PAVEMENT MATERIALS SELECTION: A DECISION THEORY APPROACH

BY

SARKI, UMARU PETER

A THESIS SUBMITTED TO THE POSTGRADUATE SCHOOL AHMADU BELLO UNIVERSITY IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF MASTER OF SCIENCE (HIGHWAY AND TRANSPORTATION ENGINEERING)

DEPARTMENT OF CIVIL ENGINEERING
AHMADU BELLO UNIVERSITY
ZARIA

AUGUST, 1996

DECLARATION

I hereby declare that this thesis has been written by me and that it is a record of my own research work. It has not been accepted in any previous publication for a higher degree.

All sources of information are specifically acknowledged by means of references.

Umaru Peter Sarki

SARKI, UMARU PETER

August, 1996.

CERTIFICATION

This thesis entitled "Pavement Materials Selection:A Decision Theory Approach" by SARKI, UMARU PETER meets the regulations governing the award of the degree of Masters of Science in Highway and Transportation Engineering, and is approved for its contribution to knowledge and literary presentation.

Professor O. Adebisi
Chairman, Supervisory Committee — 23/8/96 Date

Dr. K. J. Osinubi
Member, Supervisory Committee — 23.8.96 Date

Dr. S. P. Ejeh
Head of Department — Date

Dean, Post Graduate School — 27/5/97 Date

DEDICATION

This research work is dedicated to my entire family.

ACKNOWLEDGEMENT

I wish to express my sincere and profound gratitude to my supervisor, Professor O. Adebisi, for the guidance, useful suggestions, and constructive criticisms readily offered to me, inspite of his tight schedule of duties, and sometimes, under unpleasant circumstances.

I also wish to express my gratitude to Dr. J. O. Afolayan and Dr. K. J. Osinubi, both of the department of Civil Engineering, Ahmadu Bello university, for their keen interest in this research work, and for the suggestions and encouragement given to me.

I express my appreciation to my family, friends, colleagues, and course mates, all too numerous to mention, for their prayers and support for me.

Above all, I acknowledge the steadfast love and faithfulness of the Almighty God, through whose grace and mercy, I have been able to carry out this research work; and to him be the glory and honour. Amen.

ABSTRACT

In highway pavement design, the main factors considered are the expected traffic, subgrade strength, and the constituent materials for the pavement structure. Associated with each of these factors, however, are elements of uncertainty. Ignoring these uncertainties, means that either high factors of safety are applied in design, with its cost implications, or a high risk of failure must be accepted.

A model is developed, using the Bayesian approach to statistical decision making, which accounts for the uncertainties in the pavement materials selection process. The main elements of the pavement materials selection model include the alternative materials for a given pavement layer; the states of nature associated with each alternative (material strength states); an estimation of the likelihood associated with each state of nature; an assessment (through utility functions) of the consequences of each combination of alternative material and state of nature; and a systematic evaluation of all alternative materials to determine the optimal material selection.

The model is illustrated by its application to a case study involving the selection of a base course material from among alternative materials for use in a flexible pavement structure. With the uncertainties regarding the possible range of material strength states being expressed as probabilities, the materials selection process, proceeds by a prior analysis

with existing information, which, after the acquisition of additional information, is updated by a posterior analysis. The optimal material selection is made on the basis of maximum expected utility.

The case study shows that, the magnitudes of the expected utilities depend on the magnitudes of the prior, and, or posterior probabilities associated with the respective strength states; while any change in the magnitudes of these probabilities, depend on the sample likelihoods estimated from experimental results.

It is recommended, as improvements to the model,that the utility functions be further refined, such that the effects of costs on materials selection are explicitly reflected in the decision analysis; and that, the possibility of incorporating procedures for the selection of surfacing materials be investigated.

The design approach proposed, when fully developed,is expected to bring about not only monetary savings, which otherwise would not have been made due to less than optimal selection of materials; but also result in more efficient use of scarce resources, to be reflected through a satisfactory return on the investment of highway funds.

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LIST OF SYMBOLS

m - Alternative Material

θ - State of Nature

z - Experimental or Sample Observation

$P'(\theta)$ - Prior Probability of State

$P''(\theta)$ - Posterior Probability of State

$P(\theta)$ - Acceptance Probability of State

U - Utility Value

$E[U]$ - Expected Utility Value

ABBREVIATIONS

DM_i - Decision Maker i; i=1,2,3

CBR - California Bearing Ratio

MDD - Maximum Dry Density

OMC - Optimum Moisture Content

SN - Structural Number

A - Acceptable

N-A - Not Acceptable

AASHTO - American Association of State Highway and Transportation Officials

ASCE - American Society of Civil Engineers

TRRL - Transport and Road Research Laboratory

CHAPTER 1

INTRODUCTION

1.1 GENERAL

A decision problem exists whenever a choice must be made from a set of alternative actions. Decision problems vary in magnitude, scope, and complexity, from the most trivial ones to the most complex ones. A necessary part of engineering planning and design is the making of technical decisions. Very often, such decisions are based on predictions and information that invariably contain uncertainty due to randomness of events. For example, benefits from a highway construction may be dependent on population growth, meteorological conditions, or other such variables whose future values cannot be predicted with complete accuracy or certainty, Thus, the decision maker is forced to risk resource commitments against uncertain future gains.

Decision making problems are classified and analysed according to whether a decision is made by an individual or group, and according to whether it is effected under conditions of certainty, risk, or uncertainty. Under conditions of certainty, each action is known to lead invariably to a specific outcome. In a risk condition, each action leads to one of a set of specific outcomes, each with a known probability of occurrence. Thus, while the chances of a particular outcome may be known, no assurance can be given about which particular outcome will take place. While under condition of uncertainty, each action has as its consequence a set of possible specific

outcomes, but where the probabilities of these outcomes are completely unknown.

Decision making problems may also be single or multi-attributed.

Statistical decision theory, consists of the techniques and methods for dealing with decision problems in the face of uncertainty. The theory provides a framework for obtaining a decision rule which specifies the alternative action to choose, according to a given criterion, in order to obtain a particular consequence or result.

In the field of pavement engineering, formal decision analysis, through the application of statistical decision theory can be incorporated into the pavement design process by making subjective, as well as, quantitative assessments of uncertainties. This will provide a means of accounting for the lack of precision that is inherent in pavement engineering, and it is more likely to result in more efficient use of available resources (Ellis,1975). Thus, unlike in the case of some civil engineering structures (e.g., a bridge or dam) whose failure is catastrophic and the acceptable risk of failure is very low, the failure of a pavement is not catastrophic and the costs of accepting different levels of risk of failure can be quantified, and should be inherent in the design process (Ellis,1979).

1.2 BACKGROUND TO RESEARCH

Pavement design aims at providing a pavement structure that will serve traffic safely, conveniently, and economically. To properly select materials for the pavement structure, the *representative material characteristics for the subgrade soil* and the materials available for the pavement section must be determined. These characteristics, which are pertinent to the performance of the pavement, include efficiency of stress-distribution characteristics, resistance to transient *deflections, resistance to permanent deformations, and* susceptibility to composition and environmental changes. The evaluation of material characteristics often times, require the preparation of alternate designs, or it may require extensive studies of locally available materials to achieve the most *satisfactory and most economical design (Committee on* structural design of highways,1969).

A major contributing factor to pavement failures in the tropics is that of poor material selection (DeGraft-Johnson,1975). This may be due to inadequacy of material *specifications, inappropriate methods of evaluating materials,* use of statistically non-representative soil strength values for design, use of inferior materials, and the deterioration of sound materials over time in the pavement structure.

It follows therefore, that the choice of materials for the *different layers of the pavement, substantially affect the* performance level and overall construction cost for a given

roadway; and perhaps represents the aspect of pavement design in developing countries where significant financial savings can be made (Ellis,1975). This is mainly because of the large difference in cost between a material that will be probably suitable and one that would definitely be suitable. The issue of material treatment also becomes pertinent in this context.

Modern methods for the design of road pavements(e.g., Road Note 29, Road Note 31, TRRL LR1132, and Asphalt Institute methods), take account of three main factors. These are, traffic, subgrade strength, and pavement layer materials. Each of these factors however, is associated with elements of uncertainty.

Also, the field of pavement design is dynamic, in that, concepts are continuously changing as new data become available and the vast body of past experiences increase, especially in relation to pavement materials.

Thus, a probabilistic and systematic approach is imperative in order to account for the inherent uncertainties, and to evaluate and incorporate new data into the design and decision process of highway pavements.

1.3 AIM, OBJECTIVES, AND SCOPE OF RESEARCH

The research is aimed at developing a model by which optimal decisions in the pavement materials selection process are attained by the evaluation of alternative actions in a rational and systematic manner.

The primary objective is to illustrate a systematic method that accounts for the available information and experiences of the decision maker as inputs to the design and decision process by the application of probabilistic concepts. Other objectives are to formulate a utility function that is consistent with the decision maker's preferences among the outcomes of the alternative actions involved in the materials selection process; and to identify the optimal course of action that is logically consistent with the decision maker's preferences.

The scope of the research will cover the analysis of a single attribute, individual decision making problem under condition of uncertainty; using the Bayesian statistical decision theory approach. This approach to materials selection will be illustrated by applying the model to a case study involving the selection of subgrade and layer materials, to be used in the flexible pavement structure for a proposed expressway in Kaduna, Nigeria.

1.4 METHODOLOGY OF RESEARCH

The method to be employed, will involve the following tasks:

a) Identification of the random variable or decision variable associated with the materials selection process.

b) Identification of the alternative actions applicable to the selection process.

c) Identification of the possible experiments (or sources of additional information) in the selection process.

d) Identification of the possible states of nature.

e) Formulation of a utility function to facilitate the numerical assessment of the decision maker's preferences among outcomes of alternative actions.

f) Decision analysis with available information (Prior analysis).

g) Decision analysis with new or additional information (Posterior analysis).

h) Selection of the optimal decision on the basis of the prescribed decision criterion.

CHAPTER 2

LITERATURE REVIEW

2.1 ANALYSIS OF DECISION PROBLEMS

The analysis of decision problems is dependent upon the condition under which it is effected. That is, whether under conditions of certainty, risk, or uncertainty.

Before proceeding with any decision analysis, it is essential that the set of decision variables are first identified and clearly defined.

The analysis of decision problems under certainty simply involves the ranking of the consequences of the various alternatives and choosing the alternative with the most preferred consequence or outcome. In risk situations, the evaluation of alternative courses of action usually entails the definition of an objective function in terms of the decision variables. This function is often expressed in monetary units representing total benefit or total cost. The optimal decision is determined by the values of the decision variables that will maximize the benefit or minimize the cost function, and are obtained by such optimization techniques as linear, integer, or dynamic programming. An illustration of such an analysis is a pavement design algorithm which minimizes the total cost of the pavement system for the selected unit costs of the pavement materials, using linear programming techniques (Hejal et al,1971). In some cases, in which the objective function is a continuous function of the decision variables,

the calculus of maximization and minimization is employed for optimization. Under condition of uncertainty, analysis of decision problems involves the application of statistical decision theory, with which the remainder of this chapter shall be concerned.

2.2 STATISTICAL DECISION THEORY

Statistical decision theory, as formulated by Wald (1950), consists of the techniques and methods for dealing with decision problems in the face of uncertainty. It is a synthesis of the theories of Preference and Utility (Von Neumann and Morgenstern,1944), Probability, and Inference.

A basic necessity in the formulation of decision making problems under condition of uncertainty is the evaluation of a scale of preference for the available courses of action. This evaluation must not only take account of the basic objectives of the problem, but must also weigh benefits against costs for each of the available actions in every possible circumstance. Therefore, to make rational decisions between alternative courses of action, a clear picture of the decision maker's preferences is required.

The theory of probability provides the mechanism by which a model for any given event or situation can be formulated. Through such a model, the degrees of uncertainty associated with the given situation can be quantified. Thus, the notion of an uncertain event can be related to its probability.

The subject of probability as applied to statistical decision making was pioneered with the discovery of the basic laws of conditional probability (Bayes',1763), the propounding of the viewpoint of probabilities as "degrees of belief" (Keynes,1921; Jeffreys,1939), and the presentation of the concept of subjective probability (DeFinetti,1937).

The concepts and principles of statistical inference *(hypothesis testing, estimation, and confidence intervals)* are applied in extracting information from available data for the purpose of decision making. The modern approach to inference was mainly due to Fisher (1925); in which the principles underlying the problem and methods of estimation were evolved, and a large number of standard statistical tests derived. Later, the principles of testing hypothesis were clearly formulated by Neyman and Pearson (1930-1940).

2.3 REQUIREMENTS FOR ANALYZING DECISION PROBLEMS UNDER UNCERTAINTY

The analysis of decision making under uncertainty requires that the decision maker recognize certain important aspects of the problem (Aitchison,1970). These are:

a) The assessment of the consequences of the possible actions in the face of possible, but unknown states of nature.

b) The quantitative expression of the uncertainties of a given situation.

c) The appreciation of the quantitative aspects of decision making and hence the study of competitive situations; and

d) The use of data or information to provide some reduction in the amount of uncertainty present.

2.4 FORMULATION OF DECISION MAKING PROBLEMS UNDER UNCERTAINTY

In formulating a systematic framework for the analysis of decision problems under uncertainty, certain essential components are required (Chernoff and Moses,1959; Benjamin and Cornell,1970; Ang and Tang,1984). These include a list of all feasible alternative courses of action; the sources of acquisition of additional information, if necessary; a list of all possible outcomes associated with each alternative (states of nature); an estimation of the probability associated with each state of nature; an evaluation of the consequences associated with each combination of alternative action and state; the criterion for decision; and a systematic evaluation of all alternatives to determine the optimal action.

2.4.1 THE ALTERNATIVE ACTIONS

A decision problem exists when a choice between alternative actions must be made. More often than not, especially under condition of uncertainty, a large number of alternatives are possible. It is therefore necessary for the decision maker to select the most feasible ones to be

subsequently used in the decision analysis.

2.4.2 THE EXPERIMENTS

It is sometimes necessary, in order to improve the existing state of information or to reduce the levels of uncertainty, to carry out experiments. "Experiment" here, broadly covers any method of gathering additional information which, of course, will require the expense of time, energy, and financial resources. Inspite of these added costs, the additional information may be valuable and will generally improve the chance of making a better decision; and the added cost for this new information should be included or reflected in the decision analysis. For example, before deciding on the number of lanes in a highway, an engineer might consider calling for an additional origin-destination survey to help predict the demand on the future road. Costs of this experiment may include delay of design and construction, apart from the actual costs of conducting the experiment.

2.4.3 THE STATES OF NATURE

The state of nature is the factor upon which the consequences of a decision depend and which are not known with certainty. The difficulty, more often than not in deciding which action to take, is due to the fact that it is not known which of the possible states of nature is the true one. Thus, while each action has as its consequence a set of possible

specific outcomes, the probabilities of these outcomes are completely unknown. The state of nature can be made to reveal itself partially through an experiment, which results in either a set of possible observations, the probabilities of which depend on what the true state of nature actually is, or a table of frequency responses. With respect to any decision problem, the set of "states of nature" is assumed to be mutually exclusive and collectively exhaustive.

2.4.4 PROBABILITIES OF OUTCOMES

The relative likelihood associated with each state of nature may be estimated from probability models of engineering analysis and design. The exact model used, depends on the nature of the attribute of the physical process or the attribute being analysed in the decision problem. For example, for the purpose of settlement predictions, the t-distribution has been used to establish the probability distribution of the mean of the random variable associated with the decision analysis. This was because the t-distribution is the joint distribution of the mean and variance of the normal distribution (which is widely used to model uncertainties associated with soil properties) where both parameters are unknown (Folayan et al,1970). Other sources of assigned probabilities are observed frequencies of events, and measures of a decision maker's subjective degree of belief regarding the possible states of nature.

2.4.5 CONSEQUENCES OF OUTCOMES

For each combination of action and state, there will be a resulting consequence; and to proceed with the decision analysis, the relative desirability or preference of the decision maker with respect to the consequences, must be known. To facilitate the evaluation of the consequences associated with each combination of action and state, the concepts of utility and utility functions must be well understood.

2.4.6 UTILITY

Utility is defined as a true measure of value to the decision maker; and utility theory provides a framework whereby values may be measured, combined, and compared consistently with respect to a decision maker. In practical situations, the desirability of an alternative may depend on several factors such as monetary value, time factors, prestige, social acceptance, pleasure, and others. Except perhaps for monetary value, the majority of the factors do not have well established scales of measure. Thus, the concept of utility may be used to establish a uniform scale for measuring the overall value of an alternative. Therefore, if the utility values of all the alternatives are available, the alternative with the highest utility will be preferred.

2.4.7 UTILITY FUNCTIONS

The utility function is a relation between the magnitude of the decision variable (or attribute) and the relative preference or utility derived by the decision maker. Thus, it makes a quantitative assessment of the benefit (or disbenefit) derivable from each action in the face of each possible though unknown state of nature. Mathematically, the function represents the mapping of the degree of preference onto the real number line, thus permitting preference to be expressed numerically (Sinha,1972).

There is no single set of rules that have been laid down that invariably result in a utility function. In general, assessment of utility functions require repeated interviews conducted with carefully phrased questions (Oguara,1982), or choices among hypothetical lotteries, to determine levels of indifference by which the actual shape of the functions may be revealed (Schlaifer,1978).

Thus, apart from probability assignments, the application of statistical decision theory to decision problems rests fundamentally on an individual's preferences, whether it be extracted by utility assignments or implied by the adoption of a particular utility function.

For a preference relation like a utility function to be unique, it must satisfy certain consistency requirements. These requirements are presented in axiomatic form in Appendix A1.

Common types of utility functions depicting the risk-aversiveness, risk-affinity, and risk-neutral behaviour of a decision maker are shown in Fig.2.1.

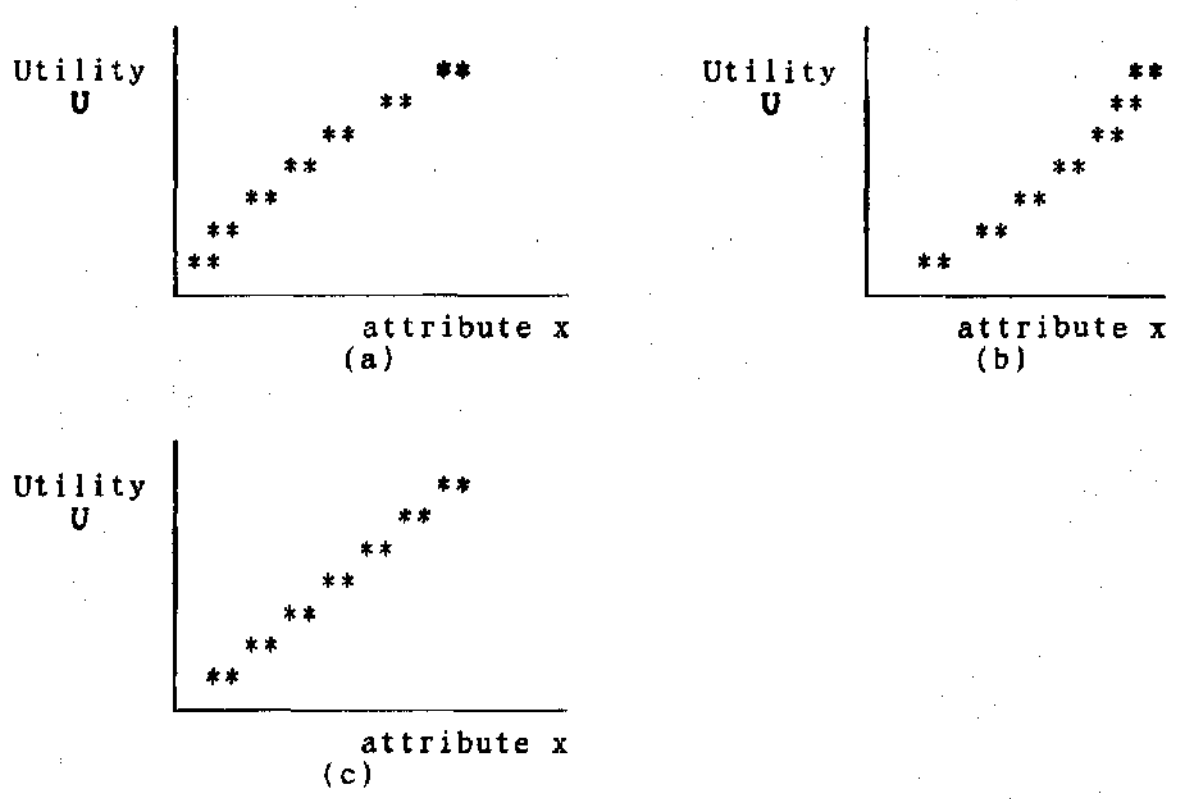


Fig. 2.1 - INDIVIDUAL UTILITY FUNCTIONS
(a) RISK-AVERSE, (b) RISK-AFFINITIVE, (c) RISK-NEUTRAL

2.5 THE DECISION CRITERIA

A decision criterion is a prescribed and precise procedure which, for any decision problem, unambiguously chooses the action which is considered "optimal" according to the criterion. For decision problems in which the consequences of the various alternatives are obtained deterministically, for example, monetary values, some of the decision criteria

commonly used are Mini-max, Maxi-max, Equal likelihood, and Minimum Regret.

2.5.1 MINI-MAX CRITERION

The decision maker chooses the alternative that minimizes the maximum loss, that is, one with the maximum lowest outcome. The criterion is however, unduly pessimistic, in that, it rules out the choice of alternatives with any likelihood, however slight, of a high loss.

2.5.2 MAXI-MAX CRITERION

The decision maker selects the alternative that maximizes the maximum possible gain. The criterion however, may preclude the selection of the realistic alternative provided that there is a finite chance for a higher gain.

2.5.3 EQUAL LIKELIHOOD CRITERION

This criterion assumes that each consequence is equally likely to occur. It therefore determines the average consequence of each alternative, and chooses the one with the highest average. The assumption of equal likelihood is however, unjustifiable, and there is no restraint imposed when very high losses are involved.

2.5.4 MINIMUM REGRET CRITERION

The criterion chooses the alternative with the smallest maximum regret. This criterion has the characteristic of concentrating on the opportunity loss especially when there is significant competition among alternatives. However, the prospect of high gains may be lost because of slightly higher opportunity loss.

Under conditions of uncertainty, the possible consequences of a given alternative may be stated only in terms of their respective probabilities. Hence, the decision criteria for such problems must consider the relative likelihoods associated with the consequences of each alternative. A general criterion for decision making under uncertainty, is the maximum expected utility criterion (Benjamin and Cornell,1970).

2.5.5 MAXIMUM EXPECTED UTILITY CRITERION

The maximum expected utility criterion states that "for any two alternatives, A and B, if A is preferred to B, then A is chosen if and only if the expected utility given alternative A, E[U/A], is greater than the expected utility given alternative B, E[U/B]". That is, the alternative with the maximum expected utility is chosen.

Thus, for any number of alternatives, n, once the utility of each consequence is known, the expected utility value of each alternative is given by:

$$E(U_i) = \sum_j P_{ij}U_{ij} \; ; \; i = 1.....n \quad .....(2.1)$$

where U_{ij} *= utility of the jth consequence associated with the ith alternative, and* P_{ij}*, its corresponding likelihood*

U_i *= utility of alternative i*

In cases where the consequences are expressed in monetary terms, C_{ij}, then,

$$E(U_i) = \sum_j P_{ij}U(C_{ij}) \; ; \; i = 1.....n \quad .....(2.2)$$

where $U(C_{ij})$ *= utility function of money*

Therefore, if the maximum expected utility criterion is used to evaluate all the alternatives in a given decision situation, and if these alternatives are independent, then one has the greatest probability of maximizing the total utility from the whole set of alternatives (Rose,1976).

2.5.6 SENSITIVITY OF EXPECTED UTILITY TO THE UTILITY FUNCTION

The expected utility is relatively insensitive to the form of the utility function at a given level of risk-aversion, and it does not change significantly over a wide range of risk-aversion coefficients (Ang and Tang,1984). The risk-aversion coefficient is the ratio of the second derivative of the utility function to the first derivative. Hence, the exact form of the utility function may not be a crucial factor in the computation of an expected utility, as long as the utility

function is consistent in expressing the decision maker's preferences among the consequences of a given action.

2.6 THE DECISION TREE

The decision tree is a formal layout which integrates the relevant components of the decision analysis in a systematic manner suitable for an analytical evaluation of the optimal alternative. It consists of the sequence of decisions that must be evaluated before the optimal decision is obtained. A typical example of a decision tree, with two alternative courses of action is shown in Fig.2.2.

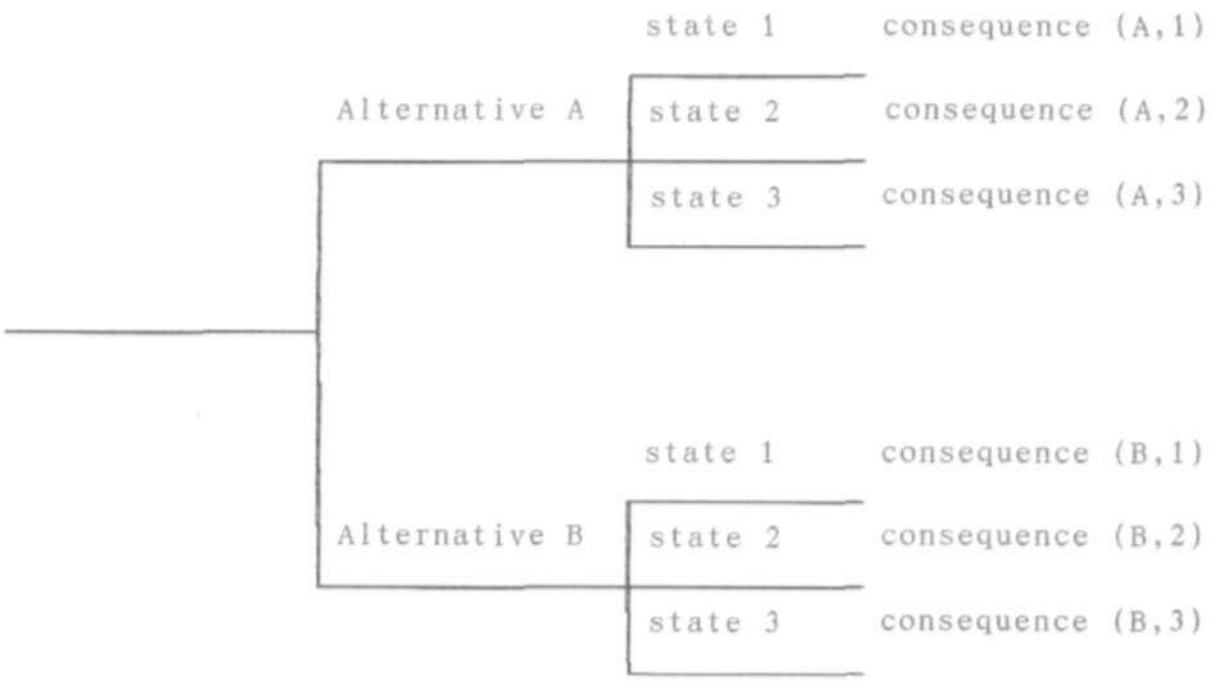


Fig. 2.2 - A TYPICAL DECISION TREE

2.7 BAYESIAN STATISTICS

Bayesian statistics was put forward by Bayes' (1763) in which he derived the statement that the probability of a "certain cause" will be subject to change given that certain events will occur. There are two essential characteristics of Bayesian statistics. These are, probability evaluation is based on experience, intuition, and personal assessment combined with a number of consistency criteria relating to a rational person; and treatment of statistical data is continuously revised on the basis of new information, or evidence that is available to the decision maker.

Classical statistics, in comparison to Bayesian statistics, utilizes only objective information through sampling, to make inferences about population characteristics when dealing with the problems of estimation or testing a hypothesis concerning some population characteristic or parameter. In Bayesian statistics however, an inference about a population parameter can be made exclusively on the basis of subjective information. If objective information becomes available later, both subjective and objective information are combined together, and inferences about the population parameter are based on this combined information.

Bayesian statistics, in combination with utility theory, has developed into an integrated theory for decision making - the Bayesian statistical decision theory. This is presented in the next section.

2.7.1 THE BAYESIAN APPROACH

The Bayesian statistical decision theory, provides a method that accounts for both the available information and "engineering judgement" (based on experiences of the decision maker), as inputs to the design and decision making process in the face of uncertainty. The decision analysis may be undertaken using the information available to the decision maker (a prior analysis), or new information may be combined with previous probability assignments before proceeding with the decision analysis (a posterior or terminal analysis).

With available information, a prior probability, $P'(\theta)$ which reflects the experiences and judgement of the decision maker, is assigned to each state of nature (θ). The state of nature represents the factor upon which the consequences of the decision depend, and which is unknown. Depending on the action (a) taken from among alternative actions $a_1, a_2, \ldots a_n$, a value $u(a,\theta)$ is received. This value is a numerical measure (assigned by a utility function) of the consequences of the action-state pair. The decision making process may be represented by a decision tree as shown in Fig.2.3.

If however, additional information or data is obtained through an experiment (e), the prior probability assignments are modified and posterior probabilities of state $P''(\theta)$ are obtained. This is achieved by means of Bayes' theorem as follows:

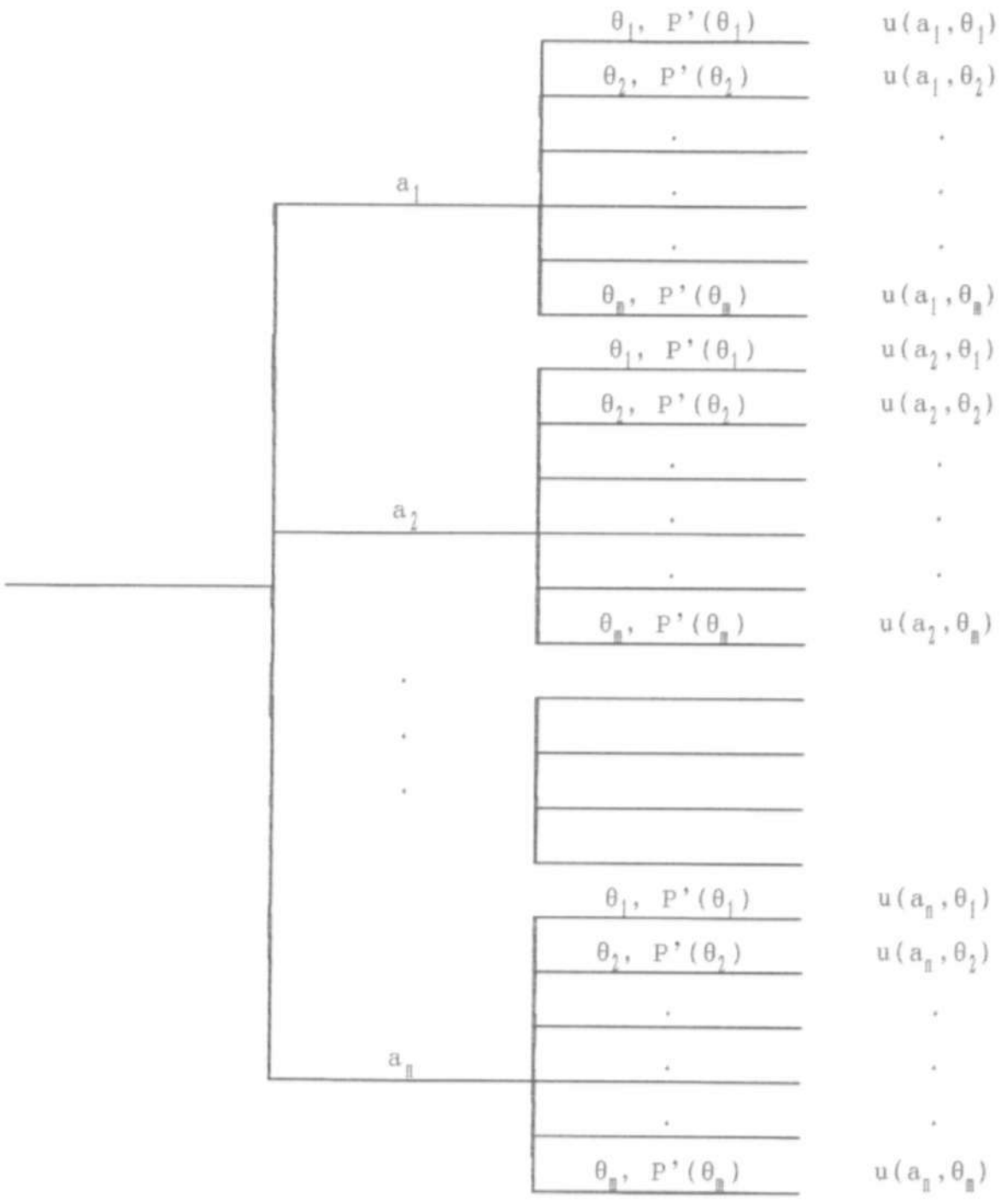


Fig. 2.3. - DECISION TREE FOR PRIOR ANALYSIS

Let θ_i = ith state of nature; i=1....m

Let $P'(\theta_i)$ = initial or prior probabilities

Let $P''(\theta_i)$ = modified or posterior probabilities

Let z_k = kth outcome of experiment; k=1....q

From Bayes' theorem,

$$P(\theta_i/z_k) = \frac{P(\theta_i, z_k)}{P(z_k)} \quad \ldots\ldots(2.3)$$

$$\Rightarrow P(\theta_i, z_k) = P(\theta_i/z_k) \cdot P(z_k)$$

$$\text{Also, } P(z_k/\theta_i) = \frac{P(\theta_i, z_k)}{P'(\theta_i)} \quad \ldots\ldots(2.4)$$

$$\Rightarrow P(\theta_i, z_k) = P(z_k/\theta_i) \cdot P'(\theta_i)$$

$$\therefore, \; P(\theta_i/z_k) \cdot P(z_k) = P(z_k/\theta_i) \cdot P'(\theta_i) \quad \ldots\ldots(2.5)$$

Thus, the probability of state θ_i given the experiment outcome z_k, or the posterior probability is,

$$P''(\theta_i) = P(\theta_i/z_k) = \frac{P(z_k/\theta_i) \cdot P'(\theta_i)}{P(z_k)} \quad \ldots\ldots(2.6)$$

By the theorem of total probability,

$$P(z_k) = \sum_r P(z_k/\theta_r) \cdot P'(\theta_r) \; ; r = 1 \ldots m$$

$$\rightarrow P''(\theta_i) = \frac{P(z_k/\theta_i) \cdot P'(\theta_i)}{\sum_r P(z_k/\theta_r) \cdot P'(\theta_r)} = N \cdot P(z_k/\theta_i) \cdot P'(\theta_i) \quad \ldots\ldots\ldots\ldots (2.7)$$

where

$$N = \frac{1}{\sum_r P(z_k/\theta_r) \cdot P'(\theta_r)} \; ; \quad \textit{a normalizing constant.}$$

$P(z_k/\theta_i)$ = *sample likelihood given* θ

The decision analysis then proceeds with the prior probabilities in Fig.2.3, being replaced with posterior probabilities.

A preposterior analysis may be used to choose among available experiments or sources of information (e), including the "do nothing" experiment (e_0). By such an analysis, the best choice of experiment is determined, and, in the process, the best choice of action given that a particular experiment has been chosen and a particular outcome of it has been observed, will also be determined. The analysis requires a terminal analysis of each experiment and each possible outcome of that experiment. The complete decision tree for such an analysis is shown in Fig.2.4.

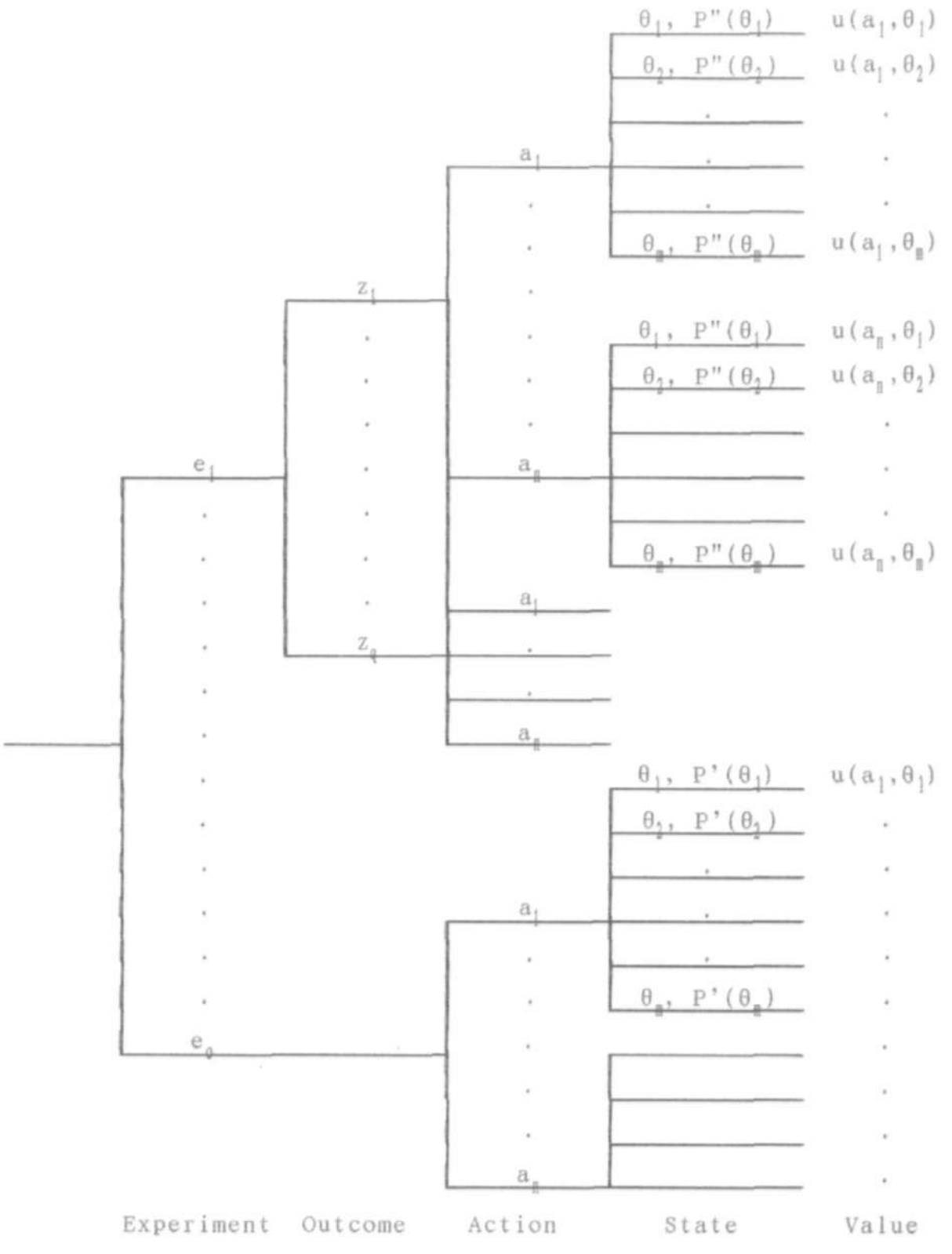


Fig. 2.4 - DECISION TREE FOR PREPOSTERIOR ANALYSIS

2.7.2 THE OPTIMAL DECISION

The selection of the optimal decision is based on the chosen decision criterion. For the Bayesian analysis, the prescribed criterion is maximization of expected utility (Benjamin and Cornell,1970). The analysis of the decision situation reduces then to calculation of expected utilities and selection of the action which shows the largest expected value.

Thus, the expected value given any action a_j, is,

$$E(u/a_j) = \sum_{all\ i} u(a_j,\theta_i).P'(\theta_i)\ ;\ \textit{for a Prior analysis}$$

$$E(u/a_j) = \sum_{all\ i} u(a_j,\theta_i).P''(\theta_i)\ ;\ \textit{for a Posterior analysis}$$

the optimal action,

$$a_{opt} = \max_j [E(u/a_j)] \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.8)$$

For the preposterior analysis, the terminal analyses for each experiment-action outcome pair, results in a corresponding optimal action and its associated expected utility, $u^*(e,z)$.

$$u^*(e,z) = \max_a [E[u(e,z,a)/e,z]] \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2.9)$$

where,

$$E[u(e,z,a)/e,z] = \sum_{all\ i} u(e,z,a,\theta_i).P(\theta_i/e,z)$$

$$P(\theta_i/e,z) = P''(\theta_i)$$

To choose from among available experiments, the expected value of each is calculated thus:

$$E[u(e)] = \sum_{all\ k} u^*(e, z_k) \cdot P(z_k/e) \quad \text{.....(2.10)}$$

where,

$$P(z_k/e) = \sum_{all\ i} P(z_k/e, \theta_i) \cdot P'(\theta_i)$$

(the probabilities of the various outcomes given that experiment (e) is taken).

The experiment with the maximum expected value is selected.

That is,

$$e_{opt} = \max E[u(e)] \quad \text{.....(2.11)}$$

The Bayesian approach to statistical decision making thus presents a useful tool by which subjective judgement and objective information are combined to arrive at an optimal decision, which is consistent with the decision maker's preferences.

2.8 APPLICATION OF STATISTICAL DECISION THEORY TO CIVIL ENGINEERING DECISION PROBLEMS

With the recognition that deterministic procedures were inferior to probabilistic concepts of analysis and design in informational content, modeling of reality, refinement of analysis and design, and decision making (Benjamin,1968); and with the development of the Bayesian methodology, the decision theory approach has met with good measure of applicability to civil engineering decision problems.

The probability of success of an engineering analysis (e.g., foundation settlements), has been shown to depend on the amount and nature of the decision maker's previous experience with similar problems (Folayan et al,1970). The formulation of a probabilistic approach to the design of soil structures has also been illustrated by development of models for the design and decision process of typical problems as sub-soil exploration, stability of foundations on clay soils, and settlement on sand (Wu,1974).

In transportation planning, the Bayesian methodology has been shown to be useful in making improved decisions regarding population forecasts (Sinha,1971). This methodology was also suggested for updating parameters of discrete choice models of travel behaviour (Ben-Akiva and Atherton,1976), and was subsequently used for updating trip generation parameters (Mahmassani and Sinha,1981).

Statistical decision theory is applicable not only to single-attribute, but also to multi-attribute decision problems. Such an application was shown by a multi-attribute decision making process that uses utility theory in selecting an optimum technique for asphalt pavement recycling (Oguara,1982).

CHAPTER 3

MATERIALS SELECTION FOR PAVEMENT DESIGN

3.1 LAYER MATERIALS IN PAVEMENT DESIGN

Many methods of pavement design are available. The suitability of a particular design method however, varies from one locality to another. The principal reasons being those of differences in factors of safety and the criteria for pavement failure, and variability of construction materials and quality of construction.

However, as new data or information become available, the concepts of pavement design continue to change, reflecting improvements to existing data or the development of new concepts based on new information.

In the early stages of development, the design of pavements consisted of rule-of-thumb procedures based on previous experiences. With the awareness that pavement performance was dependent to a large extent upon the types of soils over which it was constructed, correlations of pavement performance with subgrade types were established. Thus not withstanding, standard cross-sections were utilized for most highway construction, with the implication that a road, even though crossed several soil types was constructed using a constant thickness. This practice was often justified on an economic basis (Yoder and Witczak,1975).

With immeasurable increase in vehicular traffic over the years coupled with insufficient information about the behaviour

of pavements under traffic, and the physical properties of pavement materials and subgrades that determined this behaviour, a more rational approach to pavement design was required.

This subsequently led to the construction and systematic observation of a number of test roads (notably, the AASHTO and Alconbury Hill road tests), for the purpose of evaluating the effect of traffic loading and materials of different compositions and strengths on pavement design. The data and observations from the analysis of these test roads form the basis upon which many of the modern methods of pavement design have been conceptualized.

Noteworthy of such methods, is the TRRL LR 1132 method for the design of bituminous roads. The method employs a set of standard design curves for roads with bituminous, lean concrete and wet-mix road bases, and shows how they may be adapted to incorporate improved materials (Lister and Porter,1984). These curves were developed from extensive data of observed performance from full scale road experiments, and the application of structural concepts to interpret the results from the experimental sections. This provided a theoretical basis for design that, because it fitted observed behaviour, could be used for design situations beyond the limits of experimental data. Thus, the method provides a base-line against which new materials and designs could be assessed.

With the advances in design and construction brought about by heavier wheel loads, higher traffic levels, and the recognition of various independent distress modes contributing to pavement failure (such as rutting, shoving, and cracking), there has been the introduction and increased use of stabilized materials, reinforced soils, and geotextiles, to meet the ever increasing challenge of providing pavement materials that will perform satisfactorily as well as being economical.

3.2 THE PAVEMENT

Pavements are generally classified as being flexible or rigid. A "flexible" pavement may consist of a wearing surface built over a base course and, or, sub-base course, and they rest upon the compacted subgrade. Whereas, a "rigid" pavement is made up of portland cement concrete and may or may not have a base course between the Pavement and subgrade. The essential difference between the two types of pavements is the manner of load-distribution over the subgrade. Flexible pavements rely on the load-distribution characteristics of the layered system to distribute traffic loads to the subgrade. Rigid pavements however, because of their rigidity and high modulus of elasticity, distribute the load over a wide area of subgrade due to the bending action of the concrete slab.



Fig. 3.1 - Typical Pavement Cross-sections
(a) Flexible pavement; (b) Rigid Pavement

3.3 PAVEMENT COMPONENTS

The strength of a pavement is derived from the composite effect of the various layers (i.e., surfacing, base, and sub-base). These layers are arranged therefore, such that the layer strength, and thus the quality and cost, increases from the subgrade upwards. The overall pavement thickness and the thicknesses of the individual layers should be that the stresses in any layer or at the pavement-subgrade interface will not cause permanent deformation of the material (Ashworth, 1979). Thus, it is essential that the functions and characteristics of pavement components are such that imposed traffic loads are transmitted with diminished unit pressures to the subgrade without overstressing or deforming it.

The Pavement components include the surfacing, base course, sub-base course, and the subgrade.

3.3.1 THE SURFACING

The main function of the surfacing is to provide a running surface capable of carrying traffic loads without undue discomfort. It protects the underlying layers from undue environmental effects (such as, damage by water by acting as a waterproof membrane), and provides the necessary skid resistance for ensuring safety characteristics in braking operations. The surfacing normally consist of a bituminous "surface dressing" or of premixed bituminous material.

3.3.2 THE BASE COURSE

This is the primary load-distributing layer of the pavement. It resists and distributes stresses induced by traffic loads to the underlying layers. The base normally consist of crushed stone or gravel, or of gravelly soils, decomposed rock, sands and sand-clays stabilized with cement, lime, or bitumen.

3.3.3. THE SUB-BASE COURSE

This is the secondary load-distributing layer underlying the base course. It provides a working platform on which base materials are transported, laid, and compacted. It also acts as a level regulating course and protects the base layer from

adverse soil conditions that may prevail in the subgrade. Sub-base materials normally include naturally occurring gravel or gravel-sand-clays.

3.3.4 THE SUBGRADE

The subgrade represents the natural undisturbed local material or soil excavated from elsewhere and placed as a fill, upon which the pavement structure is constructed. The strength of the subgrade determines the thickness of the pavement which is required to transmit the wheel loads to the subgrade.

3.4 INSTITUTIONAL PROCESS OF PAVEMENT MATERIALS SELECTION

Pavement materials selection is based on materials evaluation results to establish their suitability for use as layer materials for a given pavement structure. In other words, tests are carried out to establish the conformity or otherwise of the materials to given specifications.

The main objective of pavement materials selection is to select materials that will require minimum haul and that will preferably require no treatment to improve their strengths. Ideally, the most economic road construction material would be that which lies along the proposed road alignment or adjacent to it. However, this is not always the case, usually because of inadequate quantities of suitable material along the proposed alignment. The selection process will then involve identifying quarries that are likely to yield suitable material

in adequate quantity, and from which representative samples are taken for testing under standard conditions, as stated in various testing procedures. A basis must however, be established for deciding on the tests that are essential for design purposes, as different design procedures have different test data requirements. An example of test procedures for the purpose of materials selection is given in Appendix A2. It is important at this point, to recognize that the material characteristics are only significant if they are measured in samples having the same characteristics as those materials used in actual construction. Thus, in assessing the applicability of a particular test procedure for materials evaluation, consideration must be given to how the test samples were obtained, how well they simulate the probable condition of the material in the actual pavement, how the effects of possible changes in characteristics can be evaluated, and how well the test methods used, measure all the pertinent characteristics of the materials involved. The materials selection process may be diagrammatically represented as in Fig. 3.2.

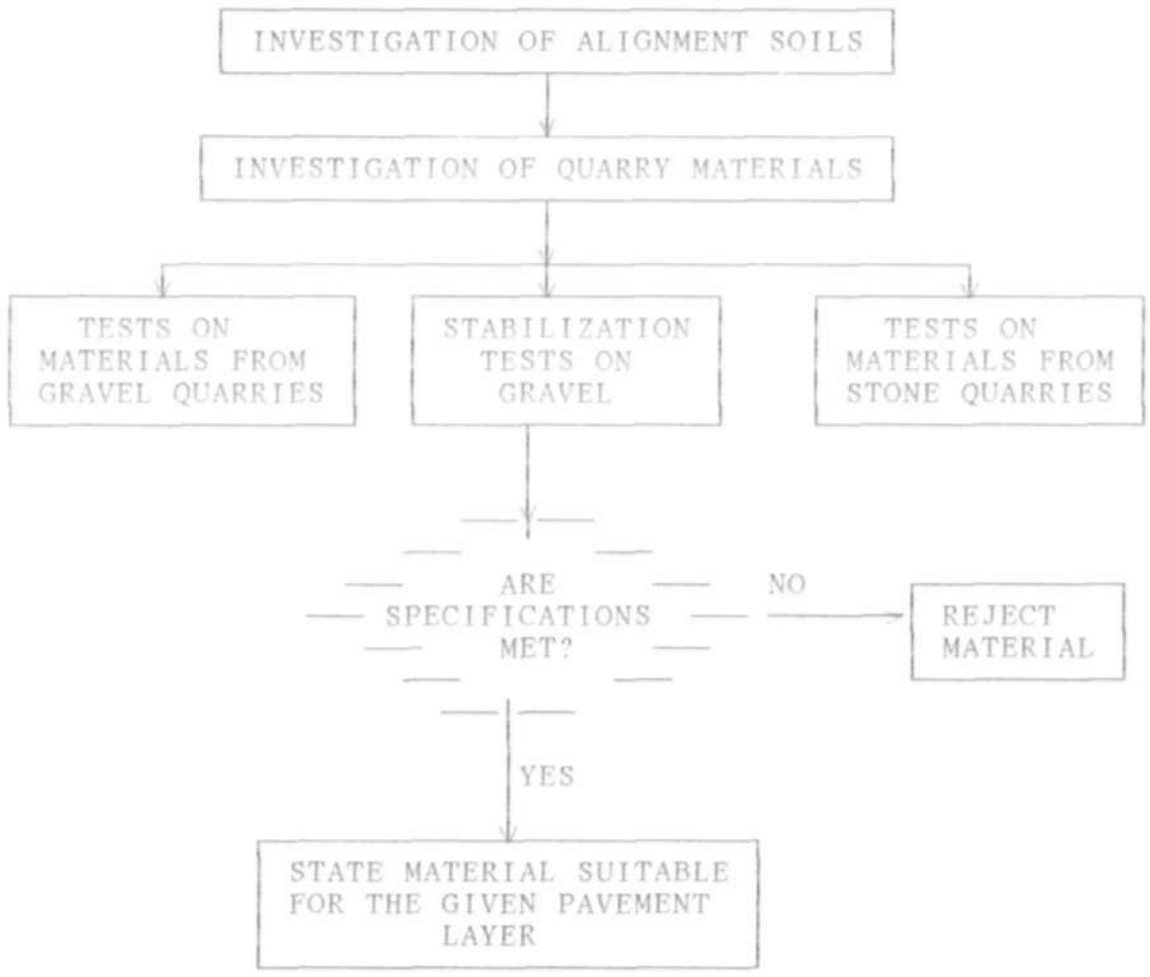


Fig.3.2 - THE PAVEMENT MATERIALS SELECTION PROCESS
(Source: Gichaga and Parker,1988)

Perhaps, one of the most challenging tasks of the pavement engineer in the tropics, is that of selection and use of tropical soils. This is so because many of the so called tropical soils are problem soils with no reasonably constant properties. The most common of these tropical soils are lateritic soils, and their use in pavement construction is briefly discussed next.

3.5 LATERITIC SOILS IN PAVEMENT CONSTRUCTION

Lateritic soils are the most common reddish surface deposits occurring widely in the tropics. Lateritic soils and gravel generally make good materials for base and sub-base construction for light and medium trafficked roads, being easy to win and to manipulate on the road surface and having naturally stable grading with suitable proportion of clayey materials to act as binder.

Some of the factors that govern the behaviour of lateritic soils in road pavements are grading, plasticity, shrinkage and swelling characteristics, degree of compaction and strength attained, the nature and mechanical strength of the concretionary aggregates, climatic and environmental conditions, design and construction procedures, as well as traffic conditions, and age of the pavement (DeGraft-Johnson et al,1972(a),1972(b)).

The main types of failure on lateritic roads are the cracking of the pavement, stripping off of the surface, and waviness of pavement surface. These failures may be attributed to poor material selection, inadequate pavement design, poor subgrade condition (which may lead to reduction in soil strength with time), and the role of moisture when it enters the pavement structure causing progressive decrease in density and strength, and increase in plasticity and possibly swell or shrinkage.

Thus, knowledge of the geotechnical properties of these soils is a critical input to their successful use as subgrade, sub-base, and base on paved roads and as surface courses on unpaved roads.

3.6 MATERIALS SPECIFICATIONS AND SELECTION

Specifications give data upon which a material is supposed to be accepted for the construction of a given pavement layer.

Stipulated specifications, as those given in Appendix A3, are usually intended for high-standard roads (surfaced or unsurfaced). Lower standards however, may be justified for roads where staged construction that requires upgrading of the construction when the traffic volume justifies such an action, could be incorporated. In this respect, there are materials in various parts of the tropics which, although may not meet specifications, could be used to construct judiciously selected low-volume or low-cost roads. Such materials include lateritic gravel (tropical belt of Africa), sabkha sands (Arabian gulf), and soft marly limestones (Caribbean and central America) (Gichaga and Parker,1988). This assertion is further confirmed by recent findings from the performance of a full-scale road experiment in Jamaica (Rolt,1994), which have shown that with good quality asphaltic concrete surfaces, marly limestones can be used to make successful road pavements, the performance of which can be predicted reasonably accurately by means of deflection measurements.

Where locally available materials do not meet specifications, consideration is usually given to the possibility of stabilizing such materials to improve their stability and load-bearing and distribution characteristics.

Of importance to the adoption and use of materials specifications is the economics of material procurement. This takes into account the type and location of insutu soils, their geotechnical properties and suitability for use in the pavement structure, their potential for stabilization, their compaction characteristics, and a comparative analysis of their likely structural behaviour when subjected to expected traffic loads, with the behaviour of the "conventional" road construction materials such as crushed stone and gravel, for which there are well established testing and design procedures.

The adequacy of materials specifications in a particular situation must be well understood, so that specifications that are unrealistic or inappropriate to the prevailing environmental conditions are not adopted. For example, the most serious limitation of the temperate zone materials specifications for tropical conditions is that their acceptance has not been based on correlation with local performance records (DeGraft-Johnson,1975).

From the foregoing discussions on pavement materials selection, it is apparent that elements of uncertainty are inherent in the selection process, and these as identified, are outined in the following section.

3.7 ELEMENTS OF UNCERTAINTY IN PAVEMENT MATERIALS SELECTION

3.7.1 VARIABILITY OF THE NATURAL SOIL

All natural soils show variations in properties from point to point in the ground, because of the inherent variations in composition and consistency during formation. Thus, soils cannot be treated as inert engineering materials with fixed properties, but careful consideration must be given to all details of their geologic history and present environment, and to the potential consequences of chemical, biological, and physical changes brought about by geotechnical activities.

3.7.2 INACCURACIES AND SHORTCOMINGS IN SOIL STRENGTH TESTS

These are introduced through methods of pretest sample preparations, types and condition of testing equipment, as well as the testing procedures and conditions of testing.

3.7.3 APPROPRIATENESS OF THE GIVEN SPECIFICATIONS FOR THE PARTICULAR CIRCUMSTANCES

This pertains to the understanding of the implications of material specifications and relevance to a particular project. An obvious example is the use in tropical areas of specifications written for temperate climates, with the possible result that a material may be rejected for use because it fails a strength criterion devised for temperate areas. This further highlights one of the weaknesses of empirical design

methods and specifications; that is, they cannot be applied with confidence outside the set of conditions in which they were derived.

3.7.4 SUITABILITY OF THE SPECIFICATION FOR A GIVEN PAVEMENT LAYER ("DOES THE MATERIAL SATISFY THE SPECIFICATION"?)

This lies in the identification of the conditions quoted in the specification (e.g., "what is the equilibrium moisture content?" or "what dry density corresponds to 95% modified AASHTO compaction?").

3.7.5 PERFORMANCE OF ALTERNATIVE MATERIALS IN SIMILAR CIRCUMSTANCES

This situation arises when, say, untreated natural materials such as lateritic gravel, sabkha sands, or soft marly limestones, fail to meet specifications, but local experience suggests that the material will perform adequately.

CHAPTER 4

MODEL FORMULATION

4.1 THE DECISION PROBLEM

The decision problem envisaged is that in which an engineer (herein referred to as the "decision maker"), is faced with the task of selecting a material from among alternative construction materials for use as layer material in a highway pavement structure, that is expected to attain a specified level of service over a given design life.

This decision must, however, be made under conditions of uncertainty due to some or all of the factors enumerated in the previous chapter.

4.2 THE DECISION VARIABLE

The objective of the materials selection process is to select materials that will preferably require no treatment to improve their strengths; and which are within economic haul distances (Gichaga and Parker,1988). A common measure of material strength for the purpose of pavement design, is the California bearing ratio (CBR).

The CBR is an index of material strength for a given pavement component (i.e., the subgrade, sub-base, or base). It also gives an indication of material thickness that will be required above a given pavement layer, such that it is not overstressed and is protected from adverse environmental conditions.

CBR is determined empirically by a penetration test, and is expressed as a percentage of the penetration resistance to that of a standard value for crushed stone (CBR=100%). The main factors that affect the CBR of a given material, are the nature of the material, density, and moisture content. For the purposes of pavement design, when more than one soil type is encountered, the CBR values are determined for each soil area. If the soil areas are clearly separated and distinguishable, then individual pavement designs may be made for each area. If however, the soil areas are randomly distributed, the CBR for design depends on the particular design method being used, but commonly, a uniform design based upon the lowest value, or the 10th percentile value may be made.

As a result of its empirical nature, and the inherent variabilities in the factors that influence the CBR values, the CBR is considered a random variable in this work, and is treated as such in the formulation and analysis of the decision problem.

4.3 MODEL COMPONENTS

The components of the decision model as applied to the materials selection process, include the experiments (or sources of additional information), alternative actions, states of nature, probability assignments, and the utility assignments.

4.3.1 EXPERIMENTS

Experiments are carried out for the purposes of evaluating the suitability or otherwise of available materials in the pavement structure, and for obtaining additional information before a given material is selected for use.

Basically, these experiments cover the investigations on alignment soils, investigations on quarry (stone, and, or, gravel) materials, and stabilization tests on alignment soils and gravel materials to determine the potentials of the materials for treatment. The experiments may involve some or all of the tests presented in Table 4.1.

Table 4.1 - TESTS FOR MATERIALS EVALUATION

INVESTIGATIONS ON ALIGNMENT SOILS	INVESTIGATIONS ON GRAVEL QUARRIES	INVESTIGATIONS ON STONE QUARRIES
Particle size distribution	Particle size distribution	Particle size distribution
Plasticity tests	Plasticity tests	Specific gravity tests
Standard compaction tests	Heavy compaction tests	Abrasion tests
Strength (CBR) tests	Strength (CBR) tests	Crushing tests
Stabilization tests	Stabilization tests	Flakiness tests
-	-	Soundness tests

(Source: Gichaga and Parker, 1988)

Particularly essential to this work, is the determination of the values of the decision variable (CBR values). These values may be obtained by CBR tests on materials in their

natural states (e_1) and, or on materials in their treated (stabilized) states (e_2). However, a "do nothing" experiment (e_0), exists when available information about the CBR values before obtaining new or additional information through experimentation, is sufficient for the decision problem to be analysed.

4.3.2 ALTERNATIVE ACTIONS

Based on either available information or experimental results, a material (m_j), is expected to be selected from among a set (m_1, m_2,.....m_n) of alternative materials, that can be used as a subgrade, sub-base, or base material.

4.3.3 STATES OF NATURE

The consequences of selecting a material (m_j), depend on the states of nature (which are not known with certainty). For the decision problem to be analysed, the states of nature include (a), the possible range of strength values that can be assumed by a given material and (b), the acceptability of the pavement material with regards to its strength (measured or estimated) for a particular application.

To determine the states of strength values therefore, a continuous state space of these values, defined on the real number line from 0 to 100, is considered. The upper limit of 100 is adopted, as this is the strength value (CBR) of the standard material (crushed stone) against which the strength

values of other materials are measured. Thus, if

x_1 = specified lower bound of strength value

x_2 = specified upper bound of strength value

x = any strength value (0 to 100)

then, the possible states of strength values are:

θ_1 (i.e., $0 < x \leq x_1$)

θ_2 (i.e., $x_1 < x \leq x_2$)

θ_3 (i.e., $x_2 < x \leq 100$)

And the states of acceptability of a pavement layer material may either be:

θ_{i1} (i.e., acceptable) or

θ_{i2} (i.e., not acceptable)

where i = 1, 2, 3 correspond to the states of strength values.

4.3.4 PROBABILITY ASSIGNMENTS

Probability assignments may be objective as obtained from observed frequencies of events, and from predictive mathematical models; or subjective as expressed by a decision maker's degree of belief regarding the possible states of nature. The probability assignments required for the decision analysis are:

4.3.5 PRIOR PROBABILITIES OF STATE, $P'(\theta_i)$

Prior probabilities of state are assigned by the decision maker prior to the acquisition and analysis of actual data; that is, before any experiments are performed.

These assignments are based on previous experience or knowledge of the decision variable, historical records, or literature survey.

For the selection of subgrade and layer materials, typical sources of prior information concerning alignment soils and quarry materials include, geological and topographical maps and reports, agricultural soil maps and reports, aerial photographs, and the results of previous soil surveys in the area of the proposed road works. The prior probabilities may be presented as in Table 4.2.

Table 4.2 - PRIOR PROBABILITIES OF STATE

ALTERNATIVE MATERIAL	MATERIAL STRENGTH STATE		
	$\theta_1(0 < x \leq x_1)$	$\theta_2(x_1 < x \leq x_2)$	$\theta_3(x_2 < x \leq 100)$
m_1	$P_1'(\theta_1)$	$P_1'(\theta_2)$	$P_1'(\theta_3)$
m_2	$P_2'(\theta_1)$	$P_2'(\theta_2)$	$P_2'(\theta_3)$
.	.	.	.
.	.	.	.
.	.	.	.
m_n	$P_n'(\theta_1)$	$P_n'(\theta_2)$	$P_n'(\theta_3)$

4.3.6 SAMPLE LIKELIHOODS, $P(z_k/\theta_i)$

Sample likelihoods are the probabilities of experimental outcomes or observations (z_k), given the state of nature (θ_i). These probabilities may be assigned by, or obtained from predictive probability distributions on the states of nature. For example, the t-distribution has been used in establishing probability distributions for the mean of a random variable for

the purpose of settlement predictions (Folayan et al,1970). The beta distribution has also been applied to explicitly represent the uncertainty concerning the estimates of decision variables (Oguara,1982).

For the case of soil properties, the variations in properties of certain typical natural soils (e.g., normally consolidated marine soft clay, sandy clay, residual silty sand, and clayey silt) are random variations about a mean or linear trend, related to the 'normal' or 'gaussian' distribution. Thus, it is often assumed that the soil strength follows the normal or gaussian distribution or some simple transform such as the 'log-normal' distribution (Lumb,1966; Wu and Kraft,1967). Some soils however (e.g., residual decomposed granite and residual decomposed volcanic rock), have been found not to follow the normal distribution, but rather, the more general 'beta' distribution (Lumb,1970). Despite such exceptions, the assumption of normality of soil properties is suitable for most practical problems in the analysis and design of pavements (Yoder and Witczak,1975).
Hence, the CBR values in this work are assumed to be normally distributed.

4.3.7 POSTERIOR PROBABILITIES, $P''(\theta_i)$

Posterior probabilities of state represent updated probabilities based on new information or experimental results. Thus, given the prior probabilities of state and the sample

likelihoods, the corresponding posterior probabilities are obtained by the application of Bayes' theorem. That is,

$$P''(\theta_i) = N.P(z_k/\theta_i).P'(\theta_i) \quad \ldots\ldots(4.1)$$

where,

$P'(\theta_i)$ = *prior probability*

$P(z_k/\theta_i)$ = *sample likelihood or the probability of the kth experimental outcome given the ith state of nature*

N = *normalising constant*

4.3.8 ACCEPTANCE PROBABILITIES, $P(\theta_{ik})$

The overall strength of a pavement is a function of the individual layer strengths. The probability of its adequacy will therefore depend on the strength states of all the layer materials.

The adequacy or acceptability of a given layer is qualitative and depends upon the rating of the given layer by an individual, and also on the intended use of the pavement, or the purpose of the layer relative to the pavement because of the need to use a variety of materials that will fulfill the desired objectives.

The assignment of acceptance probabilities can, therefore, only be subjective reflecting the decision maker's judgement about the reliability of the particular circumstances. Here, related experiences, professional knowledge, and the level of service required represent the only sources of information. The acceptance probabilities may be presented as in Table 4.3.

Table 4.3 - ACCEPTANCE PROBABILITIES

ALTERNATIVE MATERIAL	MATERIAL STRENGTH STATE		
	$\theta_1(0 < x \leq x_1)$	$\theta_2(x_1 < x \leq x_2)$	$\theta_3(x_2 < x \leq 100)$
m_1	$P_1'(\theta_{1k})$	$P_1'(\theta_{2k})$	$P_1'(\theta_{3k})$
m_2	$P_2'(\theta_{1k})$	$P_2'(\theta_{2k})$	$P_2'(\theta_{3k})$
.	.	.	.
.	.	.	.
.	.	.	.
m_n	$P_n'(\theta_{1k})$	$P_n'(\theta_{2k})$	$P_n'(\theta_{3k})$

It should be noted that, for acceptance, k=1; non-acceptance, k=2

4.3.9 UTILITY ASSIGNMENTS

Utility values are assigned by a utility function, formulated such that it assesses numerically, the decision maker's preference among the consequences of alternative actions. The utility function establishes an operational relationship between the magnitude of the decision variable and the relative preference or utility derived by the decision maker. The following examples illustrate the formulation of utility functions:

(a) Population forecasting (Sinha,1972)

Reliability in population forecasting can be measured by the ratio of actual census population for a given target year to the predicted population for the same year, ♦. Assuming that the extent of a proposed plan for transportation facilities depends on the predicted

population estimates, an appropriate utility function could be formulated in the following manner:

In a given year, if the variable, ϕ, is less than 1, this implies that the transportation facilities designed on the basis of the associated population estimates are overdesigned and the resources of the community will be unnecessarily spent. Thus, utility decreases as ϕ decreases due to increased costs of overdesign. Therefore, the utility function may be stated as:

$$U(\epsilon) = -k(\epsilon) \quad \text{.....(4.2)}$$

where,

k = *severity constant*

ϵ = *percentage error in estimating* ϕ

On the other hand, if ϕ *is greater than 1, the facilities* will be underdesigned and will not be capable of providing the required service, causing added cost due to increasing traffic congestion, accidents, air pollution, and other associated effects. Considering the cost of the system's inadequacy, severe penalties should be involved; therefore, the utility function may be represented as:

$$U(\epsilon) = -k(\epsilon)^2 \quad \text{.....(4.3)}$$

(b) Traffic sampling (Benjamin and Cornell,1970)

From a study of the dispersion of traffic throughout a city, a decision as to whether to restrict a particular street to a one-way traffic is going to be based on the

total number of cars, x, that enter the city via a certain route, A, and then use this street during a given interval of time. If the decision to restrict the street is made, and the traffic on it is high, the performance of the total traffic system will be improved. If the traffic on this street from route A is small, and the street is restricted, the inconvenience to others will outweigh the gain for route A commuters. These potential consequences may be expressed in a utility assignment of:

$$U(a_1) = -c_1 + b_1x + b_2x^2 \quad \text{.....(4.4a)}$$

where,

a_1 = *action to restrict street*

c_1 = *incurred cost to others*

b_1x, b_2x^2 = *benefits to route A users*

x = *total number of cars that enter the city via route A*

If however, a decision is taken to leave the street as it is, then no utility is derived. That is,

$$U(a_2) = 0 \quad \text{.....(4.4b)}$$

where,

a_2 = *action to do nothing*

(c) Utility functions may also be assigned by the extraction of preference statements from the decision maker. For example, in the selection of an optimum technique for asphalt pavement recycling (Oguara,1982), the utility function was assessed in the following manner:

For a given decision variable, x, boundary limits for the utility function are set, where values of 0 and 1 can be assigned to the least desirable (lower bound) and most desirable (upper bound) magnitudes of the decision variable, respectively. The magnitudes of the decision variable for which the decision maker would feel like assigning a mean utility value of 0.34, 0.50, and 0.66, are determined through carefully phrased questions. The basic attitudes of the decision maker towards risk may then be assessed by determining the portion of the utility function that is risk-averse (concave), risk-prone (convex) and risk-neutral (linear). In this manner, S-shaped functions, with points of inflection at the points where the preferences turn from prone to averse, or vice-versa, can be developed.

In the pavement materials selection process, the strength state which a material may assume, provides a means of estimating the possible layer thickness (since layer thickness decreases with increasing material strength), and consequently, the adequacy of the layer relative to the pavement structure. The aim of formulating a utility function is, therefore, to relate the magnitude of material strength to the utility derived. The utility derived from the selection of a material is assessed in this work, in terms of the contribution of a given layer to the structural strength of the pavement, which

is expressed as the pavement structural number. These individual contributions thus, provide the basis for an assessment of the desirability of various alternative materials that can be used for a given pavement layer, with regards to structural requirements (such as strength) and quality (or cost). For an appreciation of the magnitude of individual layer contributions to the structural strength of a pavement, the structural number concept is briefly discussed next.

THE STRUCTURAL NUMBER, SN

The structural number, SN, is an index of the strength of a given pavement based on the strength parameters of its individual layers. It is based on an analysis of traffic, subgrade strength, and a regional factor to allow for its use in different climatic environments. Thus, the structural number, which is measured on a scale of 1 (lowest strength) to 6 (highest strength), expresses the relative ability of a material to function as a structural component of the pavement, and is given by:

$$SN = \sum_{i=1}^{n} a_i T_i \quad \text{.....(4.5)}$$

where,

a_i = *strength coefficients of the respective layers*

T_i = *thickness of the respective layers*

and

$a_i T_i$ *represents the contribution of material j to the pavement structural strentgh*

In order to account for the contribution of the subgrade material to the overall pavement strength, the structural number is modified as:

$$MSN = \sum_{i=1}^{n} a_i T_i + [3.51(\log_{10} CBR) - 0.85(\log_{10} CBR)^2 - 1.43] \quad \text{....(4.6)}$$

where,

MSN = modified structural number

CBR = California bearing ratio of subgrade

Hence, apart from facilitating the quantitative assessment of the relative contribution which different materials can make to the structural strength of a pavement, the structural number concept also provides a means of measuring the relative efficiency of different design methods by comparing the pavement thickness recommended by various methods, as well as, the materials costs for a specified structural number (Ellis,1979).

4.3.10 THE UTILITY FUNCTION

Based on the individual layer contributions to the structural strength of a pavement (as discussed in section 4.3.9), the utility, $U_j(\theta_j,\theta_{jk})$, derived from the selection of a material j, is stated as:

$$U_j(\theta_j,\theta_{jk}) = a_iT_i \quad \text{.....(4.7)}$$

where,

a_iT_i = *contribution of material j to the pavement strength*

The T_i values are, for the purpose of this work, taken to be the minimum thicknesses of the respective layers of a pavement, as recommended by the appropriate pavement design methods. The strength coefficient, a_i, of a pavement layer is a function of the layer material strength (CBR), as well as, the layer thickness, position of the layer in the pavement, moduli and thicknesses of the other layers, and the subgrade characteristics (Ellis,1979). Values of layer strength coefficients have been established from extensive studies in tropical Africa and the caribbean, and used in the derivation of pavement deterioration relationships for developing countries. These relationships are incorporated in the TRRL Road Investment Model for Developing Countries, RTIM 2 (Parseley and Robinson,1982), and the Highway Design and Maintenance Model, HDM-III (The World Bank,1985). These strength coefficients as presented in Table 4.4, are adopted for use in this work.

TABLE 4.4 - PAVEMENT LAYER STRENGTH COEFFICIENTS

i	LAYER	STRENGTH COEFFICIENT, a_i
1	SURFACE COURSE	
	Surface treatments	0.20 - 0.40
	Asphalt mixtures (cold or hot of low stability)	0.20
	Asphalt concrete (hot premix of high stability) MR_{30}^{*} = 1500MPa MR_{30} = 2500MPa MR_{30} ≥ 4000MPa	 0.30 0.40 0.45
2	BASE COURSE	$[29.14(CBR)-0.1977(CBR)^2+0.00045(CBR)^3]*10^{-4}$
3	SUB-BASE COURSE	$0.01+0.065\log_{10}(CBR)$

* MR_{30} = Resilient Modulus at 30 days
[Source: Highway Design and Maintenance Model
(The World Bank,1985)
(extracted from Gichaga and Parker,1988)]

Hence , from equation (4.7), the utility derived from the selection of a given material varies directly with the contribution of the given layer material to the pavement strength. Thus, the utility increases (or, decreases) with an increase (or, decrease) in the layer contribution to pavement strength. The utility functions for the pavement layers and the subgrade are derived by substituting the respective strength coefficients into equation (4.7). That is,

For the base course,

$$U_j(\theta_i,\theta_{ik}) = a_2T_2$$
$$= T_2[29.14(x_B) - 0.1977(x_B)^2 + 0.00045(x_B)^3] * 10^{-4} \ldots (4.8a)$$

For the sub-base course,

$$U_j(\theta_i,\theta_{ik}) = a_3T_3$$
$$= T_3[0.01 + 0.065\log_{10}(x_{SB})] \ldots (4.8b)$$

For the subgrade,

$$U_j(\theta_i,\theta_{ik}) = a_4$$
$$= 3.51(\log_{10}x_{SG}) - 0.85(\log_{10}x_{SG})^2 - 1.43 \ldots (4.8c)$$

where,

a_2T_2 = *contribution of base layer to pavement strength*

a_3T_3 = *contribution of sub-base layer to pavement strength*

a_4 = *contribution of subgrade to pavement strength*

x_B = *CBR of base material*

x_{SB} = *CBR of sub-base material*

x_{SG} = *CBR of subgrade material*

The utility functions in equations (4.8a-4.8c) express numerically the utility derived in selecting a given material which is acceptable to the decision maker given that the specified range of strength values is the true state of strength.

If however, a given material is acceptable for a given state of

strength other than the true state of strength, the utility derived is considered to be negative, but equal in magnitude to the utility derived if the given state of strength was the true one. That is,

$$U_j(\theta_i, \theta_{ik}) = -(a_i T_i) \quad \text{.....(4.9)}$$

This is the utility which otherwise would have been derived by accepting a given material with the true state of strength. Irrespective of what the true state of strength is, if a given material is not acceptable to the decision maker, it is most likely not going to be selected. Hence, the utility derived will be nil.

That is,

$$U_j(\theta_i, \theta_{ik}) = 0 \quad \text{.....(4.10)}$$

$$\textit{since } T_i = 0$$

4.4 MODEL ANALYSIS

By the Bayesian method of analysis (Benjamin and Cornell,1970), the materials selection problem is analysed as an individual decision making problem under conditions of uncertainty, and the optimal material selection made on the basis of maximum expected utility for selected fixed costs of the pavement materials. The decision analysis may be made with existing information, a prior analysis; or with additional information, a posterior analysis.

4.4.1 PRIOR ANALYSIS

This is the analysis of the decision problem strictly on the basis of existing information about the states of nature. Thus for a given set of alternative materials (m_j; $j=1.....n$), prior probabilities of state ($P_j'(\theta_1), P_j'(\theta_2), P_j'(\theta_3)$), are assigned to each state of nature ($\theta_1, \theta_2, \theta_3$). The acceptability of the given materials relative to the overall pavement performance are expressed as probabilities ($P_j(\theta_{1I}), P_j(\theta_{2I}), P_j(\theta_{3I})$), and subsequently, the utility derived from each combination of material and state of nature are obtained through the appropriate utility function. The prior analysis situation is represented by the decision tree shown in Fig.4.1.

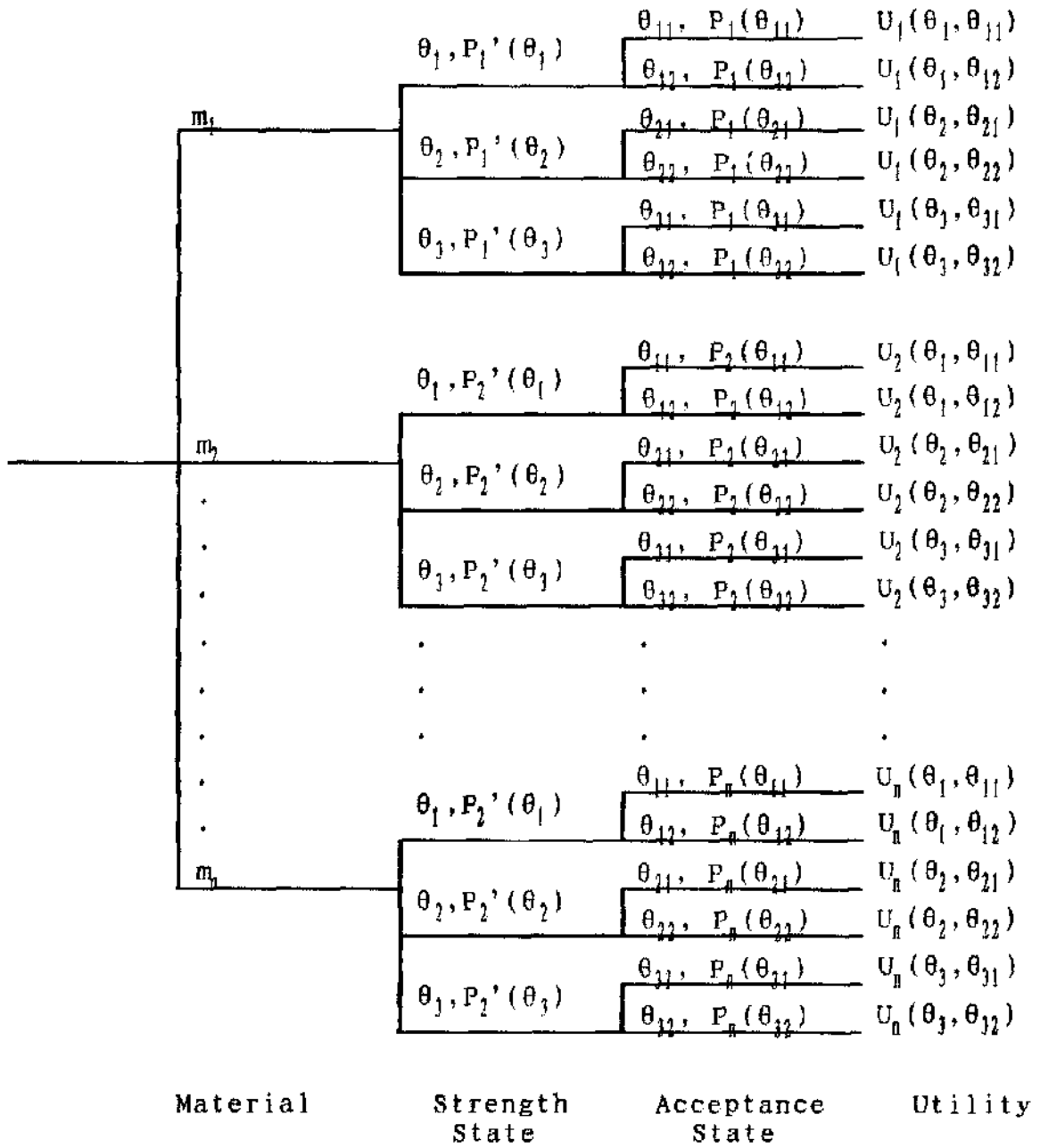


Fig.4.1 - MATERIALS SELECTION DECISION TREE (PRIOR ANALYSIS)

In order to make the optimal material selection, the expected utility must be determined for each of the alternative materials. Since for the decision problem, two groups of states are considered (strength state and acceptance state), the expected utilities are determined thus:
The expected utility given that a material, m_j, is selected and a strength state, θ_i, is observed is,

$$E[U/m_j,\theta_i] = \sum_{all\ k} P_j(\theta_{ik}) . U_j(\theta_i,\theta_{ik}) \quad \ldots\ldots(4.11)$$

and the expected utility given m_j, is,

$$E[U/m_j] = \sum_{all\ i} E[U/m_j,\theta_i] . P'_j(\theta_i) \quad \ldots\ldots(4.12)$$

The material with the highest expected utility is selected. That is,

$$m_{opt} = \max_j [E(U/m_j)] \quad \ldots\ldots(4.13)$$

4.4.2 POSTERIOR ANALYSIS

When new or added information about the states of nature are obtained by experimentation, such information is processed and incorporated into the decision problem by a posterior analysis. The analysis involves updating the prior probabilities assigned by the decision maker to posterior probabilities ($P_j"(\theta_1), P_j"(\theta_2), P_j"(\theta_3)$).
This is facilitated by Bayes' theorem:

$$P_j''(\theta_i) = N.P_j(z_k/\theta_i).P_j'(\theta_i) \quad \ldots\ldots(4.14)$$

where,

$P_j''(\theta_i)$ = *posterior probability of the ith state for the jth material*

$P_j'(\theta_i)$ = *prior probability of the ith state for the jth material*

$P_j(z_k/\theta_i)$ = *likelihood of the kth experimental outcome given the ith state of nature*

N = *normalising constant*

$$= \frac{1}{\sum_{all\ j,k} P_j(z_k/\theta_r).P_j'(\theta_r)} \quad ; \ r = 1,2,3$$

The decision analysis then proceeds in the same manner as the prior analysis, except that the prior probabilities in Fig.4.1 are replaced with posterior probabilities.

The materials selection problem may also be analysed on a complete decision tree that integrates all the elements essential for both the prior and posterior analyses. This decision tree is shown in Fig.4.2; and provides for materials selection with existing information (node 0), with additional information for materials in their natural states (node 1), and with additional information for materials in their treated (stabilized states) (node 2). A flow chart illustration of the decision model for pavement materials selection is presented in Fig.4.3.

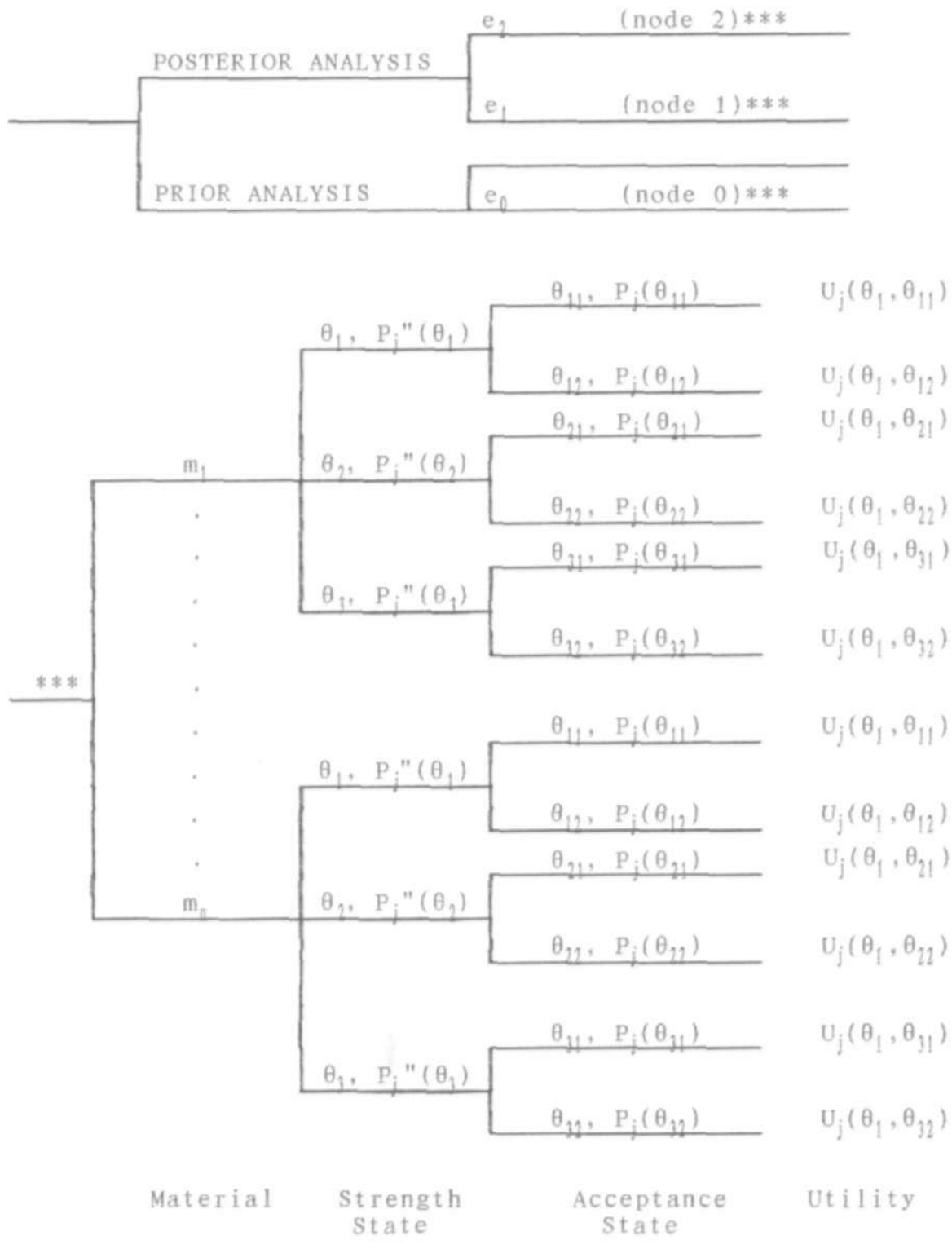


Fig.4.2 - COMPLETE DECISION TREE FOR MATERIALS SELECTION

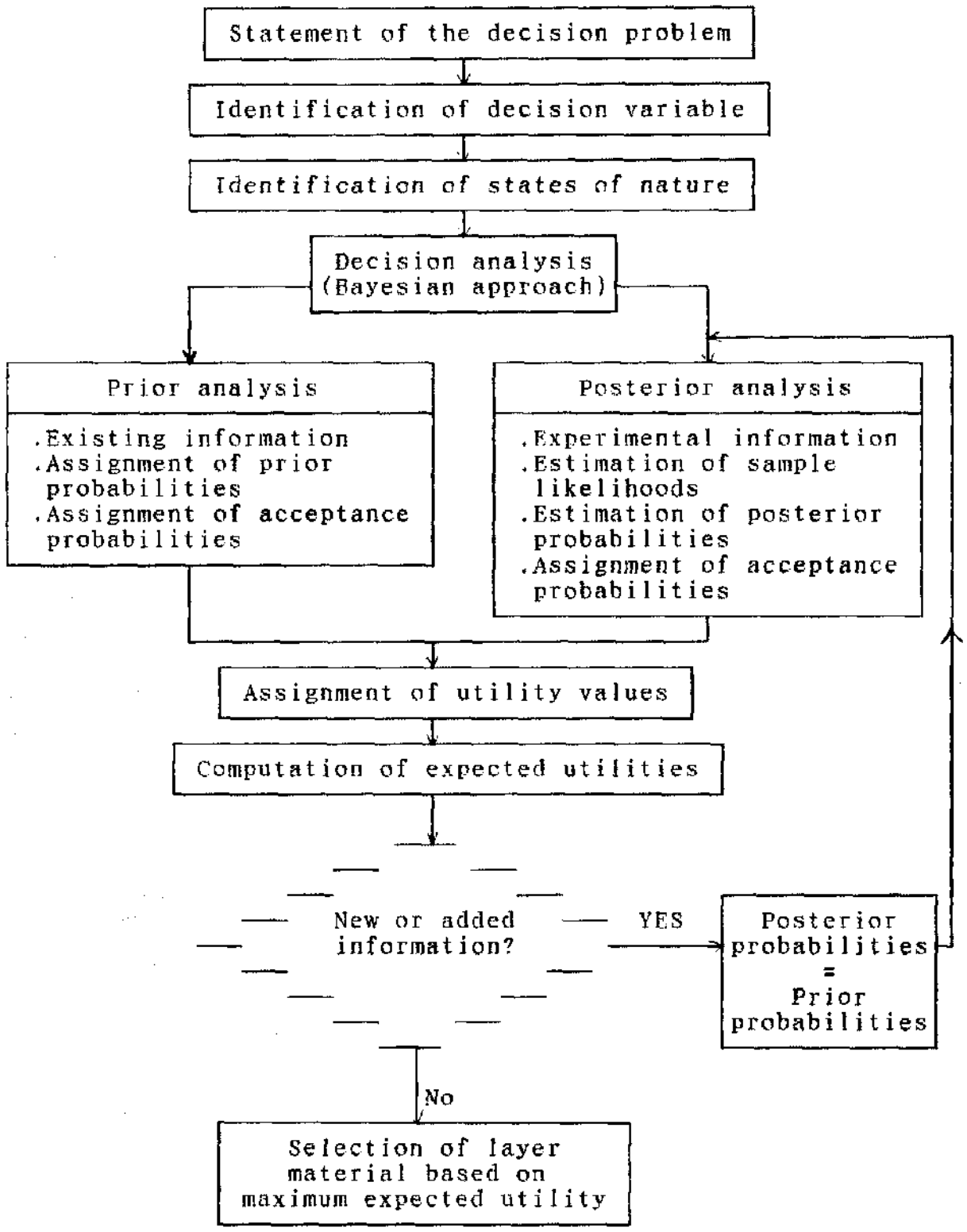


Fig.4.3 - FLOWCHART ILLUSTRATION OF DECISION MODEL FOR PAVEMENT MATERIALS SELECTION

4.4.3 INFLUENCE OF COSTS ON DECISION MAKING

Decision making under conditions of certainty or risk, has cost implications on the system. Thus, the quality of decision when weighted with the implied unit costs, gives good indication of when profits or benefits derived from the decision are maximized, or the total costs or losses involved are minimized. Hence, the unit cost(s) of the decision variable(s) significantly influence the magnitude of benefits or otherwise obtainable from a particular decision.

In the decision theory approach to the analysis of decisions under uncertainty, the costs associated with a given decision situation, are an important input to the attainment of the optimal decision, by considering the relative likelihoods of the consequences of alternative actions in the decision analysis. For the specific case of pavement materials selection, the magnitudes of unit costs of the alternative materials are a major consideration in the assignment of acceptance probabilities. For example, in addition to strength requirements, the cost per unit volume of compacted material, influences the degree to which a decision maker will be willing to accept a given layer material for use in a pavement structure. Thus, the optimal decision is attained in such a manner in which expected costs are balanced with likelihoods. A less than optimal decision will therefore, more likely result in the inefficient use of resources, due to incorrect selection of materials; while an optimum decision will most likely result

in financial savings being made, as well as, ensuring the efficient use of resources.

It should be noted, that the financial savings from an optimal decision may either be at the time the decision is made, or at some future date. This is so because, if at the time of the decision, a cheaper alternative is selected, then there is financial savings resulting from the rejection of a more expensive alternative. If however, a more expensive alternative is selected as the optimal decision, then financial savings results in terms of early repair and maintenance costs which would have had to be incurred due to the selection of a cheaper alternative.

CHAPTER 5

MODEL ILLUSTRATION

To illustrate the decision theory approach to pavement materials selection, the formulated decision model is applied to the selection of a suitable base course material, to be used in the pavement structure for a proposed expressway in Kaduna, Nigeria, as a case study. The choice of the base course for the model illustration is informed by the fact that, it is the primary load-spreading structural component of the pavement. The need therefore, to select a material that satisfactorily satisfies structural requirements for a given base course thickness, in order to achieve a specified total pavement thickness cannot be overemphasized.

5.1 BACKGROUND INFORMATION ON CASE STUDY

5.1.1 THE PROJECT

The project chosen is the engineering design of the eastern by-pass for the city of Kaduna, in Kaduna state, Nigeria. The engineering design of the 26 kilometer, 4-lane dual carriageway has, as part requirement, route soil investigation to determine the suitability of the insutu soil as subgrade material and to provide necessary data for pavement design; materials survey to locate good borrow pit materials, suitable for use as base and sub-base courses; and deep soils investigation to determine the sub-surface condition, as well as, depth to and condition of bedrock.

5.1.2 SITE DESCRIPTION AND GEOLOGY

The proposed Kaduna eastern by-pass begins from Rigachukun village along the existing Kaduna-Zaria road. It runs in an easterly direction for about two kilometers before bending southward to cross river Kaduna and meet the existing Kaduna-Kachia road. The layout of the proposed road is shown on Drwg.5.1. Thus, the road will principally serve to connect the northern part of Kaduna town with the southern part, and hence, provide a faster by-pass to the already congested urban center. The alignment, lies within the basement complex region of Nigeria and is underlain by crystalline rocks. The main rock types of this formation include the granites, gneisses, and biotites of the pre-cambrian age. Outcrops of the basement rocks are also located within the vicinity of the proposed alignment. The main products of weathering present are clayey sands, hard ferruginous sandstones, lateritic gravel, and sand deposits along river Kaduna and its tributaries. The site topography and existing geological section of the proposed road alignment are shown on Drwg.5.2 and Drwg.5.3, respectively.

5.2 ALTERNATIVE MATERIALS

By considering the economics of materials procurement and the need for overall project economy, it is desirable that suitable construction materials be located in close proximity to the project site. For the project being considered, the possible alternative materials for the base course layer of the

pavement are lateritic gravel (m_1), and crushed rock (m_2), both of which are found within the environs of the proposed alignment.

Lateritic gravel, as earlier discussed in chapter 3, generally make good material for base course construction. It is however, a material with variable behavioural trends, and with no reasonably constant properties. The nature of lateritic gravel therefore, indicates the need for care in its selection and use for any given traffic and climatic conditions. The presence of lateritic gravel along the proposed alignment is indicated by patches of the material along hill slopes. Possible borrow locations for lateritic gravel are shown on Drwg.5.2, as BP1,BP2,.....BP12.

On the other hand, crushed rock is a high quality material for use as a base course layer in a pavement structure. It may be sourced from the outcrops of granite and other basement complex rocks, which are within the vicinity of the alignment. The quality of crushed rock however, is adversely affected by the presence of vegetable or organic materials, or other deleterious materials, as well as, the width of fracture lines on the surface of the rock mass, which indicate sign of decomposition and hence, weakness of the rock material. The already existing stone quarries in the vicinity of the alignment will however be a source of useful information regarding rock quality. As will be expected, crushed rock, by virtue of its superior quality, is more expensive in terms of

θ_{i1} (i.e., acceptable) and

θ_{i2} (i.e., not acceptable)

where i is the state of strength.

5.4 PRIOR PROBABILITY ASSIGNMENTS - STRENGTH STATES

The prior probabilities of strength represent the decision maker's subjective assessment of the possible states of strength. These probability assignments were obtained through interviews with three decision makers (highway engineers). The prior information available to the decision makers were topographical and geological maps, historical records from gravel quarries that already exist in the areas close to the proposed alignment, and their respective experiences from similar projects. A profile of the decision makers, as well as, their stated preferences for the alternative materials, m_1, m_2, are given in Table 5.1; while the prior probabilities of strength, $P_j'(\theta_i)$, as extracted from the decision makers are given in Table 5.2.

TABLE 5.1 - PROFILE OF TARGET DECISION MAKERS

	DM1	DM2	DM3
EDUCATION	MSc (1979)	BEng (1987)	BEng (1992)
PROFESSIONAL STATUS	Consulting engineer	Civil engineer I (public service)	Postgraduate student
FIELD EXPERIENCE	16 years	8 years	-
STATED PREFERENCE	$m_1 > m_2$	$m_1 > m_2$	$m_2 > m_1$

TABLE 5.2 - PRIOR PROBABILITIES OF STRENGTH AS ASSIGNED BY TARGET DECISION MAKERS

ALTERNATIVE MATERIAL	DECISION MAKER	PERCEIVED MATERIAL STRENGTH STATE	
		θ_1(CBR≥80%)	θ_2(CBR<80%)
m_1	DM1	0.80	0.20
	DM2	0.95	0.05
	DM3	0.60	0.40
m_2	DM1	0.70	0.30
	DM2	0.80	0.20
	DM3	0.80	0.20

5.4.1 PRIOR PROBABILITY ASSIGNMENTS-ACCEPTANCE STATES

The decision makers expressed, as probabilities, the degrees to which they are willing to accept a base course material with a given state of strength values. In assigning the probabilities, such factors as the expected traffic intensity, subgrade conditions, drainage conditions, expected level of subsequent maintenance, haul distances, unit costs of compacted material, and related experience from the observed performance of the given materials, were considered. These acceptance probability assignments, $P_j(\theta_{ik})$, which reflect the stated individual preferences of the decision makers for the alternative materials, are given in Table 5.3.

TABLE 5.3 - PRIOR ACCEPTANCE PROBABILITIES AS ASSIGNED BY THE TARGET DECISION MAKERS

ALTERNATIVE MATERIAL	DECISION MAKER	PERCEIVED MATERIAL STRENGTH STATE			
		θ_1(CBR≥80%)		θ_2(CBR<80%)	
		STRENGTH PROB.	ACCEPT. PROB.	STRENGTH PROB.	ACCEPT. PROB.
	DM1	0.80	0.80	0.20	0.20
m_1	DM2	0.95	0.60	0.05	0.40
	DM3	0.60	0.50	0.40	0.50
	DM1	0.70	0.65	0.30	0.35
m_2	DM2	0.80	0.40	0.20	0.60
	DM3	0.80	0.80	0.20	0.20

5.4.2 PRIOR UTILITY ASSIGNMENTS

For each combination of given material, strength state, and acceptance state, utility values are assigned by the utility functions defined by equations (4.8), (4.9), and (4.10). It must be noted at this point, that it is essential that the utility values be computed on an equal thickness basis for all the alternative materials, thereby providing a common basis upon which expected utility values are evaluated and compared. Hence, for the purpose of this illustration, the base course thickness is taken to be 150mm, as recommended by the Federal Ministry of Works and Housing Design Manual (1973), and the Road Note 31 (1977).

Thus, from equation (4.8a),

$$U_j(\theta_i,\theta_{ik}) = T_2[29.14(x_B)-0.1977(x_B)^2+0.00045(x_B)^3]*10^{-4}$$

where T_2 = 150mm

For the specified CBR of 80%,

$U_j(\theta_i,\theta_{ik}) = 150[29.14(80)-0.1977(80)^2+0.00045(80)^3]*10^{-4}$

$= +19.44$

That is,

$U_1(\theta_1,\theta_{11}) = U_2(\theta_1,\theta_{11}) = +19.44$

From equation (4.9),

$U_j(\theta_i,\theta_{ik}) = -T_2[29.14(x_B)-0.1977(x_B)^2+0.00045(x_B)^3]*10^{-4}$

$= -19.44$

That is,

$U_1(\theta_2,\theta_{21}) = U_2(\theta_2,\theta_{21}) = -19.44$

From equation (4.10),

$U_j(\theta_i,\theta_{ik}) = 0$

That is,

$U_1(\theta_1,\theta_{12}) = U_1(\theta_2,\theta_{22}) = U_2(\theta_1,\theta_{12}) = U_2(\theta_2,\theta_{22}) = 0$

5.4.3 PRIOR ANALYSIS

Based on the information at hand, prior probability assignments on the strength and acceptability states, and the assigned utility values, the prior analysis situation for DM1 is represented by the decision tree in Fig.5.1.

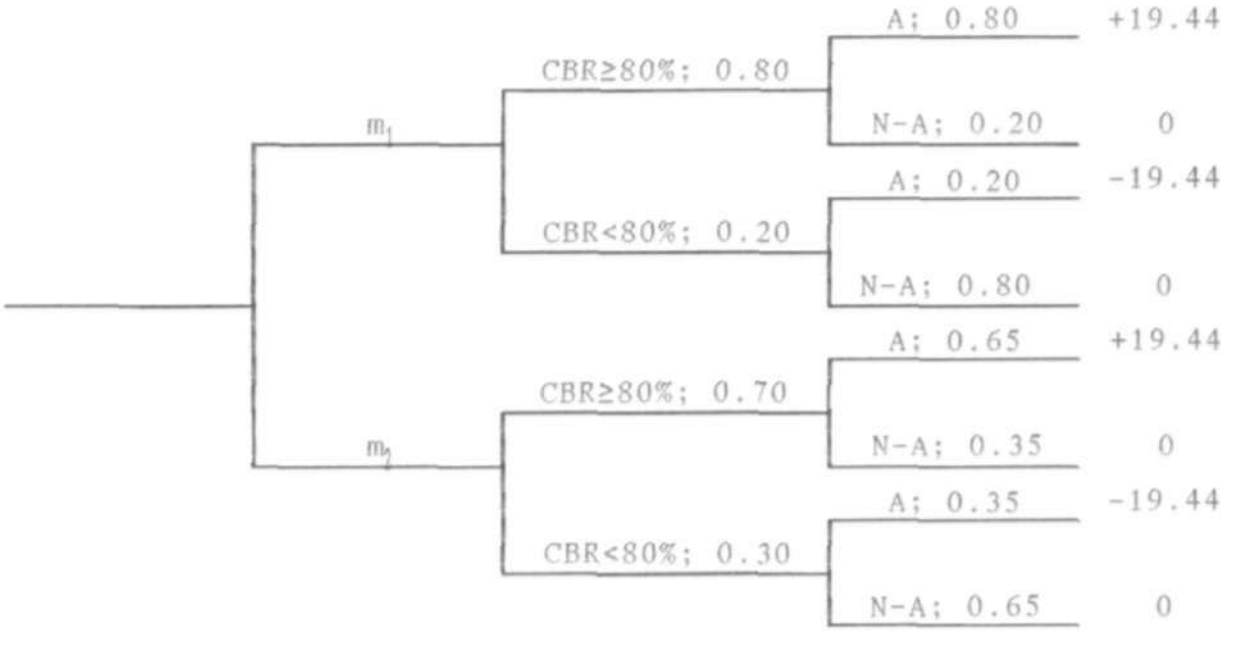


Fig.5.1 - PRIOR DECISION TREE (DM1)

To make the optimal material selection, the expected utility from each alternative material is calculated.

From equation (4.11),

the expected utility given that m_1 is selected and θ_1 observed is,

$$E[U/m_1,\theta_1] = \Sigma_{all\ k}\ P_1(\theta_{1k}).U_1(\theta_1,\theta_{1k})$$

$$= P_1(\theta_{11}).U_1(\theta_1,\theta_{11})+P_1(\theta_{12}).U_1(\theta_1,\theta_{12})$$

$$= (0.80)(19.44)+(0.20)(0)$$

$$= +15.552$$

the expected utility given that m_1 is selected and θ_2 observed is,

$E[U/m_1, \theta_2] = \Sigma_{all\ k}\ P_1(\theta_{2k}).U_1(\theta_2, \theta_{2k})$

$= P_1(\theta_{21}).U_1(\theta_2, \theta_{21}) + P_1(\theta_{22}).U_1(\theta_2, \theta_{22})$

$= (0.20)(-19.44) + (0.80)(0)$

$= -3.888$

From equation (4.12),

the expected utility given that m_1 is selected is,

$E[U/m_1] = \Sigma_{all\ i}\ E[U/m_1, \theta_i].P_1'(\theta_i)$

$= E[U/m_1, \theta_1].P_1'(\theta_1) + E[U/m_1, \theta_2].P_1'(\theta_2)$

$= (15.552)(0.80) + (-3.888)(0.20)$

$= +11.66$

Similarly, for material m_2,

$E[U/m_2, \theta_1] = \Sigma_{all\ k}\ P_2(\theta_{1k}).U_2(\theta_1, \theta_{1k})$

$= P_2(\theta_{11}).U_2(\theta_1, \theta_{11}) + P_2(\theta_{12}).U_2(\theta_1, \theta_{12})$

$= (0.65)(19.44) + (0.35)(0)$

$= +12.636$

$E[U/m_2, \theta_2] = \Sigma_{all\ k}\ P_2(\theta_{2k}).U_2(\theta_2, \theta_{2k})$

$= P_2(\theta_{21}).U_2(\theta_2, \theta_{21}) + P_2(\theta_{22}).U_2(\theta_2, \theta_{22})$

$= (0.35)(-19.44) + (0.65)(0)$

$= -6.804$

$E[U/m_2] = \Sigma_{all\ i}\ E[U/m_2, \theta_i].P_2'(\theta_i)$

$= E[U/m_2, \theta_1].P_2'(\theta_1) + E[U/m_2, \theta_2].P_2'(\theta_2)$

$= (12.636)(0.70) + (-6.804)(0.30)$

$= +6.80$

From equation (4.13),

the optimal material is selected on the basis of maxim expected utility. That is,

$m_{opt} = \max_j[E(U/m_j)]$

Now,

$E[U/m_1] = +11.66$

$E[U/m_2] = +6.80$

Therefore, $m_{opt} = \max[+11.66, +6.80] = +11.66$

Hence, m_1 (lateritic gravel) is the optimal choice.

5.5 LABORATORY INVESTIGATIONS

Investigations were carried out to locate suitable pavement construction materials, and specifically to acquire additional information about the materials strength states. Borrow areas were assessed by investigating a grid of trial pits at 60.0m centres. Bulk samples were collected using hand tools from trial pits which were of depths 1.5m below ground level. The CBR test was performed on the samples in accordance with the procedures of BS 1377 (1990), and for a soaking period of 24 hours. The summary of the test results is presented in Table 5.4.

TABLE 5.4 - SUMMARY OF CBR LABORATORY TEST RESULTS

BORROW AREA No. & LOCATION	TRIAL PIT No.	SAMPLE DEPTH	MOD. AASHTO COMPACTION			SAMPLE DESCRIPTION
			MDD	OMC	CBR	
		(m)	(Mg/m^3)	(%)	(%)	
B/P 3 Km3+800	1	1.50	2.01	12.0	80	Red-brown clayey lateritic gravel
	2	"	2.01	14.0	75	"
	3	"	1.98	14.8	85	"
	4	"	2.01	11.1	79	"
	5	"	2.05	11.0	82	"
B/P 4 Km5+550	1	1.50	1.93	10.2	88	Lateritic gravelly clay
	2	"	1.89	11.6	83	"
	3	"	1.85	10.0	80	"
	4	"	1.89	10.2	88	"
	5	"	1.85	10.2	88	"
B/P 5 Km10+550	1	1.50	1.85	10.7	76	Red brown clayey lateritic gravel
	2	"	2.01	10.2	80	"
	3	"	2.06	10.5	80	"
	4	"	2.01	10.6	85	"
	5	"	2.04	9.3	87	"
B/P 6 Km13+000	1	1.50	1.98	10.2	79	Brown clayey lateritic gravel
	2	"	2.02	9.8	80	"
	3	"	1.99	10.0	85	"
	4	"	2.00	10.3	82	"
	5	"	2.01	10.7	81	"

TABLE 5.4 (CONTINUED)

B/P 10 Km22+000	1	1.50	2.06	10.6	83	Brown lateritic gravel
	2	"	2.10	9.7	90	"
	3	"	2.06	10.8	85	"
	4	"	2.08	10.6	85	"
	5	"	2.08	9.8	79	"
B/P 11 Km23+200	1	1.50	2.07	10.5	75	Brown lateritic gravel
	2	"	2.05	9.5	95	"
	3	"	1.97	14.2	85	"
	4	"	1.99	14.0	85	"
	5	"	2.07	13.0	86	"
B/P 12 Km25+600	1	"	2.05	10.3	80	Brown lateritic gravel
	2	"	2.06	10.0	79	"
	3	"	1.99	12.8	83	"
	4	"	1.98	13.6	87	"
	5	"	2.04	13.2	81	"

Using the laboratory results, we have:

Mean CBR value ($x_{\bar{}}$) = 82.88%

Standard deviation (σ) = 4.34

Coefficient of variation (COV) = 0.052

5.5.5 SAMPLE LIKELIHOODS

The values of soil strength are assumed to be normally distributed (section 4.3.8). Therefore, the likelihood of an experimental outcome given a particular strength state may be

calculated from the probability density function of the 'normal' distribution. The probability density function of a normally distributed random variable (x) with a mean (μ) and standard deviation (σ) is given by:

$$f_x(x) = \frac{1}{\sigma\sqrt{2\pi}}\exp\left[-\frac{1}{2}\left(\frac{x-\mu}{\sigma}\right)^2\right] \; ; \quad -\infty<x<\infty \qquad \text{.....(5.1a)}$$

and the probability,

$$P(a<x\leq b) = \frac{1}{\sigma\sqrt{2\pi}}\int_a^b \exp\left[-\frac{1}{2}\left(\frac{x-\mu}{\sigma}\right)^2\right] \qquad \text{.....(5.1b)}$$

$$= \phi\left(\frac{b-\mu}{\sigma}\right) - \phi\left(\frac{a-\mu}{\sigma}\right) \qquad \text{.....(5.1c)}$$

where,

a,b = lower and upper limits that define the range of the random variable

ϕ = standard normal distribution function

Since by definition, the decision variable (CBR) ranges from 0 to 100 (section 4.3.3), it is necessary that the probability distribution function is normalized before being applied. This may be achieved by evaluating the integral in equation (5.1b) with the limits a, b set to 0 and 100 respectively, to yield a scaling factor. This scaling factor, when applied to the computed sample likelihoods, ensures that they form a proper set of probabilities. The scaling factor obtained for this particular case, is 0.999959, which is approximately taken as 1.

Now,

if z_1 = experimental outcome that CBR≥80% (i.e., θ_1), then,

$$P(z_1/\theta_1) = P(X \geq 80\%)$$
$$= P(80 \leq X < 100)$$
$$= \phi\left(\frac{100-\mu}{\sigma}\right) - \left(\phi\frac{80-\mu}{\sigma}\right) \quad \ldots\ldots(5.2a)$$

and,

$$P(z_1/\theta_2) = P(X < 80\%)$$
$$= \phi\left(\frac{80-\mu}{\sigma}\right) \quad \ldots\ldots(5.2b)$$

From the summary of test results presented in Table 5.4,

the mean, $\mu = \bar{x} = 82.88\%$

standard deviation, $\sigma = 4.34$

substituting these values into equations (5.2a) and (5.2b),

$$P(z_1/\theta_1) = \phi\left(\frac{100-82.88}{4.34}\right) - \phi\left(\frac{80-82.88}{4.34}\right)$$
$$= \phi(3.945) - \phi(-0.663)$$
$$= \phi(3.945) - [1-\phi(0.663)]$$
$$= 0.745 \text{ (from normal distribution tables)}$$

and,

$$P(z_1/\theta_2) = \phi\left(\frac{80-82.88}{4.34}\right)$$
$$= \phi(-0.663)$$
$$= 1-\phi(0.663)$$
$$= 0.255$$

5.5.2 POSTERIOR PROBABILITIES

From equation (4.14), posterior probabilities of state are obtained as follows:

$$P_j''(\theta_i) = N.P_j(z_k/\theta_i).P_j'(\theta_i)$$

where,

$$N = \frac{1}{\sum_{all\ xjk} P_j(z_k/\theta_r).P_j'(\theta_r)}$$

Thus,

$$P_j''(\theta_1) = \frac{P_j(z_1/\theta_1).P_j'(\theta_1)}{P_j(z_1/\theta_1).P_j'(\theta_1).+P_j(z_1/\theta_2).P_j'(\theta_2)} \quad \ldots\ldots\ldots\ldots (5.3a)$$

$$P_j''(\theta_2) = \frac{P_j(z_1/\theta_2).P_j'(\theta_2)}{P_j(z_1/\theta_2).(P_j'(\theta_2)+(P_j(z_1/\theta_1).P_j'(\theta_1)} \quad \ldots\ldots\ldots\ldots (5.3b)$$

For material m_1,

$P_1(z_1/\theta_1) = 0.745$

$P_1(z_1/\theta_2) = 0.255$

and for DM1,

$P_1'(\theta_1) = 0.80$

$P_1'(\theta_2) = 0.20$

For material m_2,

$P_2(z_1/\theta_1) = 1.000$

$P_2(z_1/\theta_2) = 0.000$

($P_2(z_1/\theta_1)$ = 1.000 because CBR tests are seldom carried out on crushed rock as it is the standard material (CBR=100%) against which the CBR values of other materials are measured).

and for DM1,

$P_2'(\theta_1)$ = 0.70

$P_2'(\theta_2)$ = 0.30

Therefore,

$$P_1''(\theta_1) = \frac{(0.745)(0.80)}{(0.745)(0.80)+(0.255)(0.20)} = 0.921$$

$$P_1''(\theta_2) = \frac{(0.255)(0.20)}{(0.255)(0.20)+(0.745)(0.80)} = 0.079$$

and,

$$P_2''(\theta_1) = \frac{(1.000)(0.70)}{(1.000)(0.70)+(0)(0.30)} = 1.000$$

$$P_2''(\theta_2) = \frac{(0)(0.30)}{(0)(0.30)+(1.000)(0)} = 0.000$$

5.5.3 POSTERIOR ACCEPTANCE PROBABILITY ASSIGNMENTS

The posterior acceptance probabilities were assigned by the decision makers based on the additional information about the states of strength of the alternative materials, as reflected through the updated or posterior probabilities of state. These probabilities as assigned by the target decision makers are given in Table 5.5.

TABLE 5.5 - POSTERIOR ACCEPTANCE PROBABILITIES AS ASSIGNED BY THE TARGET DECISION MAKERS

ALTERNATIVE MATERIAL	DECISION MAKER	PERCEIVED MATERIAL STRENGTH STATE			
		θ_1(CBR≥80%)		θ_2(CBR<80%)	
		STRENGTH PROB.	ACCEPT. PROB.	STRENGTH PROB.	ACCEPT. PROB.
	DM1	0.921	0.90	0.079	0.10
m_1	DM2	0.984	0.60	0.016	0.40
	DM3	0.814	0.75	0.186	0.25
	DM1	1.000	0.70	0.000	0.30
m_2	DM2	1.000	0.40	0.000	0.60
	DM3	1.000	0.85	0.000	0.15

5.5.4 POSTERIOR UTILITY ASSIGNMENTS

From equation (4.8a); m_1; T_2=150mm; x_B=82.88%

$$U_j(\theta_i,\theta_{ik}) = T_2[29.14(x_B)-0.1977(x_B)^2+0.00045(x_B)^3]*10^{-4}$$

$$=150[29.14(82.88)-0.1977(82.88)^2+0.00045(82.88)^3]*10^{-4}$$

$$= +19.70$$

That is,

$$U_1(\theta_1,\theta_{11}) = +19.44$$

From equation (4.9),

$$U_j(\theta_i,\theta_{ik}) = -T_2[29.14(x_B)-0.1977(x_B)^2+0.00045(x_B)^3]*10^{-4}$$

$$= -19.70$$

That is,

$$U_1(\theta_2,\theta_{21}) = -19.44$$

From equation (4.10),

$$U_j(\theta_i,\theta_{ik}) = 0$$

That is,

$U_1(\theta_1,\theta_{12}) = U_1(\theta_2,\theta_{22}) = 0$

Similarly, for m_2; T_2=150; x_B=100.00%

$U_2(\theta_1,\theta_{11}) = +20.80$

$U_2(\theta_2,\theta_{21}) = -20.80$

$U_2(\theta_1,\theta_{12}) = U_2(\theta_2,\theta_{22}) = 0$

5.5.5 POSTERIOR ANALYSIS

The posterior analysis proceeds in the same manner as the prior analysis except that the prior probabilities are replaced with posterior probabilities. The posterior analysis situation for DM1 is represented by the decision tree in Fig.5.2.

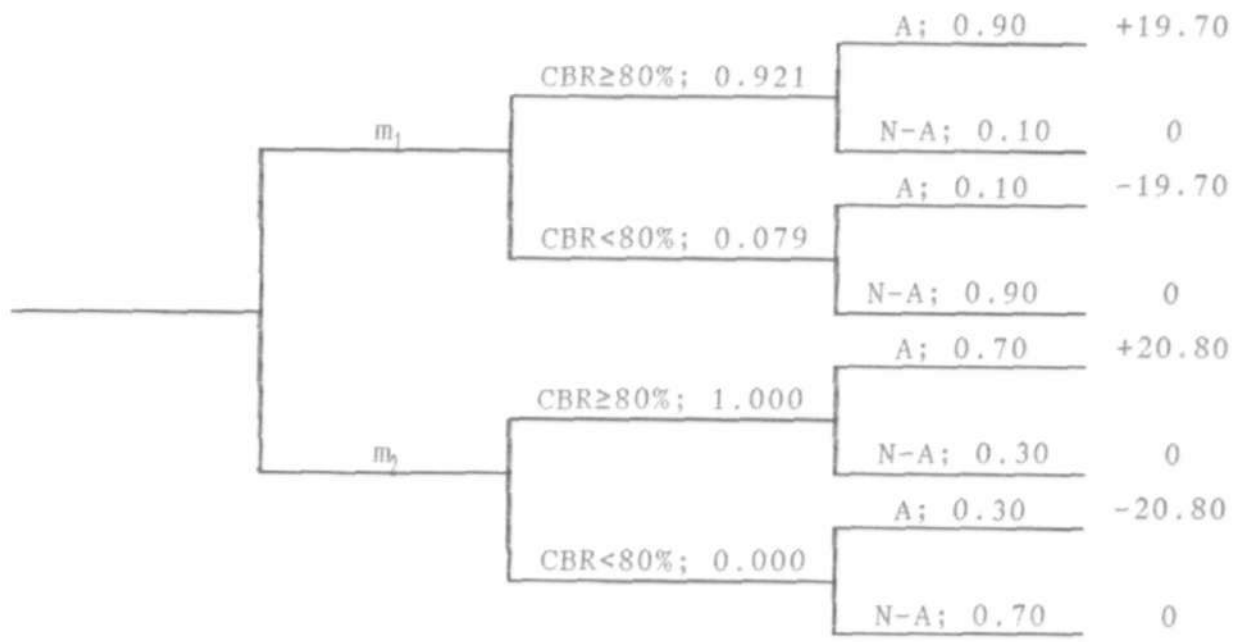


Fig.5.2 - POSTERIOR DECISION TREE (DM1)

From equation (4.11),

the expected utility given that m_1 is selected and θ_1 observed is,

$$E[U/m_1,\theta_1] = \Sigma_{\text{all } k}\ P_1(\theta_{1k}).U_1(\theta_1,\theta_{1k})$$

$$= P_1(\theta_{11}).U_1(\theta_1,\theta_{11})+P_1(\theta_{12}).U_1(\theta_1,\theta_{12})$$

$$= (0.90)(19.70)+(0.10)(0)$$

$$= +17.730$$

the expected utility given that m_1 is selected and θ_2 observed is,

$$E[U/m_1,\theta_2] = \Sigma_{\text{all } k}\ P_1(\theta_{2k}).U_1(\theta_2,\theta_{2k})$$

$$= P_1(\theta_{21}).U_1(\theta_2,\theta_{21})+P_1(\theta_{22}).U_1(\theta_2,\theta_{22})$$

$$= (0.10)(-19.70)+(0.90)(0)$$

$$= -1.970$$

From equation (4.12),

the expected utility given that m_1 is selected is,

$$E[U/m_1] = \Sigma_{\text{all } i}\ E[U/m_1,\theta_i].P_1'(\theta_i)$$

$$= E[U/m_1,\theta_1].P_1'(\theta_1)+E[U/m_1,\theta_2].P_1'(\theta_2)$$

$$= (17.730)(0.921)+(-1.921)(0.079)$$

$$= +16.17$$

Similarly, for material m_2,

$$E[U/m_2,\theta_1] = \Sigma_{\text{all } k}\ P_2(\theta_{1k}).U_2(\theta_1,\theta_{1k})$$

$$= P_2(\theta_{11}).U_2(\theta_1,\theta_{11})+P_2(\theta_{12}).U_2(\theta_1,\theta_{12})$$

$$= (0.70)(20.80)+(0.30)(0)$$

$$= +14.560$$

$$E[U/m_2, \theta_2] = \Sigma_{\text{all } k}\ P_2(\theta_{2k}).U_2(\theta_2, \theta_{2k})$$

$$= P_2(\theta_{21}).U_2(\theta_2, \theta_{21}) + P_2(\theta_{22}).U_2(\theta_2, \theta_{22})$$

$$= (0.30)(-20.80) + (0.70)(0)$$

$$= -6.240$$

$$E[U/m_2] = \Sigma_{\text{all } i}\ E[U/m_2, \theta_i].P_2'(\theta_i)$$

$$= E[U/m_2, \theta_1].P_2'(\theta_1) + E[U/m_2, \theta_2].P_2'(\theta_2)$$

$$= (14.560)(1.000) + (-6.240)(0.000)$$

$$= +14.56$$

From equation (4.13),

the optimal material is selected on the basis of maximum expected utility. That is,

$m_{opt} = \max_j[E(U/m_j)]$

Now,

$E[U/m_1] = +16.17$

$E[U/m_2] = +14.56$

Therefore, $m_{opt} = \max[+16.17, +14.56] = +16.17$

Hence, m_1 (lateritic gravel) is the optimal choice.

The complete decision tree, which integrates all the model components of both the prior and posterior analyses for DM1, DM2, and DM3, are presented in Figs. 5.3, 5.4, and 5.5 respectively.

In the same way as was illustrated for the selection of a base course material in this chapter, the model may be applied to the selection of alternative materials for the sub-base layer, as well as, fill material for the subgrade.

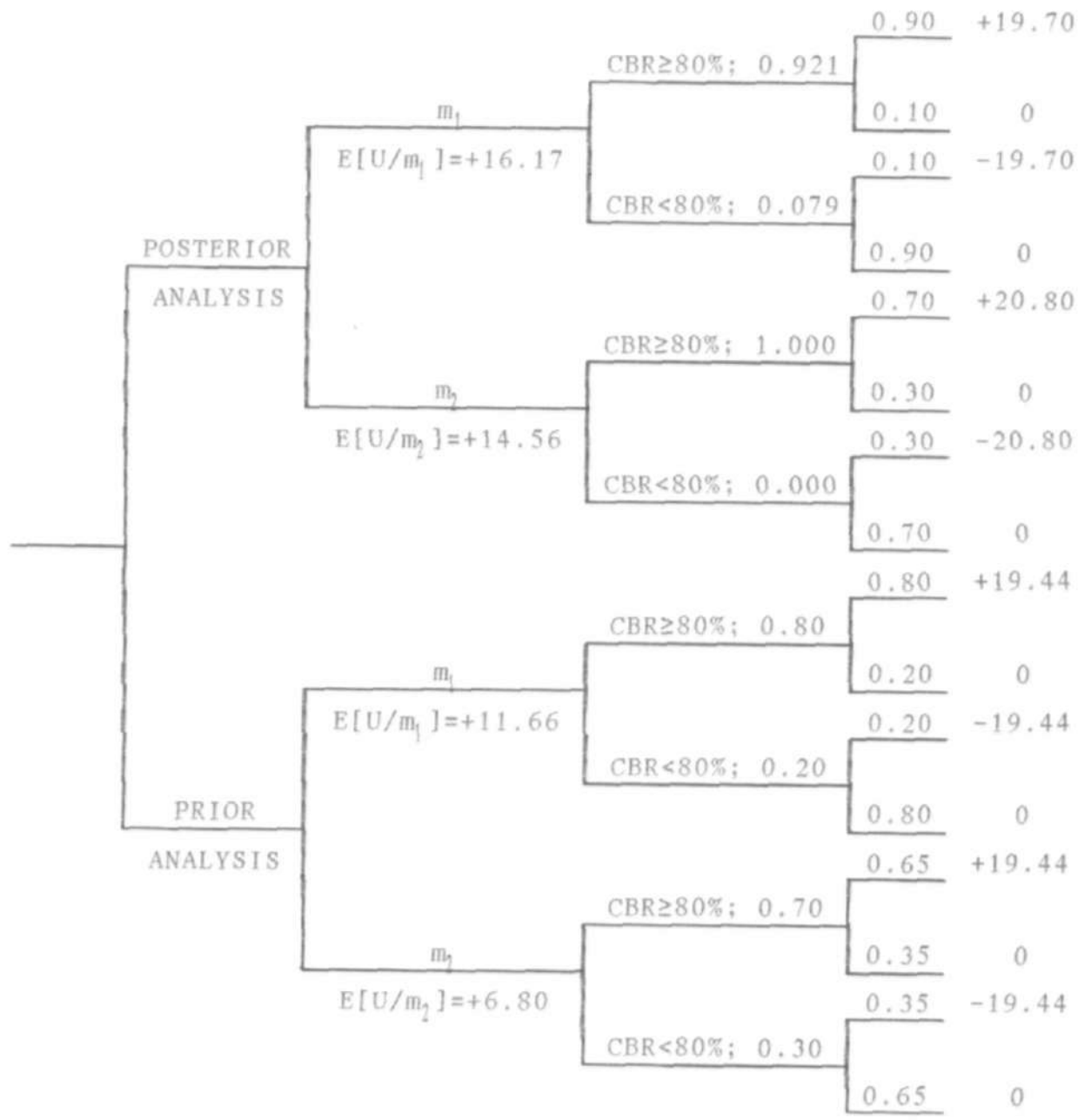


Fig.5.3 - COMPLETE DECISION TREE (DM1)

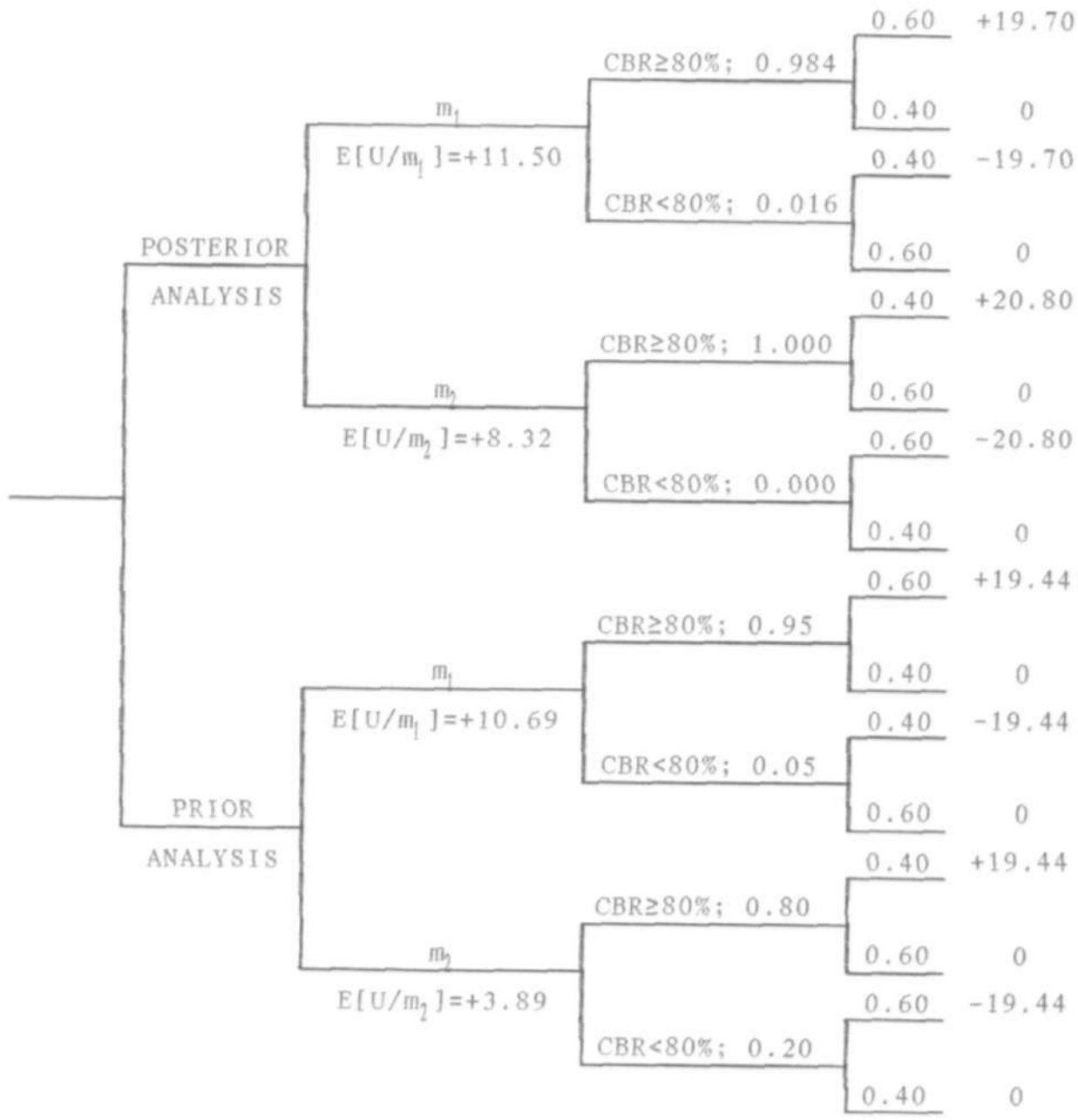


Fig.5.4 - COMPLETE DECISION TREE (DM2)

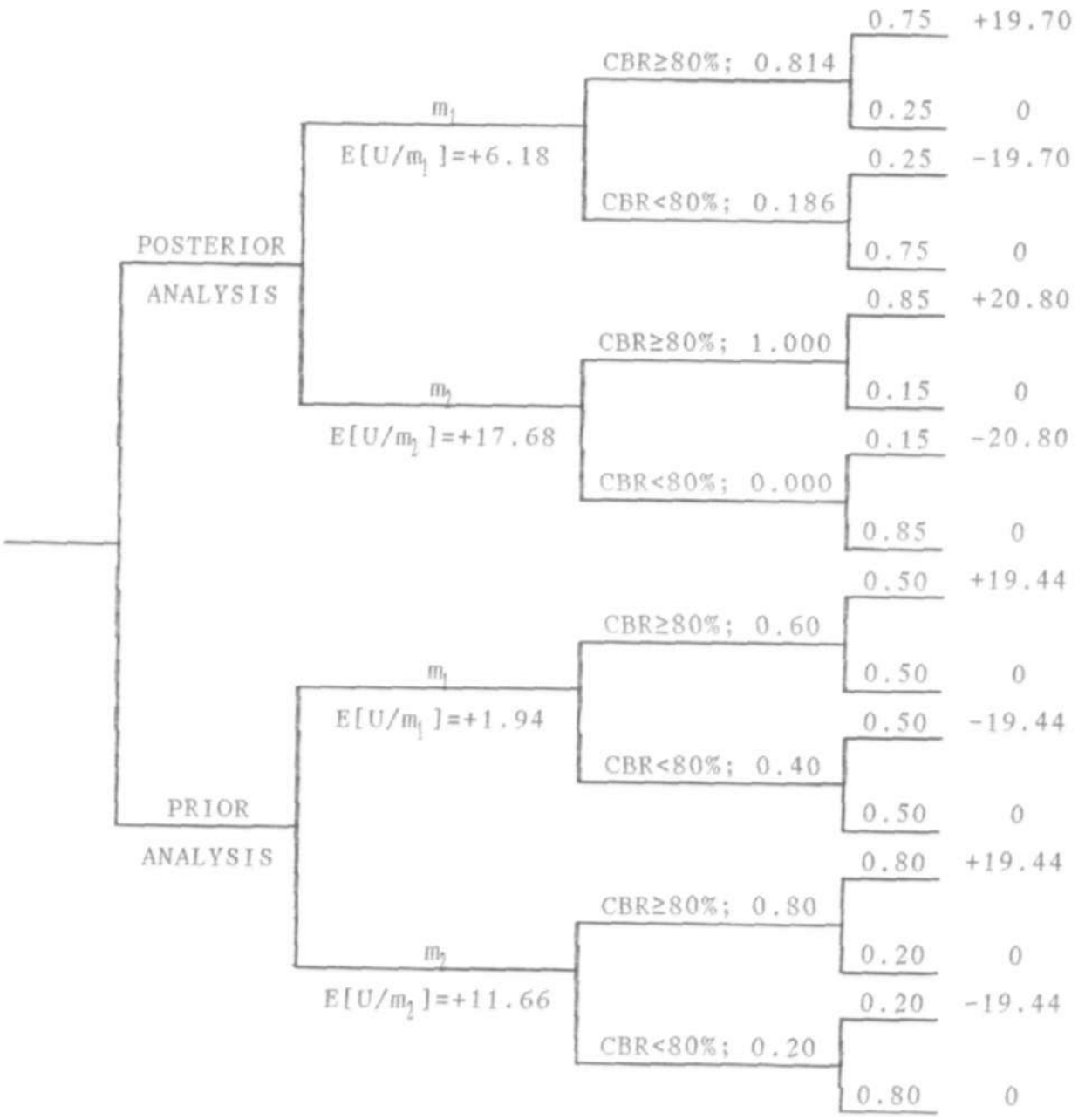


Fig.5.5 - COMPLETE DECISION TREE (DM3)

CHAPTER 6

DISCUSSION, CONCLUSIONS, AND RECOMMENDATIONS

6.1 DISCUSSION OF RESULTS

The model which has been developed, provides a means of evaluating engineering judgement and combining it with objective information to obtain an optimal selection of pavement materials. Thus, a decision maker can make decisions more confidently, based on information available at a given moment (Fig.4.3). It does not replace the institutionalized process of materials selection (Fig.3.2), but rather, enhances its reliability by accounting for the states of uncertainty involved. This singular feature, underscores the superiority of the probabilistic approach to pavement materials selection over the institutionalized deterministic procedures of materials evaluation and selection based solely on compliance with given specifications.

The decision criterion in the probabilistic approach, is the maximum expected utility criterion. The expected utility represents the weighted consequences of alternative actions, and is a measure of value to the decision maker after due consideration of the relative likelihoods associated with the respective states of nature. Thus, the optimal decision is selected as the alternative with the highest expected utility.

The acquisition of additional information, in seeking to reduce the degree of uncertainty, necessitates the updating of prior (or, existing) probability assignments, and hence,

expected utilities of the alternative actions. The effect of additional information on the decision analysis may be clearly understood by observing the influence of sample likelihoods on the prior probabilities of strength states, for the case study considered in this work. The sample likelihood is the probability of obtaining the observed experimental outcome, z_k, as a function of the true state of nature, θ_i. Sample likelihoods can also be interpreted as the relative likelihoods of the various states of nature given the experimental observation, z_k. If this likelihood is relatively higher for θ_i than the other states, the posterior probability of this state, $P''(\theta_i)$ will be increased over its prior probability, $P'(\theta_i)$. If θ_i is a relatively unlikely state to be associated with observation z_k, its posterior probability will reflect this by being smaller than its prior probability. The results obtained for the case study are presented in Tables 6.1(a) and 6.1(b).

TABLE 6.1(a) - SUMMARY OF DECISION ANALYSIS RESULTS FOR MATERIAL M1

DECISION MAKER	PRIOR ANALYSIS			POSTERIOR ANALYSIS		
	STRENGTH PROBABILITY		EXPECTED UTILITY	STRENGTH PROBABILITY		EXPECTED UTILITY
	$P'(\theta_1)$	$P'(\theta_2)$	$E[U/m_1]$	$P''(\theta_1)$	$P''(\theta_2)$	$E[U/m_1]$
DM1	0.80	0.20	+11.66	0.921	0.079	+16.17
DM2	0.95	0.05	+10.69	0.984	0.016	+11.50
DM3	0.60	0.40	+1.94	0.814	0.186	+11.11

Sample likelihood: $P_1(z_1/\theta_1)=0.745$; $P_1(z_1/\theta_2)=0.255$

TABLE 6.1(b) - SUMMARY OF DECISION ANALYSIS RESULTS FOR MATERIAL M2

DECISION	PRIOR ANALYSIS			POSTERIOR ANALYSIS		
MAKER	STRENGTH PROBABILITY		EXPECTED UTILITY	STRENGTH PROBABILITY		EXPECTED UTILITY
	$P'(\theta_1)$	$P'(\theta_2)$	$E[U/m_2]$	$P''(\theta_1)$	$P''(\theta_2)$	$E[U/m_2]$
DM1	0.70	0.30	+6.80	1.00	0.00	+14.56
DM2	0.80	0.20	+3.89	1.00	0.00	+8.32
DM3	0.80	0.20	+11.66	1.00	0.00	+17.68

Sample likelihood: $P_1(z_1/\theta_1)=1.000$; $P_1(z_1/\theta_2)=0.000$

The values of $P''(\theta_1)$ are higher than the values of $P'(\theta_1)$, since the sample likelihoods, 0.745 for M1 and 1.000 for M2, are relatively higher for $\theta_1(\theta \geq 80\%)$ than for $\theta_2(\theta<80\%)$, with sample likelihoods of 0.255 for M1 and 0.000 for M2. Because θ_2 is a relatively unlikely state to be associated with the experimental outcome, z_1, the values of $P''(\theta_2)$ reflect this by being smaller than the values of $P'(\theta_2)$.

Tables 6.1(a) and 6.1(b) also reveal the change in the magnitudes of the expected utilities from the values at the prior analysis stage to those at the posterior analysis stage. Because the expected utilities are weighted utility values, the changes may be attributed to the updating of prior probabilities of state to posterior probabilities after additional information is acquired. The magnitudes of these changes will therefore be dependent on the degrees to which the respective likelihoods change. For example, in Table 6.1(a),

the expected utility for DM2 increases from +10.69 to +11.50 (a change of +0.81), for a change in relative likelihood from 0.95 to 0.984; while for DM3, a more significant change in likelihood 0.60 to 0.814, results in an increase in expected utility from +1.94 to 11.11 (a change of +9.17).

It must be noted however, that the expected utility criterion is sufficient only to obtain a ranking of actions. This is so because, the assignment of a different linearly transformed utility function could yield a different proportion between the expected values, although, not a different conclusion as to the better decision. Therefore, it should not be concluded from the expected utility values that one action is better than the other by say, x%.

The formulated procedures are based on the Bayesian approach to decision making. They provide a framework that assists the decision maker in organizing, accumulating, and evaluating experience. It also serves as a valuable aid to the inexperienced individual who has little feelings for the uncertainties involved in any decision situation. For example, the uncertainty concerning the estimates of a decision variable, such as material strength, can be explicitly represented by a probability distribution function, like the 'normal' distribution. This representation, ensures more confidence in the decisions made under uncertainty as the decision variable is described as a distribution of values instead of being treated as single values.

Although an individual decision making process is dealt with, multiple decision makers were independently interviewed, in order to observe the subjective opinions of different individuals to the same decision problem under conditions of uncertainty. This difference in subjective opinions is clearly observed from the prior probability assignments of the decision makers (Table 5.2). Depending on the circumstances, such subjective probability assignments may not only differ from individual to individual, but even from time to time for one individual. Thus, by the comparison of prior and posterior acceptance probabilities of the decision makers (Tables 5.3 and 5.5), the difference in probability assignments for each individual with changing circumstances (before and after additional information is acquired) is seen. These observations are consistent with the fact that different decision makers have had different experiences and use different reasoning processes (Benjamin and Cornell,1970).

Despite the fact that different individuals may have different probability assignments, the individuals with the same preferences always attain the same optimal decision. This may be attributed to a requirement of the Bayesian method of decision analysis, in which subjective probability assignments must be consistent with stated preferences. As can be seen from Figs.5.3, 5.4, and 5.5, DM1 and DM2 attained the same optimal decision although their respective probability assignments were different; while DM3 attained an optimal decision which was

consistent with his stated preferences.

Furthermore, the assignment of subjective probabilities has an educational effect on the decision maker, who now has to concentrate on his own attitudes to the uncertainties inherent in a decision situation, which he may never have done before. Hence, the decision maker evaluates his attitude to uncertainties more carefully, in attaining an optimal decision for his problem.

Equations (4.8), (4.9), and (4.10), are utility functions which express the relationship between utility and the magnitude of material strength. The formulation and application of the utility functions, illustrates a means of assessing a decision maker's preferences for the various consequences of alternative actions, and by which it is logically consistent to choose between alternative actions by comparing the expected values of the utilities derivable from the respective actions.

The model as formulated is however, not without its limitations. The most apparent of which is that, it cannot be applied to the selection of pavement surfacing materials, which are mainly bituminous materials. This is because of the marked differences in the nature, composition and behaviour between materials that may be used as surfacing materials, and those that may be used for the rest of the pavement structure. Hence, the decision variable, as well as, the formulation of the utility functions as contained herein, will be inappropriate for the selection of pavement surfacing materials.

Finally, it is noteworthy to mention that the pavement materials selection model is a normative, rather than a descriptive model. That is, it prescribes the action to take in order to obtain a specific consequence for a given state of nature. It does not describe how decisions are made, as a descriptive model would.

6.2 CONCLUSIONS

The following conclusions are made, from the work presented herein:

(a) The problem of pavement materials selection may be modelled as a single attribute, individual decision making problem under conditions of uncertainty.

(b) The application of probabilistic concepts to decision problems under uncertainty, not only allows uncertainties regarding the possible states of nature in the pavement materials selection process to be quantified and expressed as probabilities, but also facilitates the combination and updating of these probabilities in the light of new or additional information regarding the states of nature.

(c) The utility function, in terms of the contribution of a given pavement layer to overall pavement strength, provides a useful relation between utility and the magnitude of the decision variable (material strength), which facilitates the numerical assessment of the decision maker's preferences among outcomes of alternative actions.

(d) The maximum expected utility criterion is an appropriate decision criterion for decision problems under conditions of uncertainty, as it ensures that consequences of alternative actions are weighted against their respective relative likelihoods in the evaluation of the optimal decision.

6.3 RECOMMENDATIONS

Based on the work done in the formulation of the pavement materials selection model, and the observations made from its application, the following recommendations are made:

(a) A source of difficulty in the assignment of subjective probabilities is the non availability of historical data upon which to base these assignments. Further, the versatility of probabilistic procedures depend on the quantity and quality of the available information. There is therefore the need for the relevant highway authorities to establish a system of accumulating, maintaining, and updating highway pavement evaluation data, especially data relating to the performance of pavement materials.

(b) The utility function, as formulated in the pavement materials selection model, relates utility to the magnitude of material strength, but does not provide for the explicit representation of the costs involved in the materials acquisition and haulage. There is, therefore, the need for further development of the utility function

in which the utility may be evaluated in monetary terms, so that costs, and their direct influence on decision making, can be explicitly included in the decision analysis.

(c) As already mentioned, the limitation of the pavement materials selection model is that, it cannot be applied to the selection of surfacing materials. It is therefore recommended that based on this work, further research be carried out with the aim of developing procedures to facilitate the optimal selection of surfacing materials.

(d) Uncertainties in pavement materials selection include those relating to the adequacy and relevance of specifications. In tropical regions like Nigeria, this may be attributed to the adoption of empirical design methods developed in temperate regions, and which cannot be applied with confidence in areas outside these regions. There is, thus, an urgent need for the development of an indigenous pavement design method. Preferably, such a design method should consist of a catalogue of designs that may be applied to areas with distinct climatic, environmental, and subgrade conditions.

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APPENDICES

APPENDIX A1

AXIOMS OF UTILITY THEORY

The following symbols define the degrees of preference between two events:

$A \succ B$ denotes "A is preferred to B"

$A \sim B$ denotes "indifference between A and B"

$A \succeq B$ denotes "A is preferred atleast as much as B"

AXIOM 1 - ORDERABILITY

The states of preference between A and B can be arranged in order of increasing preference toward B as follows:

$A \succ B$, $A \succeq B$, $A \sim B$, $A \preceq B$, $A \prec B$

AXIOM 2 - TRANSITIVITY

If $A \succ B$ and $B \succ C$, then $A \succ C$

AXIOM 3 - CONTINUITY

If $A \succ B \succ C$, a value of p(between 0 and 1) exists such that a person is indifferent between (a) obtaining B definitely and (b) getting A or C with probabilities p and (1-p), respectively. That is,

$B \sim [pA, (1-p)C]$

AXIOM 4 - SUBSTITUTABILITY

From Axiom 3, since B and [pA, (1-p)C] are equally preferred, it follows that, B is substitutable for [pA, (1-p)C] without affecting preference.

AXIOM 5 - MONOTONICITY

If $A \succ B$, then

$[pA, (1-p)B] \succ [p'A, (1-p)B]$

if and only if $p > p'$

APPENDIX A2

DETERMINATION OF THE CALIFORNIA BEARING RATIO

A2.1 GENERAL

This method covers the determination of the California bearing ratio (CBR) of a soil, which is obtained by measuring the relationship between force an penetration when a cylindrical plunger of cross-sectional area 1935 mm^2 is made to penetrate the soil at a given rate. At any value of penetration, the ratio of the force to a standard force is defined as the California bearing ratio (CBR). On account of the plunger size, the test is appropriate only to materials having a maximum particle size of 20mm.

The method of sample preparation influences markedly the results obtained form CBR tests. It has not ben found practicable to standardize test procedures sufficiently closely to obtain reproducible results on the wet cohesive soils which occur widely in Great Britain (CBR values below 5%). The test is more applicable in those tropical and sub-tropical areas where drier conditions prevail under roads and airfields.

A2.2 APPARATUS

The following apparatus is required:

(1) 5mm and 20mm BS test sieves.

(2) A cylindrical metal mould having an internal diameter of 152mm and an internal effective height of 127mm, with

deterchable base-plate and top-plate and a collar 50mm deep. The colar is for attachment to the top of the mould during compaction of a soil specimen.

(3) Three metal plugs 150mm in diameter and 50mm thick, for static compaction of a soil specimen. A handle which may be screwed into the plugs facilitates removal after compaction.

(4) A cylindrical metal plunger, the lower end of which shall be of hardened steel and have a cross-sectional area of 1935mm^2. A convenient size would be approximately 250mm long.

(5) A machine for applying the test force through the plunger, consisting of a force-measuring device and means for applying the force at a controlled rate.

(6) A means of measuring the penetration of the cylinder into the specimen and enabling the rate of penetration to be controlled.

(7) Three annular surcharge discs each having a mass of 2Kg, an internal diameter of 52mm to 54mm and an external diameter of 145mm to 150mm.

(8) A metal rammer having a 50mm diameter circular face, and weighing 2.5Kg. The rammer shall be equipped with a suitable arrangement for controlling the height of drop to 300mm.

(9) A metal rammer having a 50mm diameter circular face, and weighing 4.5Kg. The rammer shall be equipped with a

suitable arrangement for controlling the height of drop to 450mm.

(10) A compression machine for static compaction with horizontal platens not less than 200mm square and capable of a separation of not less than 300mm. Maximum force 300KN.

(11) An electric vibrating hammer having a power consumption between 600W and 750W and operating at a frequency between 25Hz and 45Hz.

(12) A steel rod 15mm to 20mm in diameter and 400mm long.

(13) A steel straightedge. Suitable dimensions are 300mmX25mmX3mm.

(14) A spatula having a steel blade approximately 100mmX20mm.

(15) A means of measuring the movement of the top o the specimen during soaking.

(16) A balance capable of weighing up to 25Kg readable and accurate to 5g.

(17) Apparatus for moisture content determination.

(18) Filter papers 150mm in diameter.

A2.3 PREPARATION OF TEST SPECIMEN

The test shall be carried out on material passing the 20mm BS test sieve. The moisture content of the soil shall be chosen to represent the design conditions for which the test results are required. Alternatively, where a moisture range is to be investigated, water shall be added as required to previously

air-dried soil in which soil aggregates greater than 5mm BS test sieve have been broken down.

Thorough mixing is essential and the soil shall, where practicable, be sealed and stored for 24h before compaction.

One of the alternative methods of compaction, static compaction or dynamic compaction, shall be used.

A2.3.1 STATIC COMPACTION

The mass of wet soil, m_1 in g, required for one specimen to be compacted to the appropriate dry density shall be calculated from the equation:

$$m_1 = 23.05(100+\omega)\rho_d$$

where,

ω = *moisture content of the soil* (%);

ρ_d = *dry density of the soil* (Mg/m^3)

Static compaction shall be carried out by one of the following methods:

Method 1

The mould shall be assembled with its collar and base-plate, the latter covered by filter paper. The weighed soil shall be poured slowly into the mould, whilst being tamped continuously by the steel rod. The amount of tamping shall be such that when all the soil has been added, the level is about 5mm to 10mm above the top of the mould. After tamping, a filter

paper shall be placed on the top of the soil followed by a 50mm thick plug and the specimen shall be compressed in the compression testing machine until the top of the plug is flush with the top of the collar. The load shall be held constant for atleast 30s and then released. With some soils, rebound may occur and a more prolonged re-application of the load may be necessary.

After compaction, the plug and filter paper shall be removed, together with the collar. Unless the sample is to be tested immediately, the top-plate of the mould shall be screwed on to prevent evaporation. Where the air content is less than 5% in the compacted soil, the specimen shall be allowed to stand for 24h before testing to facilitate the dissipation of pore pressures set up by the compaction process.

Method 2

The mould shall be assembled as in method 1. The required quantity of wet soil shall be divided into three equal parts, each sealed in a container to prevent loss of moisture. One third of the soil shall be poured into the mould. The three plugs shall then be fitted, with handles removed, and the soil compressed using the compression machine until the thickness of the soil, after removal o the load, is one third the depth of the mould. The second and third layers shall be added in a similar manner to the first. In this last operation, one plug only shall be used and it shall be pushed in until the top

surface is level with the top of the collar.

Storage and curing of the samples shall be as in method 1.

A2.3.2 DYNAMIC COMPACTION

Three procedures using dynamic compaction are in use. The first two are used where a required density is to be obtained by a process of dynamic or vibrational compaction (as an alternative to static compaction methods, where a compression machine is not available). The third is used where a given amount of comparative effort is to be applied to the soil as in the standard compaction tests. Dynamic compaction shall be carried out by one of the following methods.

Method 1

The mould shall be assembled with its base-plate and be weighed to the nearest 5g (m_2). The collar shall then be fitted and a filter paper placed in the bottom.

The mass of wet soil necessary to give the required dry density shall be prepared as for the static compaction procedure. This quantity shall be divided into five equal parts by mass, each sealed in a container to prevent loss of moisture by evaporation. The soil shall then be compacted into the mould with the collar attached in five equal layers using either the BS 2.5Kg rammer or 4.5Kg rammer. Each layer shall be given sufficient evenly distributed blows over the surface to ensure that the layer after compaction occupies one fifth of the

height of the mould. When compacted in this manner, the final level of the soil shall be just above the top of the mould. The collar shall the be removed and the soil trimmed flush with the top of the mould, using a steel straightedge. The mass of the mould with base-plate attached and containing the specimen shall then be measured to the nearest 5g (m_3).

Preliminary tests, in which adjustments shall be made to the number of blows used, may be required to fill the mould sufficiently closely with the calculated mass of soil.

Storage and curing shall be as for samples compacted by static compaction.

Method 2

The mould an the wet soil shall be prepared exactly as in method 2 for static compaction. One third of the wet soil shall be poured into the mould and compacted by a vibrating hammer. Vibrational compaction shall be continued until the thickness of the layer is equal to one third of the depth of the mould. The second and third layers shall be added and compacted in the same manner so that the final surface of the soil is level with the top of the mould (m_3).

Storage and curing shall be as for samples compacted by static compaction.

Method 3

The mould with its base-plate shall be prepared and weighed as in method 1. The collar shall then be fitted and a filter paper placed in the bottom. Approximately 6Kg of the soil at the required moisture content shall be prepared.

Either of two levels of compaction may be used; the first employs the BS 2.5Kg rammer and the second BS 4.5Kg rammer.

Using the light rammer sufficient soil shall be placed in the mould, with the collar attached, so that after 62 blows with the rammer, uniformly spread over the area of the soil, the surface level is one third of the distance up the mould or slightly above this level. Using in each case the same mass of soil, two further layers shall be added and compacted in the same manner so that the final level is less than 6mm above the top of the mould. Preliminary trials may be necessary to judge the amount of soil required for each layer. The collar shall then be removed and any excess soil trimmed flush with the top of the mould, using a steel straightedge. The mould with base-plate shall then be weighed (m_3).

For the higher level of compaction, the same procedure shall be used except that 62 blows of the 4.5Kg rammer shall be employed and the soil placed and compacted in five equal layers.

Storage and curing shall be as for samples compacted by static compaction.

A2.4 SOAKING PROCEDURE

Soaking of CBR test specimens is not normally employed in the U.K., but in certain circumstances it may be employed elsewhere. If soaking is employed it shall be carried out as given below.

After compaction in accordance with one of the alternative methods, the base-plate shall be replaced by one perforated. The collar shall be fitted to the to the other end of the mould and the thread packed with petroleum jelly. The specimen shall then be placed in a bath of water and the water level kept just below the top of the collar. Surcharge masses as required shall then be placed o the specimen. The time taken for the water to reach the top of the specimen shall be observed. If this has not occurred within three days, the top of the specimen shall be flooded and the specimen left to complete its normal soaking period. The normal soaking period shall be 4 days.

During soaking, it may be required to measure whether swell takes place. To do this, a filter paper an perforated plate shall b placed on top of the specimen, and a dial gauge reading to 0.01mm shall be suitably mounted to follow the movement of the surface.

When soaking is complete, the specimen shall be removed from the bath and allowed to drain for 15mins. The base-plate shall be replaced, the collar and perforated plates shall be removed and the specimen shall then be weighed to the nearest 5g (m_t).

A2.5 TEST PROCEDURE

The procedure is as follows.

(1) The mould, containing the specimen, with the base-plate in position, but the top face exposed, hall be placed o the lower platen of the testing machine. Surcharge masses as required shall b placed on the specimen. The plunger shall b seated under a force of 50N for a bearing ratio of up to 30% or 250N for bearing ratio above 30%, and shall be made to penetrate the specimen at a uniform rate of 1mm/min. Readings of the force shall be taken at intervals of penetration of 0.25mm to a penetration not exceeding 7.5mm.

(2) If it is intended to test both sides of the specimen, the plunger shall be raised, and the face of the specimen trued up by filling any depression left by the plunger and cutting away any projecting material with a straightedge. The base-plate shall then be removed from the lower end of the mould, and fixed on the upper end, and the mould and contents shall then be inverted. The testing procedure described above shall be repeated on the other end of the specimen.

(3) After the penetration tests have been completed, samples of the soil each about 350g shall be taken penetrated surfaces of the two ends of the specimen and the moisture contents determined. Filling material used in the end first tested shall not be included in the moisture content

sample.

A2.6 CALCULATIONS

A2.6.1 FORCE PENETRATION CURVE

A graph showing force on the plunger against penetration shall be plotted and a smooth curve drawn through the points. The curve will normally be convex upwards as in Test 1, Fig.A2.2 If however, the initial portion is concave upwards as in test 2, correction is necessary. In this case, a tangent shall be drawn at the point of greatest slope an shall be projected to cut the penetration axis as indicated in Fig.A2.2 The curve shall then be shifted to the left so that the point of intersection of the tangent with the penetration axis coincides with the origin. This gives the corrected curve from which the CBR shall be deduced.

A2.6.2 CALCULATION OF CALIFORNIA BEARING RATIO

The standard force penetration curve corresponding to 100% CBR is shown on Fig.A2.2. The forces corresponding to this curve are 11.5KN at 2mm penetration, 17.6KN at 4mm, 22.2KN at 6mm, 26.3KN at 8mm, 30.3KN at 10mm, and 33.5KN at 12mm. The force, read from the smoothed curve, required to cause a given penetration expressed as a percentage of the force required to cause the same penetration on the standard curve is defined as the CBR value at that penetration. The CBR value is calculated at penetrations of 2.5mm and 5mm, and the higher value taken.

It is convenient to plot the force-penetration curve on a diagram in which curves corresponding to various CBR levels have been printed. The CBR value can then be read directly without further computation if no correction is required.

A2.6.3 CALCULATION OF DRY DENSITY OF UNSOAKED SPECIMENS

The bulk density, ρ in Mg/m^3, of the specimen shall be calculated.

In the case of statically-compacted or vibratory specimens, this shall be done from the equation:

$$\rho = \frac{m_1}{2305}$$

In the case of dynamically-compacted specimens, this shall be done from the equation:

$$\rho = \frac{(m_3 - m_2)}{2305}$$

The dry density, ρ_d in Mg/m^3, of the specimen shall be calculated from the equation:

$$\rho_d = \frac{100\rho}{100 + \omega}$$

where

ω = *moisture content of the soil* (%)

A2.7 *REPORTING OF RESULTS*

The California bearing ratio shall be reported to the nearest 1% for CBR's up to 30, to the nearest 5% for CBR's between 30 and 100 (i.e., 30,35,40,....) and to the nearest 10% for CBR's over 100 (i.e., 100,110,120,....). The report shall also include relevant details of the method of compaction, dry soil density, the moisture content after test and, if soaked, the time of soaking. Normally, the results of tests from the top and bottom of the specimen shall be reported separately, together with the appropriate moisture content. Where the results are within 10% of the mean bearing ratio, the result shall be averaged.

(Source: Methods of test for soils for civil engineering purposes (BS 1377), 1990)

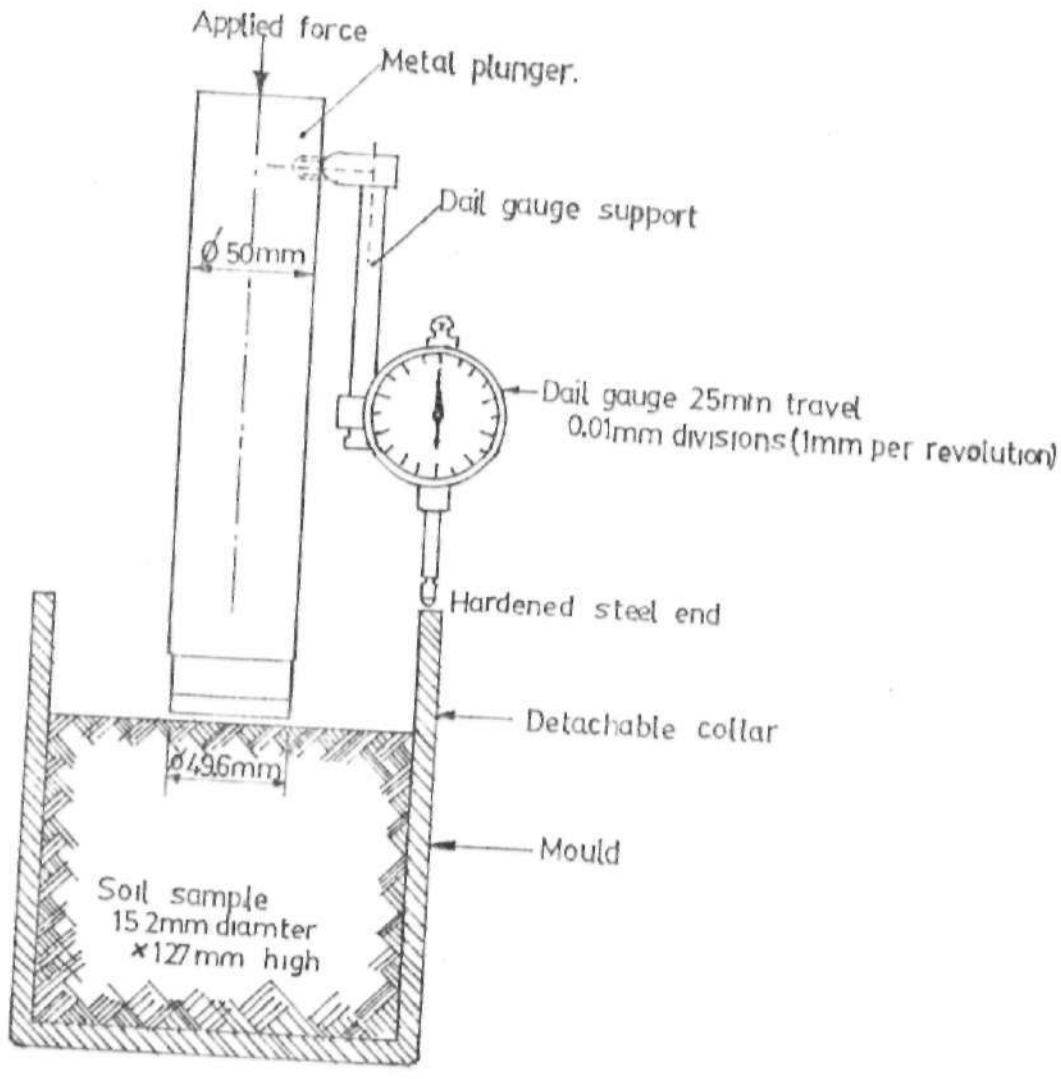


Fig.A2.1 - ARRANGEMENT FOR CALIFORNIA BEARING RATIO TEST

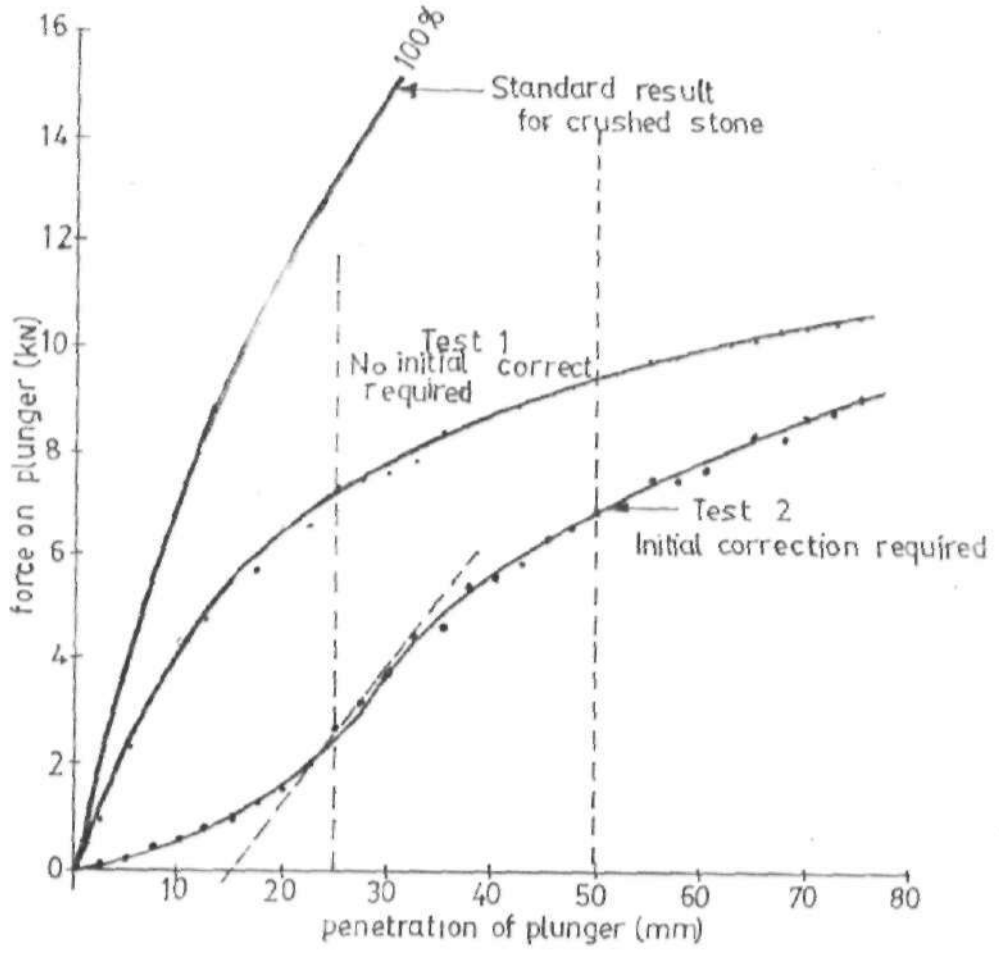


Fig.A2.2 - TYPICAL CBR TEST RESULTS

APPENDIX A3

MATERIALS SPECIFICATION

A3.1 GENERAL

A3.1.1 QUALITY OF MATERIALS

All materials used in the works shall be new and of the qualities and kinds specified. They shall comply with the requirements, unless modified herein, of the current amended editions, at the date of invitation to tender, of the British Standards (hereinafter abbreviated to BS) published by the British Standards Institution, or any other recognised Standards which may be quoted. All materials may b checked at both the source and on site and approval of any material at its source does not necessarily imply that it will be approved on site. All manufactured or propietory material shall be fixed or applied in accordance with the recommendations of the manufacturer, except where alternative requirements are specified herein.

A3.1.2 SUPPLY OF MATERIALS

The supplier of all manufactured materials shall be subject to approval. All imported materials shall be shipped wherever possible to Nigeria in Nigerian ships if so required.

A3.1.3 DEFECTIVE MATERIALS

All materials which do not comply with the requirements of this specification shall be rejected and all such materials, whether in place or not, shall be immediately removed from site by the contractor, at his own expense.

A3.1.4 HANDLING AND STORAGE OF MATERIALS

(a) The contractor shall make is own arrangements for storage space or yards and may not make use of Government or Native Authority facilities without prior approval.

(b) All material for use in the works shall be handled with due care and whenever not in immediate use shall, subject to (d) and (e) below, be stored in well ventilated, water-tight buildings with raised floors. In the case of cement and hydrated lime, each consignment shall be kept separately and the contractor shall use the consignments in the order in which they are received. The siting of these buildings shall be approved before construction commences and on completion of the works they shall be removed.

(c) All stores buildings shall be adequate for the complete protection of the materials to be kept therein and precautions shall be taken against damage by fire, particularly in the case of inflammable materials.

(d) Approved materials may be stockpiled at approved locations and, prior to stockpiling, the site shall be cleared,

cleaned, and levelled by the contractor who shall if required, also lay suitable hard surfacing racks, depending on the type of materials to be stored. Stockpiling of differently graded aggregates or materials shall be kept separate.

The method of stockpiling aggregates shall be such that no segregation will take place during storage. Under no circumstances shall materials be stockpiled on the carriageway or margins.

(e) The contractor may use bulk storage for bitumen, cement and lime provided he can satisfy the engineer's representative that adequate supplies are available and also that suitable bulk storage facilities can be provided on site.

A3.2 NATURALLY OCCURRING MATERIALS

A3.2.1 MATERIAL FOR FILL

Fill material shall consist of approved suitable material obtained from the general excavations or from borrow pits and shall be free from all logs, vegetation or any other deleterious matter. Stones larger than 6 inches in greatest dimension may be required to b removed from certain, or all sections of fill.

The material shall fulfil the following requirements:

Liquid Limit.....0-45%

Plasticity Index.....0-20%

Linear Shrinkage.....0-10%

Tests

Liquid Limit

Plasticity Index

Linear Shrinkage

A3.2.2 MATERIAL FOR SUB-BASE

Sub-base material shall consist of approved suitable material obtained from the general excavations or from approved borrow pits and shall be free from all logs, vegetation and any deleterious matter. Stones larger than 3 inches in greatest dimension shall be removed.

The material shall fulfil the following requirements:

Liquid Limit.....0-45%

Plasticity Index.....0-15%

Linear Shrinkage.....0-8%

CBR value at BS Heavy

Compaction shall be not less than.....30%

Tests

Liquid Limit

Plasticity Index

Linear Shrinkage

BS compaction

CBR test

A3.2.3 MATERIAL FOR SURFACING SIDE SLOPES AND MARGINS

Surfacing material for side slopes and margins shall consist of approved, suitable material obtained from the general excavations or from approved borrow pits and shall be free from logs and stones of 3 inches in greatest dimension. There shall be no objection to the presence of small amounts of vegetable matter but the amounts shall be subject to approval. The material shall fulfil the following requirements:

Liquid Limit.....0-50%

Plasticity Index.....6-25%

Linear Shrinkage.....0-12%

Tests

Liquid Limit

Plasticity Index

Linear Shrinkage

A3.2.4 MATERIAL FOR BASE COURSE

(a) All base course material shall be subject to approval and shall be free from all logs, vegetation or any other deleterious matter. If a water table or standing water will exist within 4 feet of the finished road surface then all CBR tests on unstabilized materials shall be carried out o specimens soaked for 24 hours if required. All CBR values quoted hereinafter shall be obtained from specimens prepared at maximum dry density and optimum moisture content using BS Heavy compaction, after 7 days and 28 days curing respectively for cement and lime stabilized specimens.

(b) Granular base course material shall be gravel-sand-clay or any other suitable material or any combination thereof which shall comply with one of the three gradations shown:

Sieve Size	Percentage Passing		
	Nominal Maximum Size		
	3"	1 1/2"	3/4"
3 inch	100	-	-
1 1/2 inch	80-100	100	-
3/4 inch	60-80	80-100	100
3/8 inch	45-65	55-80	80-100
3/16 inch	30-50	40-60	50-75
No.7	-	30-50	35-60
No.25	10-30	15-30	15-35
No.200	5-15	5-15	5-15

Not less than 10% shall be retained between each pair of successive sieves specified for use, except the largest pair.

The material shall also fulfill the following requirements:

Liquid Limit.....0-25%

Plasticity Index.....0-10%

Linear Shrinkage.....0-6%

C.B.R. value shall not be less than.....80%

(c) Sand-clay base material shall comply with the gradations shown below:

Sieve Size	Percent Passing
3 inch	100
No.7	80-100
No.14	50-80
No.25	30-60
No.52	20-45
No.200	10-25

Not less than 10% shall be retained between each pair of successive sieves specified for use, excepting the largest pair. The material may have up to 35% of stones not larger than 1 1/2" provided that the material passing the 3/16" sieve is within the specified limits.

The material shall also fulfill the following requirements:

Liquid Limit.....0-25%

Plasticity Index.....0-10%

Linear Shrinkage.....0-6%

C.B.R. value shall not be less than.....80%

(Source: Specification for road and bridge works, 1986)