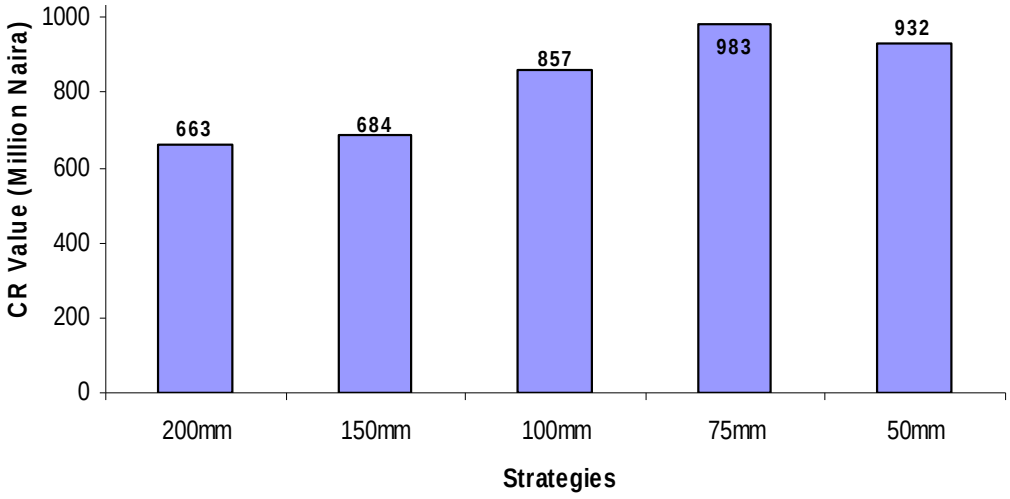


Fig: 4.30 Plot of Capital Recovery for the various Strategies

The graphs of the capital recovery cost for the various strategies shown above indicate that fifth (75mm) strategy has the highest recovery rate followed by the sixth (50mm) strategy.

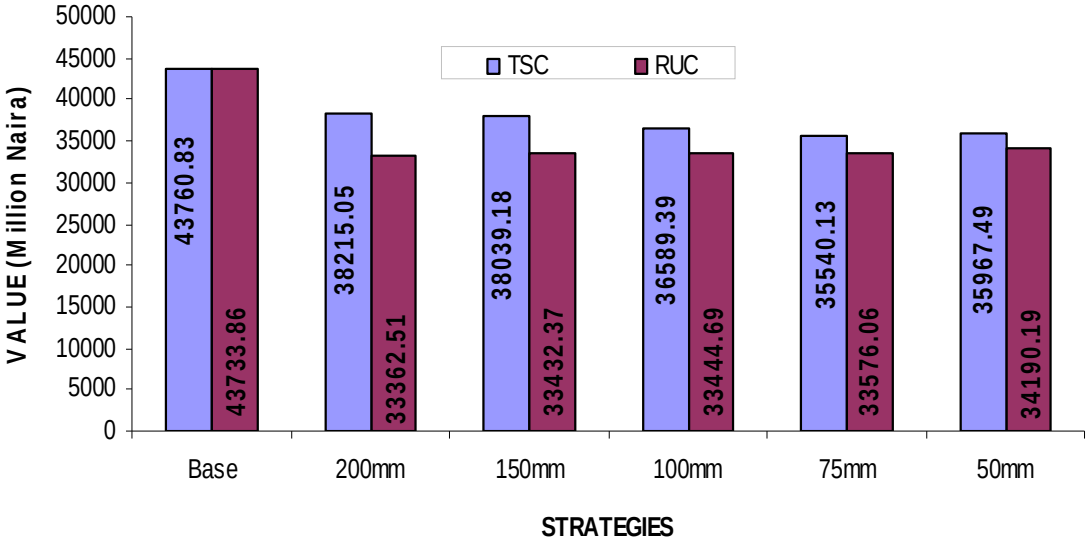


FIG. 4.31 PLOT OF TOTAL SYSTEM AND ROAD USER COSTS FOR THE VARIOUS

Figure 4.31 is the bar graphs of the total systems and user costs for the various strategies. The graphs show that the fifth strategy has the least total systems cost but second least user cost, while the sixth strategy has the second least total systems cost but least user cost. The choice under these two values, therefore remain between the fifth (75mm) and the sixth (50mm) strategies.

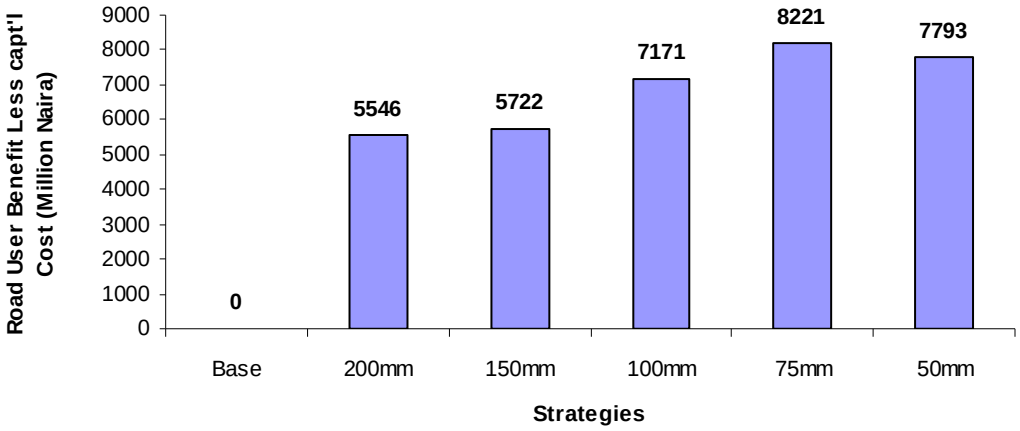


Fig: 4.32 Plot of Road User Benefit Less captl. Cost for the Various Strategies

The plot of user benefit less agency cost shown in Figure 4.32 indicates a similar trend as that of the Net Present Value (NPV) with the 75mm strategy having the highest value followed by the 50mm strategy.

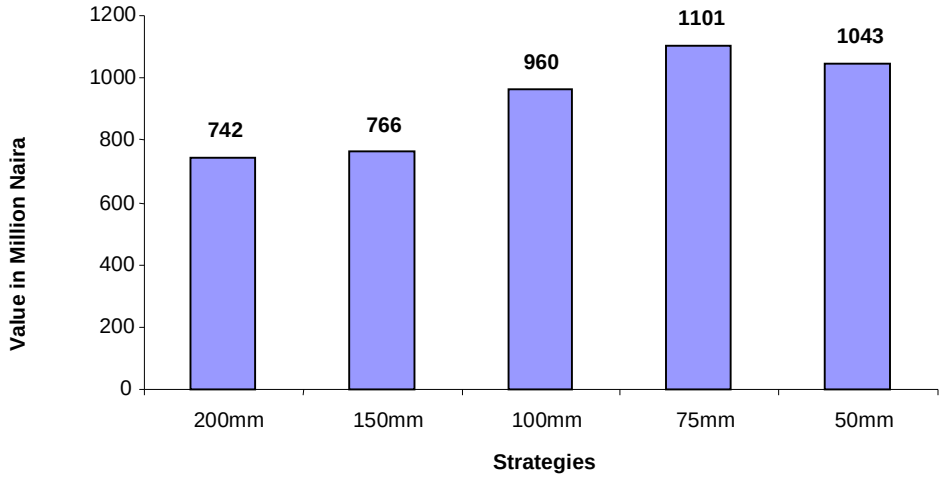


Figure 4.33 :Plot of Differences Between Total Annualised Systems Costs of the Base and other strategies

The plot of the Annualised Marginal Difference of Total Systems Cost (Figure 4.33) shows that the fifth strategy (75mm) has the highest value of ₦1101 Million followed by the sixth strategy (50mm).

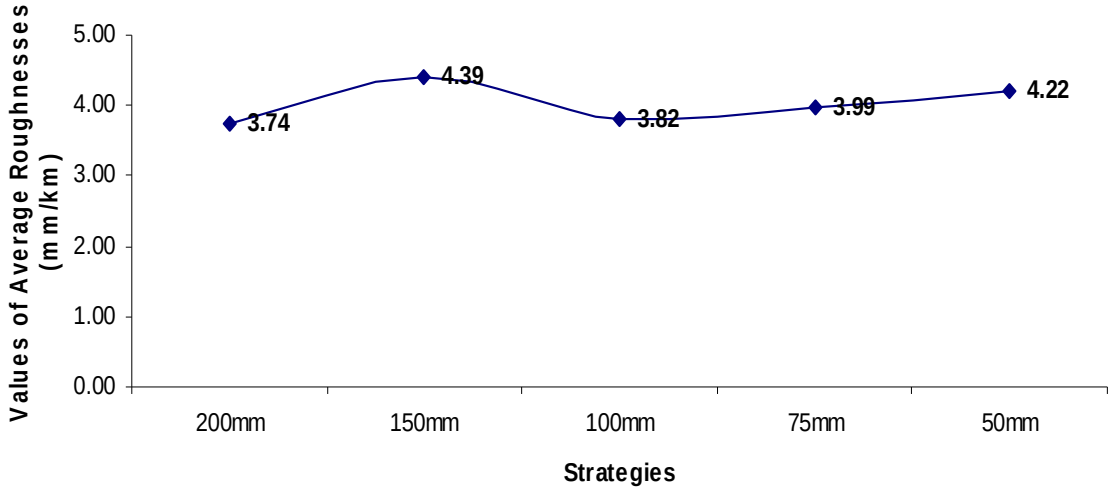


Figure 4.34 Plot of Average Roughness for the Various Strategies

A plot of roughness (a measure of performance) for the various strategies shown above indicates that all the strategies , except the base strategy perform well, however, the third strategy has the lowest average value of roughness (3.33m/km) as a result we should expect that strategy to have the lowest value of user cost.

4.4 Comparison of the Measures of Economic Performance

This section compares the various measures of economic performance of the various strategies. The Net Present Value has been selected as the criteria for evaluating the appropriateness of the other economic values. This is because the NPV has often been in many cases consistent, having just one disadvantage of not relating the benefit to the capital investment needed to produce it.

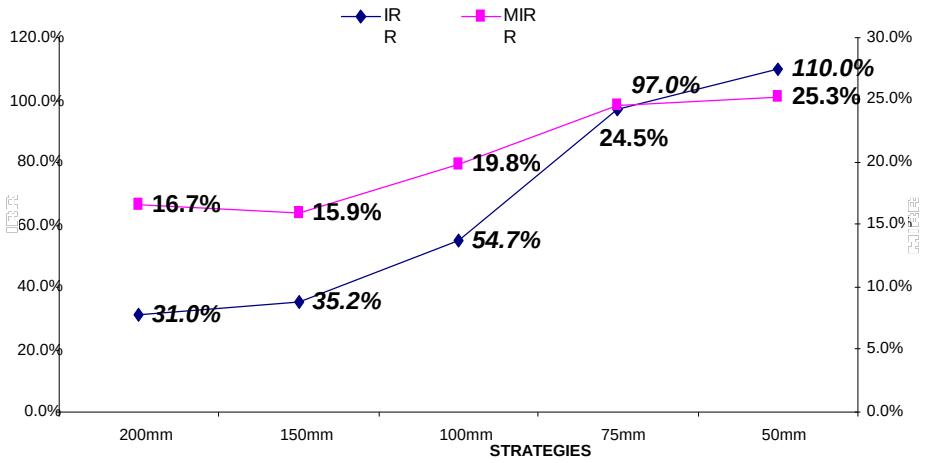


FIG: 4.35 PLOT OF IRR AND MIRR FOR THE VARIOUS STRATEGIES

Figure 4.35 shows the plots of internal rate of return (IRR) and the Modified Internal Rate of Return (MIRR). The plots show similar trends for both economic measures, except in the case of the third strategy

(150mm),where the IRR showed an improvement while the MIRR showed otherwise. The IRR measure plot however shows more responsive to minor differences.

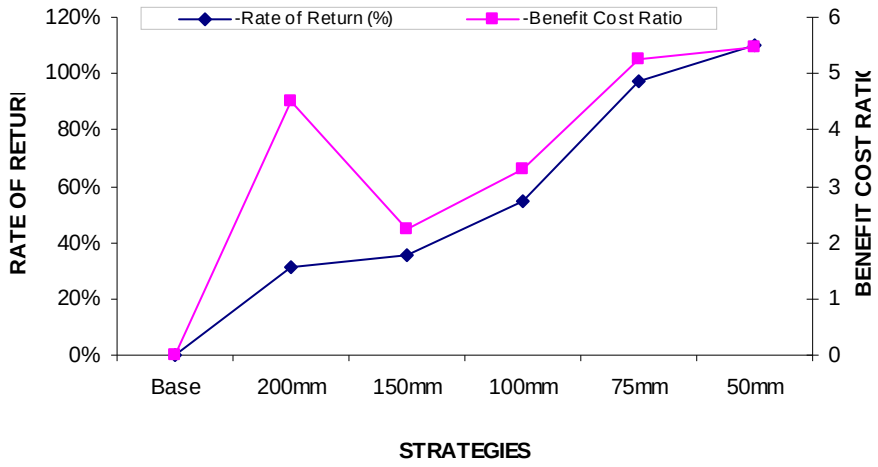


FIG. 4.36 PLOTS OF RATE OF RETURN AND BENEFIT COST RATIO FOR THE VARIOUS STRATEGIES

Figure 4.36 shows the economic measures of Benefit Cost Ratio (BCR) and Internal Rate of Return (IRR) for the various strategies on the same graph area. Their pattern of are similar except for the third strategy where there is great deviation .

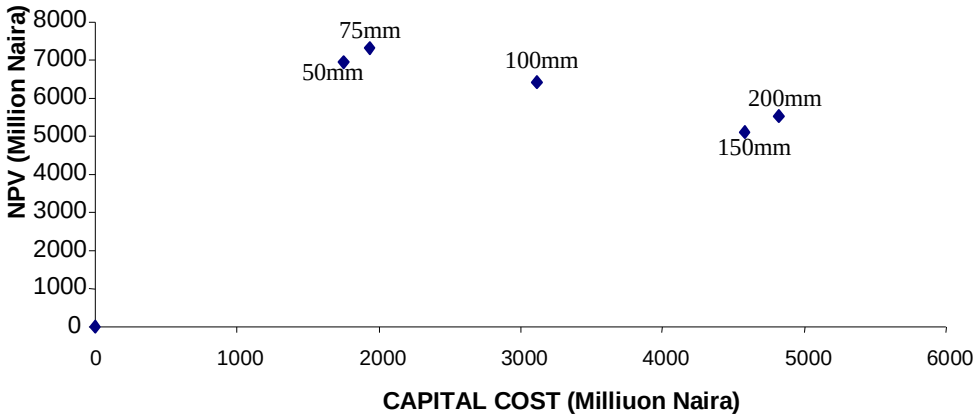


FIG. 4.37 PLOT OF NET PRESENT VALUE AGAINST CAPITAL COST

A plot of Net Present Value (NPV) against capital cost shows the same trend as the plot of Figure 4.37. The two most competing alternatives are the fifth and the sixth strategies.

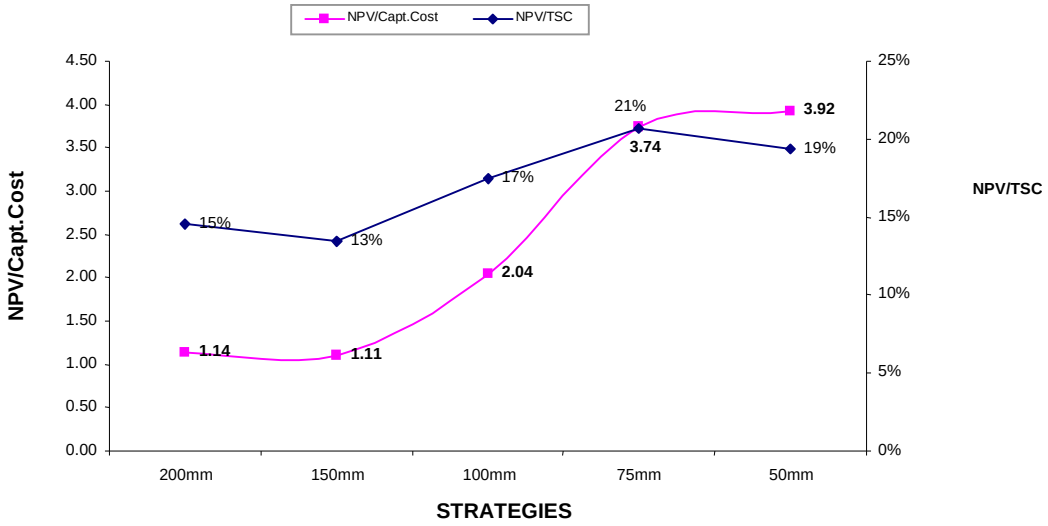


FIG. 4.38 PLOTS OF NPV Per TSC AND NPV Per CAPT. COST

Plots of NPV/Capt. Cost and that of NPV/TSC for the various strategies shown above (Fig.4.38), indicates that the most contending alternatives are the

fifth (75mm) and the sixth (50mm) strategies having the highest and second highest values of NPV and second lowest and lowest values of capital costs.

TABLE 4.19: SUMMARY OF THE ECONOMIC PERFORMANCE MEASURES FOR THE VARIOUS STRATEGIES

Strategies				**200mm**	**150mm**	**100mm**	**75mm**	**50mm**
Project NPV at 12.00 % Discount Rate:				4951.58	5108.61	6403.07	7339.91	6958.34
Road User Cost				33362.51	33432.37	33444.69	35540.13	34190.19
Capital Recovery Cost				662.91	683.93	857.23	982.66	931.57
PV of Capital Cost				4825.57	4579.84	3117.73	1937.10	1750.33
Total Systems Cost				38215.05	38039.18	36589.39	35540.13	35967.49
PV of Agency Cost				4852.54	4606.81	3144.70	1964.07	1777.30
Equivalent Annual Value				662.91	683.93	857.23	982.66	931.57
Project NPV/Total Systems Cost				13.0%	13.4%	17.5%	20.7%	19.3%
Project Internal Rate of Return (%):				31.0%	35.2%	54.7%	97.0%	110.0%
Modified Internal Rate of Return MIRR(%)				16.7%	15.9%	19.8%	24.5%	25.3%
Benefit Cost Ratio BCR :				4.51	2.25	3.30	5.24	5.45
Average Roughness,IRI (m/km)				3.74	4.39	3.82	3.99	4.22
Pav'mt Performance Quality Index				1.17	1.13	1.20	1.12	1.03
Incremental Cost Benefit Ratio								
Strategies	4 -- 5	3.29	Accept 4 and Reject 5					
	3 -- 4	0.11	Accept 4 and Reject 3					
	2 -- 4	0.05	Accept 4 and Reject 2					
	1 -- 5	0.07	Accept 4 and Reject 1					

4.5 Comments

Civil engineering projects usually take the form of extremely expensive, long life investment such as roads, railways, dams etc. They rarely involve selling a product, and the concept of price and demand seldom arise, although capacity has a definite meaning. The projects are usually undertaken for communal or societal reasons, and so the idea of profit is not essential.

In place of these missing objectives are two goals; firstly “minimum cost”- that is total payment of capital and running cost that must be made during the project life, and secondly “worthiness” – that is, the benefit that the community will get from the project will cover the total effect necessary to carry it out.

The Net Present Value (NPV) method calculates the total amount flowing into an organization as a result of a project, after paying all the capital used at an acceptable interest rate. A disadvantage of the NPV method is that profitability is not referred to the capital required to produce it. Therefore, a project may have a high NPV but can still be unattractive because it requires a very high capital outlay.

The Internal Rate of Return (IRR) relates the NPV to the capital involved but has two disadvantages; firstly, the function can mathematically have two or more solutions, so that under certain circumstances is not applicable. Secondly, a high IRR is a hypothetical analysis unless monies recovered in the project are invested in another project of equal profitability.

These disadvantages can be overcome using the Benefit- Cost Ratio (BCR) method. In this case, the ratio of NPV of the benefits to discounted cost (Capital) is calculated. The discount factor used is the estimated cost of capital.

The Equivalent Annual Value (EAV) is the equivalent constant annual sum that a project will contribute to a firm's finances, after which the capital has been paid for. This is more reasonable when viewed from the financial side of a business, because company finances are carried out on a yearly basis. According to (Rose,1976), methods like the payback period, and IRR fit only for single capital investment.

4.5.1. Basis of Comparisons of the Methods

When there is need to choose between alternatives we use a method to evaluate the economics of all of them and, if there is nothing to choose between non quantitative factors of the project, the choice is made according to the outcome of the economic evaluation. If all the economic criteria would result in the same preference order there will be no problem. However, this is not the case. Infact it is possible to choose methods of evaluation and get different preference order.

This problem arises because all these evaluation methods are merely rules of thumb or guidelines. They do not clearly defined the real objective and then evaluate according to that criteria. The objective obviously concerns the future of the firm or agency, however, the way these methods have been used indicates that there is still confusion on what the objective of the firm or agency

is. It may be to maximize benefits to share holders, or to stay in existence. For “Staying in Existence” simply a well control cash flow to maintain liquidity may be the primary objective. “Satisfying the share holders” involves dividend and capital growth criteria.

Another reason is that some methods are more easily understood by a particular group or professionals; thus the economist is particularly interested in the efficient use of capital and therefore would prefer IRR. The accountant is interested in the behavior of the firm, for which the effect on the yearly account is most meaningful and therefore prefers the average yearly evaluation (EAV). The manager who would most likely be conservative and less interested in technical accuracy would prefer the pay back or IRR. The engineer who is more averse to inaccuracy would prefer the methodological accuracy of discounting to averaging methods and so he will prefer the simplicity of handling NPV. In the same vein, the engineer is primarily occupied with capital cost; the relation of IRR to capital is therefore significant to him. The BCR combines the best of NPV and IRR.

The characteristic of the project have a further important influence on the choice of evaluation criteria. Some methods are best for single capital investments, e.g. payback and IRR; some methods are suitable only when data for the average year can be realistically defined. If there is limitation on capital then the IRR or BCR can be used. If capital is not the problem, as may often be the case of a big firm or governmental agency, there is no point of using methods which use the ratio of profit to capital, hence the NPV or EAV are better as unconstrained wealth maximizers.

The Project Balance (PB) is another method which could be regarded as a member of the unconstrained wealth maximizers described in section 3.

Based on the above arguments, the best method of economic evaluation of pavement overlay are; the TSC/NPV, EAV, NPV, NPV/TSC and CRF.

CHAPTER FIVE

SUMMARY, CONCLUSION AND RECOMMENDATION

5.1 Conclusion

In the light of the above discussion the following conclusion have been drawn:

1. **The P.E.U –TRDF** rehabilitation design method is appropriate for investigating rehabilitation requirement and design only but not appropriate for deterioration prediction it must be complimented with HDM model for comprehensive economic analysis, and that the performances of all the options (strategies) are adequate (average roughness less than 6.0).

2. **The TSC/NPV, NPV, EAV, NPV/TSC, AND CRF** are the most appropriate methods of economic evaluation of overlay designs, because economic justification and not profit making is what is needed for government public utility projects.

3. Construction cost increase with increases in overlay thickness, while road user cost decreases with increase in overlay thickness.

4. Economic evaluation indices such as BCR, IRR, MIRR etc. are inferior evaluation methods than the unconstraint profit maximizers for pavement maintenance prioritization.

5. All the options considered in this work have been found to be worthwhile economically (BCR greater than 1 in all the options).

6. The overlay thickness that maximizes user benefit is the 200mm strategy.

7. The strategy that minimizes user cost is the 200mm strategy, while the strategy that minimizes total systems cost is the fifth (75mm) overlay.

8. The TSC/NPV ratio for the five strategies were found to be 7.7, 7.4, 5.7, 4.8, and 5.2 for 200mm, 150mm, 100mm, 75mm, and 50mm respectively. The ranking then follows; 75mm, 50mm, 100mm, 150mm and lastly 200mm.

5.2 Recommendation

Based on the study conducted (200mm, 150mm, 100mm, 75mm 50mm,) the 75mm strategy has the least total systems cost (TSC) and the highest net present value (NPV). It is therefore recommended for use in the economic analysis of overlay design in the study area and the Nigerian overlay design procedure should be the basis for requirement development while the HDM shall constitute the economic analysis aspect.

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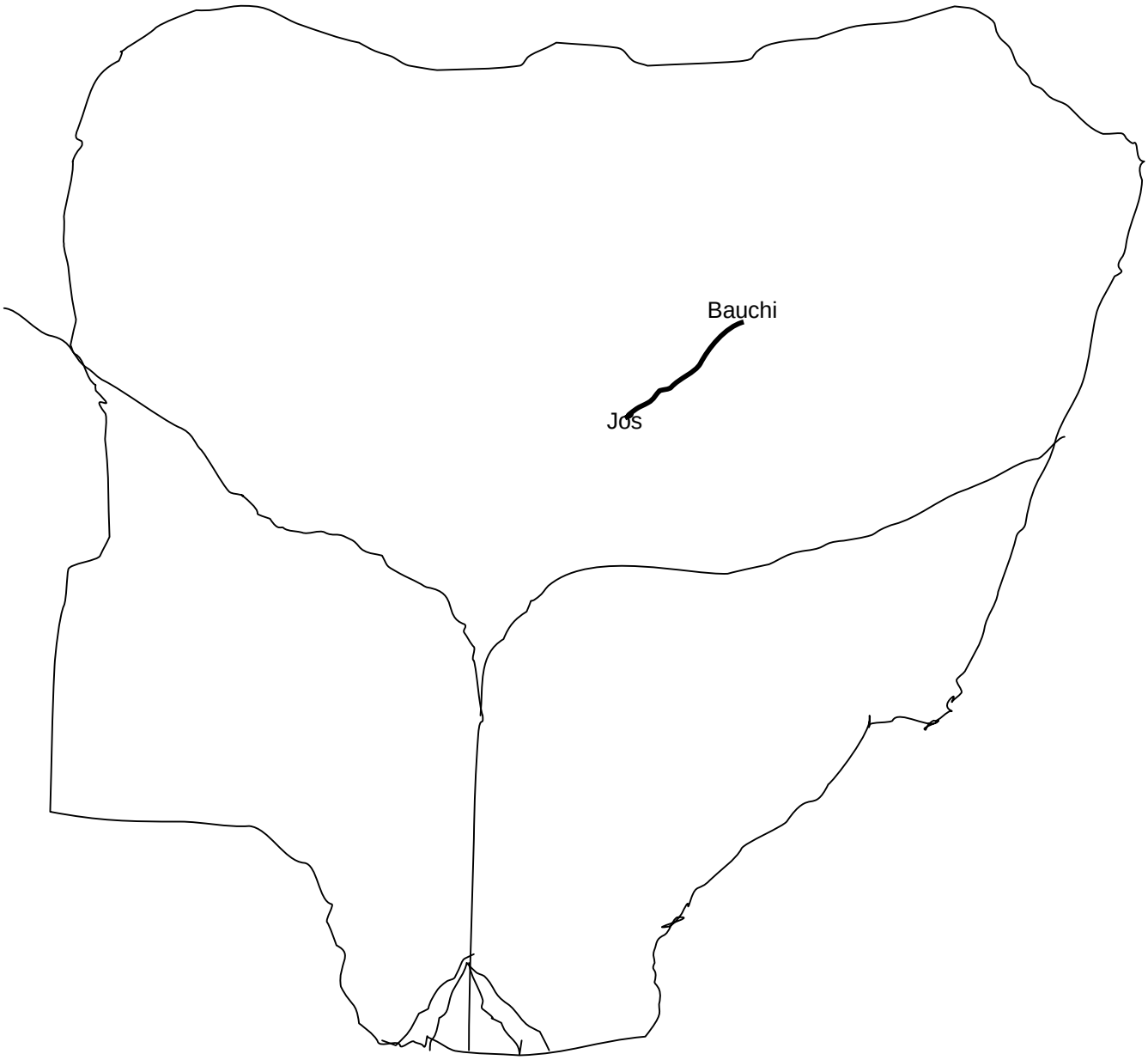
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APPENDIX I

Map of Nigeria showing the Bauchi-Jos road



APPENDIX II

Table showing data collected from the Pavement Evaluation Unit, Kaduna

Section Designation	Section Length(m)	Road Roughness QI	IRI(m/km)	All Crack			Surface Separation(m
				I	II	III	
1	13.2	81.02	6.23	23.5	32.6	20.6	42
2	9.5	108.20	8.32	21.1	25.6	6.6	17
3	13.8	77.29	5.95	24	34.2	10.2	23
4	8.8	80.48	6.19	20.6	23	3.8	4
5	11.7	130.05	10.00	25.1	37.1	38.6	18
6	7.2	73.02	5.62	32.1	30.7	16.5	2
7	10.4	97.54	7.50	21.2	25.4	36.3	29
8	11.7	119.39	9.18	19	24	16.3	7
9	6.90	53.30	4.10	18.0	18.9	6.2	4
Average		**91.14**	**7.01**	**22.73**	**27.94**	**17.23**	**16.22**