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A METHODOLOGY TO ASSESS DATA VARIABILITY AND RISK IN A PAVEMENT DESIGN SYSTEM FOR BANGLADESH

by

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Abstract

This study aimed to develop a generic methodology to quantify the risk and reliability of pavement and embankment design for Bangladesh considering the variability in design data. The study also aimed to develop a construction quality control procedure to reduce the variability in data.

To achieve this aim, data were collected from field and laboratory testing from four of the country's representative roads and a database was developed. The collected data were studied and their variability was quantified. To develop a suitable risk quantification methodology for Bangladesh, the existing methods were investigated and compared for their appropriateness in connection with the proposed analytical pavement design method and the prevailing conditions. The method proposed in this research utilizes the first order second moment theory and an analytical model based on the method of equivalent thickness. For the risk analysis of embankments the first order second moment method was also identified as suitable in the context. An integrated example of the proposed procedure is given, using the data from one of the roads tested. Existing quality control methods and techniques were also reviewed to develop a suitable quality control procedure for Bangladesh. For pavements, a performance based quality control procedure considering their load carrying capacity as an acceptance criterion was also suggested in this research, together with a quality control procedure for embankments.

THIS WORK IS DEDICATED TO MY PARENTS

Late Md. Rezaul Karim and

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Glossary of Symbols and Non-standard Abbreviations

A	Axles per Truck
AADT	Annual Average Daily Traffic
AADTT	Annual Average Daily Truck Traffic
AASHTO	American Association of State Highway and Transportation Officials
AC	Asphaltic Concrete
c'	Drained Cohesion
C_c	Compression Index
C_α	Secondary Compression Index
CBR	California Bearing Ratio
COV	Co-efficient of Variation
D	Directional Distribution
DCP	Dynamic Cone Penetrometer
e_0	Initial Void Ratio
e_p	Void Ratio at the End of Primary Consolidation
EALF	Equivalent Axle Load Factor
ESAL	Equivalent Standard Axle Loads
f	Strength Reduction Ratio
F	Factor of Safety
F_R	Reliability Design Factor
F_i	Equivalent Axle Load Factor for the *i*th Load Group
FEM	Finite Element Method
FOSM	First Order Second Moment
FWD	Falling Weight Deflectometer
G	Growth Factor
GPR	Ground Penetrating Radar
H	Layer Thickness
HDM	Highway Design and Management
L	Lane Distribution
MET	Method of Equivalent Thickness

M-E	Mechanistic-Empirical
MSA	Million Standard Axles
n	Sample Size
$N_{critical}$	Pavement Load Carrying Capacity
NCHRP	National Co-operative Highway Research Project
P_0	Initial Pressure
p_i	Percent of Axles in the *i*th Load Group
PAF	Payment Adjustment Factors
PD	Percent Defective
PSI	Pavement Serviceability Index
PWL	Percent Within Limit
r	Growth Rate
RHD	Roads and Highways Department
S	Settlement of Soil
S_N	Standard Deviation
S_0	Overall Standard Deviation of Variation
t_1	Time for End of Primary Consolidation
t_2	Design Period
T	Percent of Truck in the Traffic Mix
TRL	Transport Research Laboratory
TRRL	Transport and Road Research Laboratory
QC	Quality Control
$V[x]$	Variance of Parameter x
w_T	Predicted Traffic
W_t	Predicted Pavement Performance
$\bar{x}$	Sample Mean
Y	Design Period
β	Reliability Index
μ	Population Mean
γ	Density
Φ'	Drained Angle of Internal Friction

Chapter 1 Introduction

1.1 Background

The research reported in this thesis is part of a major research programme which sought to develop a methodology for pavement design suitable for Bangladesh. It focuses on an examination of the variability of data associated with pavement and embankment design and suggests methods to quantify and control it in both the design and construction phases.

The principal goal of any engineering design process is to produce a system which performs its intended function in a clear, swift and accurate manner. But the success of any design method depends on the accurate characterization of the uncertainties in preparing the design inputs. The design of pavements involves a number of input data. Consequently, the quality of these input data has significant effects on the design of the pavement. To address this problem, the concepts of reliability and probability were employed first in the early 1970s by researchers and engineers such as Lemer and Noavenzadeh [1971] and Kher and Darter [1973]. A number of items may contribute to the reliability of the design and the variability of data. In Bangladesh such items include overloading [Khan, 2005], poor construction practices and seasonal variation in the moisture content of granular materials and subgrade soil. Hence, for the methodology of new pavement design it was felt necessary for Bangladesh, where road pavements are usually built on embankments, to incorporate the concept of reliability. Their satisfactory performance depends on the performance of the embankment. Therefore the quality of embankment design data also needed to be considered in the country's proposed design system, together with a quality control process to reduce the variability of pavement properties during construction.

1.2 Problem definition

The design of a highway pavement and embankment in Bangladesh should consider the variability of the input data. This variability may influence the success of the pavement and embankment design and consequently can lead to premature failure. To date, no study has been conducted in Bangladesh to assess such variability in design data and its effects on the design. The assessment of variation in design data necessitates a thorough investigation of the design data collected from field and laboratory tests of representative roads. A methodology is also required for designers to quantify the variability in design data and its impact on the design produced, so that the reliability of the overall design can be determined. Moreover, a system of quality control and assurance associated with the design system is also required to reduce the variability of the construction related data. There are some methods of quantifying design risk associated with data variability available in the literature. But these were developed for particular conditions or are considered unique design models and may not be suitable for Bangladesh. Hence, an amended methodology is required which is suitable for the country's geographical, geotechnical and socio-economic conditions.

1.3 Aims and Objectives of the study

The aim of this study is to develop a methodology to assess the variability of data and associated risk in a pavement design system for Bangladesh. To achieve this aim, the following objectives have been set up for the study:

1. To assess the variability of the pavement and embankment design data
2. To develop a methodology which will quantify the variability of data
3. To quantify the risk and reliability in the pavement and embankment design system
4. To introduce a quality control and assurance process in the design system, based on the data considered by the design system.

1.4 Benefits of the study

The main beneficiary of this study will be the Roads and Highways Department (RHD) of Bangladesh, which is responsible for the design, construction and maintenance of the major road network there. The research output will help RHD in improving the quality of the design data, incorporating the desired level of reliability in designing and ultimately in obtaining a satisfactory performance from pavements and embankments. Other engineering departments which deal with the local road network of Bangladesh will also benefit from this study if they incorporate the research finding in their design system. Ultimately, the road agencies of similar developing countries may benefit from this research.

1.5 Layout of the Thesis

To achieve the above objectives this Thesis is structured as follows:

Chapter 2 presents a literature review of methods of pavement and embankment design, design data, data variability, methods of determining data variability, quantifying risk and quality control.

Chapter 3 describes the methodology followed in the present study to examine the variability of design parameters and the factors and techniques used for risk quantification and quality control.

Chapter 4 describes the quality and variability in the design data collected from field and laboratory testing carried out on four representative roads of Bangladesh.

Chapter 5 investigates the suitability of existing methods of analysing pavement design risk for the design system of Bangladesh and provides a comparative study of them. Then it discusses the logical development of the proposed method, together with a detailed description of the proposed method.

Chapter 6 presents the development of a risk quantification process for embankment design system of Bangladesh. It reviews the existing methods of risk analysis with regard to slope stability and settlement and presents a method which has been developed. The methodology intended to quantify the overall risk of this pavement-embankment design system is also presented.

Chapter 7 presents an integrated example of the proposed procedure, calculating the overall design risk using the data collected from a road of Bangladesh.

Chapter 8 deals with the development of a quality control process for pavement construction of Bangladesh. Then the development of the proposed performance based quality control system is described. A step-by-step quality control process during construction is also given.

Chapter 9 investigates the existing state of knowledge for embankment construction quality control. A comparative study of them is also provided. The Chapter then describes the quality control process for embankment construction of Bangladesh.

Chapter 10 gives an application example of the quality control process which is being proposed for Bangladesh with the field collected data.

Chapter 11 discusses the key findings of the study. It investigates the suitability of the proposed procedures through an analysis of the opinion of the RHD road engineers. It also recommends some ways of reducing risk in design and performance and specifies some areas for further research.

Chapter 12 draws some conclusions from the study.

Chapter 2 Literature Review

2.1 Introduction

This chapter reviews the process of pavement and embankment design and its associated design data, the variability in the data and the associated risk, the variability resulting from poor construction and the fundamentals of the quality control process. In more detail, this chapter first describes pavement design, in particular, analytical pavement design and its associated input parameters. It goes on to discuss the processes of embankment design in the light of slope stability and settlement and identifies the associated design input parameters. Subsequently, it reviews the methods used to quantify the variability associated with the data of pavement and embankment design and discusses the methods used to quantify risk. However, a detailed review of the existing risk quantification methods for their problems and appropriateness to Bangladesh are provided in Chapter 5 and Chapter 6 for pavement and embankment respectively. Before concluding, this chapter considers quality control systems.

2.2 Pavement design procedures and required data

There are two main types of pavement, flexible and rigid. Only the design procedures of flexible pavement will be considered in this chapter, since Bangladesh has no rigid pavements.

2.2.1 Generic design

The main goal of a flexible pavement design is to provide a structure that can carry the anticipated traffic, withstand the environmental effects and maintain a satisfactory level of service for a predefined period of time. A flexible pavement is usually designed as a system of a layered structure. The designer should consider using the locally available materials and should select the most economical combination of layer thickness and materials that performs the intended function satisfactorily. A cost analysis of pavement life cycle may be performed to evaluate the most economical option for pavement design.

A design system in general requires the input of information on cumulative traffic that it is anticipated the pavement must carry in its projected life, the properties of the materials which will be used in constructing it, the characteristic strength of the subgrade over which the pavement will be constructed and the climate where it will be located. A schematic flowchart of a generic pavement design is shown in Figure 2.1.

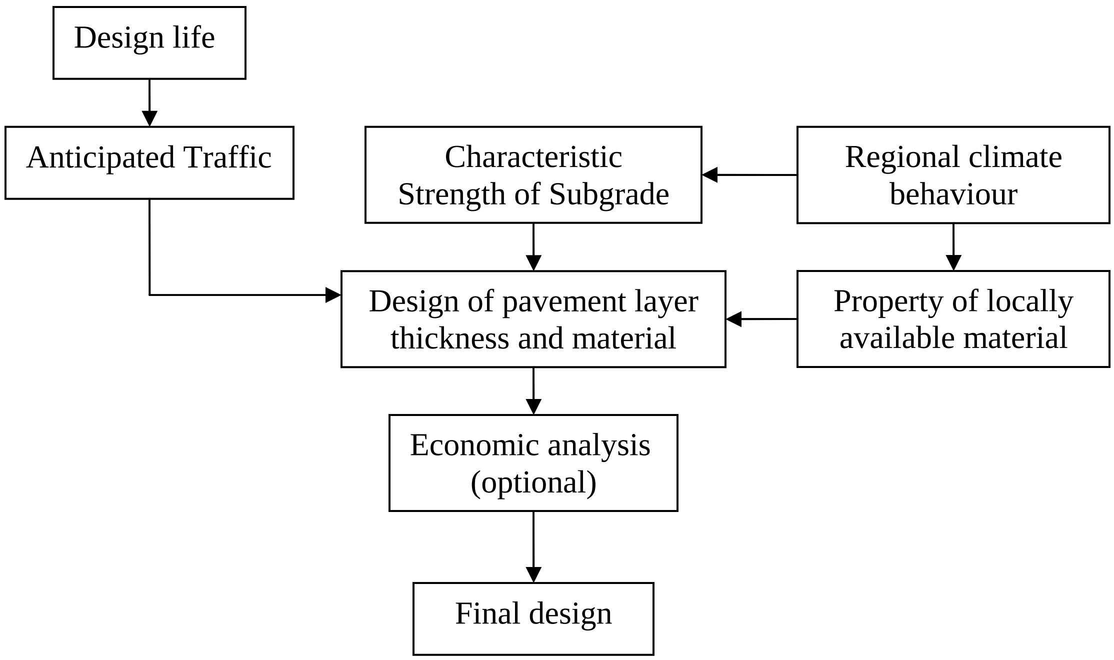


Figure 2.1: A Generic Pavement Design Flowchart

However, a pavement design system is influenced by many factors (such as traffic, available materials, costs) and therefore a systematic approach is usually followed in the design process. To achieve this, two different approaches may be followed. One is based on empirical considerations and the other follows an analytical methodology.

2.2.2 Empirical design

In the empirical approach, the design is based on the observation of historical performance of roads or on the observation of experimental roads in performance. Experimental roads may be built as public roads which are subject to normal traffic levels or as test roads where the traffic can be controlled [McElvaney and Snaith, 2002]. Examples of such methods include the U. S. Army Corps of Engineers California Bearing Ratio (CBR) method [Huang, 1993], the American Association of State Highway and Transportation Officials (AASHTO) method [AASHTO, 1986, 1993] and TRL's (Transport Research Laboratory) Overseas Road Note 31 [TRL, 1993]. But, although empirical pavement design methods have been popular in the past, it is difficult to use them accurately when the design input factors differ significantly from those used in the original design. These factors may include changes in traffic levels, climatic factors and the availability of materials. Consequently, empirical methods are ineligible in the present context and will not be discussed further.

2.2.3 Analytical design

In the analytical approach, the design is based on the structural analysis of pavements and their predicted performance in relation to measurable parameters. A significant number of analytical pavement design methods is described in the literature and commonly used ones include those developed by Shell International Petroleum Ltd [Shell 1978], the Asphalt Institute [1981]; Austroads [2004] and Nottingham University [Brunton et al. 1987]. This approach to pavement design is becoming more popular with advances in computer hardware and software technologies. As a result, many countries all over the world have partially or fully implemented analytical procedures for determining the existing strength (bearing capacity) of road pavements, for analyzing and designing new roads and for rehabilitating existing road pavements.

In analytical pavement design, two models are used. One is associated with the pavement response under traffic loads and the other concerns pavement performance. For the former, a structural model of the pavement is built and used to determine stresses, strains and deflections at critical locations in the pavement. The parameters determined at those locations are known as critical response parameters. The performance model, on the other hand, is used to estimate pavement life as a function of the critical response parameters.

A flowchart of analytical pavement design process is given in Figure 2.2.

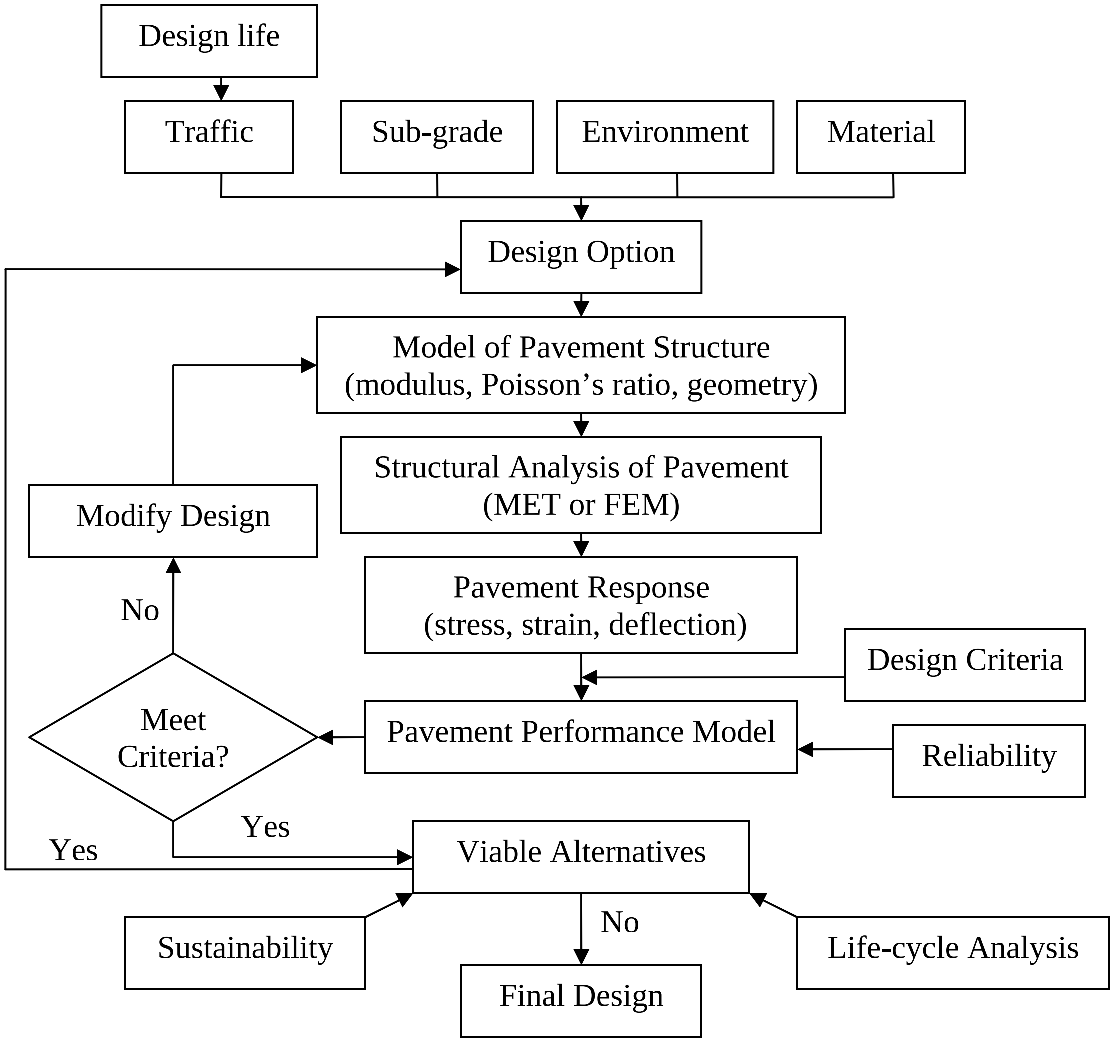


Figure 2.2: Analytical Pavement Design Method Flowchart [Evdorides, 2007]

The major steps in the iterative design process are as follows (see Figure 2.2):

1. Identify the required pavement life in terms of the equivalent number of standard axle loads
2. Determine the available and sustainable pavement materials
3. Estimate pavement layer thickness and the long-term performance properties (stiffness and/or strength) of pavement material
4. Carry out structural analysis of pavement using a pavement response model (e.g. MET or FEM) and determine critical parameters (stress, strain, deflection)
5. Compare critical stresses/strains and or deflections with allowable ones.
6. Make adjustments to thickness until the required pavement life is achieved.

2.2.3.1 Pavement design data

Traffic data are important as inputs for the analysis and design of pavement structures, because they are used to determine the loading regime to which the structure will be subject throughout its design life. Most existing design procedures, including all of the AASHTO (American Association of State Highway and Transportation Officials) Design Guides, quantify traffic in terms of equivalent standard axle loads (ESALs) [Schwartz, 2007]. This enables a single load to be used as a unit for design purposes and requires all other traffic loads to be converted to this design load. However, the mechanistic pavement response models in the Mechanistic-Empirical (M-E) pavement design guide require the magnitudes and frequencies to be specified of the actual wheel loads which the pavement is expected to bear throughout its design life. According to this guide, traffic must be specified in terms of axle load spectra rather than ESALs. Axle load spectra are the frequency distributions of axle load magnitudes by axle type (single, tandem, tridem, quad) and season (typically, per month) [Papagiannakis et al., 2006].

The traffic related information required by a standard design process includes the following [Killingsworth and Zollinger, 1995]:

- Traffic volume—base year information
    - Two-way annual average daily truck traffic (AADTT)
    - Number of lanes in the design direction
    - Percentage of trucks in the design direction
    - Percentage of trucks in the design lane
    - Vehicle (truck) operational speed
- Traffic volume adjustment factors
    - Seasonal variation / adjustment
    - Vehicle class distribution
    - Hourly truck distribution
    - Traffic growth factors
- Axle load distribution factors by season, vehicle class and axle type (single, tandem, tridem and quadruple axles)
- General traffic inputs
    - Number of axles/trucks

- Axle configuration (axle width and spacing; tyre spacing and pressure)
- Wheelbase spacing distribution

Another important input parameter for pavement design is the properties of the material to be used in the pavement layers. The information of resilient modulus and Poisson's ratio of different pavement layers is required in the mechanistic analysis of pavement structure. The resilient modulus and Poisson's ratio of subgrade soil are also required for the design of pavement structure.

In obtaining the reliable input data required for design, a major difficulty is that the required site specific information is not generally available at the design stage and sometimes has to be estimated several years in advance of construction. Further, the actual properties of the material to be used are not usually known much before construction takes place. Nevertheless, a designer should obtain as much information as possible on in-situ material properties, traffic and other inputs in order to supply a realistic design. To this end, the designer should undertake a sensitivity analysis to identify the most important factors to affect the design [Castell and Pintado, 1999; Killingsworth and Zollinger, 1995].

2.3 Embankment design procedures and required data

An embankment is designed for various purposes, such as to sustain other civil engineering structures (highways, railways) and to restrain water (dams). Only the general design procedures of highway embankments will be discussed in this chapter.

2.3.1 Generic embankment design

Highway embankments are important and costly civil engineering structures which provide an essential platform for pavements. The critical aspects of embankment design are the analysis of stability and settlement for safety of the earth structure under various operating and environmental conditions. The prime concern should be to select an economical design using locally available material and technology, so that the embankment can perform its intended function satisfactorily. A highway embankment is considered to be performing satisfactorily when it can carry the load borne by the road pavement and the environment while maintaining its stability and settlement to a tolerable limit during its service life. An embankment design system in general requires, as inputs in its design process, information on the load which the embankment will have to carry in its design life, the property of the materials which will be used in constructing the embankment, the characteristics of the foundation soil over which the embankment will be constructed and the regional environmental behaviour which the embankment will have to sustain for the length of its service life. A schematic flowchart of generic embankment design is shown in Figure 2.3.

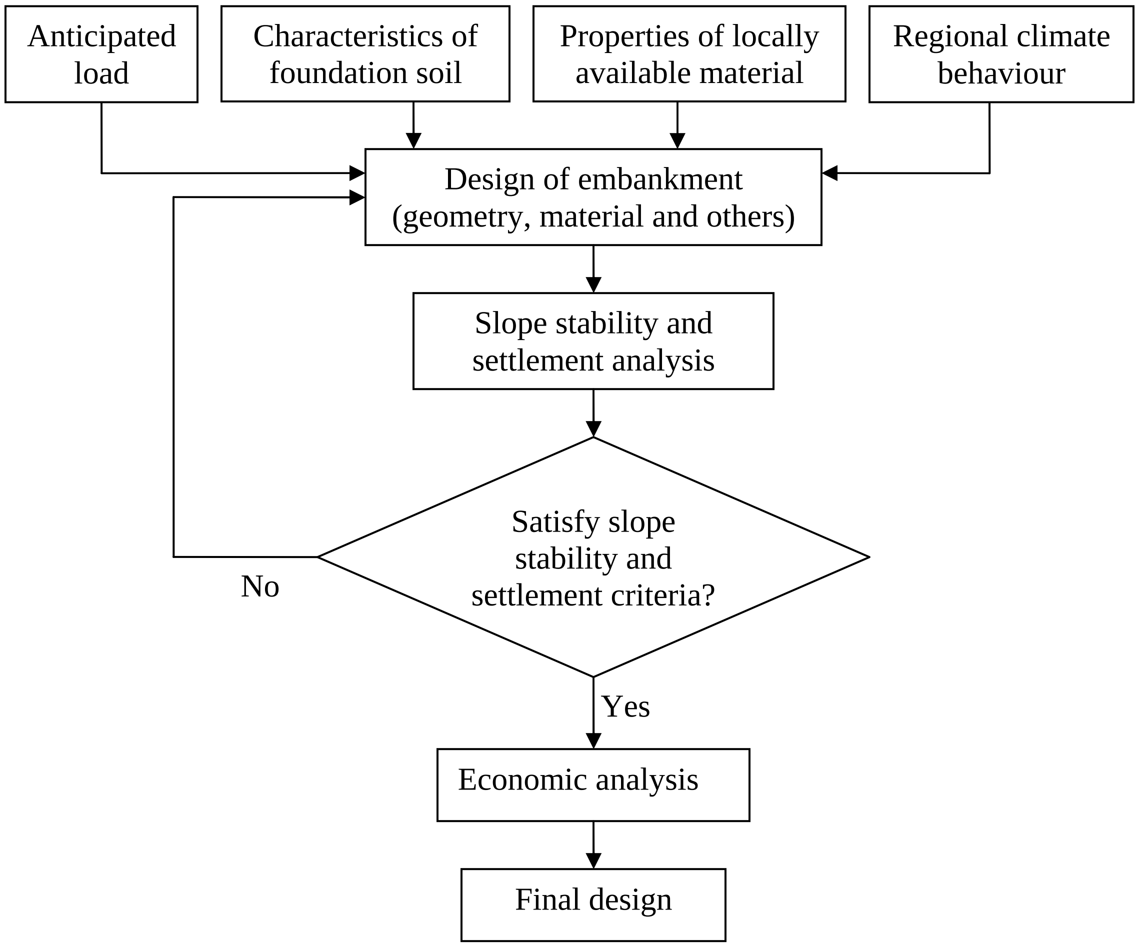


Figure 2.3: A Generic Embankment Design Flowchart

2.3.2 Embankment design data

The load data for the embankment design involves the surcharge load, water pressure and self weight of embankment. The properties of the embankment and foundation soil data involve data on their shear strength and consolidation. The shear strength data consist of drained and undrained cohesion, angle of internal friction, pore water pressure and unit weight [Christian, 1994; EI-Ramly, 2002]. The consolidation data consists of the soil modulus, Poisson's ratio, initial void ratio, density, compression index, secondary compression index, time for end of primary consolidation, design period and layer thickness of the different layers [Craig, 2004; Das, 1997; Tomlinson, 2001; Barnes, 1995]. The environmental data involve the amount of rainfall and water table height, etc. It is very difficult to obtain accurate information about these parameters in designing an embankment, since soil properties vary from one location to another and the environmental behaviour is also variable. However, to obtain an optimal design, as much information as possible should be collected by conducting a thorough site investigation and by testing.

2.4 Variability of materials and data

2.4.1 Pavement data

The main sources of the uncertainties associated with pavement design and performance are as follows [Prozzi, 2006; Dempsey et al, 2006; Ksaibati et al., 1999; Zuo et al., 2007]:

2.4.1.1 Traffic

The variation in traffic growth is an important factor, which must be quantified accurately. This growth also varies with the type of traffic. Prozzi and Hong [2006] suggested that the variation in traffic prediction parameters is one of the sources of greatest uncertainty in pavement reliability analysis. Thus, these variations in traffic prediction parameters should be taken into consideration for the proper quantification of risk in the Bangladesh's pavement design system.

2.4.1.2 Materials

No materials in nature are absolutely uniform. The inherent randomness of natural processes causes variation in material properties [Malkawi et al., 2000]. Lack of accuracy in evaluating material properties also imparts some degree of variability. Most importantly, environmental effects (precipitation, temperature, water table) on materials make a significant contribution to the variability in material properties. Some properties of materials are affected directly by the environment, such as susceptibility to the ingress of moisture, the ability to drain and the infiltration potential of the next underlying layer

[Zuo et. al 2007]. Material properties such as resilient modulus and load carrying capacity are affected by moisture content variation, which in turn is related to environmental factors and soil properties, such as gradation, Atterberg limits and suction parameters. Oh et al [2006] suggested that it is essential to evaluate the expected moisture content of the pavement layer in considering the variability in climate soil conditions for conducting proper analysis and to optimize pavement performance. Temperature, another environmental factor, markedly affects the elastic modulus of any Asphaltic Concrete (AC) layer. The results of many studies [Marshall et al., 2001; Salem et al., 2004] show that both the temperature averaging period and the temperature gradient in the asphalt affect the AC modulus and consequently the estimation of pavement life.

2.4.1.3 Construction

In order to improve the reliability of pavement design, it is necessary to have an accurate estimate of the as-constructed pavement layer thickness and its within-layer variability [Mladenovic et al. 2003; Jiang et al., 2003]. If the thickness of the layer is not the same as that specified in the design then the performance of the pavement will not be what was expected. In addition, the within-layer variability of thickness and material properties also affects pavement performance [Attoh-Okine and Roddis, 1994]. The variation of moisture content during construction spatially and temporally causes spatial and temporal variation in the strength of the pavement layer [Dempsey et al, 2006; Austroads, 2004]. Furthermore, variation in the density of pavement layer material resulting from non-uniform compaction during construction causes pavement layer strength to vary [Patel and Thompson, 1998; NCHRP, 2004].

2.4.2 Embankment data

The variability of the soil properties is the main source of uncertainties in embankment design. Geological variations, such as mineralogical composition variation, variation in stress history and variation in physical and mechanical decomposition processes result in some inherent variability in material properties [Lacasse and Nadim, 1996]. Inaccuracy in the quantification process of data on soil properties introduces further variation [Christian, 1994]. The climate factor, which varies from one location to another, from time to time variably influences the properties of the soil [Agrawal and Altschaeffl, 1991]. The climate factor also contributes to the variation in water table depth. The surcharge load, composed of the weight of the pavement and the traffic, is also variable in nature. Moreover, non-uniform and improper compaction during the construction of the embankment creates added uncertainties in embankment design [Wolff et al., 1996; Larsen, 2007].

2.5 Data variability quantification methods

An accurate design process demands the appropriate quantification of the variability of the input data so that suitable design values may be chosen. A number of standard statistical tools are available to quantify variability and these are critically considered below to assess suitability and accuracy.

The most common measure of variability is the expectation or mean value of a variable, which is determined by adding all the measurements or values in the data set and dividing the sum obtained by the number of measurements that make up the data set. It is widely recognized, however that this measure alone is not enough to describe data variation adequately. For example, two data sets with the same mean may have significantly different levels of variation. Therefore, at least one other characteristic is required to measure the variation. Statistical parameters such as the range or the standard deviation can be used to measure the extent of variation.

It is true that the range, which is defined as the difference between the largest and the smallest values in a data set, gives information about the extent of data sets, but it does not provide any measure of the dispersion of the values.

Consequently, the most commonly used parameter to measure the variation is the standard deviation, since it considers the effect of all of the individual observations. The square root of the average of the squares of the numerical differences of each observation from the arithmetic mean is known as the standard deviation. The population mean should be used in calculating the standard deviation, but as it is an unknown measure, the sample mean is what is used in practice and consequently the standard deviation is known as the sample standard deviation. To compensate for the bias involved in using the sample mean instead of the population mean, n (number of observations) in the denominator of the standard deviation equation is replaced with n-1. When n is small, the bias involved in the use of S may be fairly substantial and this tends to give too low an estimate of σ [Grant and Leavenworth, 1980; Vardeman and Jobe, 1999]. Therefore, to obtain an unbiased estimate of the population standard deviation, S is often divided by a correction factor known as c_4. When the number of observations is higher than 30, the correction factor is often assumed to be equal to 1. (The values of c_4 for a sample size from 2 to 30 are given in Appendix B-3) [Duncan, 1974; Burr, 1976; Wadsworth et al., 1986].

The standard deviation value can be used to estimate the percentage of data that will fall within selected limits. Hudson [1971] suggested that, in highway design, the difference between most values in a group and the calculated average for the group will not in most cases exceed 2 times the value of σ. i.e. 95% of all data will fall within two standard deviations of the mean.

Another important parameter, which is often used to interpret variation, is the co-efficient of variation which is defined as the ratio of the standard deviation to the mean. The co-efficient of variation is a dimensionless number and, when a comparison is needed between data sets with different units or widely different means, the co-efficient of variation is used instead of the standard deviation, since the standard deviation needs to be understood in the context of the mean of the data. However, the co-efficient of variation is sensitive to small changes in the mean when the value of the mean is near to zero and it cannot be used to construct a confidence interval of the mean.

2.6 Risk analysis methods

In the literature on engineering reliability, any occurrence of an adverse event is termed failure. The probability of occurrence of such event is known as the probability of failure. The occurrence of an adverse event is mostly related to the uncertainties involved in the process. The uncertainties in design can be treated in several ways. For example, it could be ignored, accepting the risk. It could be treated by applying a higher factor of safety to the less certain parameters. But this approach is expensive, may need unacceptable completion time and sometimes may even be impossible to implement. The uncertainties can also be treated by observing behaviour and reacting accordingly. But this method is only applicable when the design can be changed during construction on the basis of the observed behaviour. Very recently, the probabilistic reliability approach has been used to treat the uncertainties where these are quantified, using the approach of observational method.

Several methods are available in the literature to deal with reliability models, listed below:

1. First order second moment methods: The first moment about the origin is the mean or expected value and the second moment is the variance which is calculated with respect to the mean. In this method, the mean (μ) and variance (σ^2) of a function (say, g) are evaluated with the means and variances (the second moment) of the variables (x_i), using only the first order terms in a Taylor expansion [Liang et al. 1999; Alonso, 1976; Tang et al. 1976; Venmarcke,1977; Barabosa et al. 1989] as follows.

$$\mu_g \approx g\left(\mu_{x_1}, \mu_{x_2}, \dots\dots\dots, \mu_{x_n}\right) \text{ ------------------------------ (2.1)}$$

$$\sigma_g^2 = \sum_{i=1}^{n} \left(\frac{\partial g}{\partial x_i} \right)^2 \sigma_{x_i}^2 \text{ ---------------------------------- (2.2)}$$

The above equations are applicable only when the variables are uncorrelated [Li and Lumb, 1987; Christian et al, 1994; Barabosa et al. 1989].

2. First order reliability method: This method was proposed by Hasofer and Lind [1974] who suggested evaluating the derivatives of first order second moment at the critical point. Using iteration was suggested as a way of finding the critical point. The distance between the failure point and the point defined by their normalized means was defined as the reliability index β. The variables were suggested to be normalized by dividing them by their respective means.

3. Point-Estimate Methods: In this method, proposed by Rosenblueth [1981, cited in Christian, 2004; Harr, 1987], the function is evaluated at a set of combinations of high and low parameter value points and the desire moments are computed using those values to obtain an accurate approximation. The points are usually taken at plus or minus one standard deviation from the mean of each of the variables for uncorrelated variables. This method is less popular in practice, since it requires more evaluations of the performance function when the number of random variables exceeds two [Wolff et al. 1996].

4. Monte Carlo Simulation: In this method, a large number of discrete values are generated from the underlying distribution to replace each continuous variable and used to compute a large number of values of performance function and its distribution [Chowdhury and Xu, 1995; Wong 1985; Cho, 2007]. A factor of safety corresponding to each set is then calculated and plotted on a probability paper to determine their distribution. The reliability index (β) and the probability of failure (P_f) are calculated using the probability distribution of the factor of safety. The method can easily be programmed for explicit performance function with simulation software such as the Excel add-in @ RISK [EI-Ramly, 2002; Duncan, 2003], but for implicit functions (such as slope stability analysis) an additional special program is required. The accuracy of this method increases with the number of iterations, but not proportionally.

5. Other methods: Some other methods exist in the literature, such as second order second moment or the second order reliability method, where higher order approximations are considered [Christian, 2004].

2.7 Developed methods for pavement risk analysis

Some methods have been developed in the last few decades to incorporate reliability in pavement design when there is uncertainty in the design data. Some research work in this area is also available in the literature. The available methods and research studies are reviewed and briefly described below.

2.7.1 Austroads [2004]

To incorporate reliability in pavement design, Austroads [2004] used the laboratory fatigue relationship published by Shell [1978]. The relationship was adjusted in the following way to include a reliability factor corresponding to the desired project reliability.

$$N = RF\left[\frac{6918(0.856V_B + 1.08)}{S_{mix}^{0.36}\mu\varepsilon}\right]^5 \text{ --------------------------- (2.3)}$$

where N = allowable number of repetitions of the load

$\mu\varepsilon$ = tensile strain produced by the load (micro strain);

V_B = percent by volume of bitumen in the asphalt (%);

S_{mix} = asphalt modulus (MPa); and

RF = reliability factor for asphalt fatigue

Some values of the reliability factors (RF) corresponding to the different reliability levels for asphalt fatigue are presented in Table 2.1:

Table 2.1: Suggested Reliability Factors (RF) for asphalt fatigue [Austroads, 2004]

Desired project reliability				
80%	85%	90%	95%	97.5%
2.5	2.0	1.5	1.0	0.67

Permanent deformation was not considered as a distress mode in the Austroads design model due to the non availability of an appropriate model which could reliably predict the development of rutting with the passage of traffic/time, as mentioned in the guide.

2.7.2 AASHTO [1993]

This guide for the design of pavement structures determined the overall standard deviation of variation by considering errors in traffic predictions and in pavement performance prediction to analyse risk and reliability in the design. A factor known as reliability design factor was determined, using the overall standard deviation of variation and was incorporated in the design traffic. The reliability of design was defined as:

Reliability, R (percent) = 100 × Probability [Actual pavement performance, $N_t \geq$ Actual design period traffic in ESAL, N_T]

The overall variance ($S_0^{\ 2}$) was defined as the sum of the variance in traffic prediction ($S_w^{\ 2}$) and the variance in prediction of pavement performance ($S_N^{\ 2}$).

$$S_0^{\ 2} = S_w^{\ 2} + S_N^{\ 2} \quad \text{-------------------------------------} \quad (2.4)$$

The following equation for the reliability design factor (F_R) was derived:

$$\text{i.e. } F_R = 10^{-Z_R \times S_0} \quad \text{--------------------------------------} \quad (2.5)$$

where, S_o is the overall standard deviation of variation and Z_R is the standard normal deviate, the value of which for different reliability levels is presented in Appendix E-4.

To estimate the variance in traffic prediction ($S_w^{\ 2}$) and the variance in pavement performance prediction ($S_N^{\ 2}$) the following approach was proposed by Noureldin et al. [1994, 1996] and Huang [1993].

2.7.2.1 Noureldin et al. [1994, 1996]

Noureldin et al. [1996] estimated the variance of traffic prediction (S_W^2) using the first order second moment approximation approach on the AASHTO's traffic prediction equation and the following was derived:

$$S_W^{\ 2} = \frac{\left[(C.O.V.ADT*D_d)^2 + (C.O.V.P)^2 + (C.O.V.L_d)^2 + (C.O.V.TF)^2\right]}{5.3} \text{ ---- (2.6)}$$

where ADT*D_d represents average daily traffic in a heavier direction; P is the percentage of trucks in the traffic mix; L_d is the lane distribution; TF is the truck factor (number of ESALs per truck). The growth factor and the design period were assumed to be constants.

To estimate the variance of the pavement performance prediction ($S_N^{\ 2}$) Noureldin et al. [1994] used AASHTO's flexible pavement performance prediction model, as follows:

$$S_N^2 = \overline{COV(MR)}^2 + P_2 \quad \overline{SN}^2 \cdot \overline{COV(SN)}^2 \text{ --------------------- (2.7)}$$

P_2 = variance component of *SN*

To determine the *COV* (*SN*), the variance of *SN* was estimated in the following way;

$$\begin{aligned} Var(SN) \cong\ & \overline{a}_1^2 Var(D_1) + \overline{D}_1^2 Var(a_1) + \overline{a}_2^2 \overline{m}_2^2 Var(D_2) + \overline{a}_2^2 Var(m_2)\overline{D}_2^2 \\ & + Var(a_2)\overline{m}_2^2 \overline{D}_2^2 + \overline{a}_3^2 \overline{m}_3^2 Var(D_3) + \overline{a}_3^2 Var(m_3)\overline{D}_3^2 + Var(a_3)\overline{m}_3^2 \overline{D}_3^2 \end{aligned} \text{ ------- (2.8)}$$

The COVs for AASHTO's layer coefficients (a_i) were estimated in the following way:

$$COV(a_1) \cong (0.33 - 0.5) COV \quad of \quad Marshall \quad Stability \text{ ----------------- (2.9)}$$

$$COV(a_2) \cong (0.33 - 0.77) COV \quad of \quad CBR \text{ -------------------------------- (2.10)}$$

$$COV(a_3) \cong (0.33 - 0.9) COV \quad of \quad CBR \text{ --------------------------------- (2.11)}$$

To estimate the COVs for AASHTO's drainage coefficients (m_i), the range of the drainage coefficient values as recommended by AASHTO [1986] was used [see Appendix C-4].

2.7.2.2 Huang [1993]

Huang estimated the variance in traffic prediction (V [logW_T]) using the first order approximate approach on the following traffic prediction equation;

$$w_T = \left(\sum_{i=1}^{m} p_i F_i\right)(ADT_o)(T)(A)(G)(D)(L)(365)(Y) \quad \text{----------------} \ (2.12)$$

where, the growth factor $G = \frac{1}{2}\left[1 + (1+r)^Y\right]$ ---------------------------- (2.13)

Huang applied first order approximation approach on AASHTO's performance prediction model to estimate the variance in pavement performance prediction (V [logW_t) and derived the following:

$$V[\log W_t] = \left(\frac{\partial \log W_t}{\partial SN}\right)^2 V[SN] + \left(\frac{\partial \log W_t}{\partial p_0}\right)^2 V[p_0] + \left(\frac{\partial \log W_t}{\partial M_R}\right)^2 V[M_R] \quad \text{---} \ (2.14)$$

Reliability was defined in the following ways:

Reliability = Probability (log W_T –log W_t <0)

2.7.3 NCHRP [2004]

The NCHRP [2004] guide for the Mechanistic-Empirical (M-E) design of new and rehabilitated pavement structures analyzes the reliability of flexible pavement design for individual pavement distresses, such as asphaltic concrete fatigue (bottom up) cracking, longitudinal (top down) cracking, rutting or asphaltic concrete thermal cracking. The reliability (R) in general is defined as the probability that the particular distress of a design project is less than the critical level of distress over the life of the design.

2.7.4 Kim [2006]

Kim [2006] presented a practical probabilistic design format to incorporate reliability in the M-E flexible pavement design procedure. It was suggested that uncertainties due to spatial variation and imprecision in quantifying parameters should be integrated as parameter uncertainties and quantified in terms of the standard deviation (S_p) of pavement performance. Similarly, it was suggested that model bias and statistical error should be integrated as systematic error and quantified in terms of the standard deviation (S_m) of pavement performance. The overall standard deviation (S_0) was determined as follows:

$$S_0 = \sqrt{S_p^2 + S_m^2} \quad \text{----------------------------------- (2.15)}$$

The study suggested the following reliability based pavement design equation with a target reliability, R, using a rut prediction model:

$$RD_{\max} = S_0 * \beta_{target} + RD_{predicted} \quad \text{------------------------- (2.16)}$$

where S_0 = overall standard deviation, as discussed above

β_{target} = the target reliability index

The rut depth of 12.7 mm was considered as a limit state ($RD_{threshold}$). The value of $RD_{threshold} - RD_{max}$ was computed and compared with the specified tolerance level. It was suggested that the design should be changed until this criterion is satisfied.

2.7.5 TRRL [1975]

In the TRRL report, Ellis [1975] mentioned various uncertainties in traffic prediction and subgrade strength. The report suggested giving more consideration to satisfactory return on the investment of highway funds and less to criteria for success or failure since road failure seldom has disastrous economic and social consequences. This means that both technical and financial risks should be considered when making decisions in this regard. To take this approach in choosing between alternative designs, the worst outcomes for all design options were considered using a decision tree and the total expenses of all design options, including the maintenance of the worst outcome during service life, were calculated with a view to find the least costly one. This approach was considered to be conservative, since it always takes into account the worst conditions. However, an improvement to this situation was suggested: that is, integrating the probability of different outcomes (the sum of probabilities of all the outcomes of any decision has to be 1.0) according to the judgement of experienced engineers.

2.7.6 Chua et al. [1992]

Chua et al. [1992] proposed a reliability model based on mechanistic pavement design principles, which took into account component reliability, fatigue cracking and subgrade rutting to construct a system's reliability. The probability of failure of the pavement section was defined as:

$$P_F = P[g(x) \le 0] = \int_{g(x)\le 0} f_x(x)dx$$

where g(x) is the limit state function which was derived from the limit-state equations for the individual distress modes. The limit state equation for fatigue cracking, considering 45% surface cracking (limiting criteria) by year T after being opened to traffic, was defined as:

$$g_{fT}(x) = L_f - \sum_{t=1}^{T}\sum_{i}\left(\frac{n_{it}}{N_{fi}}\right) \quad \text{-------------------------- (2.17)}$$

where L_f = a damage index which takes the value unity for 45% surface cracking, n_{it} and N_{fi} = the actual and allowable number of load applications in the *ith* year, corresponding to the maximum tensile strain ε_i. The limit state function for rutting by year T was defined as:

$$g_{RT}(x) = L_R - \sum_{t=1}^{T}\sum_{i}\left(\frac{n_{it}}{N_{Ri}}\right) \quad \text{-------------------------- (2.18)}$$

where L_R = a damage index equal to unity at the limiting criteria of rutting control, n_{it} and N_{Ri} = the actual and allowable number of load applications in the *ith* year, corresponding to the maximum compressive strain ε_i.

2.7.7 Alsherri and George [1988]

Alsherri and George [1988] proposed a simulation model for evaluating the reliability of pavements, based on an analytical formulation of pavement performance and the basic design parameters. The following reliability equation based on the present serviceability index was used to formulate the model:

$$R = P\lfloor p_f \ge p_t \rfloor$$

where p_f = present serviceability index at time *t*; and p_t = limiting (terminal) serviceability index, generally set at 2.5 for AASHTO's design and 3.0 for premium design. The design model proposed in AASHTO's interim guide [1972] and AASHTO's guide [1986] was used to calculate the PSI as a function of time. One computer model based on the Monte Carlo simulation method was used in this method to calculate reliability. The following expression was used to estimate reliability under the assumption that both the p_f and p_t are normally distributed:

$$R = \phi\left[\frac{\mu_{pf} - \mu_{pt}}{\left(\sigma_{pf}{}^{2} + \sigma_{pt}{}^{2}\right)^{1/2}}\right] = \phi(z_0)$$

where Φ = standard normal distribution; μ_{pf}, μ_{pt} = mean value of p_f and p_t; $\sigma_{pf,}$ σ_{pt} = standard deviations of p_f and p_t; and z_0 = standard normal deviate.

The following formulation was suggested to calculate the reliability defined above.

$$R = \frac{1}{\sqrt{2\Pi}}\int_{-\infty}^{z_0} \exp\left(\frac{-z^2}{2}\right)dz \quad \text{--------------------------- (2.19)}$$

2.7.8 Kulkarni [1994]

Kulkarni [1994] presented a methodology which chose traffic as a design element for incorporating reliability in evaluating the reliability of alternate pavement designs with different types of pavements. The reliability *R* of a pavement design was defined as:

R = probability [actual traffic load capacity, *N* > actual cumulative traffic, *n*], or

R = probability [ln *N* > ln *n*] = probability [ln *N* – ln *n* > 0]

ln N and ln n, it was suggested, would follow normal distribution since N and n are log-normally distributed. The safety margin (SM) of design was defined as:

SM = ln *N*- ln *n* ----------------------------------- (2.20)

The reliability index (β), defined as the ratio of mean (E) and standard deviation (SD) of safety margin (SM), was then used as a suitable measure for design reliability.

$$\beta = \frac{E[SM]}{SD[SM]} = \frac{E[\ln N] - E[\ln n]}{\sqrt{\mathrm{var}[\ln N] + \mathrm{var}[\ln n]}} \quad \text{----------------------- (2.21)}$$

2.7.9 Zhang [2006]

Zhang [2006] investigated the applicability of the method of moments as a technique for estimating the probability of failure in order to develop a reliability function for pavement infrastructure. A limit state function, where the functional forms of strength and stress were defined separately, was considered in the investigation, which was as follows:

G (**X**, *t*) = strength – stress (*t*)

where G (**X**, *t*) = time dependent limit state function for *n* basic random variables **X** = $[x_1 ... x_n]^T$. Strength was defined as allowable use before failure and stress as accumulated use at time t. The failure event was defined as $\{G(\mathbf{X}, t) \leq 0\}$ and the probability of failure pF (t) was expressed as a time-dependent multidimensional failure state integral for the failure event as follows:

$$pF(t) = \Pr ob[G(X,t) \leq 0] = \int_{G(X)\leq 0} f(X,t)dX$$

where f (**X,** t) was defined as a time dependent joint probability density function of the basic random variables **X**. The method of moments (first four moments) was used to evaluate the above integral.

2.7.10 Lua et al. [1996]

Lua et al. [1996] investigated the effects of spatial variability on finite-element pavement response analysis and on the predicted service life, for airfield pavements in particular. Theories of statistics and structural reliability were combined in the investigation and it was termed probabilistic finite element analysis. Random spatial variability in the layered material properties was characterized by multiple random fields and random variables were used to characterize the uncertainties in external loads and in pavement geometry.

2.7.11 Brown [1994]

Brown [1994] reviewed the advantages and disadvantages of the reliability methodology. Reliability methodology was defined as the process of estimating and correctly combining all the uncertainties associated with a particular design model into an overall variance. It was suggested that the overall uncertainty in the mathematical system could be determined if the uncertainty in each portion could be estimated. In this procedure, the sensitivity of the output to each input variable (considering other uncertainties also) was compared by means of a sensitivity analysis. It was also suggested that a safety factor should be applied as convenient. The process required only information on the number of deviations away from the mean that the design should consider. For this purpose, consulting the opinions of expert designers was suggested. The mean plus two standard deviations was suggested as the best design option for rural secondary roads and mean plus four standard deviations for urban freeways [Brown et al., 1970; cited in Brown, 1994].

2.8 Developed methods for embankment risk analysis

Geotechnical engineers are now recognizing the fact that some significant uncertainties inherent in embankment design cannot be removed by reasonable investigation effort or expenditure [Chowdhury and Xu, 1995]. Hence, some methods of risk analysis have been developed in the last few decades with regard to embankment slope stability and settlement. The present study reviewed the existing risk analysis methods and presents a brief summary of them below.

2.8.1 Developed methods of slope stability risk analysis

The uncertainty in geotechnical engineering was conventionally a matter of applying factors of safety (deterministic analysis) or of implementing local experience and engineering judgment. But the safety factor is not a consistent measure of risk. Historical analysis shows that such apparently conservative design is not proof against failure. In particular when proper consideration is not given to uncertainty, considering the safety factor alone gives a misleading sense of safety [Li and Lumb, 1987]. Probabilistic concepts and methods are excellent tools to quantify uncertainty and incorporate it in the design process. Although the benefits of probabilities analysis were identified long ago, few designers have implemented it. EI-Ramly [2002] identified the following reasons for designers to choose deterministic analysis over probabilistic:

1) Designers do not usually feel confident in dealing with probabilities, due to their lack of sufficient statistical knowledge.
2) It is wrongly assumed that probabilistic analysis requires more data, time and effort than deterministic.
3) Lack of sufficient documents in the literature demonstrating the merits and use of probabilistic analysis.
4) The tolerable range of unsatisfactory performance is not well defined and there is no link between deterministic and probabilistic assessment.

In geotechnical engineering, the probabilistic theory started to be used in the 1970s [Juang et al., 1998]. Since then, numerous studies of probabilistic slope stability analysis have been undertaken to deal with the uncertainties of soil properties in a systematic manner. They include work by Alonso [1976], Li and Lumb [1987], Tang et al. [1976], Barabosa et al. [1989], Chowdhury and Xu [1995], Wong [1985], Juang et al. [1998] and Xu and Low [2006] etc. In probabilistic slope stability analysis, the factor of safety, defined as the ratio of resistance to driving forces on a potential sliding surface, is determined deterministically and probabilistic reliability analysis is used to assess the uncertainties in the factor of safety [Bhattacharya et al., 2003]. The factor of safety is expressed in terms of its mean and variance [Malkawi et al., 2000]. The degree of uncertainty in the calculated factor of safety is usually expressed with the reliability index (β). A number of approaches is available in the literature for performing probabilistic risk analysis, as discussed in section 2.6. The existing methods of probabilistic slope stability risk analysis as found in the literature are briefly described below.

2.8.1.1 First-order second moment method

The first-order second moment method is mostly used for probabilistic slope stability analysis. It includes Wu and Kraft [1970], Tang et al. [1976], Venmarcke [1977], Li and Lumb [1987], Barabosa et al. [1989], Christian et al. [1994], Liang et al. [1999] and many others. In the first order second moment reliability method, the expected value of the performance function is calculated by evaluating the performance function using the expected values of the parameters. The variance of the performance function is computed by summing the products of the partial derivatives of the performance function at the mean parameter values and the variance of the corresponding parameters. In slope stability risk analysis with this method, the slope geometry and the probability distribution of soil properties (Φ, c and γ) are specified first. Then the critical slip surface and its associated factor of safety are determined, using limit equilibrium methods [Bishop, 1955; Janbu, 1968 and Spencer, 1967]. The partial derivatives of the factor of safety with respect to each of the soil properties are then evaluated. Next, the mean and variance of the factor of safety are calculated. Finally, the reliability index (β) and the probability of failure (P_f) of the slope are calculated. The Reliability index is defined as

$$\beta = \frac{E[F]-1.0}{\sigma[F]} \quad \text{OR} \quad \beta = \frac{E[F]-1.0}{E[F].Cov[F]} \quad \text{----------------------} \ (2.22)$$

where β is reliability index and F is computed factor of safety. The Reliability index normalizes the factor of safety with respect to its standard deviation. It defines the number of standard deviations *F* (safety) is away from its failure value of 1.0. A higher value of β means lower probability of failure.

2.8.1.2 Point estimation method

The point estimation method developed by Rosenblueth [in Harr, 1987] could be used as an alternative to Taylor's expansion [Liang et al., 1999; Li and Lumb 1987; Barabosa et al., 1989] where the moments of the performance function are determined by evaluating it as a set of combinations of high and low parameter values and weighting the results by factors. Although this method better captures the behaviour of the nonlinear functions, it is not popular in practice since many evaluations of the performance are required for more than two random variables. In addition, both Taylor's series and the point estimate method are not invariance for nonlinear performance functions and yield different values for the reliability index when the performance function and limit state can be expressed in different equivalent ways (say, force/resistance=1 or force-resistance=0). Hasofer and Lind [1974] developed an invariant reliability index determination procedure, where Taylor's series was expanded about an unknown point termed the failure point and an iterative process was used to solve it. However, many researchers were unwilling to use it, probably due to its complexity [Tang et al., 1976; Christian, 1994; Venmarcke, 1977].

2.8.1.3 Monte Carlo simulation

Chowdhury and Xu [1995], Cho [2007] and some others have recently used the Monte Carlo simulation in probabilistic slope stability analysis. In the Monte Carlo simulation method, the slope geometry and the probability distribution of soil properties (Φ, c and γ) are specified first. Then independent sets of soil properties (Φ, c and γ) are generated from their assigned probability distributions. The Limit equilibrium method [Bishop, 1955; Janbu, 1968; or Spencer, 1967] is used to calculate the factor of safety for each set, after which the mean, the standard deviation and associated probability distribution of the factor of safety are determined. Accordingly, the Reliability index (β) and the probability of failure (P_f) are evaluated.

Some other research work on probabilistic slope stability risk analysis also available in the literature; in this the first order second moment method is used in a different way; or the Monte Carlo simulation method is used with the finite element method; or fuzzy sets are used to address the uncertainties. Such as Christian et al. [1994], Liang et al. [1999], Griffiths and Fenton [2004], Juang et al. [1998], and Xu and Low [2006]. A brief summary of these methods is presented below.

2.8.1.4 Mean first order reliability method

Christian et al. [1994] simplified the general first order reliability method by proposing the mean first order reliability method. In this method, the derivatives of the first order second moment method are evaluated at the mean value of variables. To apply this method, Christian et al. [1994] divided the uncertainties which affect the stability of slope into the two following categories:

1) Data scatter: uncertainties due to real spatial variation and random testing error were termed data scatter. The spatial variability is averaged over the failure surface in case of a long failure surface and creates less uncertainty. Hence a reduction factor f was suggested by many researchers [EI-Ramly et al., 2002].

2) Systematic error: two types of systematic error were identified:

a) Statistical error in the mean due to a limited number of tests. Larger numbers of tests yield different results.
b) Bias due to the experimental procedure itself.

It was suggested that another source of uncertainty or error could result from the simplification and assumptions made in the design, for example, the use of two-dimensional analyses instead of three-dimensional, failure to find the most critical failure surface and error associated with numerical and rounding. An investigation carried out by Azzouz et al. [1983] showed that all these factors increased the mean factor of safety by 5% and impart a variation which could be estimated as COV of 0.07.

2.8.1.5 Risk analysis algorithm with Fellenius limit equilibrium method

Liang et al. [1999] proposed a reliability-based algorithm for calculating the risk in slope design where the first order second moment (FOSM) probabilistic approach was implemented into the Fellenius limit equilibrium method so as to derive the performance function and reliability index. The probability of failure was defined as:

$$P_f = P\{g(X) < 0\}$$

The performance function $g(X)$ was formulated using the modified Fellenius method in the following way;

$$g(X) = F - 1.0$$

Then the reliability index, β was defined taking into account the performance function and all the random variables normally distributed as:

$$\beta = \frac{\mu_g}{\sigma_g}$$

where μ_g and σ_g are the mean and standard deviation of the performance function, g(X). The corresponding probability of failure was then described by:

$$P_f = \phi(-\beta)$$

where Φ (.) was the standard normal cumulative probability. The following equation of β based on log normal distribution was proposed:

$$\beta = \frac{\ln\left(\mu_F / \sqrt{1+V_F^2}\right)}{\sqrt{\ln\left(1+V_F^2\right)}} \quad \text{---------------------------------- (2.23)}$$

where μ_F and V_F are the mean and co-efficient of variation of the factor of safety respectively.

2.8.1.6 Deterministic approach using fuzzy set

Juang et al. [1998] proposed an easy deterministic approach to incorporate uncertainty in soil parameters in the slope stability analysis: that of expressing the uncertainty parameters as a fuzzy set. The fuzzy set was defined as pair values [x, μ(x)] where a member x belongs to the set in a degree of μ(x), ranging from 0 to 1. A sub set of a fuzzy set called a fuzzy number was used for routine geotechnical uncertainty modelling. Since, in routine geotechnical practice, a statistical significant database is rarely available or too costly to obtain, a fuzzy number, it was suggested, could be used to reflect the uncertainty. The uncertainty could be incorporated in a fuzzy number by means of engineering judgment based on available information.

2.8.1.7 Random finite element method

Griffiths and Fenton [2004] combined a nonlinear finite element analysis with random field generation techniques for the reliability analysis of simple homogeneous slopes and termed the method the random finite element method. The method fully accounts for spatial correlation and averaging and a powerful slope stability analysis tool which does not require previous assumptions about the shape or location of the failure surface. The Monte Carlo simulation was used to calculate the failure probability.

Wong [1985] also used the Monte Carlo simulation together with a finite-element method to calculate the probability of failure of a homogeneous slope. However, the computation of the failure probability was reported as costly when there are many input variables.

2.8.1.8 Finite-element method with the first-order reliability method

Xu and Low [2006] proposed a procedure for probabilistic slope stability analysis for embankments by integrating a finite-element method and first-order reliability method (FORM), which is not restricted to any specific stability analysis method.

2.8.2 Developed methods of settlement risk analysis

Very few methods have been developed for the risk analysis of embankment settlement. A brief summary of a developed method as found in the literature for the risk analysis of soil settlement is presented below.

2.8.2.1 Fenton and Griffiths [2002]

Fenton and Griffiths [2002] proposed a probabilistic approach to a settlement risk analysis where the reliability of the foundation was assessed against the probability of excessive settlement for a single spread footing and the probabilities of excessive differential settlement for a paired spread footing. The elastic modulus of soil was considered as the main variable in soil properties. In this approach, the deterministic settlement (δ_{det}) was determined first, by using a finite element analysis including an invariant soil modulus. Then the variance of log-elastic soil modulus ($\sigma^2_{\ln E}$) was determined. Next, using these values and the variance function of width (W_f) and depth

(H) of foundation [$\gamma(W_f, H)$], the mean ($\mu_{\ln\delta}$) and the variance of log-settlement ($\sigma_{\ln\delta}$) were determined as follows:

$$\mu_{\ln\delta} = \ln(\delta_{\det}) + \frac{1}{2}\sigma^2_{\ln E} \quad \text{-------------------------- (2.24)}$$

$$\textit{and} \qquad \sigma_{\ln\delta} = \sqrt{\gamma(W_f, H)\sigma^2_{\ln E}} \quad \text{------------------------- (2.25)}$$

They used a computer program to estimate the covariance between local averages of log-elastic modulus ($C_{\ln\delta}$). Then the mean, standard deviation and correlation coefficient of actual settlement under each footing was calculated and, using these values, the mean and variance of differential settlement (Δ) were calculated. Finally, the mean absolute settlement ($|\Delta|$) was predicted and the risk of structure failure for settlement was described as the probability of mean absolute settlement greater than the limiting value. That is,

Risk of settlement failure = P [$|\Delta|$ > limiting value] -------------- (2.26)

2.9 Quality control and quality assurance

Quality Control in highway construction is a process which is undertaken to ensure that a highway pavement or embankment is constructed according to the specifications given in the design. In quality control, the degree of compliance with the standard specifications of the completed works is also assessed, providing a means by which the work can be accepted or rejected in accordance with the prescribed standards. Not all materials and construction are exactly the same and they are always subject to some variation, which may or may not be acceptable. Therefore, this variability should be taken into account in formulating the methodology of quality control procedure so that a high level of confidence can be assumed so long as sampling and testing have been carried out properly. Consequently, the highway quality assurance methods based on mathematical models and statistical concepts have progressed from the early materials-and-methods specifications through statistical end-result specifications to the current trend towards performance-related specifications (PRS) [Weed, 2006]. A number of such methods for highway pavement are described in more detail in Chapter 8 and for highway embankment in Chapter 9 with a view to developing methodology suitable for the conditions in Bangladesh.

In many cases, the data from quality-control tests are not used to correct deficiencies if they are present during construction. As mentioned previously, quality control enables the specification compliance of the pavement being constructed to be determined. This quality check should be made continuously during the construction stage. It enables modifications to be made to the original design to ensure that the pavement will perform its intended function without reaching the terminal serviceability level before the end of the design period.

Statistical end-result specifications usually suggest some sort of pay adjustments as recompense in cases where the specified level of quality has not been achieved [Pathomvanich, 2000; Douglas et al., 1999; Dobrowolski and Bressette, 1998]. In such cases a contractor's pay may be adjusted according to the level of quality actually achieved in relation to what was specified. The objective of such schemes is to cover the extra cost expected to remedy work which is initially deficient in quality by withholding sufficient payment at the time of construction. For example, if the construction work is below an acceptable standard a pavement may not be capable of carrying the design loading and thus has more chance of failing before the end of the design period. The resulting unplanned maintenance activities result in an additional expense to the highway agency and society as a whole because such repairs generally occur some time after the contractual obligations, if any, have expired.

Since no materials and construction processes are entirely homogeneous, any particular property of a material or work can be described by a large number of individual values, which will vary according to some type of distribution. Usually, these individual values can be represented by a population with a normal distribution, having a mean value and a standard deviation, with sufficient practical accuracy. The acceptance decision is based on a small number of tests made on samples or made at selected locations, since no highway agency can test the entire material or construction. The true mean of the results of all possible tests that could be made on an entire material or construction will seldom, or never, coincide with the computed average of test results from the small number of test samples. Further, there is a possibility that the test may belong to a population which is either acceptable or unacceptable in terms of specification. The probability of rejecting acceptable work based on a small number of tests is known as Type I error and designated by the Greek letter α (alpha). Conversely, the probability of accepting unacceptable work based on small number of tests is known as Type II error and designated by the Greek letter β (beta). The best way to reduce these two types of error is to increase the number of measurements in the sample [Duncan, 1974; Barker, 1994]. The following values for α and β risks are suggested by AASHTO Standard R 9[2] as stated in AASHTO's implementation manual for quality assurance [1996]:

Table 2-2: Suggested Risk Levels: AASHTO R 9[2] [After AASHTO, 1996]

CRITICALITY	α	β
Critical	5.0%	0.5%
Major	1.0%	5.0%
Minor	0.5%	10.0%
Contractual	0.1%	20.0%

2.10 Summary

This chapter presented a literature review for the important individual components of an analytical approach to road pavement and embankment design and tried to identify the variability associated with each component. It outlined various methods of data variability quantification and their advantages and disadvantages. Then various risk analysis methods for variability in design parameter were critically presented. Thereafter, the chapter focused on the quality assurance processes which can be used to assess and control the conformity of construction to requirements specified by the design. The following chapter describes the methodology used in the present research to develop models for reliability and quality assurance such as can be incorporated in a pavement and embankment design system for Bangladesh.

Chapter 3 Research Methodology

3.1 Introduction

The objective of this research, as stated in Chapter 1, is to develop a method to assess the impact of data variability and quantify risk in an integrated design system for roads and their supporting embankments in Bangladesh. The study also seeks to develop a method to introduce quality control and assurance processes in the design system. To this end, Chapter 2 presented a literature review of pavement and embankment design, design data, data variability, methods of determining data variability, quantifying risk and quality control, with a view to incorporating relevant aspects in a new procedure for analytical pavement design in Bangladesh. The review found that there are a number of techniques which can be used for these purposes and this chapter presents a methodology for using suitable techniques in a design procedure for Bangladesh.

3.2 Overall approach

The overall approach which is followed in this research to achieve the objectives is presented in Figure 3.1.

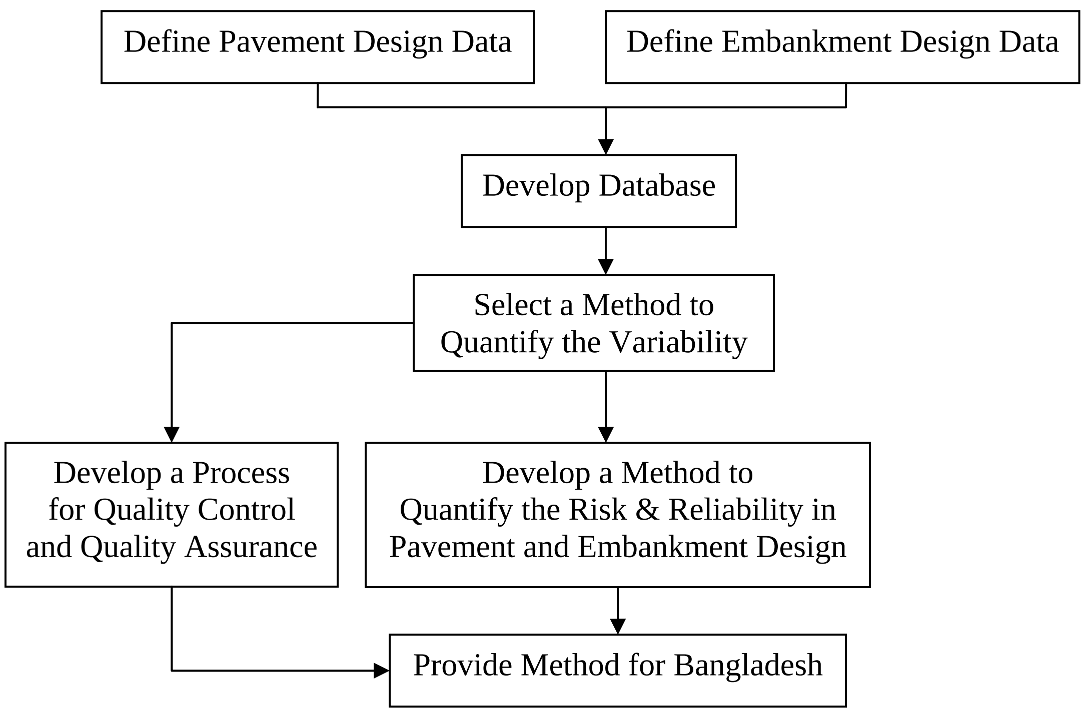


Figure 3.1: Research Methodology Flowchart

In this process the pavement and embankment design data are identified first and a database is developed with field evaluation. Then a method is selected to quantify the variability in the design data. To quantify the risk in pavement and embankment design due to variability in the design data, the available methods and techniques are investigated and a suitable method is identified and developed for Bangladesh. Similarly, a suitable quality control process is developed for pavement and embankment construction in Bangladesh by investigating the existing methods and techniques. Finally, the selected or developed methods are recommended for Bangladesh by presenting them before representative Bangladeshi road engineers, to ensure their acceptability. The detailed description of this process is presented below.

3.3 Database development

The quantification of design risk and the requirements of a quality control process are based on the variability of the design data. Hence, a database was required and was subsequently developed in this research.

3.4 Pavement design data variability analysis

As described in Section 2.4 the major uncertainties associated with an analytical design process for road pavements concern traffic, material characterisation and pavement performance prediction. Uncertainty in pavement performance prediction results from variability in climate, material properties, pavement layer thickness, pavement surface condition evaluation and sub-grade stiffness. Detailed descriptions of these factors and techniques which may be used to quantify them are summarised below.

3.4.1 Traffic prediction variation analysis

Uncertainties in traffic prediction result from variations in: the average daily traffic in heavier direction; the percentage of trucks in the traffic mix; the percentage of trucks in the design lane; and the number of ESALs per truck [Huang, 1993].

3.4.1.1 Variation in average daily traffic in heavier direction

The traffic volume and load travelling on a road in either direction varies with the time of day and season and this can have an impact on the load for which the design is calculated.

Traffic growth rates also vary from year to year. For an accurate characterization of the traffic potential, the variations should be quantified. However, this may be problematic in Bangladesh where traffic data for both directions are not usually available and it is usual to assume that equally heavy traffic travels in both directions.

3.4.1.2 Axle load composition

Heavy loaded axles are the main contributor to pavement damage [AASHTO, 1993; Sadeghi and Fathali, 2007]. The AASHTO road tests conducted in the 1950s [AASHTO, 1986] suggested that pavement damage is proportional to the axle load raised to the fourth power. Consequently, it is very important to be able to determine and quantify any potential variation in the composition of the traffic in terms of axle loads.

The problem of over-loaded vehicles causing road pavements to deteriorate faster than they were designed to is particularly apparent in many developing countries, where an increasing number of heavy and over-loaded axles move on national and regional highways. Investigation by Maheri and Akbari [1993] indicates that this problem causes a great deal of damage to road networks and results in noticeable unplanned maintenance and repair costs [cited in Sadeghi and Fathali, 2007]. Therefore, the variation in axle load should be quantified in the design risk analysis process.

3.4.1.3 Variation in percent of trucks in design lane

As mentioned previously, the traffic composition may be distributed differently across the lanes on a multi-lane road, such as a motorway. Usually, the nearside lane is most heavily loaded over the life of a pavement as it tends to carry most of the heavily laden trucks. Where it is not possible to measure the distribution of traffic across a multi-lane road, a variety of methods has been suggested for estimating the traffic using each lane from the traffic data from one or more lanes. For example, Khan [2005] has suggested lane distribution factors for use in Bangladesh. These may be used to determine the traffic in each lane from traffic surveys and are presented in Table 3.1 below. However, in practice it is unlikely that these figures are an exact representation of reality and it may be expected that there will be some variation in the traffic distribution. Such variations should be quantified since they will have an impact on the risk associated with any design which uses them.

Table 3.1: Lane Distribution Factors (LDF) for Bangladesh [After Khan, 2005]

Road Type	Lane Distribution Factor (LDF)	Remarks
Single lane	1.0	Considering AADT on both directions
2-lane single carriageway	1.0	
2-lane dual carriageway	0.75	
3-lane dual carriageway	0.60	
4-lane dual carriageway	0.45	

3.4.2 Pavement performance prediction uncertainties analysis

3.4.2.1 Variation in pavement layer thickness

Due to construction practices, there is likely to be a difference in the designed and the as constructed thickness of the pavement layers. These possible differences should be quantified to enable reliability to be incorporated in the design process. This may be achieved by calculating the relevant statistical parameters (such as the mean, standard deviation, variance, co-efficient of variation) of the thickness of the layers which constitute the road pavement. These parameters can be collected from field trials of existing roads or from existing records.

3.4.2.2 Variation in pavement layer strength

For the same reason as adduced above, pavement layer strength can also deviate from that specified in a design. To quantify pavement layer strength variation, layer strength data can be collected by field testing on existing roads and from records and in this way statistical measures of variation can be calculated.

3.4.2.3 Sub-grade stiffness variation

The sub-grade stiffness is an important parameter in the design process, being often used to characterise numerical models of the road pavement by which stresses and strains in the pavement layers are determined [Austroads, 2004]. The stiffness, however, can be expected to vary spatially as well as seasonally and such variations should be taken into account in any reliability-based approach to pavement design. Usually this may be achieved by determining statistical parameters such as the co-efficient of variation of sub-grade stiffness for the length of the new road and, if possible, the associated seasonal variation.

3.5 Embankment design data variability analysis

Uncertainty in the embankment design system, as identified in section 2.4, generally results from variability in geometry, load, soil strength parameters, sub-soil properties and consolidation factors. To quantify the risk and reliability, these uncertainties must be evaluated. The procedure for achieving this is similar to that for determining the reliability of road pavement design, as discussed above. Namely, the potential variability of each of the parameters can be calculated using standard statistical techniques and these measures of variability can then be used in an appropriate method to determine reliability.

3.6 Design risk quantification and reliability integration

To quantify the risk associated with design data variability and to incorporate reliability in a design system for Bangladesh, the research investigates the various methods of risk quantification and the processes available in the literature of incorporating reliability in pavement and embankment design. In addition, the research also studies the process of determining risk and reliability due to data uncertainty in other branches of engineering. To identify or to develop the most suitable methods for Bangladesh, a number of criteria were used, as follows:

1. **Accuracy:** Any method selected must accurately determine risk and reliability.
2. **Suitability for use in the proposed design system:** The main goal of the research project is to develop a comprehensive design system for Bangladesh. Consequently, it is important that any method chosen should be suitable for the design process being developed for the country.
3. **Appropriateness for the conditions in Bangladesh:** As the pavement design system being developed will be used in Bangladesh, the appropriateness of the method for its conditions is also an important criterion which includes the following:

a) Suitability for use with the data available in Bangladesh: that is, the methods should make use of the data available in Bangladesh.

b) In accordance with the prevailing distress mode in Bangladesh: Reliability is closely related to the predominant pavement distress modes. Thus a method incorporating reliability should be selected such that it addresses the prevailing pavement distress modes found in Bangladesh. For example, wheel track rutting is not seen in Bangladesh, but cracking is common.

c) Requiring simple computational procedure: Given the socio-economic conditions in Bangladesh, procedures which are less rigorous will suit the country better.

4. **Consider all the uncertainties in pavement design:** The area of uncertainties in pavement and embankment design was identified in section 2.4. A proper method of incorporating reliability in the pavement and embankment design system of Bangladesh should address all these uncertainties.

A detailed description of the various methods of incorporating reliability found in the literature is presented in the literature review chapter and a discussion of their relative merits with respect to the above criteria is given in Chapters 5 and 6 for pavements and embankments respectively. The most appropriate techniques identified are used in the logical design of a system to determine design reliability for Bangladesh and this is also discussed in these two chapters. Finally, field testing data are used to validate the suggested or developed method; these are presented in Chapter 7.

3.7 Overall risk and reliability

Given the ultimate objective of this research project, the study seeks to develop a procedure to combine the reliability of pavement design and the reliability of embankment design. The way in which this was achieved is presented in Chapter 6.

3.8 Quality control and quality assurance

Weed [2006] suggests that a criterion which can be used to measure the quality of construction should meet the three following requirements:

1. It should be clearly related to performance.
2. Its performance relationship should be consistent.
3. It should be convenient to measure and quantify.

In addition to these, a quality control process for the design system of Bangladesh must satisfy the following:

1. It should be suitable for the design system of Bangladesh
2. It should be easy to understand and implement.
3. It should be convenient to measure and quantify

Different organizations and manufacturers follow different approaches for the quality control of their materials and products. The prevailing quality control processes in pavement and embankment construction, as found in the literature, are studied in this research and briefly discussed, including their suitability and adaptability for Bangladesh in Chapters 8 and 9 respectively. The most appropriate techniques identified are used in the logical design of a system for the quality control of pavement and embankment construction in Bangladesh; this is also discussed in these chapters respectively.

3.9 Summary

The methods of analyzing data variability used in the present research have been discussed in this chapter. The most appropriate statistical parameters to quantify the variability of data used for the design of road pavements and embankments in Bangladesh were identified from the literature. These are to be incorporated in risk assessment tools for analytical pavement and embankment design, respectively. The components and requirements for such tools were identified and described in this chapter, whilst a detailed comparison of a number of techniques which may be used to construct these tools is given in Chapters 5 and 6. Finally, the requirements were identified for a procedure which can be used for quality control and assurance purposes. A suitable method for meeting these requirements for Bangladesh is described fully in Chapters 8 and 9 with reference to pavements and embankments, respectively.

Chapter 4 Data Quality Analysis

4.1 Introduction

This chapter considers the quality and variability of input data for the design system of Bangladesh. First, the data collection process is briefly discussed. Then the pavement design data are analysed for quality and variability. Data collected from four roads of Bangladesh Roads and Highways Department road network are considered in this regard. The acceptability of the data is also judged. The statistical parameters of the data sets, as discussed in the literature review chapter, are also evaluated. In the latter part of the chapter, the quality of the embankment design data is considered. The same procedure as discussed above is followed to assess the variability of the embankment design data.

4.2 Data Collection

The Roads and Highways Department of Bangladesh (RHD) is responsible for the construction and maintenance of the country's major road network. There are three main types of road in this network. These are National highways, Regional highways and Zilla (rural) roads. Data were collected from two representative National Highways (N302: Tongi-Ashulia-Zerabo-EPZ Road, N4: Joydevpur-Tangail-Jamalpur Road), one Regional Highway (R301: Tongoi-Kaligonj-Gorashal-Panchdona Road) and one Zilla road (Z3024: Master Bari-Mirzapur-Pirujali-Nuhashpalli-Mawna Road). Embankment design data were collected from N302 road by conducting field and laboratory tests. To collect pavement design related data, a number of tests were considered. These included coring, digging a pit (0.75m×0.75m), field density test, Dynamic Cone Penetrometer (DCP) test, Benkelman beam deflection test, condition survey (potholes, cracking, rutting, depression, edge break), roughness survey, traffic count survey and axle load survey. Some laboratory testing was also carried out in this project to determine soaked CBR (California Bearing Ratio), moisture content, maximum dry density, recovered bitumen and aggregate properties and Marshall Stability. The pavement laboratory testing was done at the Bangladesh University of Engineering and Technology and the Dhaka University of Engineering and Technology. Field and laboratory testing was carried out on the embankment materials at the Bangladesh Road Research Laboratory and at the University of Birmingham.

4.3 Pavement Data Quality Analysis

4.3.1 Traffic prediction data

4.3.1.1 Traffic data

Traffic data were collected by conducting two days (48 hrs) of traffic count surveys in roads N4, R301 and Z3024. It is understood that the quality of data collected is very limited and therefore the assessment of traffic variability over the year is not ideal. However the emphasis of this research was on the development of the methodology and the production of some preliminary results rather than on the full assessment of the traffic data variability. For N302 road, traffic data were collected from records. The average traffic for each category was calculated and the total traffic expressed in terms of annual average daily traffic (AADT). Table 4.1 shows the traffic characteristics of the different roads of Bangladesh which were surveyed. The traffic composition of these roads is presented in Figures 4.1-4.4.

Table 4.1: Summary of traffic count survey

Road No.	Heavy Truck	Medium Truck	Small Truck	Large Bus	Medium Bus	Micro Bus	Utility	Car	Auto-Rickshaw	Motor cycle	Total AADT
	nos.	nos.	nos.	nos.	nos.	nos.	nos.	nos.	nos.	nos.	nos.
N4	82	5822	1320	738	1721	1646	1409	2462	2673	698	18570
N302	158	1890	474	1082	1181	1098	956	1965	255	244	9303
R301	71	1824	446	190	546	912	420	848	910	329	6496
Z3024	0	174	4	0	136	21	44	60	558	321	1318

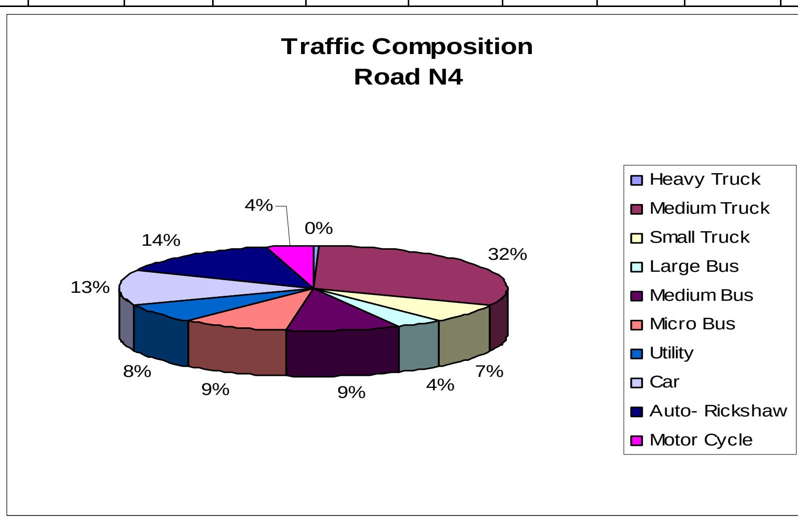


Figure 4.1: Composition of traffic in Road N4

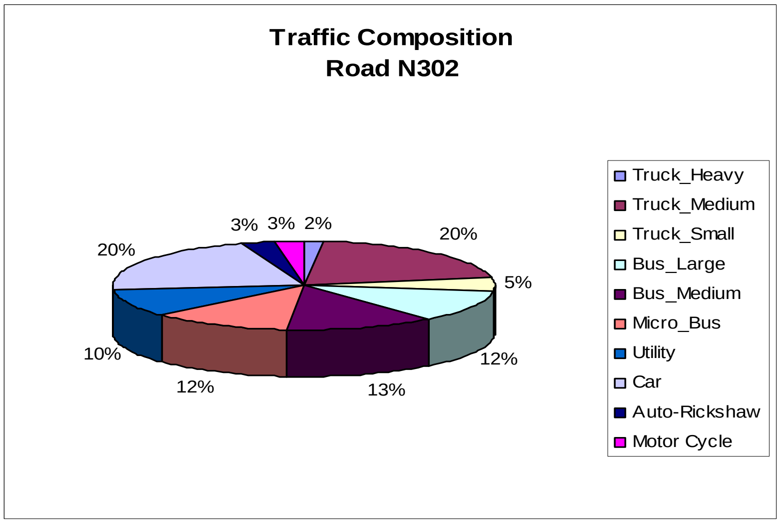


Figure 4.2: Traffic composition in Road N302

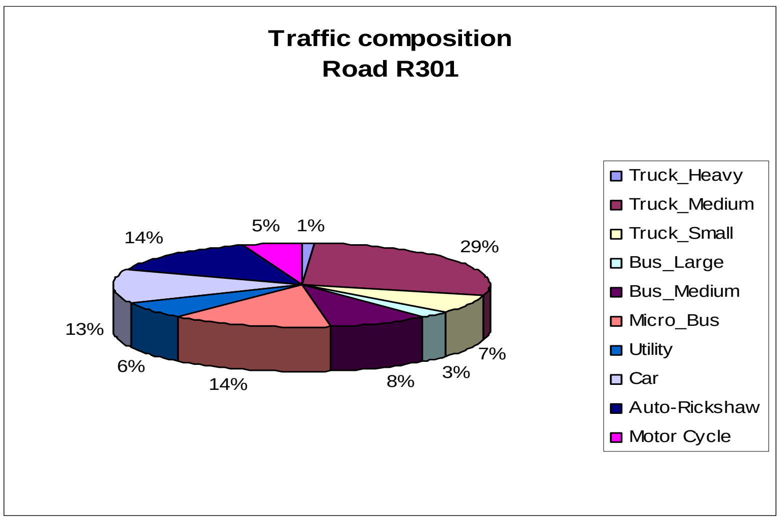


Figure 4.3: Traffic composition in Road R301

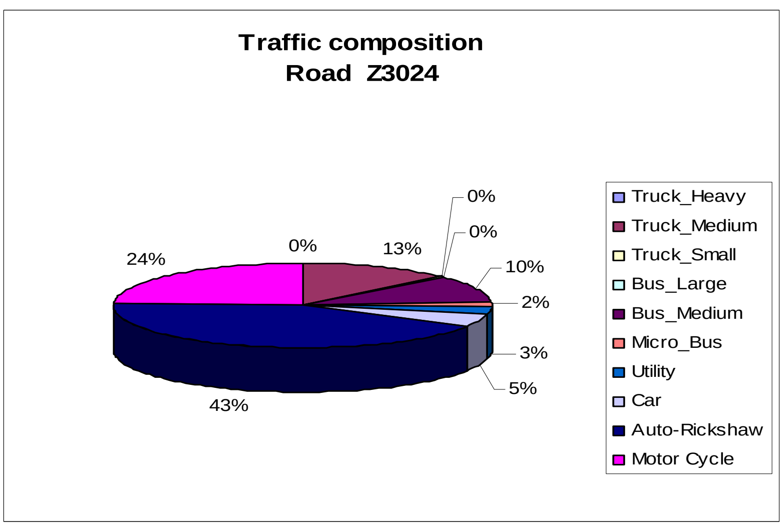


Figure 4.4: Composition of traffic in Road Z3024

Definitions of the different types of vehicle are given in Appendix D. The percentage of trucks and buses (large, medium) in the traffic mix is calculated and presented in Table 4.2 for all the four roads under investigation. The axle loads of the different types of truck and bus also vary. Table 4.3 presents the proportion of different types of vehicle groups causing more damage.

Table 4.2: Percentage of trucks and buses (large, medium) in the traffic mix

	Road N4	Road N302	Road R301	Road Z3024
Percentage of Trucks and Buses (large, medium)	52.14%	51.44%	47.37%	23.82%

Table 4.3: Proportion of different types of vehicle groups causing more damage

	Heavy Truck (%)	Medium Truck (%)	Small Truck (%)	Large Bus (%)	Medium Bus (%)
Road N4	0.84	60.13	13.63	7.62	17.78
Road N302	3.30	39.50	9.91	22.61	24.68
Road R301	2.31	59.28	14.49	6.17	17.74
Road Z3024	0.00	55.41	1.27	0.00	43.31

Nearly fifty percent of the traffic on roads N4, N302 and R301, according to Table 4.2, are trucks or heavy buses. Among them, the proportion of medium trucks is higher. On road Z3024, the percentage of trucks is less than twenty five percent and is mostly composed of medium trucks and buses.

4.3.1.2 Axle load data

An axle load survey was conducted on the sites selected for this research programme. Overloading is a major problem which makes the axle load data highly unpredictable. The statistical parameters of the axle load data sets, such as mean, median, standard deviation, co-efficient of variation, range are evaluated. Then the equivalent axle load factor of mean axle load for all vehicle categories is determined using fourth power rule and then the Asphalt Institute chart and its maximum value was used. The summarised results are presented in Table 4.4.

Table 4.4: Axle load data summary

	Heavy Truck				Medium Truck		Small Truck		Large Bus		Medium Bus	
	Front	Rear1 (Tractor)	Rear1 (Trailer)	Rear2 (Trailer)	Front	Rear	Front	Rear	Front	Rear	Front	Rear
	tonne	tonne	tonne	tonne	tonne	tonne	tonne	tonne	tonne	tonne	tonne	tonne
Mean	6.36	7.03	12.76	12.66	5.06	13.38	2.81	5.67	5.04	7.76	2.64	4.55
Median	6.30	8.1	14.7	14.2	5.2	13.85	2.9	4.5	5.2	8.0	2.6	4.2
Std Dev	1.92	2.20	5.97	6.04	1.56	5.26	1.08	3.74	1.10	1.70	0.78	1.47
COV (%)	30.15	31.32	46.76	47.71	30.7	39.32	38.53	66.03	21.77	21.86	29.6	32.2
Variance	3.67	4.85	35.59	36.47	2.43	27.67	1.17	14.02	1.20	2.88	0.61	2.15
Range	3.7-9.6	4.5-8.5	3.7-21.8	2.9-21.5	1.9-8.1	2.6-25.2	1.1-4.3	1.2-13	2.4-7.0	3.4-10.4	1.4-4.2	2.2-8.0
EALF	**0.368**	**0.551**	**5.96**	**5.77**	**0.148**	**7.205**	**0.014**	**0.233**	**0.145**	**0.816**	**0.011**	**0.097**

It may be seen from Table 4.4 that the mean equivalent axle load factor (EALF) for medium trucks is significantly high, with a co-efficient of variation (COV) of between 30% and 40%. The EALF of Heavy trucks is also significant and its COV ranges from 30% to 50%. Small trucks, although they have lower EALF, show higher COV. The EALF of large buses has a lower COV but that of medium buses is higher.

4.3.2 Deflection data

The Benkelman beam deflection testing procedure was used to collect deflection data. The RHD's manual for Benkelman beam deflection testing [RHD, 2005] was followed in the testing and calculation procedure. The deflection data were collected from the four roads mentioned in section 4.2. Figures 4.5-4.8 show the variation in deflection data. Statistical analysis is carried out to evaluate the variability. The summary of this analysis is presented in Table 4.5. In road N4, deflection data is found to be highly variable with a high COV (61.12%). The same trend is shown in roads R301 and Z3024. The values of the COV for these two roads are 38.02% and 49.48%, respectively. However, the deflection data of N302 shows less variation with a COV of less than 20%.

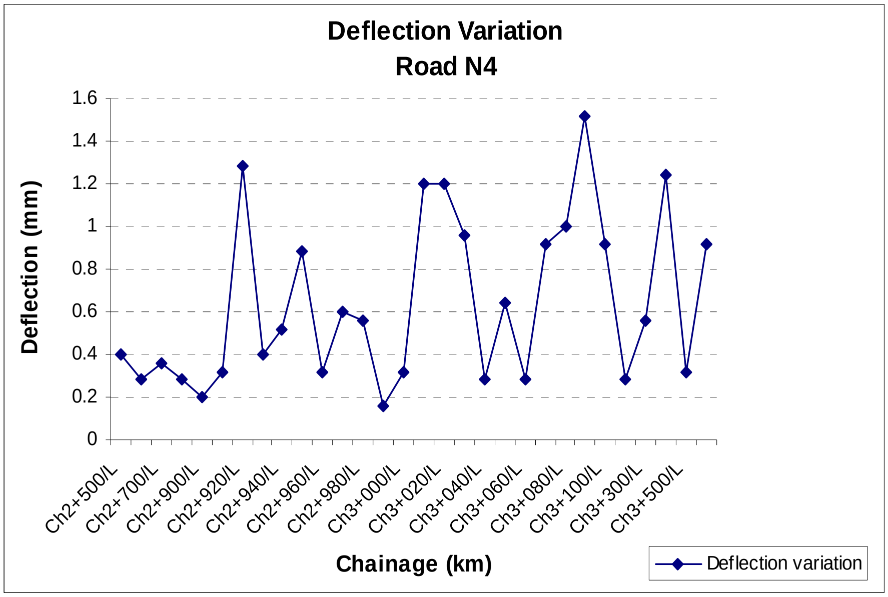


Figure 4.5: Variation of deflection data in Road N4

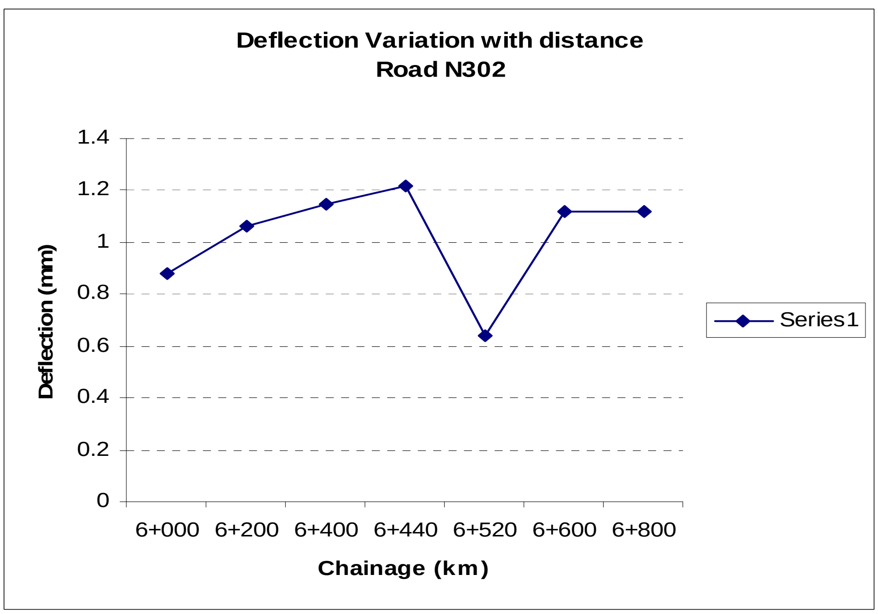


Figure 4.6: Variation of deflection data in Road N302

Table 4.5: Summary of statistical analysis of deflection data

Road	Mean	Median	Standard Deviation	COV	Variance	Mini-mum	Maxi-mum	Range
	mm	mm		%		mm	mm	mm
N4	0.64	0.54	0.39	61.12	0.15	0.16	1.52	0.16-1.52
N302	1.0	1.12	0.20	19.55	0.04	0.64	1.22	0.64-1.22
R301	2.81	2.50	1.07	38.02	1.14	1.32	4.40	1.32-4.40
Z3024	2.37	2.00	1.17	49.48	1.37	0.80	4.72	0.80-4.72

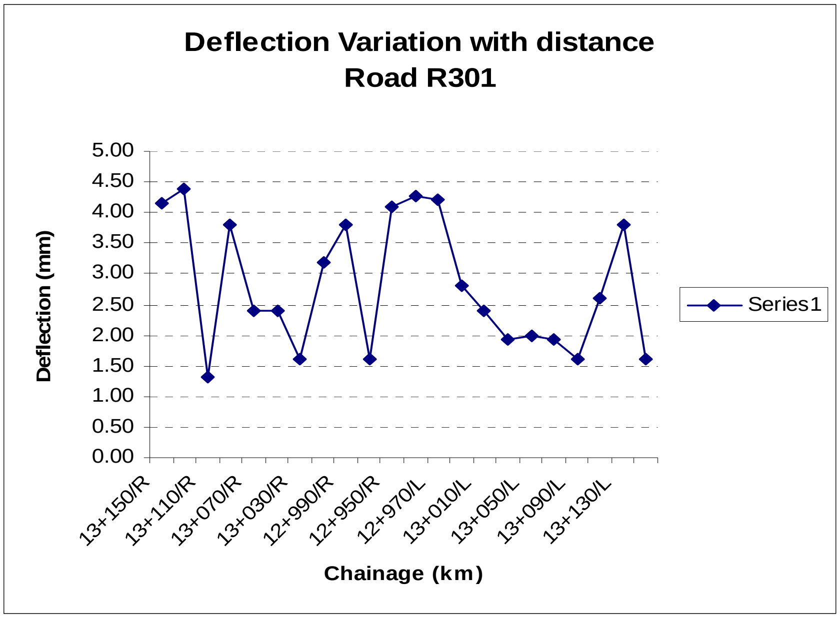


Figure 4.7: Variation of deflection data in Road R301

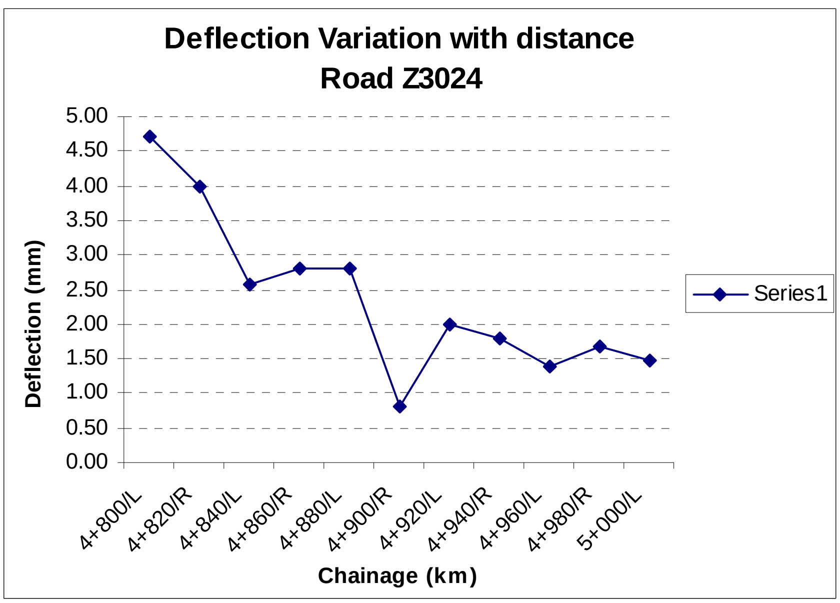


Figure 4.8: Variation of deflection data in Road Z3024

The deflection data range from 0.16 to 1.52 mm for N4, from 0.64 to 1.22 mm for N302, from 1.32 to 4.40 mm for R301 and from 0.80 to 4.72 for Z3024, showing the high variability of the deflection data between them. To assess the quality of the data collected a comparison was made between these data and a data set already used by RHD. Deflection data were taken from the HDM (Highway design and Management) database of the Roads and Highways Department (RHD) of Bangladesh. The summary statistics of these data is presented in Table 4.6. High variation is also found in the HDM deflection data with COV ranging from 15.21% to 44.44% in different roads.

Table 4.6: Summary statistics of HDM deflection data

Road	Mean	Median	Standard Deviation	COV	Variance	Mini-mum	Maxi-mum	Range
	mm	mm		%		mm	mm	mm
N4	1.38	1.00	0.61	44.44	0.38	1.00	3.00	1.00-3.00
N302	1.48	1.00	0.37	25.20	0.14	1.00	2.00	1.00-2.00
R301	1.29	1.00	0.23	17.53	0.05	1.00	2.00	1.00-2.00
Z3024	1.32	1.00	0.20	15.21	0.04	1.00	1.00	1.00-1.46

4.3.3 Pavement layer thickness data

To evaluate the pavement layer thickness, coring and a trial pit of 0.75m×0.75m were applied in the selected roads. Dynamic Cone Penetration (DCP) tests could be used to determine pavement layer thickness but the DCP data were found unacceptable, as discussed later in this chapter. Hence the DCP data were not used to calculate pavement layer thickness. The variability in pavement layer thickness data is shown in Figures 4.9-4.12 and the summary of the statistical analysis is presented in Table 4.7.

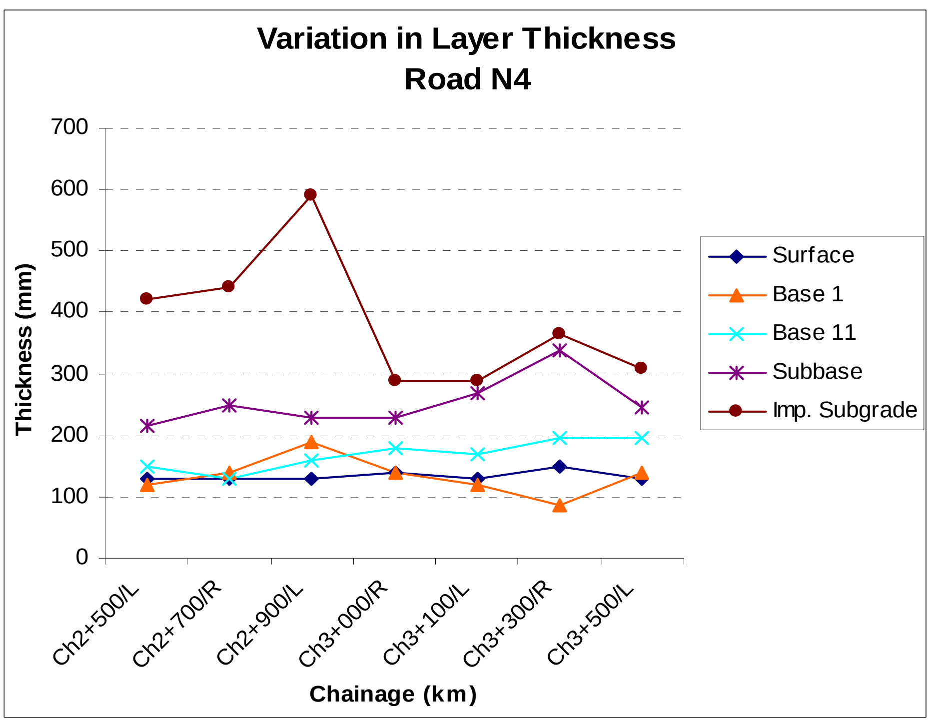


Figure 4.9: Variation in pavement layer thickness in Road N4

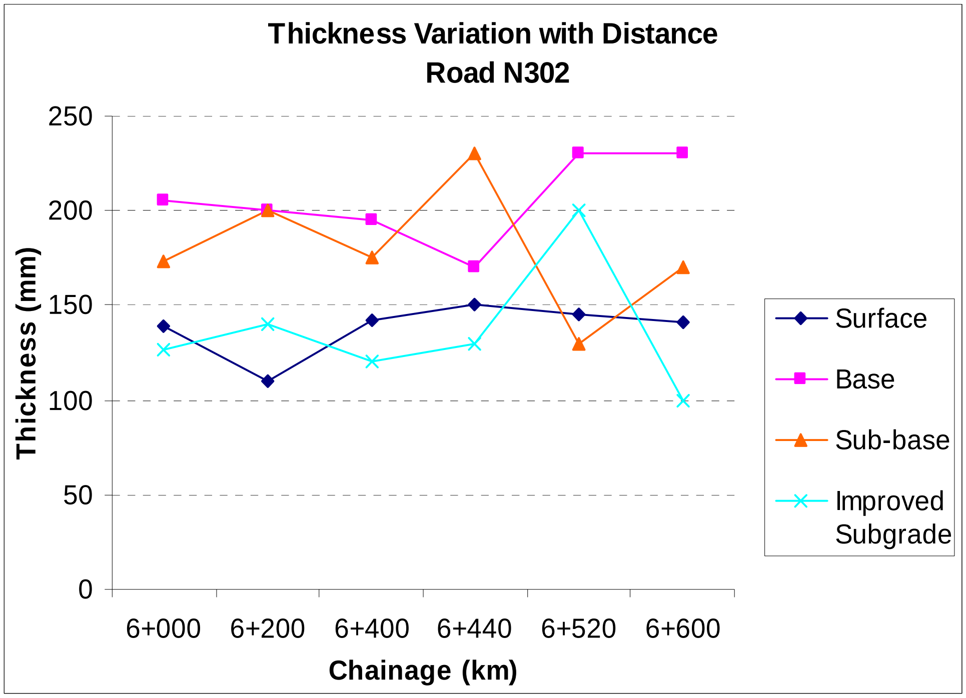


Figure 4.10: Variation in pavement layer thickness in Road N302

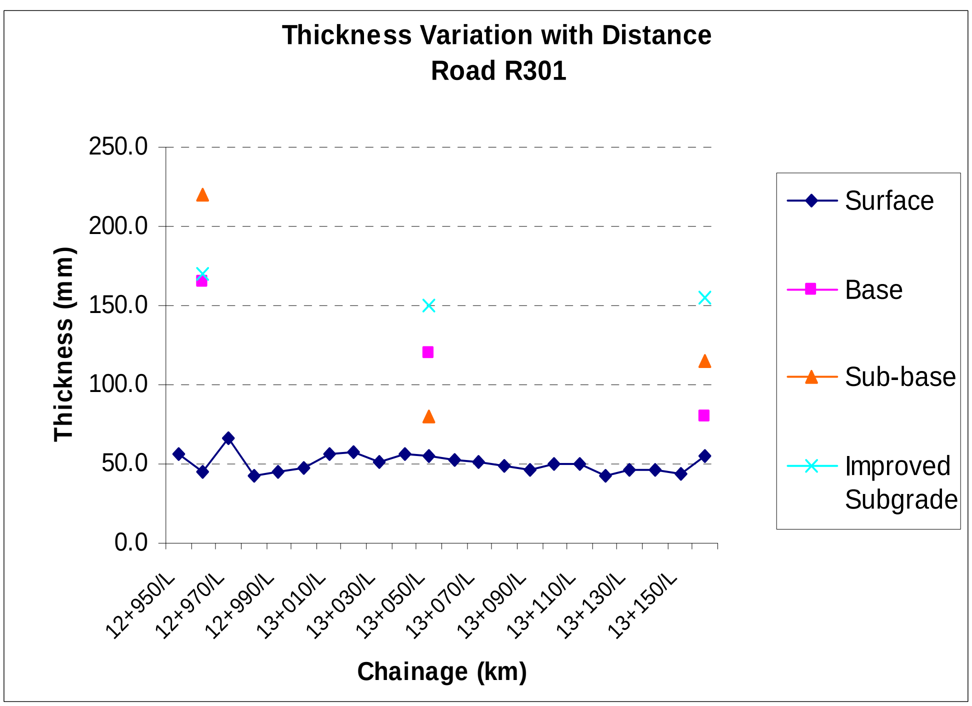


Figure 4.11: Variation in pavement layer thickness in Road R301

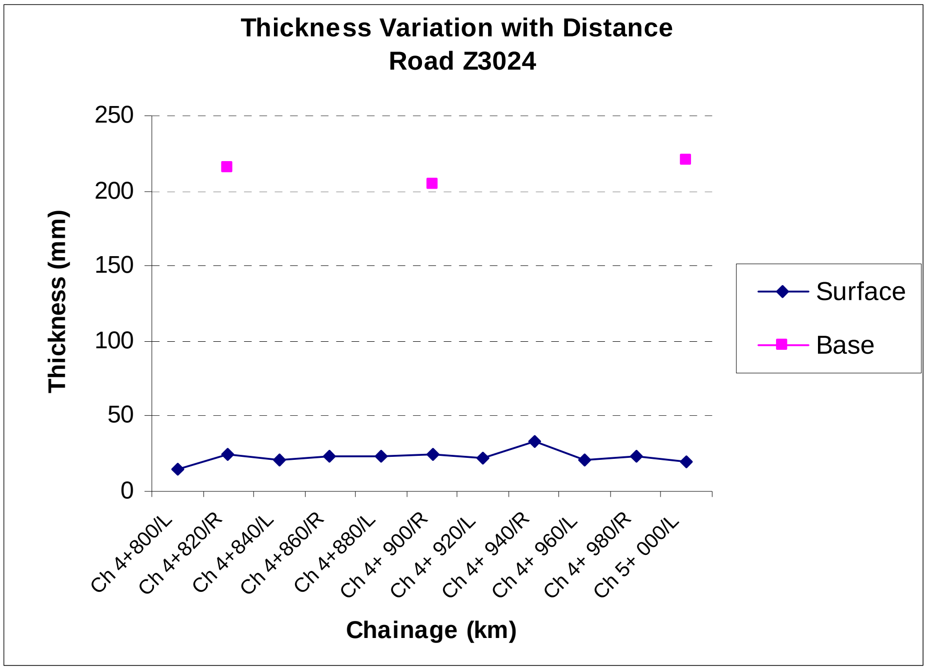


Figure 4.12: Variation in pavement layer thickness in Road Z3024

Table 4.7: Summary statistics of pavement layer thickness data

Road	Layer	Mean	Median	Standard Deviation	COV	Variance	Range
		mm	mm		%		mm
N4	Surface	134.29	130	7.87	5.86	61.90	130-150
	Base 1	133.57	140	31.72	23.75	1006	85.0-190
	Base 11	168.57	170	23.93	14.20	572.62	130-195
	Sub-base	254.29	245	41.68	16.39	1736.9	215-340
	Imp. subgd	386.43	365	108.27	28.02	11722.62	290-590
N302	Surface	137.83	141.5	14.16	10.27	200.57	110-150
	Base	205.00	202.5	22.80	11.12	520	170-230
	Sub-base	179.67	174	33.39	18.58	1114.67	130-230
	Imp. subgd	136.17	128.5	34.00	24.97	1156.17	100-200
R301	Surface	50.63	50	5.93	11.71	35.15	42.7-66.7
	Base	121.36	120	32.15	26.49	1033.77	80.0-165
	Sub-base	127.73	115	59.7	46.74	3563.64	80.0-220
	Imp. subgd	156.82	155	8.53	5.44	72.73	150-170
Z3024	Surface	23.04	23.70	4.52	19.63	20.45	14.3-33.0
	Base	213.33	215	7.64	3.58	58.33	205-220

In road N4, the base and improved subgrade layer thickness data are found highly variable, with a co-efficient of variation of 23.75% and 28.02% respectively. Reasonable variability is shown by the surface, base and sub-base layer thickness data in N302 with a COV of less than 20%. However, significant variability is found in the base and sub-base layer thickness data in R301 with COV of 26.49% and 46.74% respectively.

The pavement layer thickness data also collected from the HDM database of RHD were analyzed statistically to find their variability and compared with the data from the four roads selected by this research. The results are summarized in Table 4.8. The HDM data on the selected roads also shows substantial variation with a COV in most cases ranging between 20% and 30%. High variability is found in N4 with a COV of more than 28% in all layers.

Table 4.8: Statistical analysis summary of pavement layer thickness (HDM data)

Road	Layer	Mean	Median	Standard Deviation	COV	Variance	Range
		mm	mm		%		mm
N4	Surface	88.13	87.00	31.50	35.74	992.25	6-120
	Base	253.02	241.00	70.94	28.04	5032.41	60-340
	Sub-base	175.75	200.00	49.72	28.29	2472.14	13-272
	Imp. subgd	220.43	210.00	63.84	28.96	4075.06	75-345
N302	Surface	105.00	105.00	25.50	24.28	650.00	40-145
	Base	158.41	150.00	43.20	27.27	1866.26	101-255
	Sub-base	197.41	195.00	46.39	23.50	2151.88	128-307
	Imp. subgd	229.76	228.00	41.54	18.08	1725.94	157-303
R301	Surface	59.33	58.00	8.84	14.90	78.14	50-80
	Base	153.79	155.00	32.26	20.97	1040.52	100-215
	Sub-base	168.08	183.00	39.48	23.49	1558.69	70-210
	Imp. subgd	199.00	196.00	51.15	25.70	2616.09	126-320
Z3024	Surface	41.00	39.00	8.55	20.85	73.07	30-50
	Base	167.31	170.00	37.52	22.42	1407.56	110-240

4.3.4 Variation in pavement layer strength data

Dynamic Cone Penetration testing was used to evaluate the pavement layer strength. The California Bearing Ratio (CBR) was calculated from the DCP penetration rate, using an empirical equation [Harison, 1987] the suitability of which was identified by Shahjahan [2009]. Figures 4.13-4.16 present the variation in the CBR values calculated from the DCP results.

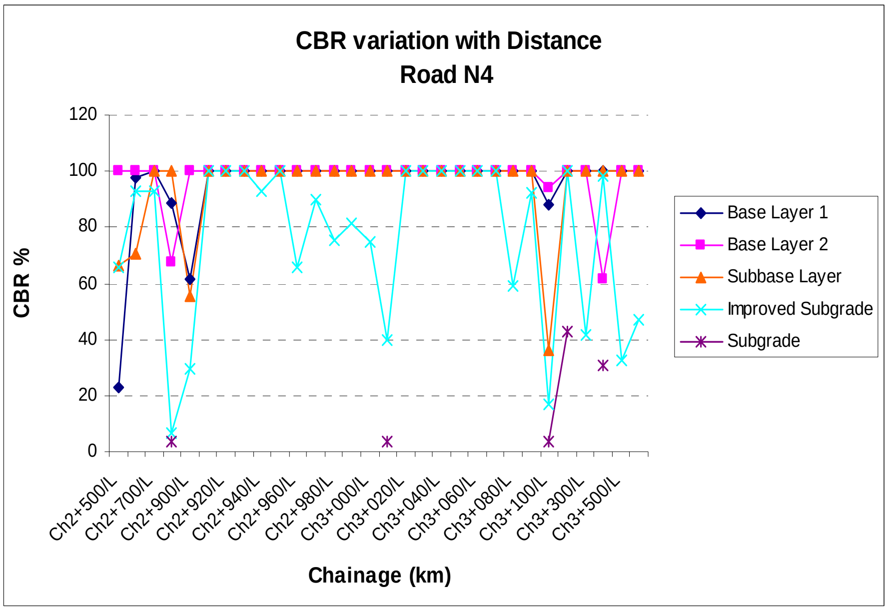


Figure 4.13: Variation in CBR estimated from DCP tests in Road N4

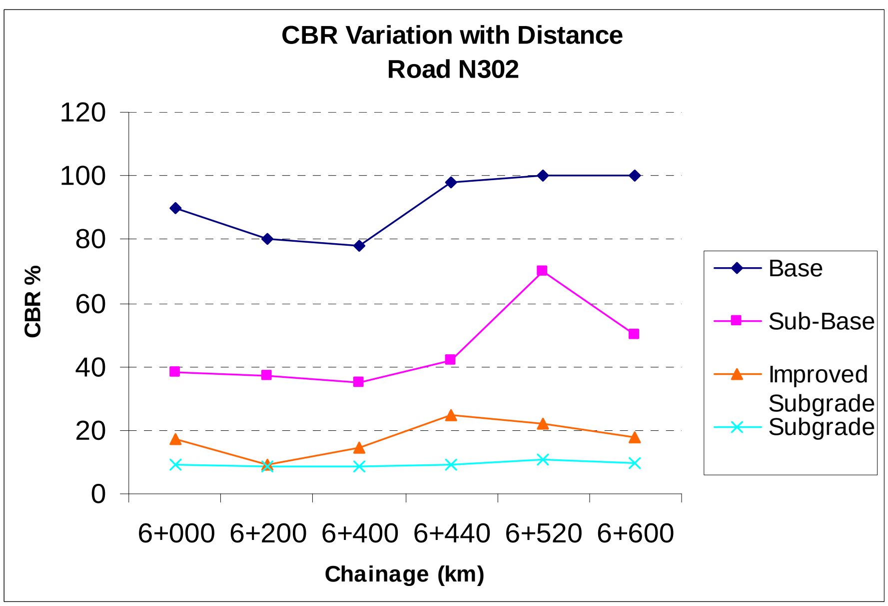


Figure 4.14: Variation in CBR estimated from DCP tests in Road N302

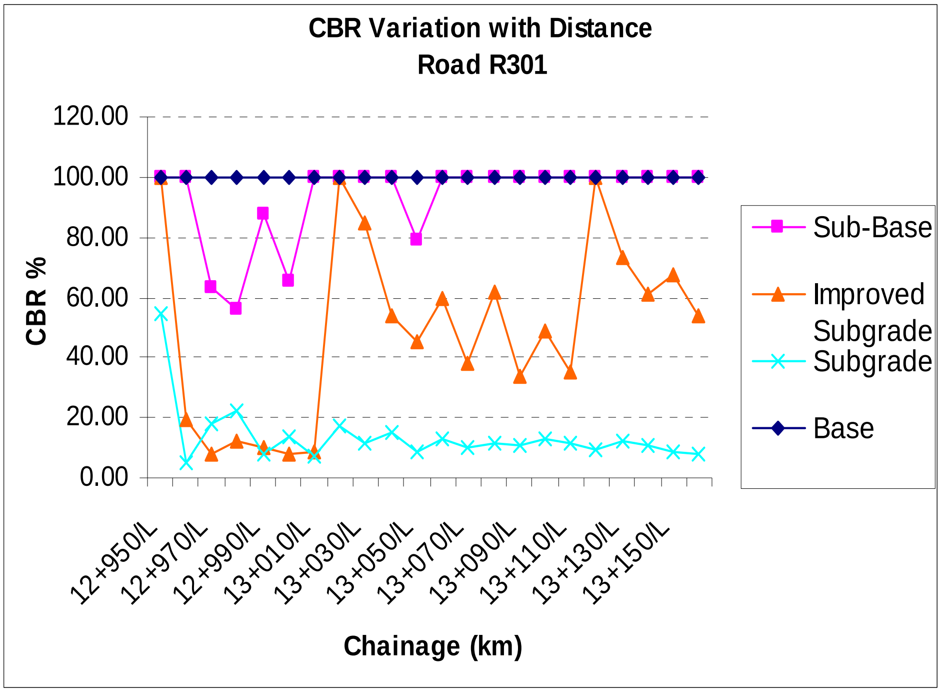


Figure 4.15: Variation in CBR estimated from DCP tests in Road R301

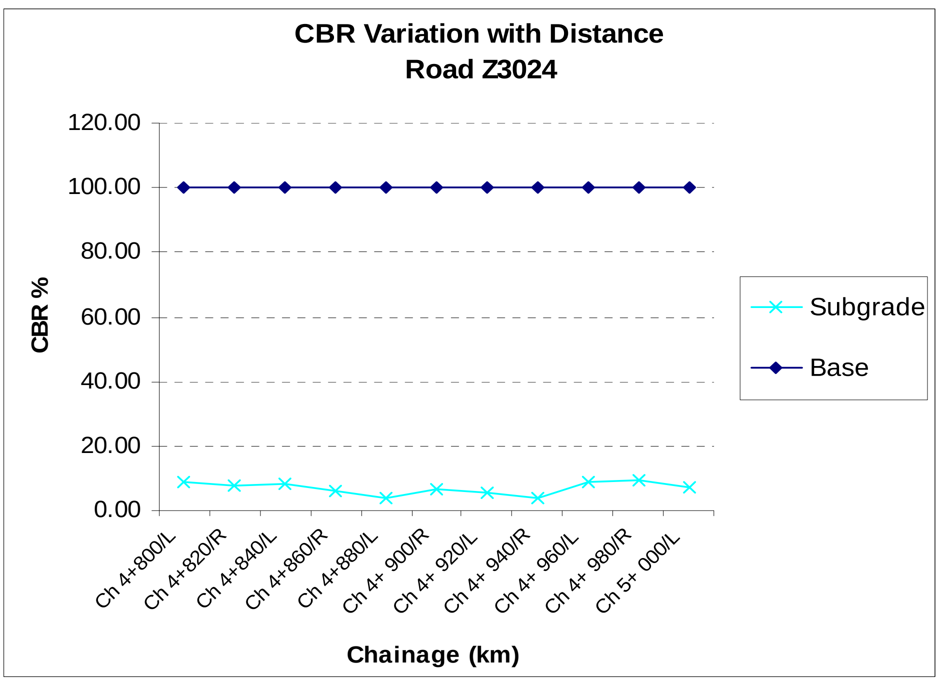


Figure 4.16: Variation in CBR estimated from DCP tests in Road Z3024

But in most cases the DCP and the corresponding CBR values give unrealistic results. For example, the CBR of base, sub-base and improved subgrade (sand) layer of road N4 took the value of 100% in almost 90% of cases, as shown in Figure 4.13, which appears to be unlikely. In road R301 the CBR value is found to be 100% at all points for the base layer and almost 90% points for the sub-base layer. Although the DCP test in road N302 gives reasonable results, in road Z3024 the DCP value gives a CBR of 100% at all points for the base layer.

The soaked CBR values determined from laboratory testing were also considered. Only a limited number of soaked CBR tests were carried for this research project, due to constraints of time. The variability of data is shown in Figures 4.17-4.20 and the statistical analysis summary is presented in Table 4.9. A significantly high COV of 89.34% is found for the base layer of road N4. The strength data of other layer of this road show reasonable variation. Significant variability in the data is also found in those on the strength of the sub-base layer (COV=29.12%) and strength of the improved subgrade layer (COV= 31.99%) of road N302. The data on the strength of the base, sub-base and subgrade layer of road R301 also show high variability with COV of 48.38%, 66.11% and 53.71% respectively. Reasonable variability is found in the data on the layer strength of road Z3024, with a COV value of less than 20%.

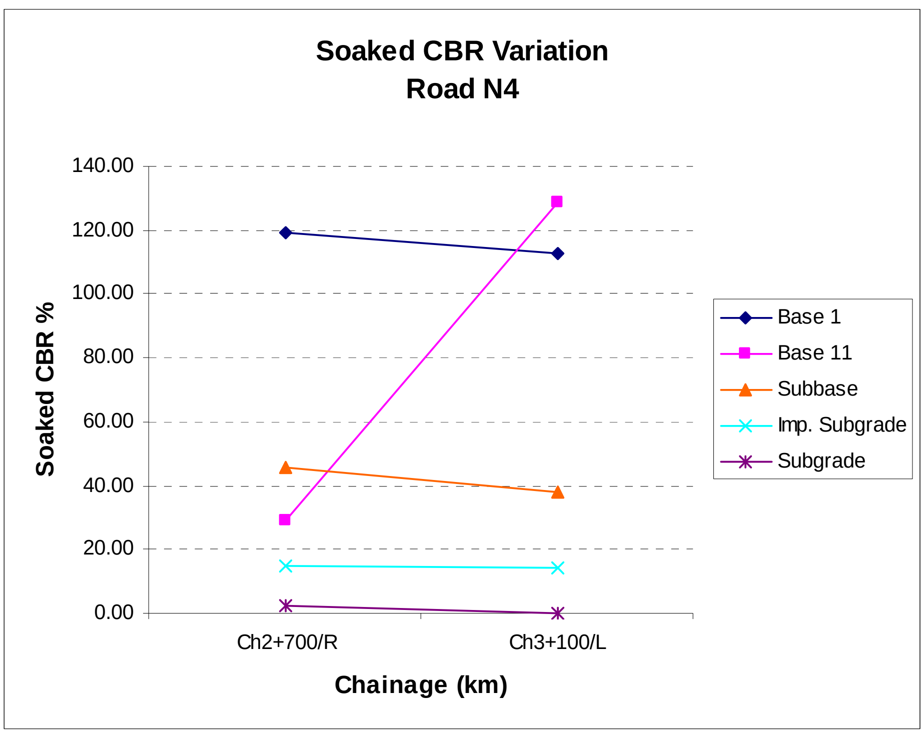


Figure 4.17: Variation in pavement layer strength (soaked CBR) in Road N4

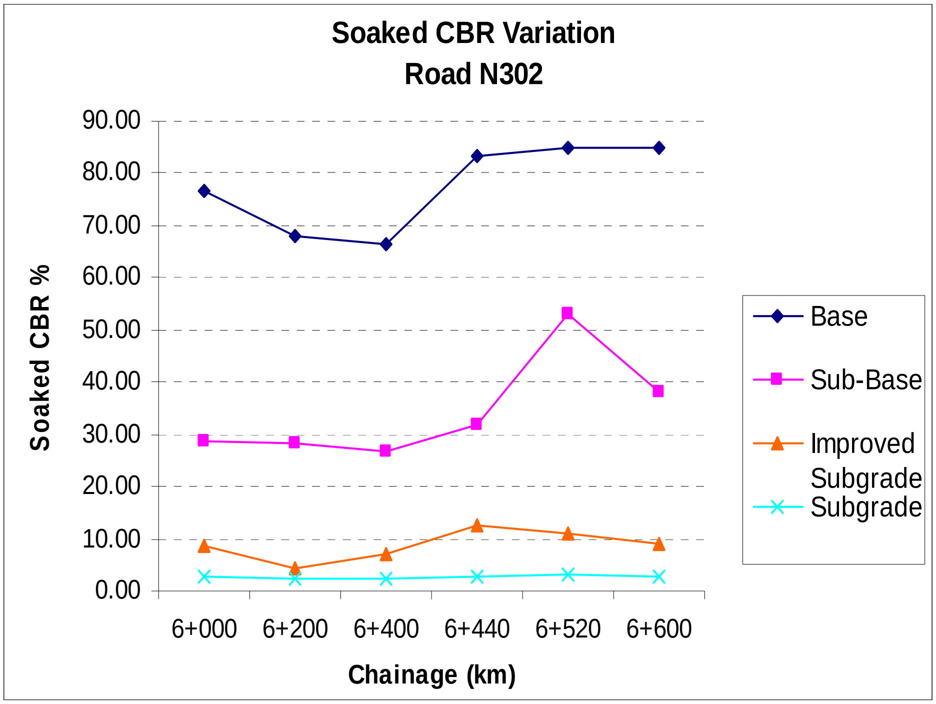


Figure 4.18: Variation in pavement layer strength (soaked CBR) in Road N302

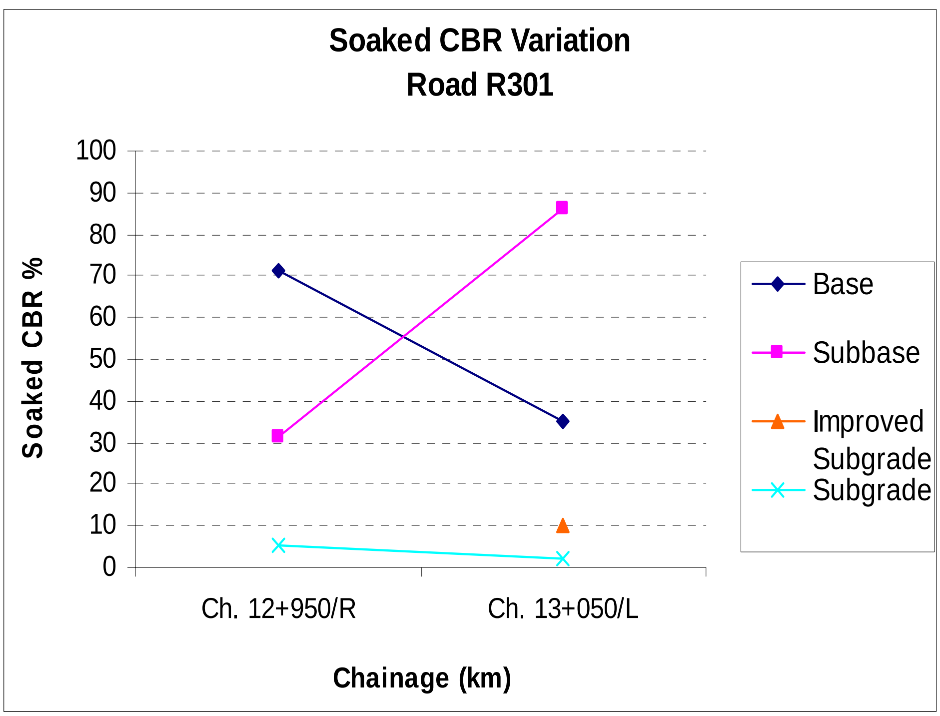


Figure 4.19: Variation in pavement layer strength (soaked CBR) in Road R301

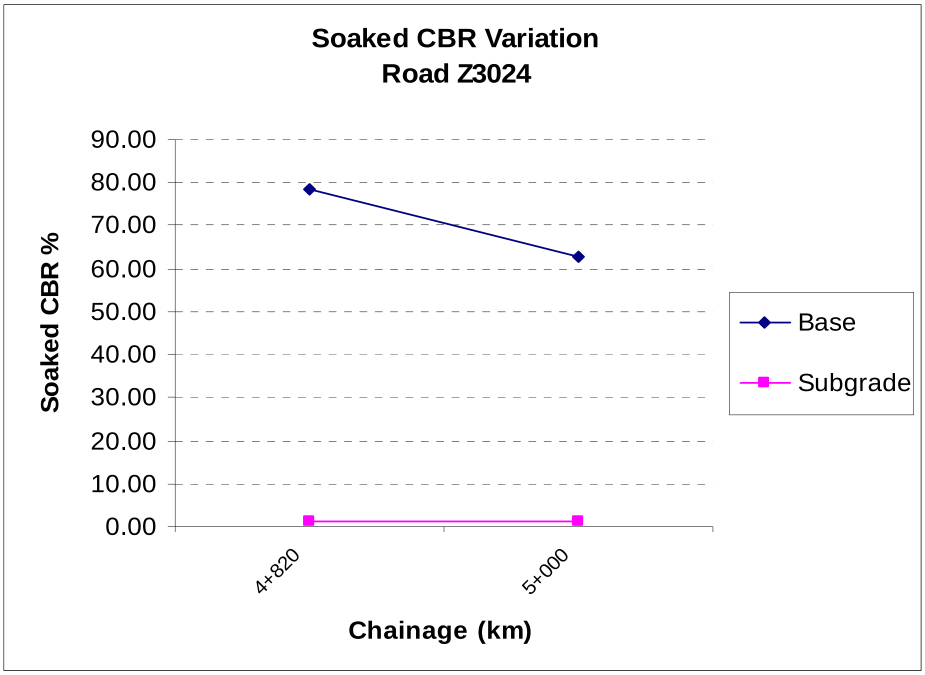


Figure 4.20: Variation in pavement layer strength (soaked CBR) in Road Z3024

Table 4.9: Summary statistics of pavement layer strength in terms of Soaked CBR

Road	Layer	Mean	Median	Standard Deviation	COV	Variance	Range
N4	Base 1	115.75	115.75	4.60	3.97	21.13	112.5-119
	Base 11	78.75	78.75	70.36	89.34	4950.13	29-128.5
	Sub-base	41.75	41.75	5.30	12.7	28.13	38-45.5
	Imp. sbgd	14.52	14.52	0.09	0.63	0.01	14.45-14.58
	Subgrade	3.01	3.01	0.69	23.02	0.48	2.52-3.5
N302	Base	77.35	79.9	8.52	11.01	72.54	66.3-85
	Sub-base	34.45	30.4	10.03	29.12	100.7	26.6-53.2
	Imp. sbgd	8.79	8.75	2.81	31.99	7.91	4.5-12.5
	Subgrade	2.77	2.68	0.29	10.60	0.086	2.5-3.28
R301	Base	53.16	53.16	25.72	48.38	661.28	34.97-71.34
	Sub-base	58.68	58.68	38.79	66.11	1504.92	31.25-86.11
	Imp. sbgd	9.85	9.85	0	0	0	9.85-9.85
	Subgrade	3.79	3.79	2.04	53.71	4.14	2.35-5.23
Z3024	Base	70.42	70.42	11.12	15.79	123.61	62.56-78.28
	Subgrade	1.34	1.34	0.11	8.19	0.01	1.26-1.42

4.3.5 Pavement Roughness data variation

Pavement roughness data were collected as part of this work. Figures 4.21-24 show the variation in pavement roughness data in roads N4, N302, R301 and Z3024 in turn. A summary of the statistical analysis conducted is presented in Table 4.10. The variability in roughness data is found significant in roads N4, N302 and Z3024 with COV values of 25%, 29.73% and 32.74% respectively. Pavement roughness data were also collected

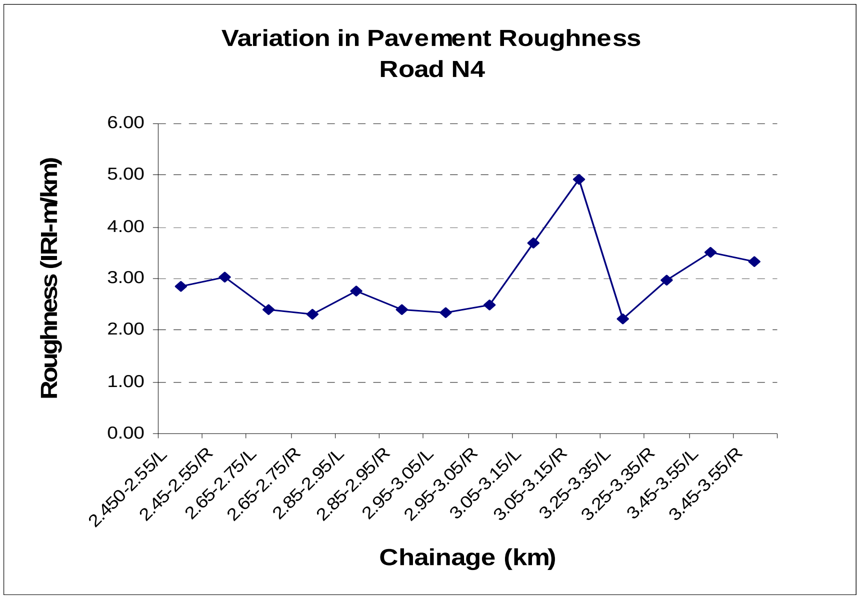


Figure 4.21: Variation in roughness (IRI) data in Road N4

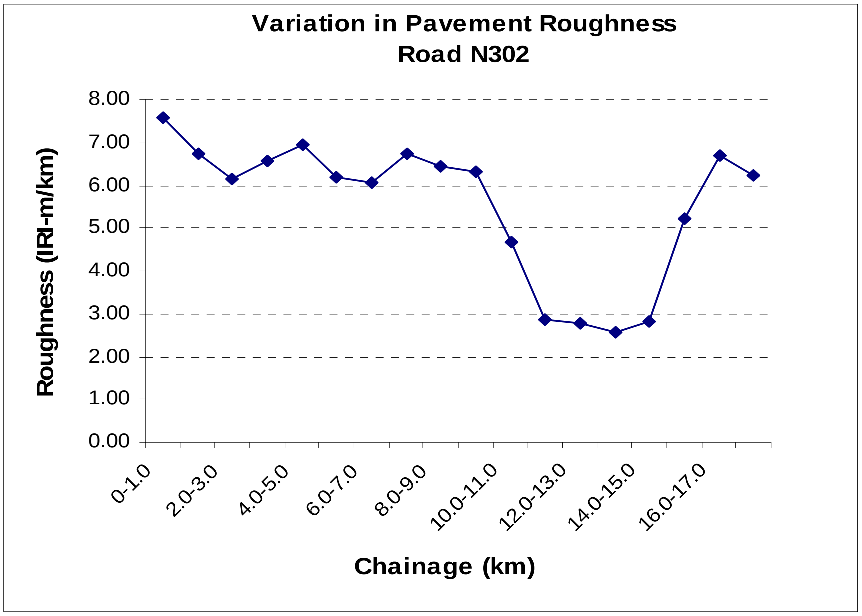


Figure 4.22: Variation in roughness (IRI) data in Road N302

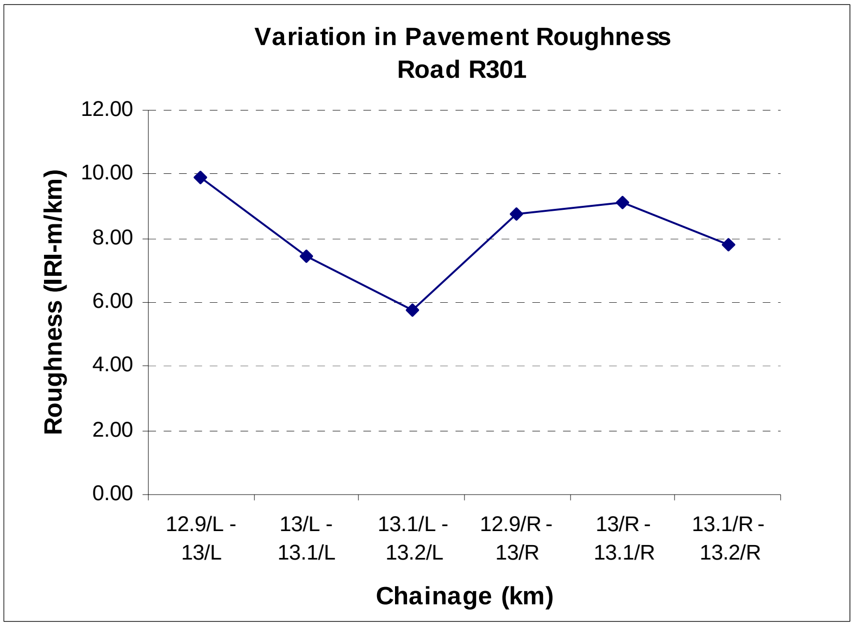


Figure 4.23: Variation in roughness (IRI) data in Road R301

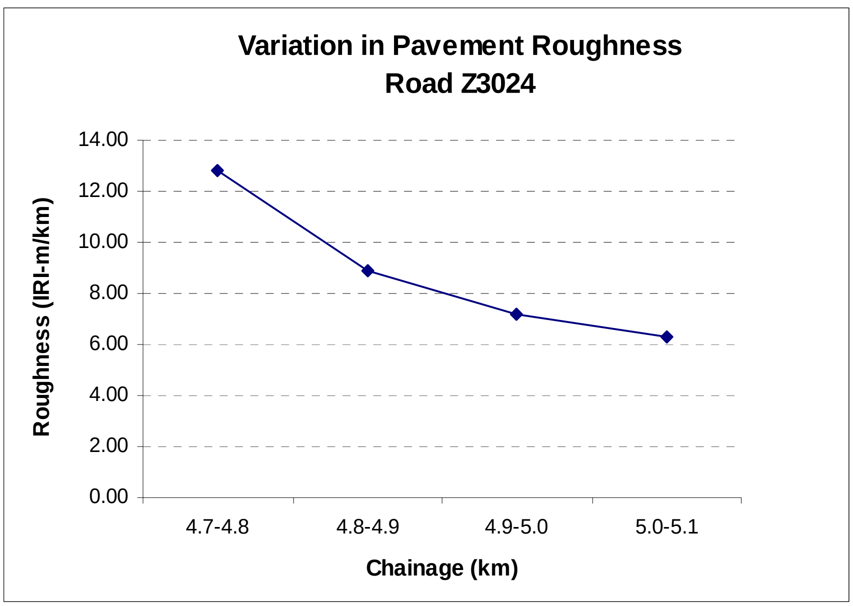


Figure 4.24: Variation in roughness (IRI) data in Road Z3024

Table 4.10: Summary of statistical analysis of pavement roughness (IRI) data

Road	Mean	Median	Standard Deviation	COV	Variance	Mini-mum	Maxi-mum	Range
N4	2.94	2.79	0.74	25.0	0.54	2.22	4.91	2.22-4.91
N302	5.53	6.195	1.64	29.73	2.71	2.56	7.58	2.56-7.58
R301	8.13	8.30	1.47	18.13	2.17	5.74	9.92	5.74-9.92
Z3024	8.78	8.02	2.87	32.74	8.26	6.30	12.78	6.30-12.78

from the HDM database and were also analyzed. The statistical parameters of these data are presented in Table 4.11. The HDM roughness data also show substantial variation with a co-efficient of variation of more than 50% in road N4 and around 30% in roads N302 and R301.

Table 4.11: Statistical analysis summary of HDM database roughness (IRI) data

Road	Mean	Median	Standard Deviation	COV	Variance	Mini-mum	Maxi-mum	Range
N4	5.01	5.00	2.59	51.64	6.69	2.00	13.00	2.00-13.00
N302	5.49	6.00	1.68	30.69	2.84	3.00	8.00	3.00- 8.00
R301	8.06	8.00	2.40	29.76	5.75	2.00	12.00	2.00-12.00
Z3024	11.53	12.00	1.36	11.80	1.85	9.00	14.00	9.00-14.00

4.4 Embankment Data Quality Analysis

Data related to the embankment design were collected from the field and laboratory testing of soil samples which were taken from road N302. These tests were performed by Shahjahan [2009] in another M. Phil research module of the project and the data were collected from his research work. The collected data are analyzed statistically to find their variability. The data consist of index properties, the shear strength parameters and the consolidation parameters. The variability of these data is analyzed and presented below.

4.4.1 Index Properties

The index properties of soil are its moisture content, liquid limit, plastic limit, plasticity index, unit weight, specific gravity and void ratio. These data are analyzed statistically to find their variability and hence their quality. Figures 4.25-4.32 show the spatial variation of the soil index properties. A summary of the statistical analysis of the variability of the soil index properties is presented in Table 4.12. The different index properties show different degrees of variability. Significantly high variability is found in the moisture content with a co-efficient of variation of 84.4%. Similarly the initial void ratio shows high variability (COV=76.82%). Sufficient variability is also found in the dry unit weight, the plastic limit and the plasticity index with COV of 39.67%, 27.27% and 22.5% respectively. The liquid limit and the unit weight data show reasonable variability with a COV of below 20%. Minor variability is found in the specific gravity data with a COV value around 6%.

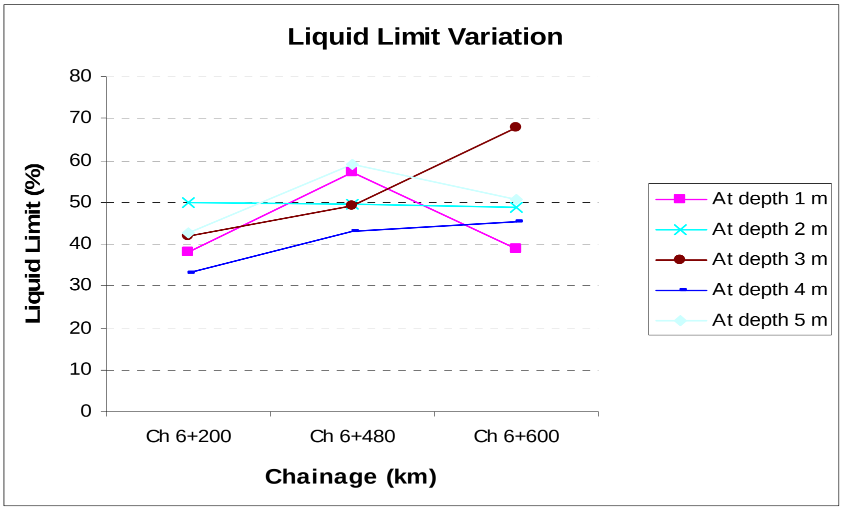


Figure 4.25: Variation in liquid limit at different depths

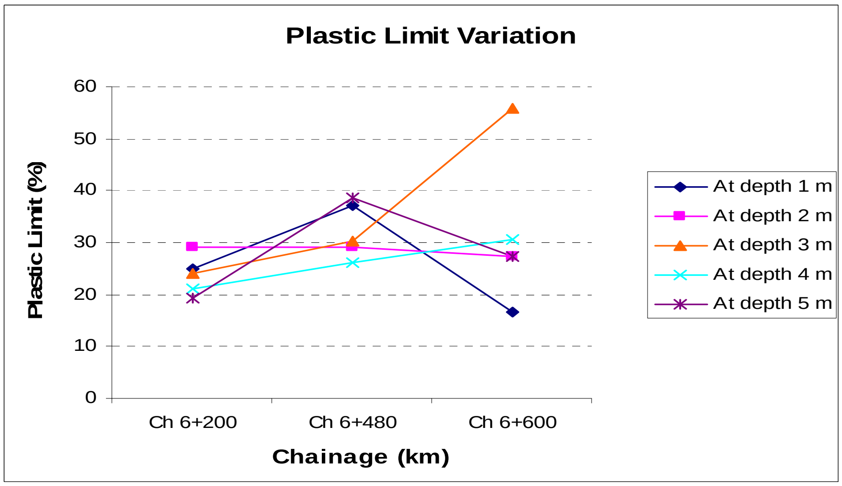


Figure 4.26: Variation in plastic limit at different depths

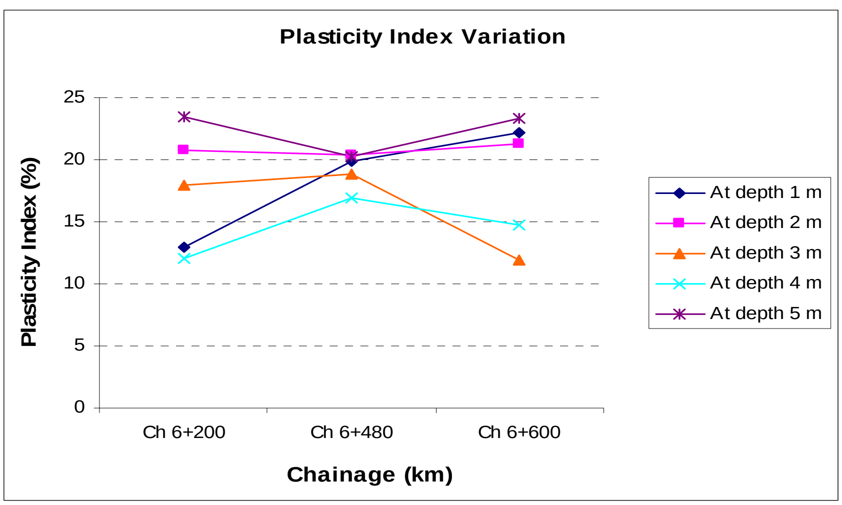


Figure 4.27: Variation in plasticity index at different depths

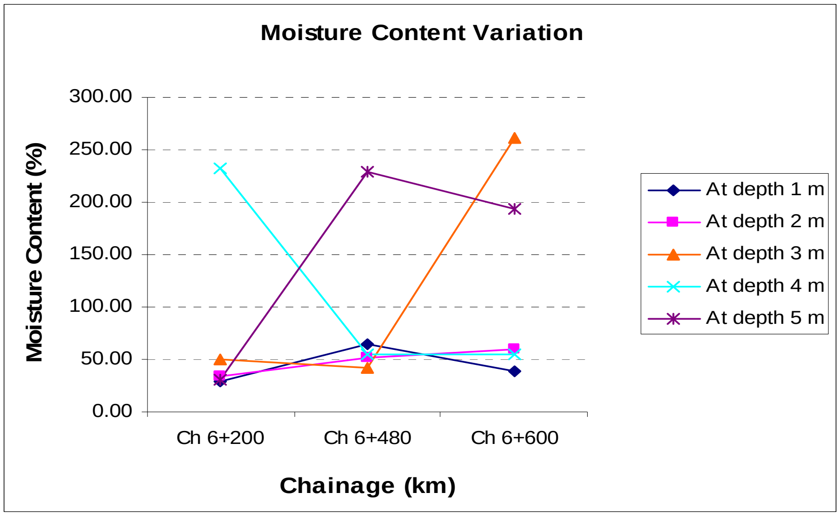


Figure 4.28: Variation in moisture content at different depths

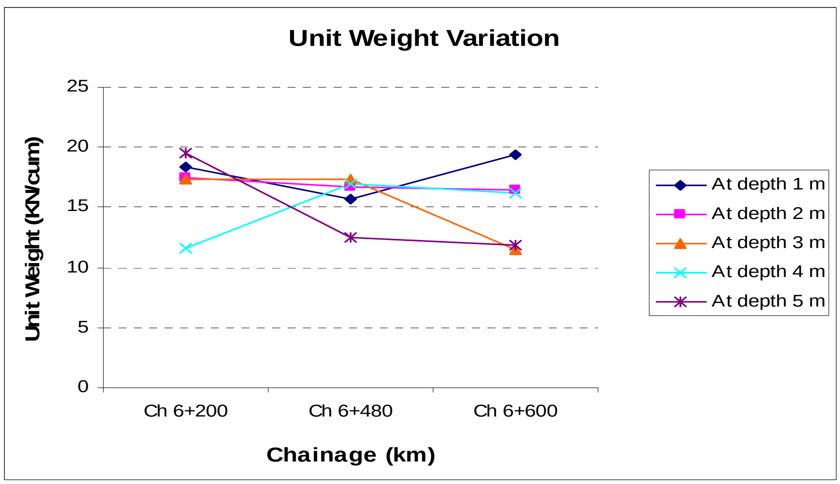


Figure 4.29: Variation in unit weight at different depths

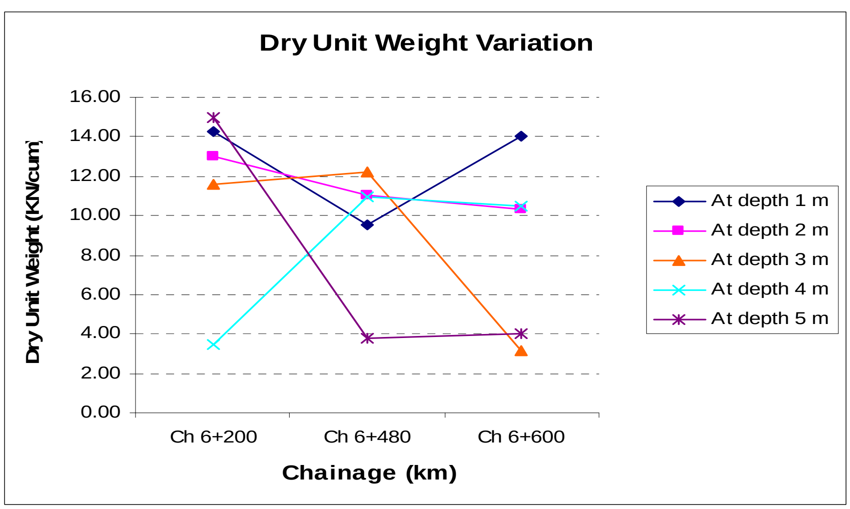


Figure 4.30: Variation in dry unit weight at different depths

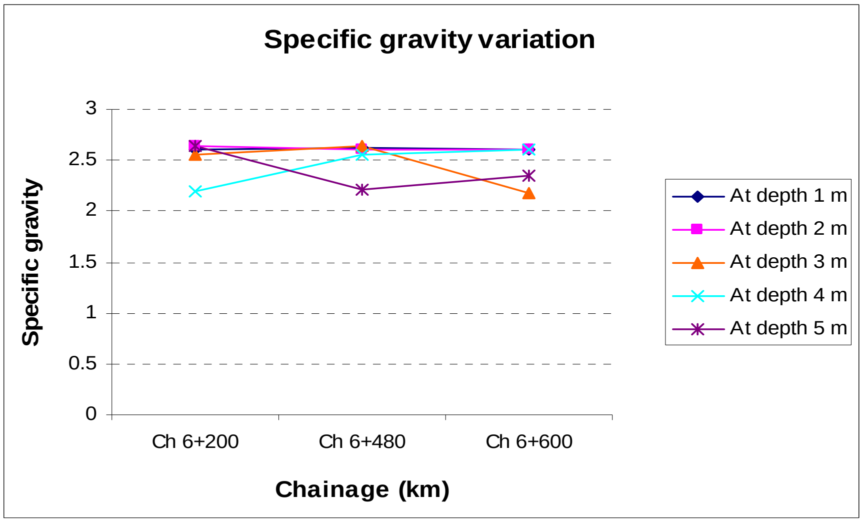


Figure 4.31: Variation in specific gravity at different depths

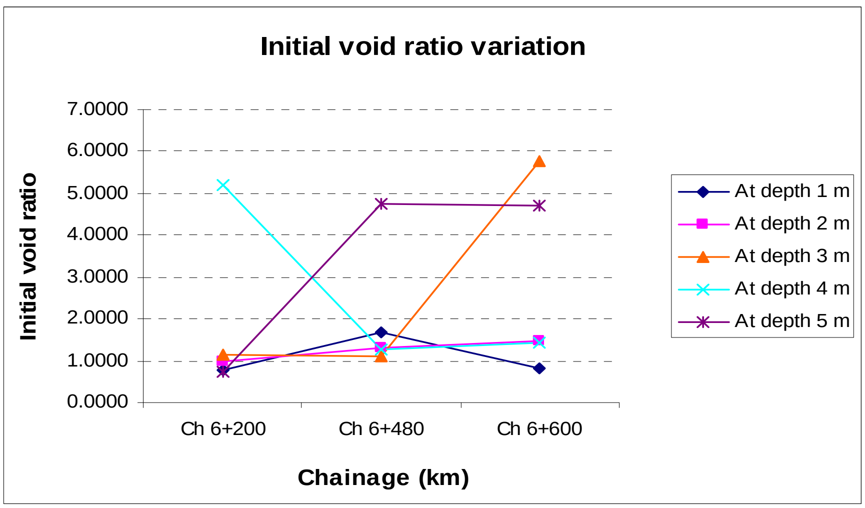


Figure 4.32: Variation in initial void ratio at different depths

Table 4.12: Statistical parameter of soil index properties

Soil index Properties	Units	Mean	Median	Standard Deviation	COV %	Variance	Range
Liquid limit	%	47.66	47.8	8.51	17.86	87.16	29.1-67.8
Plastic limit	%	28.91	27.97	7.88	27.27	121.55	16.76-55.8
Plasticity index	%	18.75	20.37	4.22	22.50	22.43	5.43-24.99
Moisture content	%	83.47	50.89	70.46	84.40	6843	26.5-260.9
Unit weight	kN/m^3	15.94	16.3	2.71	17.01	7.31	11.4-19.97
Dry unit weight	kN/m^3	10.06	10.97	3.99	39.67	16.00	3.16-15.57
Specific gravity		2.50	2.58	0.16	6.49	0.03	2.18-2.7
Initial void ratio		2.00	1.29	1.53	76.82	3.27	0.63-5.76

4.4.2 Shear strength data

The shear strength data, as determined from direct shear test and triaxial tests on soil samples collected from road embankments in various parts of N302 are analyzed statistically for their variability. Figure 4.33 shows the variation in drained and undrained soil cohesion and Figure 4.34 shows the variation in the drained and undrained soil angle

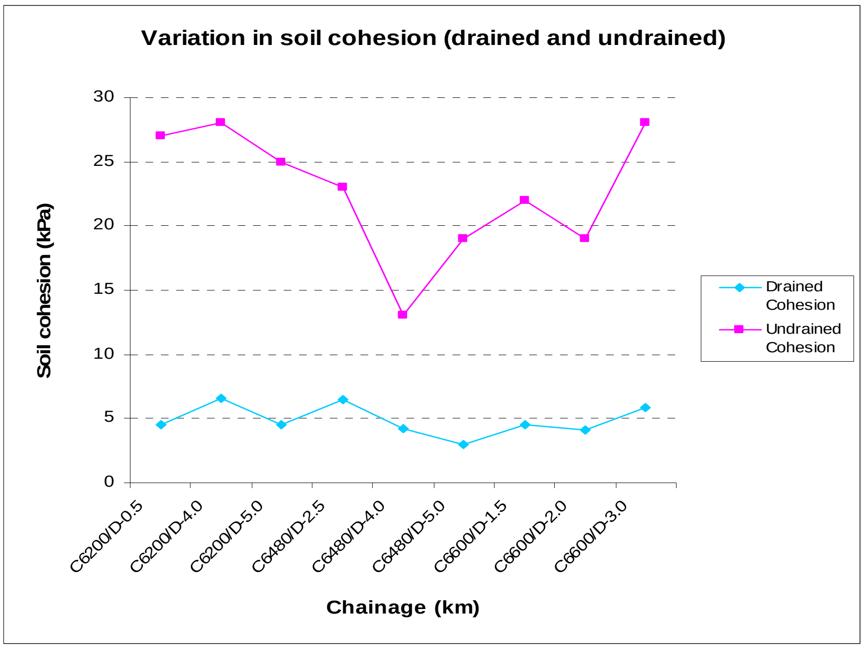


Figure 4.33: Variation in soil cohesion

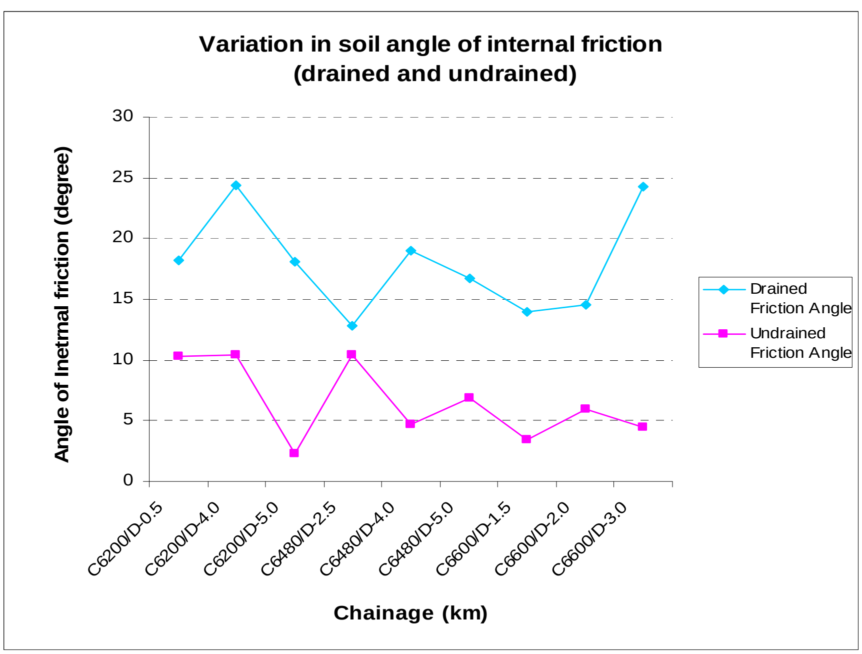


Figure 4.34: Variation in soil angle of internal friction

of internal friction. The statistical parameters of the shear strength data analysis are presented in Table 4.13. Significant variability is found in all the shear strength data. The undrained angle of internal friction exhibits the greatest variability among them, with a co-efficient of variation of 48.36%. The variability in drained and undrained cohesion data is also found to be considerable, with COV values of 24.78% and 22.17% respectively. Similar variability is found in the data on the drained angle of internal friction (COV=23.03%).

Table 4.13: Statistical analysis summary of shear strength data

Shear strength Parameter	Mean	Median	Standard Deviation	COV	Variance	Range
Drained cohesion (kPa)	4.86	4.5	1.20	24.78	1.45	2.96-6.53
Undrained cohesion (kPa)	22.67	23	5.02	22.17	25.25	13.0-28.0
Drained angle of internal friction (degree)	18.02	18.14	4.15	23.03	17.22	12.85-24.4
Undrained angle of internal friction (degree)	6.53	5.9	3.16	48.36	9.98	2.3-10.4

4.4.3 Consolidation data

The consolidation parameters obtained from consolidation tests of soil samples collected from different horizontal and vertical locations are analyzed statistically to find variability. The variation in undrained modulus as obtained from triaxial tests and the variation in the void ratio of the remoulded soil sample is shown in Figures 4.35 and 4.36 respectively. Figures 4.37 and 4.38 show the variation in data on dry density and saturated density. The variation in data on the compression index and secondary compression index is shown in Figures 4.39 and 4.40 respectively. The statistical analysis summary of these data is presented in Table 4.14. The data show high dispersion within their range. Secondary compression index data are found to be highly variable with a COV of more than 100%. Data on soil dry density also exhibit significant variability. The COV of soil dry density varies from 30% to 60% at different depths. Reasonable variability is found in the data on the compression index, swell index, modulus, saturated density and initial void ratio where COV ranges between 7% and 33%. The range of consolidation data as found in different layers runs from 0.83-1.90 for initial void ratio, 3.49-15.57 kN/m^3 for soil dry density, 11.61-19.97 kN/m^3 for soil saturated density, 0.27-0.391 for the compression index, 0.034-0.078 for the swell index, 0.002-0.061 for the secondary compression index and 1880-3600 kN/m^2 for the undrained modulus. The value of soil dry density is found low in some cases, this is due to existence of black cotton soil in those points. It is clear from the above ranges that the consolidation data is extremely variable.

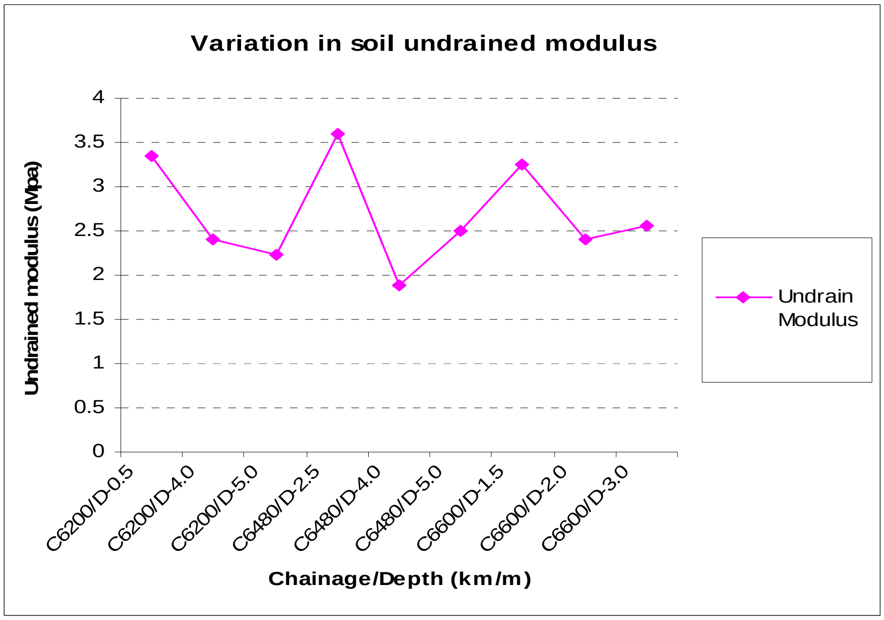


Figure 4.35: Variation in soil undrained modulus

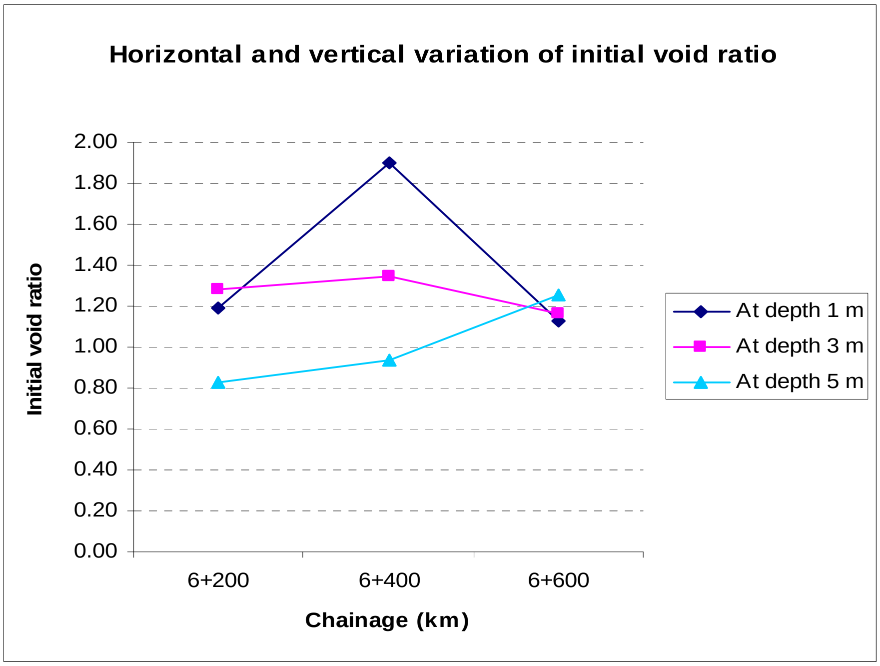


Figure 4.36: Variation in initial void ratio (remoulded soil)

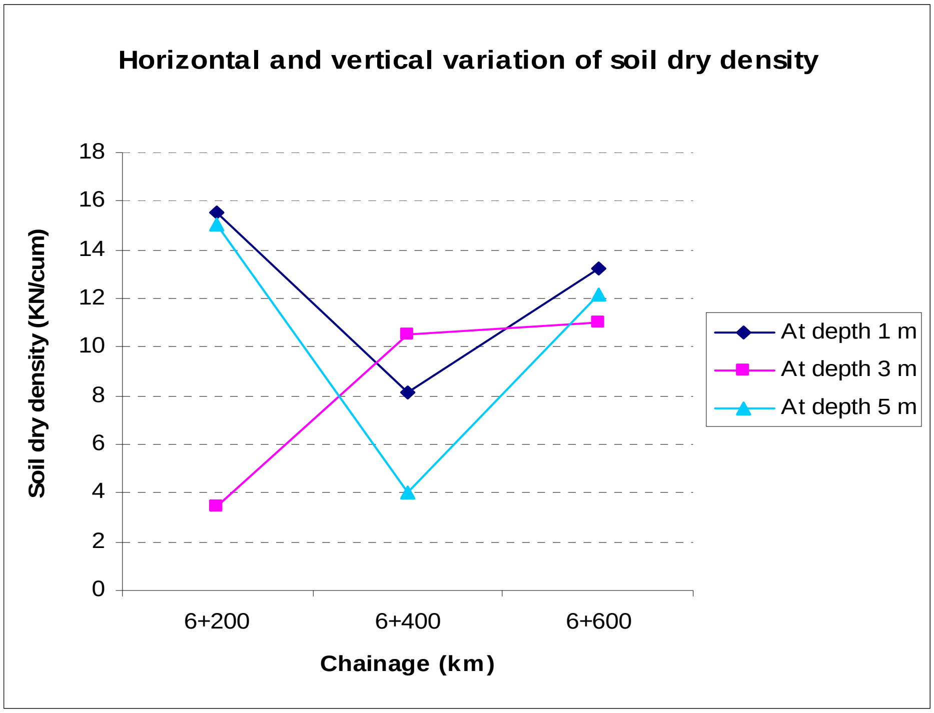


Figure 4.37: Variation in soil dry density

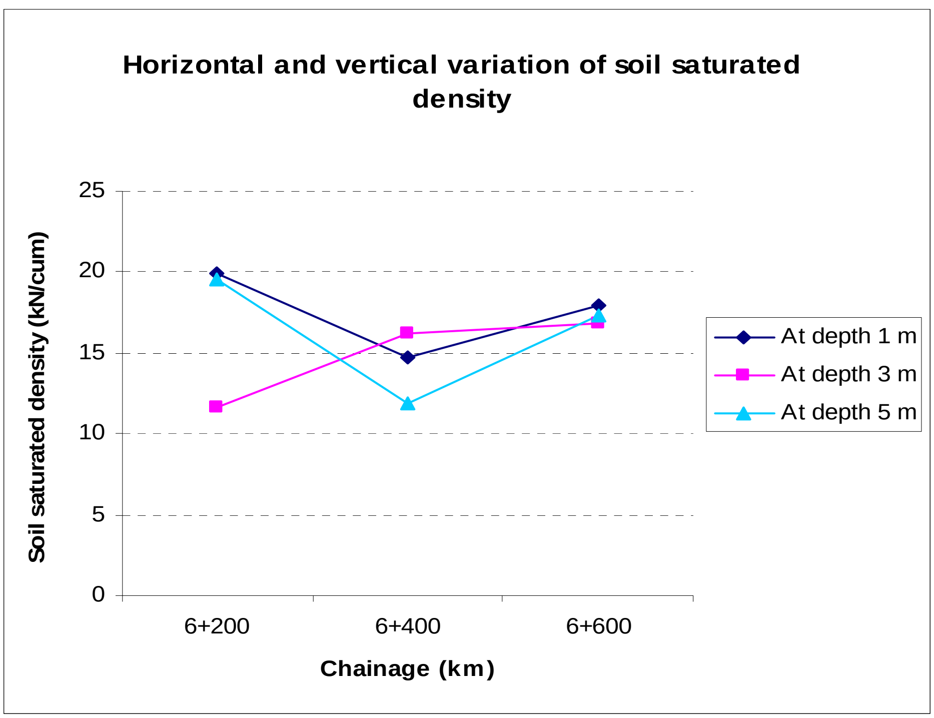


Figure 4.38: Variation in soil saturated density

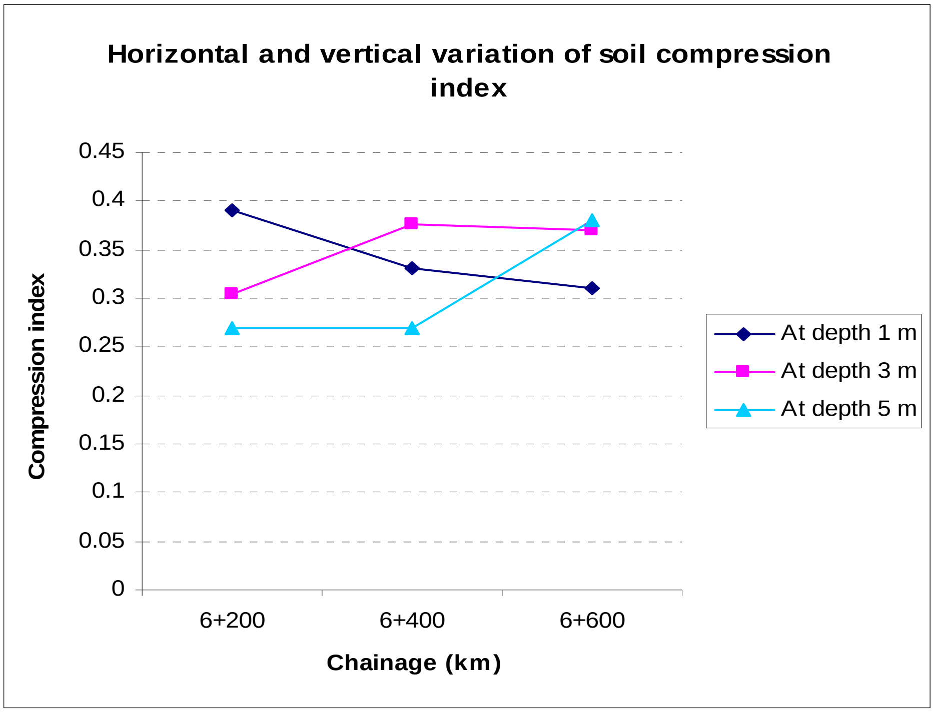


Figure 4.39: Variation in soil compression index

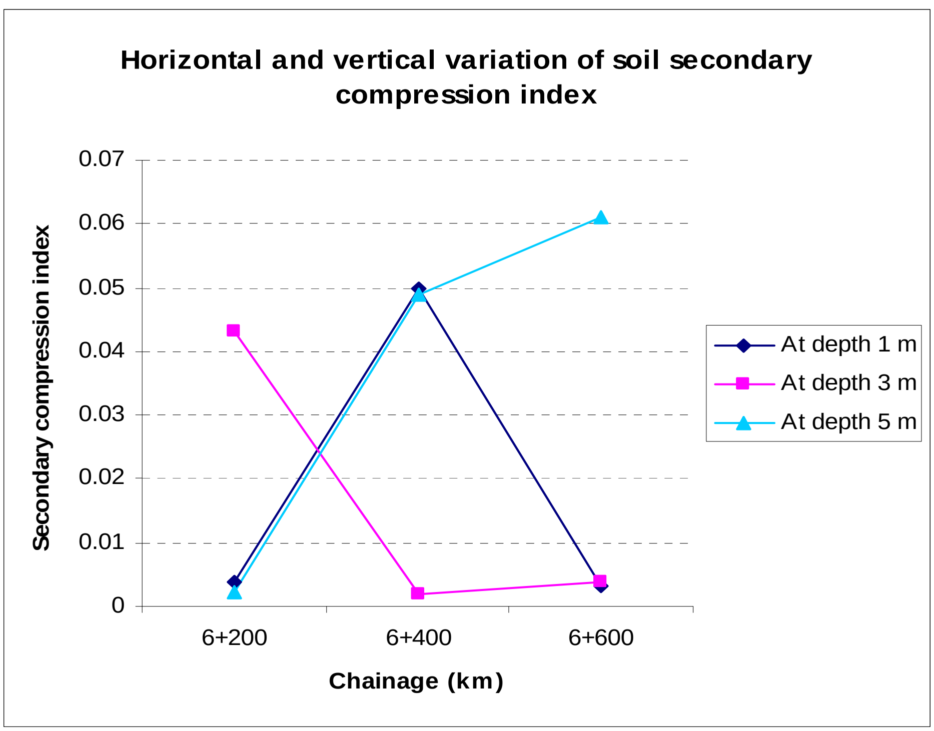


Figure 4.40: Variation in soil secondary compression index

Table 4.14: Statistical analysis summary of soil consolidation parameters

Parameter	Layer No.	Mean	Median	Standard Deviation	COV %	Variance	Range
Initial void ratio (remoulded sample)	1	1.407	1.19	0.428	30.45	0.183	1.13-1.90
	2	1.266	1.28	0.096	7.61	0.0092	1.16-1.35
	3	1.008	0.94	0.221	21.88	0.049	0.83-1.26
Soil dry density (kN/m^3)	1	12.303	13.23	3.815	31.01	14.557	8.11-15.57
	2	8.337	10.48	4.207	50.46	17.70	3.49-11.04
	3	10.41	12.19	5.702	54.78	32.516	4.03-15.01
Soil saturated dry density (kN/m^3)	1	17.53	17.92	2.657	15.15	7.057	14.7-19.97
	2	14.85	16.16	2.82	18.99	7.95	11.61-16.77
	3	16.25	17.36	3.987	24.53	15.89	11.83-19.57
Compression index	1	0.344	0.330	0.0418	12.15	0.0017	0.311-0.391
	2	0.35	0.37	0.04	11.41	0.0016	0.304-0.376
	3	0.307	0.27	0.0635	20.71	0.004	0.27-0.38
Swell index	1	0.059	0.0614	0.0156	26.22	0.0002	0.043-0.074
	2	0.071	0.076	0.0104	14.70	0.0001	0.059-0.078
	3	0.042	0.034	0.0134	32.21	0.0002	0.034-0.057
Secondary compression index	1	0.0189	0.0037	0.0269	142.11	0.0007	0.0031-0.05
	2	0.0163	0.0038	0.0232	142.43	0.0005	0.002-0.043
	3	0.0374	0.049	0.0311	83.07	0.001	0.002-0.061
Modulus (kN/m^2)		2684.4	2500	575.16	21.43	330803	1880-3600

4.5 Summary

The quality and variability of input data for the design system of Bangladesh was presented in this chapter. First, the data collection process in the field and laboratory testing of selected roads of Bangladesh is briefly described. Then the variability of data is presented with graphs and the results of statistical analyses in summary tables. The results indicate that the variability of all types of data is considerable and therefore it is important that any design system incorporating these data should cater for their variability. The following chapters seek to quantify this variability.

Chapter 5 Pavement Design Risk and Reliability

5.1 Introduction

This chapter describes the complete development process of the method of quantifying the pavement design risk for Bangladesh. The available methods for the process of quantifying pavement design risk are assessed with regard to their suitability and applicability in the design system of Bangladesh. Then the logical development leading to the design of a suitable method for Bangladesh is presented. Finally a detailed description of the proposed method is provided.

5.2 Assessment of the available methods

A number of risk and reliability methods have so far been developed. An assessment of these follows.

Austroads [2004]: The Austroads [2004] method considered the differences in laboratory test conditions and in-service conditions as well as the difference in actual and predicted service life due to variability in construction, environment and traffic. However, the in-service conditions in Bangladesh are totally different from those in Australia, since the environment of Bangladesh is totally different. The traffic characteristics and the characteristics of the pavement construction in Bangladesh differ significantly from those in Australia. Hence, the elements developed by Austroads (see section 2.7.1) seem not to be applicable for the design system of Bangladesh.

AASHTO [1993]: The risk analysis technique of this guide is simple, but considers an empirical performance prediction model to estimate the variance in pavement performance. The proposed design method for Bangladesh is analytical and considers only those design data which apply to analytical pavement design. Consequently, this method is not entirely suitable for the design system of Bangladesh.

The methods of both Noureldin et al. [1994, 1996] and Huang [1993] are variants of the AASHTO method.

NCHRP [2004]: The method proposed in this M-E design guide incorporates long term pavement distress data which are not currently available in Bangladesh.

Kim [2006]: The technique proposed in this method considers a rut prediction model. However, pavement failure due to rutting is not predominant in Bangladesh.

TRRL [1975]: The method did not consider the risk associated with the variability in the factors affecting pavement design. Hence, the method is not suitable for developing a system which could analyse the variability in the pavement design data and associated risk.

Chua et al. [1992]: The proposed method is also not suitable for the design system of Bangladesh, since it involves computational difficulties to perform multi-fold integration for a high number of variables. The method requires a computer program to perform the analysis.

Alsherri and George [1988]: The suggested method uses the present serviceability index to formulate a reliability model using the AASHTO [1972, 1986] design model and the Monte Carlo simulation. But the design system of Bangladesh requires a tool which is simple and suitable for analytical pavement design.

Kulkarni [1994]: The method is based on historical pavement performance data which are not available in Bangladesh and no arrangement has been made yet for the future collection of long term pavement performance data. Thus this method is not suitable for Bangladesh.

Zhang [2006]: The method proposed by Zhang [2006] requires time series data which are not available in Bangladesh. Hence, the method cannot be applied in the design system of Bangladesh.

Lua et al. [1996]: This reliability analysis method is based on a finite element pavement response model. But the pavement design method which is proposed for Bangladesh considers the method of using the equivalent thickness design model for the structural analysis of pavement. As a result, this reliability analysis method is not suitable for the pavement design system of Bangladesh.

Brown [1994]: The uncertainty estimation technique of this method, performing a sensitivity analysis, is simple and suitable for analytical pavement design system. A safety factor also was suggested to apply in a convenient manner. But the safety factor determination and application procedure is not well defined. Hence the procedure did not completely fulfil the task in hand.

The above points are summarised in Table 5.1.

Table 5.1: The summary of the suitability of the available methods

Available Methods	Accurately determine risk and reliability due to data variability	Suitable for use in the proposed analytical pavement design system	Suitable for the conditions in Bangladesh (available data and failure mode)	Consider all the variables in pavement design	Easy Computational procedure
Austroads [2004]	Yes	Yes	No	Yes	Yes
AASHTO [1993]	Yes	No	No	Yes	Yes
Noureldin et al. [1994, 1996]	Yes	No	No	Yes	Yes
Huang [1993]	Yes	No	No	Yes	Yes
NCHRP [2004]	Yes	Yes	No	Yes	Yes
Kim [2006]	Yes	Yes	No	Yes	No
TRRL [1975]	No	No	No	No	Yes
Chua et al. [1992]	Yes	Yes	No	Yes	No
Alsherri and George [1988]	Yes	No	No	Yes	Yes
Kulkarni [1994]	Yes	Yes	No	Yes	Yes
Zhang [2006]	Yes	Yes	No	Yes	No
Lua et al. [1996]	Yes	No	Yes	Yes	No
Brown [1994]	No	Yes	Yes	Yes	Yes

5.3 Proposed Approach

5.3.1 Development of the proposed method

Comparing the available methods with respect to the criteria as defined in the methodology chapter shows that no method can be found completely suitable for the design system of Bangladesh. Therefore this research develops a method of quantifying pavement design data variability and determining reliability for such a system. The reliability determination concept of this method is similar to the procedure of AASHTO [1993], but a different approach is developed to compute the variance in pavement performance since the empirical pavement performance equation which is used to construct the variance of pavement performance in AASHTO's method [1993] is not suitable for analytical pavement design. The analytical pavement design system being proposed for Bangladesh predicts pavement performance using a pavement structural analysis model based on layered elastic theory in association with the method of equivalent thickness. Thus, this research develops a procedure to determine the variance of this model predicting pavement performance, due to the variability in the design data. The theory of the first order approximation approach is used in this task. To determine the variance in traffic prediction, this research uses the first order approximation approach which was also used by Noureldin et al. [1996] and Huang [1993]. Then the overall variance and reliability design factor are calculated using the equations proposed by AASHTO [1993] as discussed in section 2.7.2. A detailed description of this proposed method is presented in section 5.3.2. The proposed method is suitable for designing a pavement of the desired degree of reliability. However, it is sometimes necessary to determine the reliability of an existing pavement. To this end an alternative procedure is proposed in this research. In the alternative approach, the safety factor of the pavement structure is determined first as a ratio of pavement capacity (predicted performance) and demand (predicted traffic). The same pavement structural analysis model as discussed before is used to evaluate pavement capacity. Then a risk analysis of the safety factor for different traffic and pavement performance prediction parameters is conducted. The theory of mean first order second moment method, which was used by many researchers for analysing the embankment slope stability risk, is used in the developed alternative approach. The detailed steps of this alternative method are presented in section 5.3.3.

5.3.2 Detailed description of the proposed method of analysing pavement design risk

This risk analysis method is a variant of that in the AASHTO method. The steps of this method are briefly described below with a flowchart, presented as Figure 5.1.

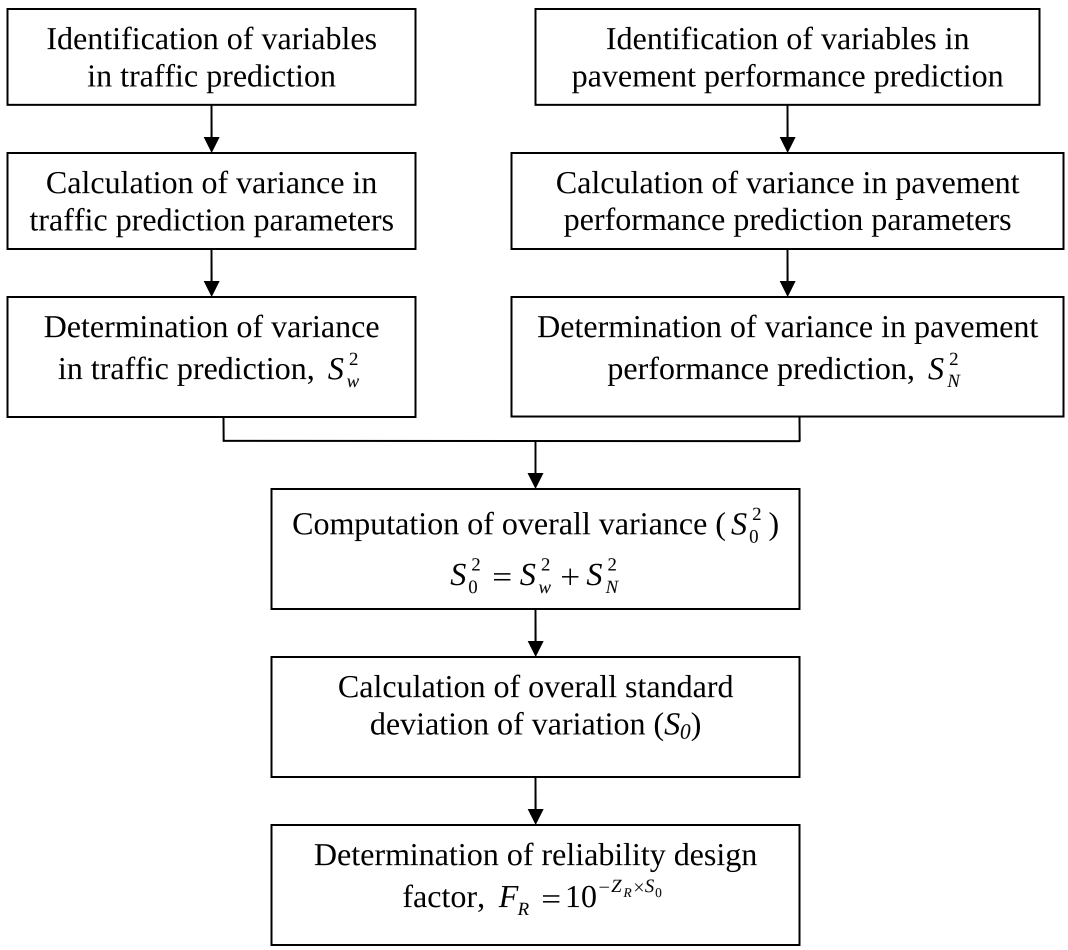


Figure 5.1: Risk analysis flowchart of the proposed first procedure

1. Identify variables in traffic prediction (w_T). The variables which may be considered are:

- i) Average daily traffic, ADT_0
- ii) Percent of trucks, T
- iii) Axles per truck, A
- iv) Growth factor, G

v) Growth rate, r

vi) Directional distribution, D

vii) Lane distribution, L

viii) Percent of different types of axle, p_i

ix) Equivalent axle load factor of different types of axle, F_i

x) Design life, Y

2. Identify variables involved in pavement performance prediction (W_t). The variables which may be considered are:

i) Thickness of surface layer

ii) Thickness of granular layer

iii) Strength of surface layer

iv) Strength of granular layer

v) Subgrade strength

3. Calculate mean, standard deviation, co-efficient of variation and variance of each variable.

4. Determine the variance in traffic prediction (S_w^2) with the following equation [Huang, 1993]:

$$V[\log w_T] = (\log e)^2 \left\{ \frac{V\left[\sum p_i F_i\right]}{\left(\sum p_i F_i\right)^2} + \frac{V[ADT_0]}{(ADT_0)^2} + \frac{V[G]}{G^2} + \frac{V[T]}{T^2} + \frac{V[A]}{A^2} + \frac{V[D]}{D^2} + \frac{V[L]}{L^2} \right\} \text{ -- (5.1)}$$

where V [log w_T] = variance of traffic prediction, V[x_i] = Variance of variable x_i and

$$V[G] = \left\{0.5Y(1+r)^{Y-1}\right\}^2 V[r]$$

5. Determine the variance in pavement performance prediction (S_N^2) with first order approximation approach, as follows (see section 2.6):

$$V[W_t] = \sum_{i=1}^{n} \left(\frac{\partial W_t}{\partial x_i} \right)^2 V[x_i]$$

Now, using the central divided partial differences as an approximation to partial derivatives [EI-Ramly, 2002] and considering systematic error, the equation for variance in pavement performance prediction is as follows:

$$V[W_t] = \sum_{i=1}^{n}\left(\frac{\Delta W_t}{\Delta x_i}\right)^2 V[x_i] + \sum_{i=1}^{n}\left(\frac{\Delta W_t}{\Delta x_i}\right)^2 V[x_i]_{systematic} \quad \text{-------------} \; (5.2)$$

where $V[W_t]$ = Variance in pavement performance prediction.

$V[x_i]$ = Variance of pavement performance prediction parameters x_i

$V[x_i]_{systematic}$ = Variance of variables x_i due to systematic error which could be estimated as $V[x_i]/n$ [Freund, 1979] where n = sample size.

ΔW_t = Change in pavement performance due to change of variable x_i

A sensitivity analysis is performed to evaluate the changes in pavement performance (ΔW_t) due to changes in each variable (Δx_i). A structural analysis model based on the method of equivalent thickness (MET) is used in the present research for this purpose.

6. Calculate the overall variance (S_0^2) as follows [AASHTO, 1993]:

$$\textit{Overall Variance } (S_0^2) = \textit{Variance in Traffic Prediction } (S_w^2) + \textit{Variance in Pavement Performance Prediction } (S_N^2) \; \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \; (5.3)$$

7. Determine the overall standard deviation of variation (S_0) as follows:

$$\textit{Overall Standard Deviation of Variation } (S_0) = \sqrt{\textit{Overall Variance}}$$

8. Calculate the reliability design factor for a desired reliability level as follows [AASHTO, 1993]:

$$F_R = 10^{-Z_R \times S_0} \quad \text{--------------------------------------} \; (5.4)$$

where the value Z_R can be found from standard normal curve area tables [Appendix E-4] for a given reliability level, R.

9. Incorporate reliability in design by multiplying the predicted traffic with the reliability design factor to estimate the design traffic

5.3.3 Proposed alternative approach of analysing pavement design risk (FOSM)

A flowchart of the proposed alternative method is presented in Figure 5.2 and the detailed steps of this method are described below:

1. Identify variables in traffic prediction (w_T) and pavement performance prediction (W_t). The variables which may be considered are described in section 5.3.2.

2. Calculate the mean, standard deviation, co-efficient of variation and variance of each variable. Also calculate the variance of mean due to systematic error (see sec. 5.3.2).

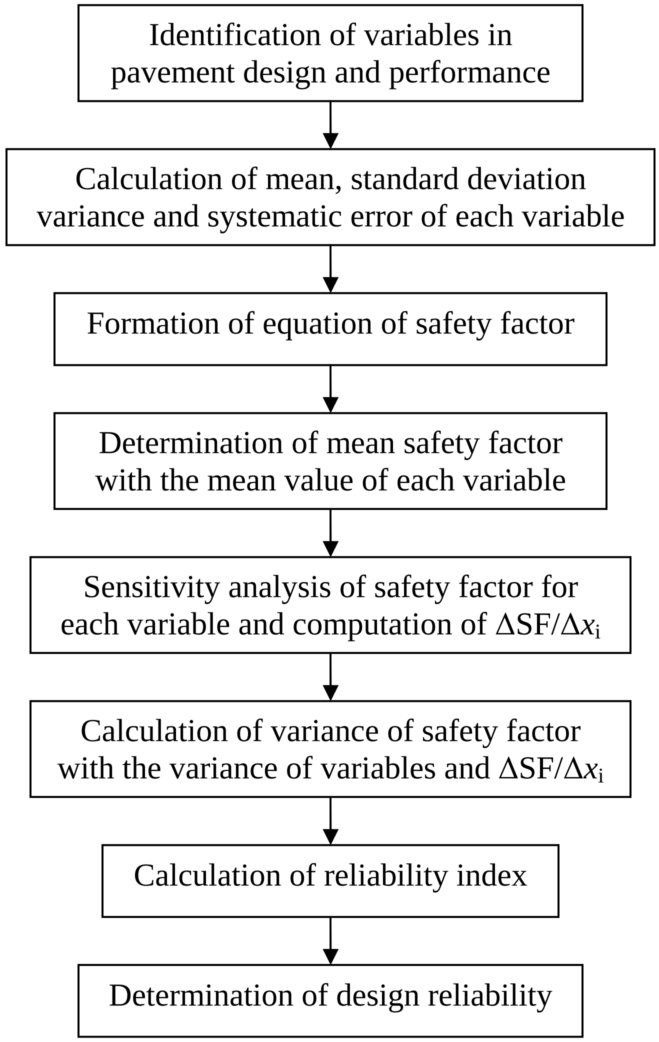


Figure 5.2: Flowchart of the proposed alternative risk analysis procedure (First Order Second Moment procedure)

3. Formulate the equation for the safety factor (SF). The safety factor represents how long pavement capacity is greater or less than demand and the equation is as follows:

$$Safety\quad Factor,\quad SF\quad = \frac{\Pr edicted\quad pavement\quad performance,\ W_t}{\Pr edicted\quad Traffic,\quad w_T} \text{ ------ (5.5)}$$

4. Determine the mean safety factor (E [SF]) with the mean value of traffic prediction and pavement performance prediction variables (x_i). A structural analysis model based on the method of equivalent thickness (MET) is used in this research to predict pavement performance.

5. Perform a sensitivity analysis to evaluate the changes in the safety factor (ΔSF) due to changes in each variable (Δx_i).

6. Determine the variance of the safety factor ($V[SF]$) with the first order approximation approach (see section 2.6) as follows:

$$V[SF] = \sum_{i=1}^{n} \left(\frac{\partial SF}{\partial x_i} \right)^2 V[x_i]$$

Now, using divided differences as an approximation to partial derivative [EI-Ramly, 2002] and considering systematic error, the equation for variance of safety factor takes the following form:

$$V[SF] = \sum_{i=1}^{n} \left(\frac{\Delta SF}{\Delta x_i} \right)^2 V[x_i]_{spatial} + \sum_{i=1}^{n} \left(\frac{\Delta SF}{\Delta x_i} \right)^2 V[x_i]_{systematicerror} \quad \text{-------- (5.6)}$$

where $V[x_i]_{spatial}$ = Variance of traffic and performance prediction variables x_i

$V[x_i]_{systematic}$ = Variance of variables x_i due to systematic error.

7. Calculate the standard deviation of safety factor ($\sigma\,[SF]$) as follows:

$$\sigma[SF] = \sqrt{Variance \quad of \quad safety \quad factor, \quad V[SF]}$$

8. Calculate the reliability index. The safety factor takes the value of 1 at limit state. Hence, the equation for reliability index could be written as;

$$\beta = \frac{E[SF] - 1.0}{\sigma[SF]} \quad \text{--------------------------------------- (5.7)}$$

9. Determine the reliability of pavement from the standard normal curve area Table for the corresponding value of the reliability index (Appendix E-1).

5.4 Summary

The chapter presented the complete development process of the proposed method of analyzing the pavement design risk for Bangladesh. First, a comparison of the existing method of analyzing pavement design risk with respect to the criteria as defined in the methodology chapter was briefly presented. Then the theory and concept used to develop the proposed risk analysis method for the pavement design of Bangladesh was discussed. Finally the procedures of two alternative proposed methods were presented in detail.

Chapter 6 Embankment Design Risk and Reliability

6.1 Introduction

A prime objective of this research project as described in Chapter 1 was to quantify the risk in embankment design due to the variability in data. To this end, the literature review chapter reviewed the embankment design procedures and design data, variability in design data, variability quantification and risk analysis methods. This chapter considers the development of a suitable procedure for quantifying the embankment design risk in the light of slope stability and settlement so as to incorporate the findings in the design system of Bangladesh. Accordingly, the chapter first briefly compares the existing slope stability risk analysis methods considering the criteria as mentioned in methodology chapter to judge their suitability for the design system of Bangladesh. Subsequently, the suggested method for Bangladesh is presented in detail. The later part of this chapter presents a similar comparison of the methods developed for analyzing settlement risk. Finally, the procedure which is proposed in this research for analyzing the embankment settlement risk in Bangladesh is discussed in detail.

6.2 Embankment slope stability risk analysis

6.2.1 Assessment of the available methods

A number of methods have been developed for analysing the stability risk of embankment slopes. An assessment of these follows.

First order second moment method: the first order second moment method requires fewer calculations and less computing time than other methods do. This method is widely used and well recognized. It is easy to implement and requires no computer program. It can consider the variability of all slope stability design parameters and can be applied using the data available in Bangladesh.

Point estimation method: the point estimation method is complex and hence not popular in practice.

Monte Carlo simulation method: the Monte Carlo simulation method has been proved powerful and offers a practical tool for more detailed analysis of slope stability when high speed computers are available. But more initial effort was required by this method to develop a spreadsheet for slope analysis than by the FOSM method combined with commercial slope stability analysis software [EI-Ramly et al., 2003]. In addition, it is not possible to program implicit function, for example, slope stability analysis with simulation software (such as the excel add-in @ RISK); this needs a special program. The accuracy of this method increases with the number of iterations but not proportionally. Since the method requires a special program for probabilistic slope stability analysis, it is not suitable for the embankment design system of Bangladesh.

Mean first order reliability method: This is a specified form of first order second moment method. Hence the limitations and advantages of first order second moment method apply to this method also. This method has the added advantage that it is simpler than the general first order reliability method.

Risk analysis algorithm with Fellenius limit equilibrium method: This method is a variant of the first order second moment method, expressing the performance function in a different way. However, the method is not widely used.

Deterministic approach using fuzzy sets: The uncertainty in this method is characterized by judgment, which is approximate and not necessary equal to the true degree of uncertainty. The method is also not well recognized or widely practised.

Random finite element method: Since the number of random variables is highly dependent on the finite-element mesh, the calculation of the failure probability or the reliability index can be complicated. Moreover, the embankment design system proposed for Bangladesh did not consider the finite element method for slope stability design. Hence, the random finite element method does not fulfill the requirements of a suitable method for Bangladesh.

Finite element method with first order reliability method: The method is based on the response surface method where a finite-element method is integrated with the first-order reliability method (FORM). Since the method is not compatible with the embankment design system of Bangladesh and requires a special program to implement it, the method is not suitable for the task in hand.

The above comparison is summarised in Table 6.1.

Table 6.1 Summary of suitability of the available methods for Bangladesh

Methods	Accuracy	Addresses all the variability of slope stability analysis	Suitable for the conditions of Bangladesh (available design and data)	Requires simple computational procedure
The first order second moment method	Yes	Yes	Yes	Yes
Point estimation method	Yes	Yes	Yes	No
Monte Carlo Simulation method	Yes	Yes	Yes	No
Mean first order reliability method	Yes	Yes	Yes	Yes
Risk analysis algorithm with Fellenius limit equilibrium method	Yes	Yes	Yes	No
Deterministic approach using fuzzy sets	No	Yes	Yes	No
Random finite element method	Yes	Yes	No	No
Finite element method with first order reliability method	Yes	Yes	No	No

6.2.2 Suggested Approach

From the comparison shown in Table 6.1, it appears that the first order second moment method is widely used, easily implementable, requires no computer program and performs its function reasonably accurately. In addition, the method can be implemented with the data available in Bangladesh. Hence this method is recommended for the design system of Bangladesh.

To perform risk analysis with the first order second moment method, a method of analysing slope stability is required. Analysis methods of two kinds, simplified methods and finite element methods, are commonly used to analyze slope stability. Although the finite element method has been increasingly used in recent years [Griffiths and lane, 1999; Zou et al. 1995; Griffiths and Fenton, 2004], the simplified methods which rely on simplifying the assumptions on the location of the slip surface, rigid body displacements, soil properties etc. have been widely practised and are widely recognized for slope stability analysis [Tang et al., 1976, Christian, 1994; EI-Ramly, 2002; Venmarcke, 1977; Li and Lumb, 1987; Barabosa et al., 1989]. Several simplified methods are available in the literature, each based on different assumptions. Among them, Spencer’s method [Spencer, 1967] is more accurate but rigorous. The simplified Bishop method [Bishop, 1955] is very simple and widely used although it satisfies only the overall moment and is applicable to a circular slip surface. The investigation carried out by Malkawi et al. [2000] showed that the discrepancies between the results of other methods and those of the Spencer method range from 0.04 to 0.1. Since the Bishop method is simple, well recognized and has associated errors which are not too large, the present research uses this method for probabilistic risk assessment.

The steps of the suggested risk analysis method are briefly described below and a flowchart is presented as Figure 6.1.

1. Identify the variables which affect the stability of the embankment slope. The variables which may be considered are:

i) Cohesion, c'

ii) Angle of internal friction, Φ'

iii) Unit weight, γ

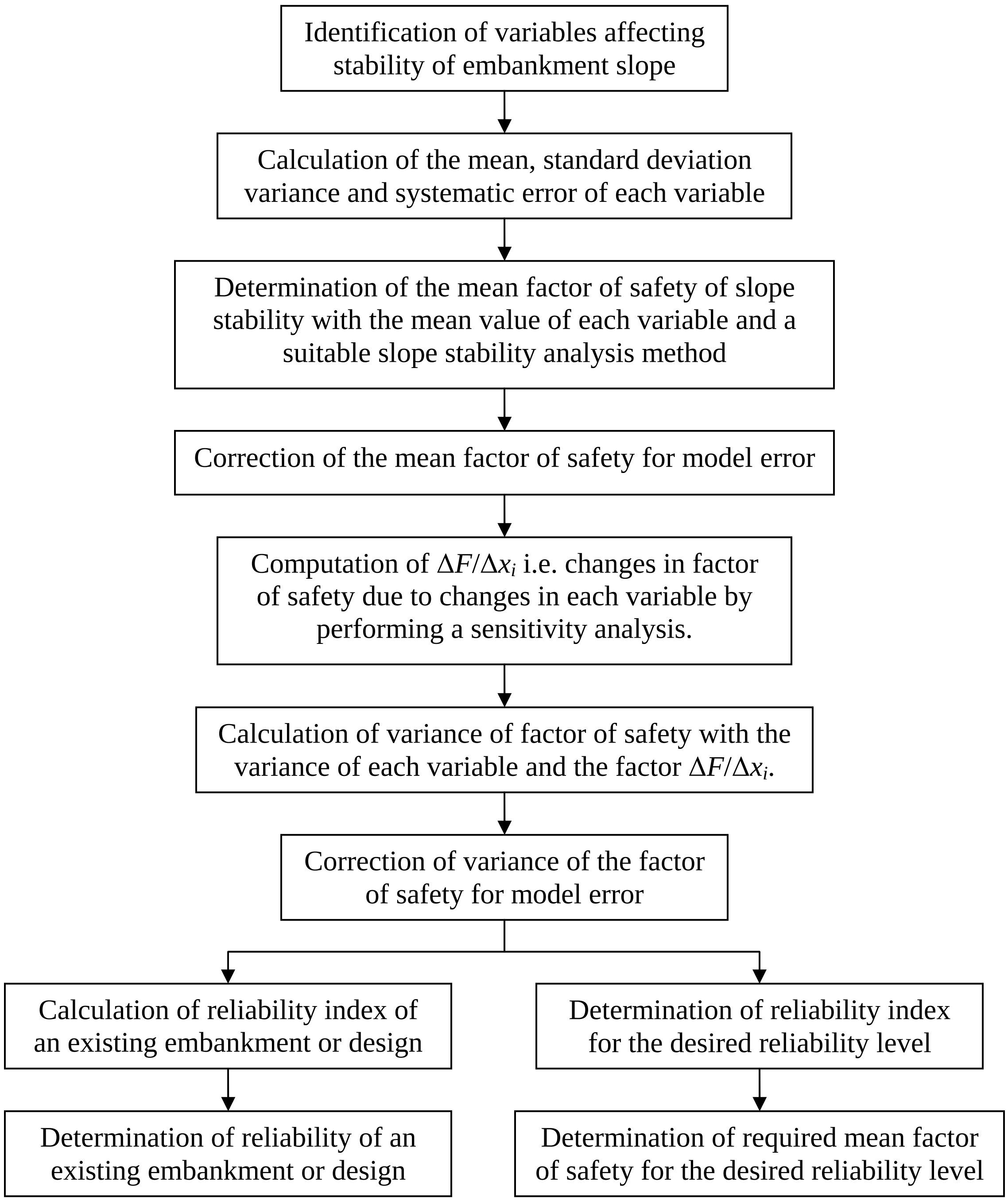


Figure 6.1: Flowchart for analyzing the stability risk of an embankment slope (suggested method).

A key factor in causing embankment failures is the change in pore water pressure likely to be associated with heavy rain which in turn causes sudden increase in the pore water pressure and a reduction in effective stresses within the embankment body. This factor should have been explicitly considered in the reliability analysis. However as such data was not available a simplified approach was considered and the reliability model considered the variability of the available data. Had the variability of pore water pressure been considered, the overall reliability would have been higher (see equation 6.1).

2. The next step is to calculate mean, standard deviation, co-efficient of variation and variance of each variable. Also calculate the variance of variables x_i due to systematic error (error due to the limited number of tests).

3. Then determine the mean factor of safety ($E[F]$) with the mean value of each variable and the following stability analysis equation [Bishop, 1955]:

$$F = \frac{1}{\sum W \sin\alpha} \sum \frac{\left[c'b + (W - ub)\tan\phi'\right]\sec\alpha}{1 + \dfrac{\tan\alpha \tan\phi'}{F}} \quad \text{-------------------- (6.1)}$$

where u represents pore water pressure, W weight of slice, b width of slice and α slice base inclination. The weight of slice can be determined as W = γbh; where γ is the unit weight of soil and h is the height of slice.

4. Correct mean factor of safety for model error [Azzouz et al., 1983]:

Corrected mean factor of safety = Mean factor of safety ×1.05

5. Perform sensitivity analysis to evaluate the changes in the factor of safety (ΔF) due to changes in each variable (Δ x_i).

6. Determine the variance of the factor of safety (V [F]) with the following first order approximation equation [Christian, 1994]:

$$V[F] \approx f\sum_{i=1}^{k}\left(\frac{\Delta F}{\Delta x_i}\right)^2 V[x_i]_{spatial} + \sum_{i=1}^{k}\left(\frac{\Delta F}{\Delta x_i}\right)^2 V[x_i]_{systematic} + V[e] \quad \text{--------- (6.2)}$$

where $V[x_i]_{\text{spatial}}$ = Spatial variance of slope stability variables x_i

$V[x_i]_{systematic}$ = Variance of slope stability variables x_i due to systematic error

f = Reduction factor due to averaging of spatial variability over the failure surface. The value of f is usually taken as 0.25.

V[e] = Variance in the factor of safety due to model error (V[e]) which could be estimated as V[e] = $(0.07\times\text{corrected E [F]})^2$ [Azzouz et al., 1983]

7. Calculate the standard deviation of the factor of safety (σ [F]):

$$\sigma[F] = \sqrt{V[F]}$$

8. Calculate the reliability index with the following equation (see section 2.8.1.1):

$$\beta = \frac{E[F] - 1.0}{\sigma[F]} \text{ --- (6.3)}$$

9. Determine the reliability of the embankment design from the standard normal curve area Table for the corresponding value of the reliability index (Appendix E-1).

<u>Incorporation of desired reliability in the design for slope stability</u>

1. To incorporate a desired reliability in a design for slope stability requires the design value of the mean factor of safety for slope stability first to be ascertained. To this end, the reliability index value corresponding to the desired reliability level is determined first from the standard normal curve area Table (Appendix E-1).

2. Then the design value of the mean factor of safety of slope stability for the selected reliability level is determined with the value of reliability index in the following way:

From equation 6.3, $\beta = \frac{E[F] - 1.0}{\sigma[F]}$ or $\beta = \frac{E[F] - 1.0}{E[F] * COV[F]}$

$$\text{or, } E[F] = \frac{1}{[1 - \beta * COV[F]]} \text{ --------------------------- (6.4)}$$

Here, the co-efficient of the variation of the factor of safety, COV [F] is determined by analyzing data collected from a similar existing structure using the procedure described in steps 1-7 above.

3. Hence, an embankment should be designed such that its mean factor of safety for slope stability is more than or equal to the value as determined above to obtain the desired reliability in design against stability.

6.3 Embankment settlement risk analysis

6.3.1 Assessment of the available methods

The existing settlement risk analysis methods, as discussed in section 2.8.2, are reviewed with respect to certain criteria (defined in the methodology chapter) with a view to selecting or developing a suitable method of analyzing embankment settlement risk for Bangladesh. A brief summary of this review is presented below.

Fenton and Griffiths [2002]: This risk determination procedure, which considers the probabilities of exceeding the limited settlement, is easily understandable and implementable for analyzing the embankment settlement risk for Bangladesh. Moreover, the mean and variance of variable are used in the calculation procedure, which is also computable with the prevailing data in Bangladesh. However, the method considers only the elastic modulus of soil as a variable parameter. Moreover, a computer program was used to determine the covariance between the local averages of the log-modulus. Hence, the method is not altogether suitable for Bangladesh. However, a suitable method for Bangladesh could be developed using the concept of risk determination procedure.

6.3.2 Proposed method

Following an assessment of available methods, it was felt that the first order second moment (FOSM) was the most appropriate for the following reasons:

A. First order second moment theory is simple, easy to understand, requires no computer program, considers all the variabilities of settlement and is suitable for conditions in Bangladesh, where all the necessary data are available.

B. In addition, the first order second moment method is recommended for analyzing the risk in embankment slope stability. Hence, this method is selected to maintain consistency in the methods of both slope stability and settlement risk analysis.

A flowchart of embankment settlement risk analysis with the proposed procedure is presented in Figure 6.2 and the detailed steps of this method are described below;

1. Identify the variables which affect embankment settlement. The variables which may be considered are:

   i) Modulus of elasticity, E
   ii) Applied pressure, p
   iii) Layer thickness, H
   iv) Initial void ratio, e_0
   v) Soil density, γ
   vi) Compression index, C_c
   vii) Secondary compression index, C_α.

2. Calculate the mean, standard deviation, co-efficient of variation and variance of each variable. Calculate also the systematic error (variance in mean due to the limited number of tests). The variance of mean of a parameter x due to systematic error could be estimated as $V[x]/n$, where n = sample size [Freund, 1979].

3. Determine the mean predicted settlement with the mean value of each variable and the settlement prediction equations. The immediate settlement (S_i) could be calculated in the following ways [Tomlinson, 2001; Foott and Ladd, 1981; Atkinson, 2007]:

$$S_i = pB\frac{1-\mu^2}{E} I_p \text{ ------------------------------------ } (6.5)$$

where p = net pressure applied; B = width of foundation; μ = Poisson's ratio; E = modulus of elasticity; and I_p = non dimensional influence factor (see Appendix F-2).

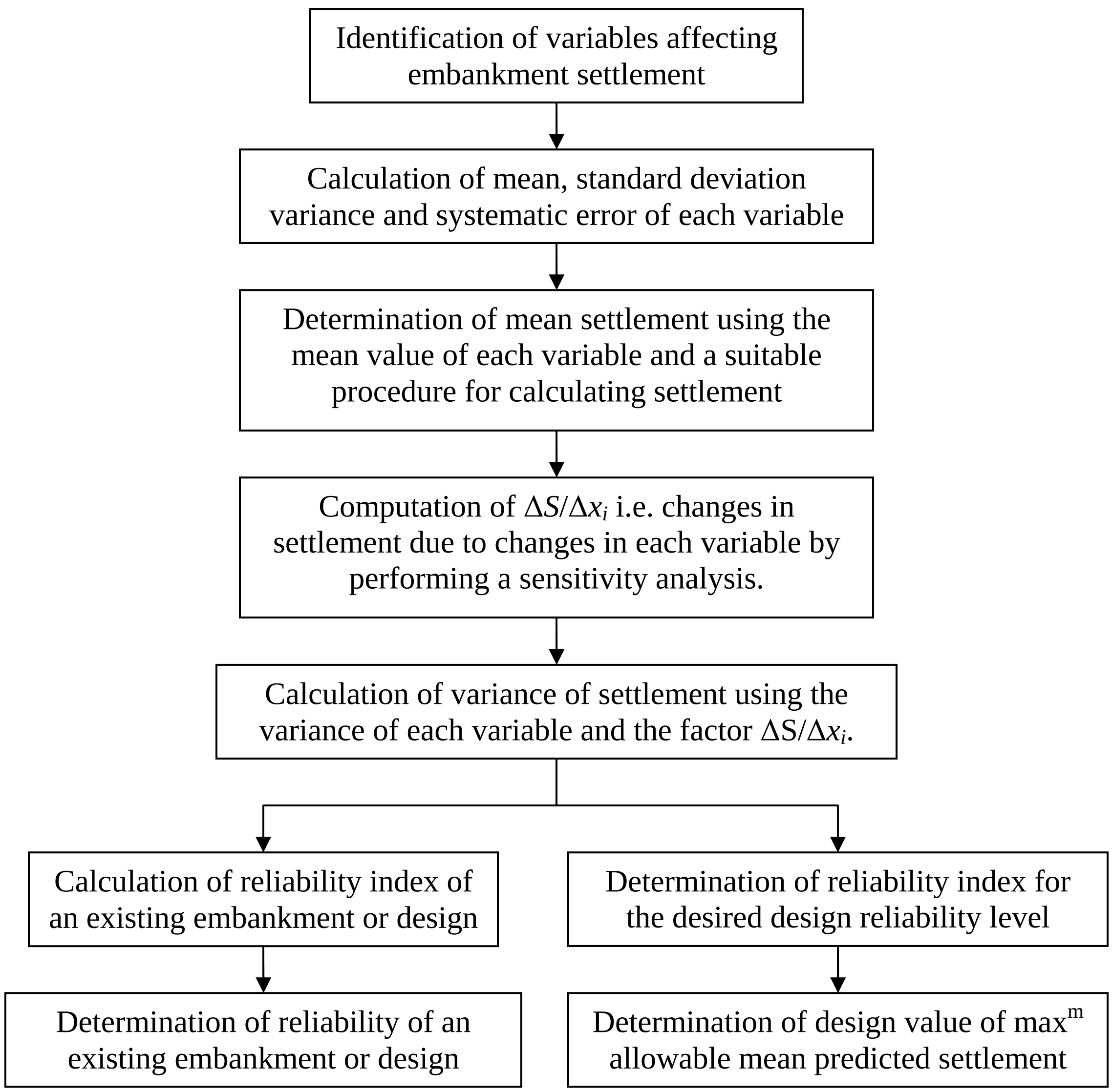


Figure 6.2: Flowchart of the proposed method of analyzing embankment settlement risk.

The primary settlement for normally consolidated soil can be determined in the following ways [Craig, 2004; Das, 1997; Tomlinson, 2001]:

$$S_p = \sum_{i=1}^{k} \frac{C_c H}{1+e_0}\left(\log \frac{p_0 + \Delta p}{p_0}\right) \text{ ----------------------------- (6.6)}$$

where k represents the number of layers, C_c compression index, H layer thickness, e_0 initial void ratio, p_0 pressure and Δp surcharge.

The equation which can be used to determine the secondary settlement in normally consolidated soil is as follows [Das, 1997]:

$$S_s = \sum_{i=1}^{k} \frac{C_\alpha H}{1+e_p}\left(\log \frac{t_2}{t_1}\right) \text{----------------------------------- (6.7)}$$

where C_α = secondary compression index; H = layer thickness; k = number of layers; e_p = void ratio at the end of primary consolidation, t_1 = time for end of primary consolidation, t_2 = design period.

Total mean settlement, E[S] = mean immediate settlement, E [S_i] + mean primary settlement, E [S_p] + mean secondary settlement, E [S_s]

4. Perform sensitivity analysis to evaluate the changes in predicted settlement (ΔS) due to changes in each variable (Δx_i).

5. Determine the variance of settlement ($V[S]$) using the first order approximation approach (see section 2.6). When the central divided partial differences is used as an approximation to partial derivatives [EI-Ramly, 2002] and systematic error is considered, then the equation for variance of settlement can be written as:

$$V[S] = \sum_{i=1}^{n}\left(\frac{\Delta S}{\Delta x_i}\right)^2 V[x_i]_{spatial} + \sum_{i=1}^{n}\left(\frac{\Delta S}{\Delta x_i}\right)^2 V[x_i]_{systematicerror} \text{ --------- (6.8)}$$

where $V[x_i]_{\text{spatial}}$ = Spatial variance of predicted settlement variables x_i

$V[x_i]_{systematic}$ = Variance of settlement variables x_i due to systematic error.

6. Calculate the standard deviation of settlement prediction (σ [S]):

$$\sigma[S] = \sqrt{V[S]}$$

7. Calculate the reliability index. In general, an embankment is considered to have failed in settlement when its settlement value exceeds 150 mm. Hence, the equation for reliability index can be written as (where $E[S]$ is in mm);

$$\beta = \frac{150 - E[S]}{\sigma[S]} \text{ --- (6. 9)}$$

8. Determine the reliability of the embankment from the standard normal curve area table for the corresponding value of the reliability index (see Appendix E-1).

<u>Incorporation of desired reliability in the design for settlement</u>

1. To incorporate a desired reliability in the design against settlement requires the limiting value of the mean predicted settlement to be determined first. To this end, the reliability index value corresponding to the desired reliability level is determined from the standard normal curve area Table (Appendix E-1).

2. Then the limiting value of the mean predicted settlement for a desired reliability level is determined with the reliability index value in the following ways:

From equation 6.9, $\beta = \frac{150 - E[S]}{\sigma[S]}$ or $\beta = \frac{150 - E[S]}{E[S] * COV[S]}$

$$\text{or, } E[S] = \frac{150}{[1 + \beta * COV[S]]} \text{ -----------------------------(6.10)}$$

Here, the co-efficient of the variation of settlement, *COV* [*S*] is determined by analyzing the data collected from a similar existing structure, using the procedure described in steps 1-6 above.

3. Hence, an embankment should be designed such that its mean predicted settlement is less than or equal to the maximum allowable value of mean predicted settlement as determined above to obtain the desired reliability in the design against settlement.

It should be appreciated that the method proposed for calculating settlement of embankment is based on various parameters derived from laboratory testing. Therefore the calculation based on the proposed method is likely to be conservative compared to in-situ settlement. In addition a significant percentage of embankment settlement tends to take place during construction. Taking these factors into account it could be concluded that the reliability calculated should be normally lower than that which is based on in-situ testing data.

6.4 Embankment risk for both slope stability and settlement

If the reliability of embankment design against slope stability is found to be 80%, it means that if 100 embankments are built with this design then 80 of them will have the chance of not experiencing slope failure. Similarly, if the embankment is designed for 60% reliability against settlement, then 60 out of 100 embankments built with this design will have the chance of not failing in settlement. So, the probability of obtaining an embankment with no slope failure in its design life is 0.8 (80/100) and the probability of obtaining an embankment with no settlement failure in its design life is 0.6 (60/100).

Since the probabilities of successful performance of embankment for slope stability or settlement are independent events, the general risk of embankment for both slope stability and settlement can be calculated using special multiplication rules of probabilities. According to this rule, the probability that two independent events will both occur is simply the product of their probabilities. Now, applying this rule, the overall probability of success of an embankment design (embankment design reliability) considering the chance of success in both slope stability and settlement can be calculated as follows:

The probability of overall success of embankment design = Probability of success of embankment design for slope stability × Probability of success of embankment design for settlement --- (7.1)

Hence, the general risk of embankment design = (1- the overall probability of success of embankment design) × 100%

6.5 Overall risk of pavement-embankment design system

As pavement performance is linked to embankment performance a simple approach is needed to reflect this in computing the joint probability of failure and risk. According to the theory of probabilities, if P(A) is the probability of event A and P(B) is the probability of event B, then the joint probability of events A and B may be determined in the following way:

$$P(A \cap B) = P(A).\, P(B)$$

There are three possible outcomes which could occur when pavement and embankment risks are considered together, since each event is independent. These are:

1. Risk of pavement failure and failure in embankment stability

2. Risk of pavement failure and failure in embankment settlement
3. Risk of pavement failure, failure in embankment stability and failure in embankment settlement.

For example, if the reliability of pavement design is 70%, the reliability of embankment design against slope stability is 80% and the reliability of embankment design against settlement is 60%, then the overall risk of the design system can be calculated in the following way:

Overall design risk (3 cases)

1. Risk of pavement failure and failure in embankment stability

$$= [1- (0.7\times0.8)] = 0.44 \text{ or } 44\%$$

2. Risk of pavement failure and failure in embankment settlement

$$= [1- (0.7\times0.6)] = 0.58 \text{ or } 58\%$$

3. Risk of pavement failure and failure in embankment stability and settlement

$$= [1- (0.7\times0.8\times0.6)] = 0.664 \text{ or } 66.4\%$$

6.6 Summary

This chapter presented the development of a suitable process for quantifying embankment design risk considering both slope stability and settlement for the design system of Bangladesh. To this end, it considered first a comparison of existing methods of analyzing slope stability risk with respect to the some predefined criteria. The first order second moment method satisfied all requirements as a suitable method for Bangladesh. Moreover, this is a widely used and well recognized method. The detailed procedure involved in this method was also presented. Similarly a suitable method was developed in this research to quantify the embankment settlement risk by reviewing the existing literature. The first order second moment theory was used in developing the method of settlement risk analysis. The detailed procedure for this method was also described. Finally, the chapter presented a methodology using special multiplication rules of probabilities to quantify the overall risk of embankment design and the overall risk of a pavement-embankment design system. The next chapter will consider an integrated example of an overall risk quantification process with field test data of one of the roads in Bangladesh.

Chapter 7 An Integrated Example

7.1 Introduction

This chapter considers an integrated example of the quantification of risk in the pavement and embankment design system for Bangladesh, based on the concepts presented in Chapters 5 and 6. The data collected from road N302 are chosen for this example. The pavement design risk is quantified first and then the embankment design risk for slope stability and settlement is determined. Next, the overall embankment design risk is calculated. Finally, the overall risk of a pavement embankment design system is determined.

7.2 Pavement design risk quantification

7.2.1 Application example of proposed method

<u>Quantification of variance in traffic prediction</u>

1. The traffic prediction data of road N302 is summarized as follows:

Annual Average Daily Traffic (AADT) = 9303

Percentage of heavy vehicles (trucks, buses) in the traffic mix = 51.44%

The proportion of different types of heavy vehicle with their equivalent axle load factors as found by the traffic and axle load survey is shown below:

Parameter	Heavy Truck			Medium Truck		Small Truck		Large Bus		Medium Bus	
	Front	Rear1	Rear2	Front	Rear	Front	Rear	Front	Rear	Front	Rear
Percent of Axles, p_i	1.62	1.62	1.62	19.43	19.43	4.87	4.87	11.12	11.12	12.14	12.14
EALF, F_i	0.368	5.960	5.775	0.148	7.205	0.014	0.233	0.145	0.816	0.011	0.097

2. The AADT considers the traffic in both directions. To estimate the traffic in the design direction, the direction distribution was taken as 50%. There is only one lane in the design direction. Hence, the lane distribution is 100%. The variance in directional distribution and lane distribution can be ignored. Traffic growth rate (r) data were taken from a previous research report [khan, 2005]. The data for the co-efficient of variation of ADT, growth rate, percentage of trucks, axles per truck are assumed in this example since the traffic count survey was conducted for a limited period of time. The summary statistics of these parameters are presented below:

Design factor	ADT_0	Growth Rate, r %	Percent-age of Trucks T %	Axles per Truck, A	Directional Distribution D%	Lane Distribut-ion L%	Axle load factor $\Sigma p_i \times F_i$
Mean	9303	8	51.44	2.03	50	100	1.76
COV	15	10	12	8	0	0	35
Variance, V [x]	1947281	0.000064	0.0038	0.026	0	0	0.3782

3. The traffic growth factor, G is calculated for a design period (Y) of 10 years as follows:

$$G = \frac{1}{2}\left[1+(1+r)^Y\right] = 0.5\ [1+ (1+0.08)^{10}] = 1.579$$

The variance in the growth factor $V[G] = \left\{0.5Y(1+r)^{Y-1}\right\}^2 V[r]$

$$= \{0.5\times10\times (1.08)^9\}^2\times0.000064 = 0.00639$$

4. The predicted traffic (w_T) is calculated as follows [Huang, 1993]:

$$w_T = \left(\sum_{i=1}^{m} p_i F_i\right)(ADT_o)(T)(A)(G)(D)(L)(365)(Y)$$

where ADT_0 represents average daily traffic, T the percentage of trucks, A axles per truck, G growth factor, r growth rate, D directional distribution, L lane distribution and Y design life. p_i represents the percentage of different types of axle and F_i their respective equivalent axle load factor. Hence, the predicted traffic

$$w_T = 1.76\times9303\times0.5144\times2.03\times1.579\times0.5\times1.0\times365\times10 = 49.2 \text{ Million ESA}$$

5. Next, the variance in the predicted traffic (V [log w_T]) is calculated in the following way [Huang, 1993]:

$$V[\log w_T] = (\log e)^2 \left\{ \frac{V\left[\sum p_i F_i\right]}{\left(\sum p_i F_i\right)^2} + \frac{V[ADT_0]}{(ADT_0)^2} + \frac{V[G]}{G^2} + \frac{V[T]}{T^2} + \frac{V[A]}{A^2} + \frac{V[D]}{D^2} + \frac{V[L]}{L^2} \right\}$$

$$V[\log w_T] = 0.1886\left[\frac{0.3782}{(1.76)^2} + \frac{1947281}{(9303)^2} + \frac{0.00639}{(1.579)^2} + \frac{0.0038}{(0.5144)^2} + \frac{0.026}{(2.03)^2} + \frac{0}{(0.50)^2} + \frac{0}{(1.00)^2}\right]$$

$$= 0.03175$$

The variance in traffic prediction is therefore (S_w^2) = 0.03175

Quantification of variance in pavement performance prediction

6. The layer thickness and modulus are as follows:

Design factor	Thickness				Strength (Resilient Modulus)				
	Surface	Base	Sub-Base	Granular	Surface	Base	Sub-Base	Granular	Sub-grade
	mm	mm	mm	mm	MPa	MPa	MPa	MPa	MPa
Mean	137.8	205.0	179.67	384.67	1679.12	289.07	93.72	197.83	27.65
Standard Deviation	14.16	22.80	33.39	57.13	1447.57	31.83	27.29	39.69	2.93
Co-efficient of variation	10.27	11.12	18.58	14.85	86.21	11.01	29.12	20.07	10.60
Variance	200.5	520.0	1114.7	3264.4	2095457	1013.1	744.76	1575.7	8.58
Systematic error	33.43	86.67	185.78	544.07	349243	168.85	124.13	262.61	1.43

The systematic error due to the limited number of tests is calculated by dividing the variance V[*x*] of each parameter by the sample size (n = 6).

7. Then the mean pavement performance is predicted with a mean value of each pavement performance prediction parameter, using the developed pavement structural analysis model (i.e. the M.E.T) as follows:

Thickness		Strength (Resilient Modulus)			f_1	he_2	R
Surface	Granular	Surface	Granular	Subgrade			
mm	mm	MPa	MPa	MPa		m	m
137.83	384.67	1679.1	197.83	27.65	1.07	0.30	227.9

ε_t	f_2	he_3	ε_c	ε_t (std ld)	$N_{cracking}$	ε_c (std ld)	$N_{Rutting}$	$N_{critical}$
		m			MSA		MSA	MSA
0.0003	0.794	1.02	0.00069	0.00027	0.3121	0.00061	0.3086	0.3086

The mean predicted pavement performance in terms of a million standard axles is (W_t) = 0.3086

8. Then a sensitivity analysis of the above pavement performance prediction model is carried out for each parameter by changing the parameter one standard deviation above and below the mean value while keeping the other parameter unchanged. Following this, the ratio of the differences in pavement performance prediction to the differences in respective parameter values is determined. The results of the sensitivity analysis are presented below:

Parameter	Average	Standard deviation	Average + 1 STD	Average - 1 STD	Predicted performance	Predicted performance	$\Delta W_t/\Delta x_i$
			x_i^1	x_i^2	W_{t1}	W_{t2}	
Surface thickness	137.83	14.16	152.00	123.67	0.42758	0.21218	0.00760
Granular thickness	384.67	57.13	441.80	327.53	0.31206	0.15547	0.00137
Surface strength	1679.12	1447.57	3126.69	231.55	0.62974	0.05440	0.00020
Granular strength	197.83	39.69	237.52	158.13	0.38429	0.20116	0.00231
Subgrade strength	27.65	2.93	30.58	24.72	0.31206	0.25513	0.00972

9. The results of this sensitivity analysis, that is, the divided difference ($\Delta W_t/\Delta x_i$), are then squared up and multiplied separately with the spatial and the systematic variance. This is done for all parameters and summed up separately with regard to the spatial and systematic variance. The detail of this analysis is presented below:

Parameter	$\Delta W_t/\Delta x_i$	Variance, V (x_i)		$(\Delta W_t/\Delta x_i)^2.V(x_i)$	
		Spatial	Systematic	Spatial	Systematic
Surface thickness	0.00760	200.57	33.43	0.0116	0.001933
Granular thickness	0.00137	3264.40	544.07	0.00613	0.001022
Surface strength	0.00020	2095457	349243	0.08275	0.013792
Granular strength	0.00231	1575.66	262.61	0.00838	0.001397
Subgrade strength	0.00972	8.58	1.43	0.00081	0.000135
	$\Sigma(\Delta W_t/\Delta x_i)^2.V(x_i) =$			0.1097	0.0183

10. The variance in the predicted pavement performance ($V[W_t]$) is then calculated as follows:

$$V[W_t] = \sum_{i=1}^{n}\left(\frac{\Delta W_t}{\Delta x_i}\right)^2 V[x_i] + \sum_{i=1}^{n}\left(\frac{\Delta W_t}{\Delta x_i}\right)^2 V[x_i]_{systematic}$$

$$= 0.1097 + 0.0183$$

$$= 0.1280$$

Therefore, the variance in pavement performance prediction $\left(S_N^2\right)$ = 0.1280

Determination of overall variance and design reliability

11. Then the overall variance of pavement design and performance is calculated:

Overall Variance (S_0^2) = *Variance in Traffic Prediction* (S_w^2) + *Variance in Pavement Performance Prediction* (S_N^2)

= 0.03175 + 0.1280 = 0.1597

12. Next, the overall standard deviation of variation (S_0) is calculated:

Overall Standard Deviation of variation (S_0) = √ *Overall variance*

= √ (0.1597) = 0.3996

13. Finally, to design for a 75% design reliability level, the reliability design factor is determined as follows:

Reliability design factor, $F_R = 10^{-Z_R \times S_0}$ = $10^{0.674 \times 0.3996}$ = 1.86

The value of Z_R is taken from statistical Tables (Appendix E-4) for the reliability level chosen. A reliability factor of 1.86 for the predicted traffic of 49.2 MSA means that the pavement should be designed for a traffic capacity of 49.2 × 1.86 = 91.51 MSA to obtain a 75% reliability in design.

7.2.2 Comparison of results with other methods

A. Comparison with the method of Noureldin et al. [1994, 1996]

To validate the proposed method, the field data of road N302 are analysed by another method proposed by Noureldin et al. [1994, 1996]. These writers used the AASHTO empirical pavement performance model in their method to calculate the variance in pavement performance which involves the mean and COV of the AASHTO structural number (SN). The mean and COV of the pavement layer co-efficients (a_1, a_2, a_3) and drainage co-efficients (m_2, m_3) are required to calculate the mean and COV of SN. The layer co-efficient values are determined by the CBR and modulus data of road N302 using the AASHTO [1993] method. The drainage co-efficient values are determined from AASHTO's Table [1993] (Appendix C-4). The COV value of the layer and drainage co-efficients are determined using the equation proposed by Noureldin et al. [1994, 1996]. The detailed steps of the calculation using this method are provided in Appendix B. For comparison, a summary of the results using both the proposed method and that of Noureldin et al. [1994, 1996] is presented below.

Table 7.1: Summary of results of the proposed method and that of Noureldin et al. [1994, 1996]

Methods	Variance in traffic prediction (S_w^2)	Variance in pavement performance prediction (S_N^2)	Overall variance (S_0^2)	Overall standard deviation of variation (S_0)	Reliability design factor, F_R
Proposed method	0.0317	0.1280	0.1597	0.3996	1.86
Noureldin et al.	0.0301	0.1076	0.1377	0.3711	1.78

From the above table it seems that the developed method gives slightly more conservative results than the method proposed by Noureldin et al. [1994], but a more insightful analysis could be effected of more data was available. In general, the following differences were observed between these two methods:

1. The suggested method considers a systematic error (variance of mean due to the limited number of tests) in addition to the spatial variance in pavement performance prediction whereas the proposed method of Noureldin et al. [1994] did not consider such an error.
2. The proposed method of Noureldin et al. [1994] considers a COV of the Marshall stability in determining the COV of the layer co-efficient a_1, whereas the developed method considers the COV of resilient modulus. The COV of Marshall stability data was found 27.7% whereas the COV of resilient modulus data was found 86.21% in N302 road.
3. The proposed method of Noureldin et al. [1994] disregards the variance of some traffic parameters.

B. <u>Comparison with the proposed alternate method (FOSM)</u>

The proposed method is suitable for incorporating the desired reliability in the design of a new pavement, while the FOSM is suitable for finding the reliability of an existing pavement. To compare the analysis results of these two methods, a back analysis process is followed in the present research where a pavement is designed first by the proposed method for a selected reliability level. Then the risk analysis of this design is carried out using the FOSM method and the results are compared. A detailed risk analysis using the FOSM method is provided in Appendix A and the summarised results are presented below:

Table 7.2: Summary of results of the proposed and alternative method

Proposed method	Alternate method
Selected Reliability = 75% Reliability design factor found = 1.86	Selected safety factor = 1.86 Reliability found = 66.2%

Since the concepts of reliability determination of these two methods are different, the analysis results will never exactly coincide. However, reasonably good agreement is found in the case in hand.

From the above comparison it is clear that the proposed method gives reasonably good results. Consequently, the method is confidently recommended for the analytical pavement design system of Bangladesh.

7.3 Embankment design risk quantification

7.3.1 Quantification of embankment design risk against slope stability

The embankment design risk against slope stability of road N302 is calculated using the proposed FOSM method. The steps of the calculation procedure are presented below;

Identification of variables and tentative variance

1. The variables which affect the stability of the embankment slope as identified in section 6.2.2 are cohesion (c´), density (γ) and angle of internal friction (Φ´). The variability of these variables is determined by analyzing data collected from soil samples taken from road N302. The detailed discussion of the variability of these data is presented in section 4.4.2. The statistical analysis summary is presented below;

Design factor	Cohesion	Friction Angle	Unit Weight
Mean	4.86	18.02	15.75
Standard deviation	1.20	4.15	3.25
Co-efficient of variation	24.78	23.03	20.62
Variance, $V[x_i]$	1.45	17.22	10.55
Systematic error	0.16	1.91	1.17

Here, systematic error is determined by dividing the variance V[x_i] of each parameter with the sample size (n = 9), as discussed above.

Determination of mean factor of safety

2. The simplified Bishop method is used in this research for slope stability analysis where the factor of safety of slope stability is determined as follows:

$$F = \frac{1}{\sum W \sin\alpha} \sum \frac{\left[c'b + (W - ub)\tan\phi'\right]\sec\alpha}{1 + \frac{\tan\alpha \tan\phi'}{F}}$$

$$\text{Or, } F = \frac{\sum A \times B}{\sum W \sin\alpha}$$

where $A = c'b + (W - ub)\tan\phi'$ and $B = \frac{\sec\alpha}{1 + \frac{\tan\alpha \tan\phi}{F}}$

Now, using the mean value of each variable (c´, Φ´, γ) and considering a critical failure arc, the mean factor of safety is determined as follows:

Slice No	Width (b)	Height (h)	Weight (W)	Slice base Inclination (α)	Wsinα	Water table height above slice base (h_w)	Pore pressure (u)	A	B	A×B
1	1.85	0.20	5.8	-16.5	-1.7	0.4	3.9	8.5	1.14	9.68
2	2.00	0.85	26.8	-9.6	-4.5	1.1	10.8	11.4	1.06	12.14
3	2.00	1.85	58.3	-3.8	-3.9	1.8	17.7	17.2	1.02	17.55
4	1.00	2.75	43.3	2.5	1.9	2.3	22.6	11.6	0.99	11.48
5	1.00	3.45	54.3	7.9	7.5	2.7	26.5	13.9	0.97	13.53
6	1.00	4.90	77.2	13.5	18.0	2.8	27.5	21.0	0.96	20.28
7	1.00	4.50	70.9	21.4	25.9	2.7	26.5	19.3	0.97	18.7
8	1.00	4.30	67.7	26.7	30.4	2.2	21.6	19.9	0.98	19.53
9	1.00	3.50	55.1	32.3	29.5	1.5	14.7	18.0	1.01	18.13
10	1.00	2.20	34.7	40.8	22.6	0.4	3.9	14.9	1.07	15.84
11	1.00	1.37	21.6	48.7	16.2	0.0	0.0	11.9	1.15	13.69
12	1.05	1.08	17.9	58.6	15.2	0.0	0.0	10.9	1.32	14.41
Summation					157.3					185.0

$$\text{Mean factor of safety, } E[F] = \frac{\sum A \times B}{\sum W \sin\alpha} = \frac{185.0}{157.3} = 1.1761$$

Correction of mean factor of safety for model error

3. The mean factor of safety is then corrected for model error (see section 2.8.1.4) by multiplying it by an adjustment factor [Azzouz et al. 1983], as follows:

Corrected mean factor of safety = Mean factor of safety × Adjustment factor

= 1.1761×1.05 = 1.2349

Calculation of variance of factor of safety

4. Then a sensitivity analysis of the factor of safety is carried out whereby the mean value of each variable is increased or decreased by one standard deviation and the value of factor of safety in each case is determined. Then the ratio is determined of the differences in the factor of safety to the differences in the respective values of variables for each parameter. The results of this analysis are presented below:

Parameter	Average	Standard deviation	Average + 1 STD	Average - 1 STD	Factor of safety	Factor of safety	$\Delta F/\Delta x_i$
			x_i^1	x_i^2	F_1	F_2	
Cohesion, c´ (kN/m^2)	4.86	1.20	6.06	3.65	1.309	1.042	0.1111
Friction Angle, Φ´ (º)	18.02	4.15	22.17	13.87	1.337	1.022	0.0379
Unit Weight, γ (kN/m^3)	15.75	3.25	19.00	12.50	1.165	1.192	0.0042

5. The square of divided difference ($\Delta F/\Delta x_i$) of each parameter is then multiplied with their respective spatial and systematic variance and summed up separately as follows:

Parameter	$\Delta F/\Delta x_i$	Spatial Variance $V[x_i]_{spatial}$	Systematic Variance $V[x_i]_{systematic}$	$(\Delta F/\Delta x_i)^2.$ $V[x_i]_{spatial}$	$(\Delta F/\Delta x_i)^2.$ $V[x_i]_{systematic}$
Cohesion, c´ (kN/m^2)	0.1111	1.45	0.16	0.161	0.018
Friction Angle, Φ´ (º)	0.0379	17.22	1.91	0.654	0.073
Unit Weight, γ (kN/m^3)	0.0042	10.55	1.17	0.044	0.005
			$\Sigma(\Delta F/\Delta x_i)^2.V[x_i]$ =	0.858	0.095

6. Then the variance of factor of safety is determined as follows:

$$V[F] \approx f\sum_{i=1}^{k}\left(\frac{\Delta F}{\Delta x_i}\right)^2 V[x_i]_{spatial} + \sum_{i=1}^{k}\left(\frac{\Delta F}{\Delta x_i}\right)^2 V[x_i]_{systematic} + V[e]$$

where the reduction ratio, f = 0.25 (see section 2.8.1.4) and the variance in the factor of safety due to model error, V[e] = (0.07×corrected E [F])^2

= (0.07 ×1.2349)^2 = 0.0075

Hence, the variance of the factor of safety,

$V[F]$ = 0.25×0.858+0.095+0.0075

= 0.3175

Determination of reliability index

7. The reliability index (β) is then calculated using the following equation:

$$\beta = \frac{E[F] - 1.0}{\sigma[F]}$$

where the mean factor of safety, $E\ [F]$ = 1.1761

The standard deviation of factor of safety, $\sigma\ [F] = \sqrt{V\ [F]} = \sqrt{0.3175} = 0.5634$, hence

$$\text{Reliability index, } \beta = \frac{1.1761 - 1.0}{0.5634}$$

$$= 0.313$$

Determination of Risk of N302 road embankment against slope stability

8. From the standard normal curve area Table [Appendix E-1], the reliability of the embankment is determined for the corresponding value of the reliability index.

The reliability of N302 road embankment is found to be 63%

Hence, the risk of embankment failure in slope stability = 1 - 63% = 37%

Incorporation of the desired reliability in design for slope stability

9. The required value of the mean factor of safety to achieve a desired reliability level in design against slope stability is determined using Equation 6.4 in the following way:

$$E[F] = \frac{1}{[1 - \beta * COV[F]]}$$

where $COV\ [F]$ is determined from the analysis results of an existing similar embankment. The $COV\ [F]$ for N302 road is determined as follows:

$$COV[F] = \frac{\sigma[F]}{E[F]} \times 100 = \frac{0.5634}{1.1761} \times 100 = 47.9\,\%$$

The reliability index, β is determined from the standard normal curve area Table [Appendix E-1]. For the desired reliability level of 80%, the value of reliability index, β = 0.842

Hence, the required value of the mean factor of safety,

$$E[F] = \frac{1}{[1 - \beta * COV[F]]} = \frac{1}{[1 - 0.841 \times 0.479]} = 1.675$$

The required value of the mean factor of safety for a different desired reliability level is also determined for the N302 road embankment and is presented in Table 7.3.

Table 7.3: The required mean factor of safety for a different desired reliability level

Desired Reliability level	Corresponding reliability index, β	The required mean factor of safety, E[F]
50%	0.000	1.000
55%	0.126	1.064
60%	0.253	1.138
65%	0.385	1.226
70%	0.524	1.335
75%	0.674	1.477
80%	0.841	1.675
85%	1.037	1.987
90%	1.282	2.592

7.3.2 Quantification of embankment design risk against settlement

The embankment design risk for the settlement of road N302 is determined using a method developed in this research, based on the first order second moment theory. The steps of the calculation procedure are presented below.

<u>Variables and its tentative variance</u>

1. The variables which affect embankment settlement as identified in section 6.3.2 are the modulus of elasticity (E), applied pressure (p), layer thickness (H), initial void ratio (e_0), density of soil (γ), compression index (C_c) and secondary compression index (C_α). The layer-wise field testing data of these parameters are analyzed statistically. A detailed discussion of the variability of these data is presented in section 4.4.3. The summary statistics of the variability of settlement parameters is presented below:

	Layer thickness	Initial void ratio	Saturated density	Dry density	Compres sion index	Secondary compression index
	H	e_0	γ_{sat}	γ_{dry}	C_c	C_α
Layer one						
Mean	2.25	1.4067	17.53	12.30	0.344	0.0189
STD	0.5	0.4283	2.656	3.82	0.0418	0.0269

COV	22.22	30.447	15.15	31.01	12.15	142.11
Variance	0.25	0.1834	7.057	14.56	0.0017	0.0007
Systematic error	0.083	0.0611	2.352	4.85	0.0006	0.0002
Layer two						
Mean	1.666	1.2633	14.84	8.34	0.35	0.0163
STD	0.382	0.0961	2.82	4.207	0.0399	0.0232
COV	22.91	7.6061	18.99	50.46	11.414	142.43
Variance	0.146	0.0092	7.95	17.70	0.0016	0.00005
Systematic error	0.049	0.0031	2.65	5.9	0.0005	0.0002
Layer three						
Mean	1.583	1.0083	16.25	10.41	0.3067	0.0374
STD	0.8036	0.2206	3.987	5.70	0.0635	0.0311
COV	50.756	21.876	24.53	54.78	20.71	83.07
Variance	0.6458	0.0487	15.89	32.52	0.004	0.001
Systematic error	0.2153	0.0162	5.29	10.84	0.0013	0.0003

The surcharge load for embankment (ΔP) is the weight of the pavement. The weight of the pavement is calculated by multiplying the layer thickness of the pavement with respective layer densities. The calculation of the surcharge load of road N302 is presented below:

Location	Layer	Layer thickness	Layer density	Surcharge	Total
		m	KN/m^3	KN/m^2	KN/m^2
Ch. 6+200	Surface	0.11	21.9744	2.41718	10.41
	Base	0.2	15.4998	3.09996	
	Sub-base	0.2	14.5188	2.90376	
	Imp.subgd	0.14	14.2245	1.99143	
Ch.6+480	Surface	0.15	21.9744	3.29616	11.12
	Base	0.17	15.4998	2.63497	
	Sub-base	0.23	14.5188	3.33932	
	Imp.subgd	0.13	14.2245	1.84918	
Ch.6+600	Surface	0.141	21.9744	3.09839	10.55
	Base	0.23	15.4998	3.56495	
	Sub-base	0.17	14.5188	2.46819	
	Imp.subgd	0.1	14.2245	1.42245	

The applied load for the embankment is the traffic load. The applied pressure at subgrade level (i.e. top of embankment) of road N302 due to standard wheel load is calculated in the following way:

$$\text{Applied pressure, } p = \frac{wheel \quad load}{\Pi R^2}$$

where R = radius of the wheel loaded area at the subgrade level. The radius of the wheel load influence area at subgrade level can be approximated in the following way:

R = tyre contact radius + √3 × total thickness of pavement layer.

Now, using the layer thickness data of road N302, the applied pressure found at different locations is as follows:

Location	Total layer thickness	Radius of load area at Subgrade	Pressure
	m	m	KN/m^2
Ch.6+200	0.65	1.276	7.82
Ch.6+480	0.68	1.328	7.22
Ch.6+600	0.64	1.260	8.02

The undrained modulus data as collected from triaxial tests are analyzed statistically. The surcharge and applied pressure data are also statically analyzed. A summary of this analysis is presented below.

	Soil modulus, E	Applied pressure, p	Surcharge ΔP
	KN/m^2	KN/m^2	KN/m^2
Mean	2684.44	7.69	10.69
STD	575.15	0.36056	0.376
COV	21.43	4.6907	3.517
Variance	330803	0.13	0.1414
Systematic error	36755.9	0.0144	0.0471

Calculation of Mean Settlement, E[*S*]

2. There are three stages of settlement: immediate, primary and secondary settlement.

i) The mean immediate settlement $E[S_i]$ of N302 road embankment is calculated first using the mean values of immediate settlement parameters and the following equation [Tomlinson, 2001: Foott and Ladd, 1981: Atkinson, 2007]:

$$E[S_i] = E[p].E[B]\frac{1-E[\mu]^2}{E[E]}.I_p$$

Where, Mean applied net pressure, E [*p*] = 7.69 kN/m^2

Mean width of foundation, E [*B*] = 10.9 m

Mean Poisson's ratio E [μ] = 0.35

Mean modulus of elasticity of soil, E [*E*] = 2684.44 kN/m^2

Non-dimensional influence factor, I_p = 1.12

$$\text{Therefore, } E[S_i] = 7.69 \times 10.9 \times \frac{1-(0.35)^2}{2684.44} \times 1.12 = 30.67 \text{ mm}$$

ii) To calculate the mean primary settlement, the layer mean predicted primary settlement is calculated first using the layer mean value of the primary settlement parameters and the following equation [Craig, 2004: Das, 1997: Tomlinson, 2001]:

$$S_p = \frac{C_c H}{1+e_0}\left(\log \frac{p_0 + \Delta p}{p_0}\right)$$

The mean value of parameters and the calculated mean predicted primary settlement of different layers of N302 road embankment are presented below:

Layer No.	Mean Layer thick-ness	Mean Initial void ratio	Mean Satu-rated density	Mean Dry density	Mean Density of water	Mean Pressure	Mean Sur-charge	Mean Com-pression index	Mean Primary settlement
	E[H]	E[e_0]	E[γ_{sat}]	E[γ_{dry}]	E[γ_w]	E[P_0]	E[ΔP]	E[C_c]	E[S_p]
	m		kN/m^3	kN/m^3	kN/m^3	kN/m^2	kN/m^2		mm
1	2.25	1.41	17.53	12.30	9.81	13.84	10.69	0.344	79.952
2	1.67	1.26	14.85	8.34	9.81	31.87	10.69	0.350	32.376
3	1.58	1.01	16.25	10.41	9.81	41.18	10.69	0.307	24.240

Then the total mean predicted primary settlement is calculated as follows:

Total mean primary settlement, E [S_p] = Σ layer mean predicted primary settlement

= (79.952+32.376+24.240) mm = 136.57 mm

iii) Similarly, to calculate the mean secondary settlement, the layer mean predicted secondary settlement is calculated first, using the layer mean value of the secondary settlement parameters and the following equation [Das, 1997]:

$$S_s = \frac{C_\alpha H}{1+e_p}\left(\log \frac{t_2}{t_1}\right)$$

The mean value of the secondary settlement parameters and calculated mean predicted secondary settlement of different layers of N302 road embankment is presented below:

Layer No.	Layer Thickness	Initial void ratio	Pressure	Surcharge	Compression index	Change in void ratio	Void ratio at end of primary consolidation.	Secondary compression index	Time for end of primary constn.	Design period	Secondary settlement
	H	e_0	P_0	ΔP	C_c	Δe	e_p	C_α	t_1	t_2	S_s
	m		kN/m^2	kN/m^2					yr	yr	mm
1	2.25	1.41	13.84	10.69	0.344	0.085	1.3211	0.0189	2	10	12.828
2	1.67	1.26	31.88	10.69	0.350	0.044	1.2194	0.0163	2	10	8.538
3	1.58	1.01	41.18	10.69	0.307	0.031	0.9776	0.0374	2	10	20.930

Then the total mean predicted secondary settlement is calculated as follows:

Total mean secondary settlement, E [S_s] = Σ layer mean predicted secondary settlement

= (12.828+8.538+20.930) mm = 42.296 mm

iv) Finally, the total mean predicted settlement E[S] of N302 road embankment is calculated in the following way:

$$E[S] = E[S_i] + E[S_p] + E[S_s]$$

= (30.67+136.57+42.296) mm = 209.54 mm

<u>Calculation of variance of predicted settlement, V[*S*]</u>

3. To calculate the variance of settlement, first a sensitivity analysis of settlement is performed, using the mean settlement calculation model. In the sensitivity analysis, the mean value of each variable is increased and decreased by one standard deviation while keeping the other variables unchanged and the corresponding settlement values in each case are determined. Then the ratio is determined of the differences in settlements to the difference in the corresponding value of variables, i.e. $\Delta S/\Delta x_i$ for each variable. The results of a sensitivity analysis of the settlement of N302 road embankment is presented below:

Parameter	Layer No.	Average +1 STD Value	Average -1 STD Value	Settlement mm	Settlement mm	$\Delta S/\Delta x_i$ $[S_1-S_2]/[x_i^1-x_i^2]$
		x_i^1	x_i^2	S_1	S_2	
Applied Pressure, p (kN/m^2)		8.050	7.33	210.98	208.10	3.9938
Modulus of elasticity (kN/m^2)		3259.6	2109.3	204.13	217.90	0.0120
Layer thickness, H (m)	1	2.7500	1.7500	208.42	212.09	3.6700
	2	2.0490	1.2848	216.91	201.91	19.640
	3	2.3870	0.7797	230.50	187.33	26.859
Void ratio, e_0	1	1.8350	0.9784	195.46	229.75	40.031
	2	1.3594	1.1672	207.87	211.36	18.160
	3	1.2289	0.7877	205.04	215.16	22.939
Density (kN/m^3)	1	16.119	8.4880	184.23	250.27	8.6545
	2	17.666	12.027	205.31	214.56	1.6403
	3	20.240	12.266	207.97	211.35	0.4239
Compression index, C_c	1	0.3858	0.3022	219.31	199.77	233.75
	2	0.3899	0.3101	213.26	205.83	92.991
	3	0.3702	0.2432	214.63	204.45	80.147
Secondary compression index, C_α	1	0.0458	-0.0080	227.77	191.31	677.54
	2	0.0394	-0.0069	221.70	197.38	524.83
	3	0.0685	0.0063	226.93	192.15	559.72

4. The results of this sensitivity analysis, that is, the divided difference ($\Delta S/\Delta x_i$) for each parameter, is then squared up and multiplied separately for their respective spatial and systematic variance. The results of this multiplication for all parameters are then summed up separately for spatial and systematic variance. The details of this analysis with N302 road embankment data are presented below:

Parameter	Layer No.	$\Delta S/\Delta x_i$	Variance, $V(x_i)$		$(\Delta S/\Delta x_i)^2.V(x_i)$	
			Spatial	Systematic	Spatial	Systematic
Applied Pressure, p		3.9938	0.1300	0.01444	2.0736	0.2304
Modulus of elasticity, E		0.0120	330803	36755.9	47.403	5.2670
Layer thickness, H	1	3.6700	0.25000	0.08333	3.3672	1.1224
	2	19.640	0.14583	0.04861	56.250	18.750
	3	26.859	0.64583	0.21528	465.91	155.30
Void ratio, e_0	1	40.031	0.18343	0.06114	293.95	97.984
	2	18.160	0.00923	0.00308	3.0450	1.0150
	3	22.939	0.04866	0.01622	25.604	8.5345
density, γ_{sat}	1	8.6545	7.05730	2.35243	528.59	176.20
	2	1.6403	7.95003	2.65001	21.391	7.1302
	3	0.4239	15.8954	5.29848	2.8561	0.9520
Compression index, C_c	1	233.75	0.00175	0.00058	95.453	31.818
	2	92.991	0.00160	0.00053	13.801	4.6004
	3	80.147	0.00403	0.00134	25.908	8.6360
Secondary compress. Index, C_α	1	677.54	0.00072	0.00024	332.33	110.78
	2	524.83	0.00054	0.00018	147.87	49.289
	3	559.72	0.00097	0.00032	302.41	100.80
$\Sigma(\Delta S/\Delta x_i)^2.V(x_i)$ =					2368.22	778.41

5. Now, the variance of settlement $V[S]$ is determined using Equation 6.8 as follows:

$$V[S] = \sum_{i=1}^{n}\left(\frac{\Delta S}{\Delta x_i}\right)^2 V\left[x_i\right]_{spatial} + \sum_{i=1}^{n}\left(\frac{\Delta S}{\Delta x_i}\right)^2 V\left[x_i\right]_{systematicerror}$$

$= 2368.22 + 778.41$

$= 3146.63$

Computation of reliability index

6. From the above calculation, the following is found:

Mean settlement, $E[S] = 209.54$

Variance of settlement, $V[S] = 3146.63$

Standard deviation of settlement, $\sigma[S] = \sqrt{V[S]} = \sqrt{3146.63} = 56.09$

Now, the reliability index (β) is calculated using Equation 6.9 as follows:

$$\beta = \frac{150 - E[S]}{\sigma[S]} = \frac{150 - 209.54}{56.09} = (-)\ 1.061$$

<u>Determination of Reliability of N302 road embankment against settlement</u>

7. From the standard normal curve area table, the reliability of the embankment against settlement for the corresponding value of the reliability index is found to equal 14.5% Hence, the risk of embankment failure for settlement = 1-14.5% = 85.5%.

<u>Incorporation of desired reliability against settlement in the design of embankment</u>

8. The maximum allowable value of mean predicted settlement to achieve a desired reliability level in design against settlement is determined using Equation 6.10 in the following way:

$$E[S] = \frac{150}{[1 + \beta * COV[S]]}$$

where *COV* [*S*] is determined from the analysis results of an existing similar embankment. The *COV* [*S*] in the present case is determined as follows:

$$COV[S] = \frac{\sigma[S]}{E[S]} \times 100 = \frac{56.09}{209.54} \times 100 = 26.7\,\%$$

The reliability index value is determined from the standard normal curve area Table.

If the desired reliability level is 80%, then the value of the reliability index, β = 0.841

Hence, the maximum allowable value of the mean predicted settlement,

$$E[S] = \frac{150}{[1 + \beta * COV[S]]} = \frac{150}{[1 + 0.841 \times 0.267]} = 122.5\,\text{mm}$$

The maximum allowable value of mean predicted settlement for a different desired reliability level is also determined for N302 road and is presented in Table 7.4:

Table 7.4: The maximum allowable settlement for a different desired reliability level

Desired Reliability level	Corresponding reliability index, β	The maximum allowable value of mean predicted settlement, E[S]
50%	0.000	150.00
55%	0.126	145.12
60%	0.253	140.51
65%	0.385	136.02
70%	0.524	131.59
75%	0.674	127.12
80%	0.841	122.50
85%	1.037	117.48
90%	1.282	111.75

7.3.3 Quantification of embankment risk for both slope stability and settlement

The reliability of the embankment (N302) against slope stability = 63% and

the reliability of embankment (N302) against settlement = 14.5% means

that the overall reliability of N302 road embankment (see section 6.4)

= Probability of success of embankment for slope stability × Probability of success of embankment for settlement

= 0.63 × 0.145

= 0.0914

Hence, the overall reliability of the N302 road embankment = 0.0914 ×100% = 9.14%

That is, the overall risk of embankment failure = 1 - 9.14 = 90.86%

7.4 Overall risk quantification

The overall risk of the pavement embankment design system (N302) is calculated, using the procedure as proposed in section 6.5 in the following way:

The reliability of pavement design = 75%

The reliability of the embankment against slope stability = 63% and

The reliability of the embankment against settlement = 14.5%

Overall design risk (3 cases)

1. Risk of pavement failure and failure in embankment stability,

$$= [1- (0.75\times0.63)] \times100\% = 52.75\%$$

2. Risk of pavement failure and failure in embankment settlement,

$$= [1- (0.75\times0.145)] \times100\% = 89.13\%$$

3. Risk of pavement failure, failure in embankment stability and settlement,

$$= [1- (0.75\times0.63\times0.145)] \times100\% = 93.15\%$$

7.5 Summary

This chapter provided an integrated example to quantify the overall risk of a pavement embankment design system for Bangladesh, considering the field and laboratory test data of road N302. The risk in pavement design was found to be 25%, the risk in embankment design for slope stability was found to be 37% and for settlement the risk was found to be 85.5%. The overall embankment risk was found to be 90.86%. The overall risk of the pavement-embankment design system considering all possible combinations of failure was also quantified and for the worst combination was found to be 93.15%. The next chapters will consider the development of a process for the quality control of pavement and embankment construction in Bangladesh to reduce the variability in the design data.

Chapter 8 Pavement Construction Quality Control

8.1 Introduction

A quality control and assurance system is essential to reduce the variability of construction related data. One of the objectives of this research, as mentioned in Chapter 1, is to develop a quality control and assurance process for the pavement design system of Bangladesh. The concepts and fundamentals of quality control and assurance process were discussed in the literature review chapter. This chapter presents the complete development process of a quality control system for Bangladesh. With this in mind, the review of developed methods and techniques for the quality control of pavement construction and the comparison of them with respect to the criteria as defined in the methodology chapter in presented first with a view to identifying a suitable method or technique for Bangladesh. Next, the logical development leading to the proposed method is discussed, followed by detailed steps for the proposed procedure. The chapter concludes by presenting the step-by-step quality control process during construction; this process was also developed in the present research to make the quality control process smooth.

8.2 Developed methods

A very few completed quality control procedures in pavement construction are available in the literature. However, significant research work is found on the different components of a quality control process, including quality control tests, quality measures, testing frequency and sampling and performance relationships. A brief summary of them is presented below.

8.2.1 Quality control tests

The commonly used quality control measures for pavement construction are density, moisture content, gradation and layer thickness. These parameters are not consistent with the performance based construction criterion where evaluating the strength and stiffness of material is important. More importantly, the stiffness values are used as an input in the design of pavement systems. A design directly connected with the construction method is a prerequisite for balancing cost effectiveness with quality criteria. Hence, a technique for measuring in situ stiffness is required.

Pavement thickness is one of the most critical parameters in pavement design procedures but is significantly affected by poor construction practices. Hence, pavement layer thickness tests are an important part of any quality control process. The direct way to evaluate pavement layer thickness is by digging a core. But this process is destructive, time consuming and is not suitable for large-scale evaluation. There are other ways of evaluating pavement layer thickness, such as using the Dynamic Cone Penetrometer [Chen et al., 2001] or Ground-Penetrating Radar [Al-Qadi et al., 2003: Saarenketo and Scullion, 2000].

8.2.2 Quality measures and conformance

Livneh [2002] mentioned that highway agencies in many countries adopt a statistical approach to evaluate the quality characteristics of pavement construction. Some advantages of this approach were also reported, such as greater production flexibility, no need for engineering judgment, decreased disputes over marginal quality work, etc. Weed [1999] recommended mean and standard deviation as a suitable quality measure in determining the performance of a pavement. Burati and Weed [2006] also suggested using mean and standard deviation jointly to determine specification compliance in the quality control process of pavement construction. Schmitt et al. [1998] proposed five different measures: average, moving average, average absolute deviation (AAD), range and quality level analysis for the same purpose. Dobrowolski and Bressette [1998] reported that the t-test for sample means with a confidence interval of 99 percent should be used for comparing quality control test results. One-sided and two-sided *t*-tests were used by Mladenovic et al. [2003] to compare the mean constructed thickness and the design values. Torres-Verdin and McCullough [1991] also suggested using the student’s t-distribution in establishing confidence interval of the mean for quality control parameters.

8.2.3 Performance relationship

A) <u>S shaped performance model</u>

Weed [2006] proposed a multi-characteristic performance relationship for use in quality control processes. Since, most quality characteristics had points of diminishing returns (points beyond which little additional performance could gain from still higher levels of quality), a more appropriate model with an S shape was suggested. An exponential expression given by the following equation was recommended as a suitable mathematical form, which could produce an S shape.

$$y = Ae^{-Bx^{C}} \text{ ---------------------------------- (8.1)}$$

The equation was recommended when zero took the best possible level of the independent variable (*x*) (say, pavement roughness or percentage defective, PD). However, when zero took the worst possible value (say, pavement thickness or percentage within limit, PWL) the following equation best represented the model form:

$$y = A\left(1 - e^{-Bx^{C}}\right) \text{ -------------------------------- (8.2)}$$

The two equations above were based on a single quality characteristic which was not in accordance with reality. Weed [2003] extended Equation 8.1 in the following way which could consider any reasonable number (k) of quality characteristics:

$$EXPLIF = e^{B_o + B_1 PD_1 + B_2 PD_2 + \cdots + B_k PD_K} \text{ ------------ (8.3)}$$

where *EXPLIF* (expected life in years) took the place of dependent variable (*y*) and *PD* took the place of independent variable (*x*). A performance matrix was suggested for use in evaluating the co-efficient of the above equation.

B) <u>Simulation method</u>

Patel and Thomson [1998] investigated the effect of variability on system performance by performing a sensitivity analysis of the quality characteristics with the simulation method and showed that deviation from the target mean value significantly affected the fatigue life and there was varied influence on the fatigue life due to deviations from the target standard deviation.

8.2.4 Pay adjustment

Highway agencies throughout the world are now increasingly using adjusted payment provision for work which fails to meet the desired quality level but is not deficient enough to warrant removal and replacement. More recently, many agencies have started to provide monetary incentives in the form of bonuses for work which substantially exceeds the desired level of quality. Research undertaken by the California Department of Transportation found that incentives provided by the pay factor encouraged a better product [Douglas et al., 1999]. However, there are no well-established methods to determine the magnitude of pay adjustment appropriate for varying levels of as built quality. Schmitt et al. [1998] investigated the pay adjustment attributes of different agencies' specifications and found two basic methods for pay adjustment: a factor (or multiplier) and a fixed rate. The factor was mentioned as the more common; it involves a predetermined pay percentage for measured test results. In contrast, an example of fixed rate adjustment was given as a $1 per ton pay reduction for a certain level of quality regardless of bid price. Weed [1998] reviewed earlier work and mentioned that many pay schedules currently in use may not fully reflect the real costs incurred by highway agencies as a result of defective work and consequently suggested the following equation to calculate the future cost:

$$C_n = C_0\,(1+R_{INF}/100)^n \quad \text{-------------------------------} \; (8.4)$$

where C_n stands for future cost after n years, C_0 for present cost and R_{INF} for annual inflation rate (percent). Then the present worth of future cost was calculated as:

$$W_0 = C_n\,(1+R_{INT})^n \quad \text{----------------------------------} \; (8.5)$$

where W_0 represents present worth, C_n represents the cost n years in the future and R_{INT} represents annual interest rate (percent). Dobrowolski and Bressette [1998] suggested determining a pay factor for each quality characteristic by statistical quality analysis and calculating a composite pay factor by using weighted individual pay factors. Individual and composite pay factors were then suggested for use as a basis for decisions whether to accept or reject work and also over paying the contractor. Weed [1999] suggested that pay equations based on the mean and standard deviation derived from the sample closely correspond the value of the as-built work determined by the life-cycle cost analysis. Weed [2001] performed a life-cycle cost analysis to develop the link between quality received and economic gain or loss to the highway agency and suggested the following equation:

$$PAYADJ = C\left(R^{DESLIF} - R^{EXPLIF}\right)/\left(1 - R^{OVLIF}\right) \quad \text{-----------------} \; (8.6)$$

where *PAYADJ* = appropriate pay adjustment for pavement or overlay (same unit as C)

C = Present total cost of resurfacing (typical value = \$20/yd^2 or \$23.92/m^2)

DESLIF = design life of pavement or overlay (typical 20 years for new pavement, 10 years for overlay)

EXPLIF = expected life of pavement or overlay (independent variable)

OVLIF = expected life of successive overlays (typically 10 years) and

R = (1+*INF*) / (1+*INT*) [*INF* is the long-term annual inflation rate and *INT* is the long-term annual interest rate, both in decimal form]

Zaghloul et al. [1998] quantified the payment adjustment factors (PAFs) by performing a life cycle analysis considering the long-term effects of the initial roughness; they strongly advised every highway agency to conduct this type of analysis for their local conditions in order to develop their own PAF.

8.3 Comparison of the developed methods and techniques

The available methods and techniques are compared with respect to the criteria as determined in section 3.8 of the methodology chapter with a view to developing a suitable method of quality control in pavement construction for Bangladesh. A brief summary of this comparison is presented below.

Quality control tests:

<u>Strength and stiffness tests</u>: The Dynamic Cone Penetrometer (DCP) test is cost effective, simple to use and easy to maintain [Abu-Farsakh et al., 2005]. There are some verified correlations in the literature to calculate pavement strength and stiffness from the DCP test results [Powell et al., 1984; Abu-Farsakh et al. 2005; Chen et al. 2001]. But these correlations are site specific and no correlation has so far been developed for Bangladesh. Besides, DCP data collected from road testing in Bangladesh shows unrealistic results, as discussed in section 4.3.4. Hence, DCP is not suitable for evaluating the strength of pavement construction in Bangladesh. Falling weight deflectometer (FWD) tests are well recognized and widely used [Damnjanovic and Zhang, 2006: Noureldin et al., 2003]. The moduli of pavement layer are easily evaluated using a back calculation process with data from FWD tests [Mehta and Roque, 2003]. The moduli thus evaluated are of higher

quality and more cost effective than lab-measured moduli [Houston et al., 1992; Kim et al., 2007]. This method of calculating pavement strength is consistent with the proposed analytical pavement design system of Bangladesh. In addition, FWD data is clearly related to pavement performance [Chen and Scullion, 2006; Damnjanovic and Zhang, 2006]. Hence FWD could be used in the country's quality control of pavement construction. However, as the cost of acquisition and maintenance of an FWD is high, the Benkelman Beam could be used as a much cheaper substitute.

<u>Thickness tests</u>: Although coring is the easiest and most straightforward way of evaluating pavement layer thickness, it is destructive and not suitable for large-scale evaluations. Ground-Penetrating Radar (GPR) technology has not been used widely for quality control or pavement assessment purposes, due to reliability issues with the technique. Various GPR performances were reported by different investigators, depending on the site surveyed and data analysis technique used [Al-Qadi et al., 2005]. Besides, GPR is sophisticated and costly, requiring skilled manpower to implement and thus is not suitable for a developing country such as Bangladesh. DCP, in contrast, is simple to understand and easy to implement. Moreover, the changing slope of the depth versus profile of the accumulated blows of DCP tests corresponds well with the thickness obtained in the test pits [Chen et al., 2001]. Hence, DCP could be used to assess the thickness of pavement layers in the quality control process.

Quality measures and conformance:

The statistical approach of evaluating the quality characteristics of pavement construction, as proposed by Livneh [2002], is suitable for Bangladesh since it is well recognized, widely used and a scientific way of deriving a universal property from a sample property. Weed [1999] and Burati [2006] suggested mean and standard deviation is the best measure to determine the population mean of quality characteristics from a sample mean and hence one which could well be used in the quality control system of Bangladesh. Dobrowolski and Bressette [1998], Torres-Verdin and McCullough [1991] and Mladenovic [2003] proposed a t-test to establish a confidence interval of sample mean which is suitable for any quality control system; this is because a t-test is applicable for a small number of tests and in the quality control process of construction usually a small number of tests is performed to make decisions about a large project.

Performance relationship:

S shaped performance model: The equation proposed in this method to evaluate pavement performance for a higher number of quality characteristics involves a number of co-efficients and it was suggested that these should be used to evaluate by means of a performance matrix. But a performance matrix is not available in Bangladesh. Besides, this performance evaluation procedure is not compatible with the proposed design system of Bangladesh.

Simulation method: The simulation method of evaluating pavement performance by performing a sensitivity analysis of quality characteristics is not suitable for Bangladesh since a computer model is required to perform the simulation.

Pay adjustment:

Adjusted payment provision could be used in Bangladesh's quality control system since many highway agencies in the world are now increasingly using it and it encourages a better product, as many researchers have noted. Pay adjustment in the form of a pay factor would be suitable for Bangladesh, since, as Schmitt et al. report [1998], it is widely used. The pay adjustment equation proposed by Weed [2001] would be best for Bangladesh, since it is based on the pavement life cycle cost and compatible with the proposed design system of Bangladesh.

8.4 Logical development of the proposed method

Performance based quality control

A suitable quality control system for Bangladesh, as defined in the methodology chapter, should be clearly related to performance; the performance must be consistent, convenient to measure and quantify and suitable for the country's pavement design system. The proposed pavement design system of Bangladesh is an analytical design system where pavement performance is measured by the critical number of load repetitions that a pavement can sustain before failure. Considering the criteria of the required quality control system and the performance measure of analytical pavement design, the critical number of road repetitions that a pavement can sustain before failure ($N_{critical}$) is selected as the quality control parameter for the proposed quality control system.

To quantify the parameter $N_{critical}$, a pavement structural analysis model based on elastic layered theory is used. The input of this analysis model is determined by testing, which involves evaluating the strength and thickness of the pavement layer. Some criteria for suitable quality control tests were defined in the methodology chapter. Investigating the existing quality control tests with respect to these criteria identifies suitable methods for Bangladesh, as discussed in section 8.3. The Falling Weight Deflectometer or Benkelman Beam is identified as a suitable method for evaluating pavement layer strength and the Dynamic Cone Penetrometer is identified as suitable method for evaluating thickness. Stratified random sampling (where the population is divided into a number of non-overlapping sub-populations, or strata and then a simple random sampling follows to select samples from each stratum) is identified as a suitable method of sampling for the quality control of pavement construction since it better represents the actual field than simple random sampling. In addition, the problem of clustering (clogging of samples close to each other) in simple random sampling is reduced to some extent in stratified random sampling. Now, data from the quality tests helps to evaluate the pavement capacity. To find the population mean of pavement capacity, the mean and standard deviation of the sample mean is used, as identified in section 8.3. The t test is used to establish confidence interval of mean as discussed in section 8.3. If the population mean deviates slightly from the design value then provision is also made in the proposed system to accept the work with adjusted payment, as discussed in section 8.3. The detailed procedure for this method is provided in section 8.5.1.

<u>Step-by-step performance evaluation</u>

In case of performance-based quality control, the road section is tested for acceptance after the completion of the work. When a completed road section is rejected due to improper quality it means a loss of resources and time. In most cases, there is a dispute between the contractor and the road agency which sometimes ends in court. To avoid these, it will be better to have a system of quality control after the construction of every layer of pavement, such as a step-by-step method of quality control. Torres-Verdin and McCullough [1991] proposed a methodology of evaluating pavement performance after the completion of every layer using the AASHTO structural number parameter value $\sum a_i D_i m_i$ for the completed layer. However, this method is not compatible with the proposed analytical pavement design system for Bangladesh. Hence, a methodology for quality control during construction is proposed in the present research which uses the critical number of load repetitions that a pavement can sustain before failure ($N_{critical}$) as a control parameter for pavement performance in order to be compatible with the analytical design system. The detailed procedure in this method is presented in section 8.5.2

8.5 Proposed Methods

8.5.1 Performance based quality control

In this method the performance of the pavement is considered the main criterion for acceptance. Pavement performance is measured by evaluating the pavement's load carrying capacity. A structural analysis model together with pavement surface deflection and layer thickness data is used to evaluate pavement capacity in terms of the critical number of load repetitions before failure ($N_{critical}$). The non-destructive Falling Weight Deflectometer (FWD) or Benkelman Beam testing is used to measure surface deflection. A Dynamic Cone Penetration (DCP) test is conducted to evaluate the thickness of the pavement layers. The evaluated pavement capacity is compared with the required one. If it is satisfactory then the pavement section being tested is accepted. Since the pavement performance is selected as the parameter for quality control, the method is named the 'performance based quality control method of pavement construction'.

A flowchart of this method is presented in Figure 8.1 and the detailed steps of this method are described below:

1. The required capacity of the pavement needs to be specified first, so that the evaluated pavement performance can be judged. A small amount of deviation is acceptable in every design. Hence, the level of tolerance should also be specified. The road agency usually specifies the level of tolerance.

2. Then the length of the road to be tested needs to be divided into a number of sections to carry out the stratified random sampling.

3. Then quality control tests are performed in each section using random sampling which involves FWD or the Benkelman beam test for surface deflection and DCP or coring for the layer thickness. The soaked CBR test is also suggested, to better represent the layer

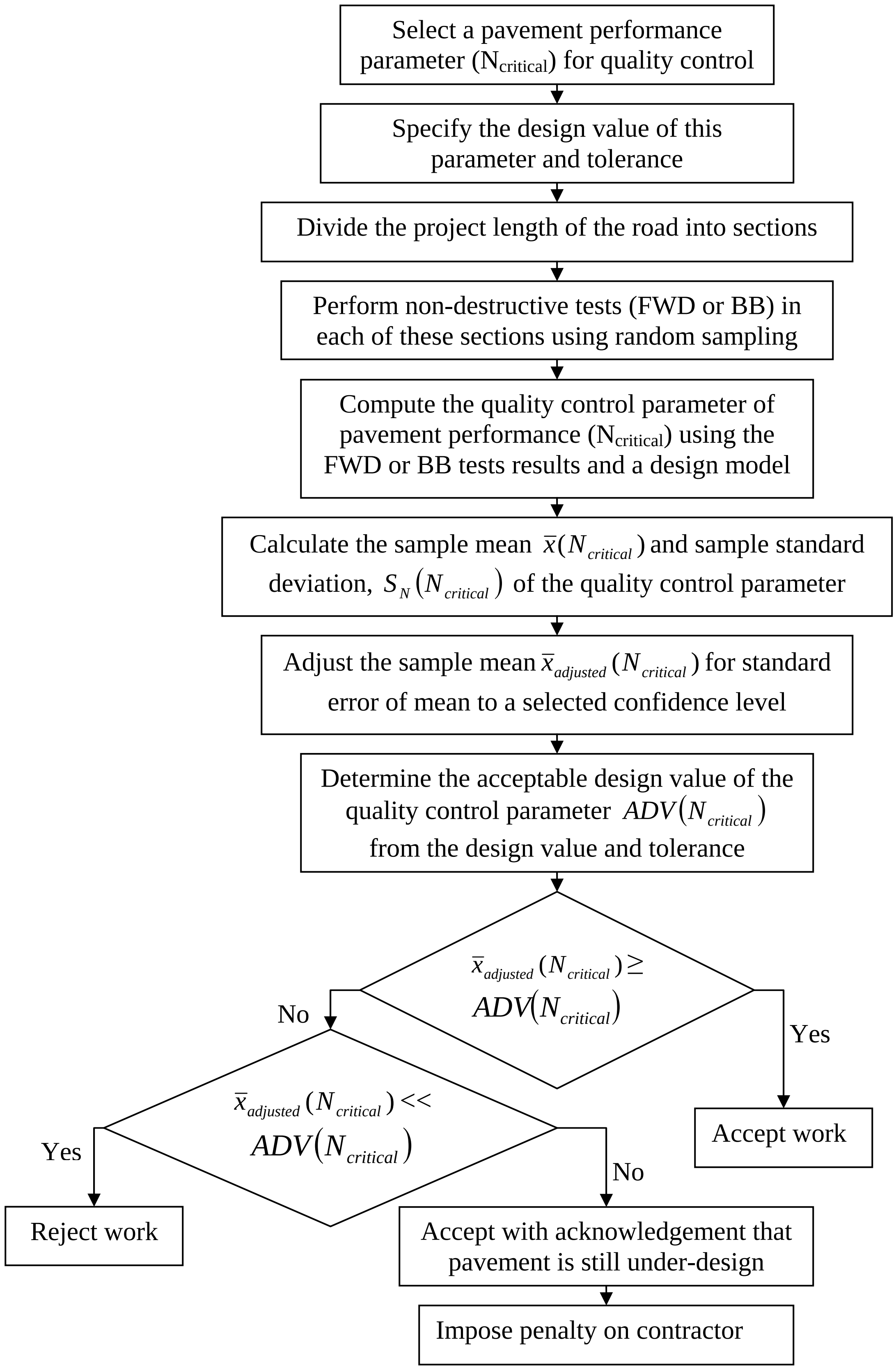


Figure 8.1: Performance based quality control flowchart

strength of pavement.

4. Now using these data from the quality tests and the back calculation procedure of the pavement structural analysis model based on the method of equivalent thickness (MET), the pavement capacity (critical number of load repetitions, $N_{critical}$) at all tests points is determined.

5. Then the mean $\bar{x}(N_{critical})$ and standard deviation, $S_N(N_{critical})$ of the pavement capacity ($N_{critical}$) is determined. However, the mean pavement capacity [$\bar{x}(N_{critical})$] as determined in this process may not represent the actual capacity of the pavement [$\mu(N_{critical})$], since it is based on a small number of tests.

6. The mean pavement capacity should be adjusted to establish a confidence level so that the adjusted pavement capacity is not less the actual pavement capacity. For this purpose, it is required to assume that the distribution of pavement capacity (population) has roughly the shape of normal distribution so that the distribution of a sample mean for a small number of tests can be approximated to follow the student's *t* distribution. Then the following can be concluded from the confidence interval of means of the student's *t* distribution:

If the probability of occurring Type I error, as discussed in section 2.9, that is, the level of significance, is selected as α, then for a one sided confidence level the probability is 1-α that $\bar{x}(N_{critical})$ will differ from $\mu(N_{critical})$ by less than $t_\alpha.\frac{S_N(N_{critical})}{\sqrt{n}}$, where *n* represents sample size. Hence, to assert a probability 1- α that the sample mean pavement capacity is not less than the actual pavement capacity [$\mu(N_{critical})$], the sample mean pavement capacity [$\bar{x}(N_{critical})$] should be adjusted using a one sided confidence interval of mean in the following way:

Adjusted sample mean pavement capacity,

$$\bar{x}_{adjusted}(N_{critical}) = \bar{x}(N_{critical}) - t_\alpha.\frac{S_N(N_{critical})}{\sqrt{n}} \quad \text{--------------------- (8.7)}$$

7. Then the acceptable design value of pavement performance $ADV(N_{critical})$ is calculated from the design value of pavement performance, $DV(N_{critical})$ and tolerance in the following way:

$$ADV(N_{critical}) = DV(N_{critical})\text{-Tolerance} \quad \text{--------------------} \ (8.8)$$

8. The pavement capacity, as determined from the quality control test and adjusted for confidence, i.e. $\bar{x}_{adjusted}(N_{critical})$, is then compared with the acceptable value of pavement performance, i.e. $ADV(N_{critical})$.

9. The work is accepted if the evaluated pavement capacity [$\bar{x}_{adjusted}(N_{critical})$] is greater than the acceptable design value [$ADV(N_{critical})$]. However, if the evaluated pavement capacity is slightly less than the acceptable design value, then the work could be accepted, while acknowledging that the work is under-design and imposing a penalty on the contractor.

8.5.2 Step-by-step Quality Control

The detailed steps of this method are described below with a flowchart, presented as Figure 8.2:

1. In this method, the same parameter as used in performance based quality control system, that is, the critical number of load repetitions that a pavement can sustain before failure ($N_{critical}$) is selected as a suitable measure of pavement performance.

2. Next, the design value of pavement performance $DV(N_{critical})$ and tolerance should be specified. The road agency usually specifies the level of tolerance.

3. Then the acceptable design value of pavement performance $ADV(N_{critical})$ is calculated as follows:

$$ADV(N_{critical}) = DV(N_{critical}) \text{ - Tolerance}$$

4. Then the length of the road to be tested is divided into a number of sections in order to conduct stratified random sampling.

5. After the completion of each layer, quality control tests are performed in each section using random sampling. The quality control tests for thickness involve Dynamic Cone Penetration (DCP) or coring, and for strength, field CBR or soaked CBR.

6. Then statistical analysis is performed to determine the mean, standard deviation, co-efficient of variation and variance of the data of measured layer thickness and strength of the constructed layer.

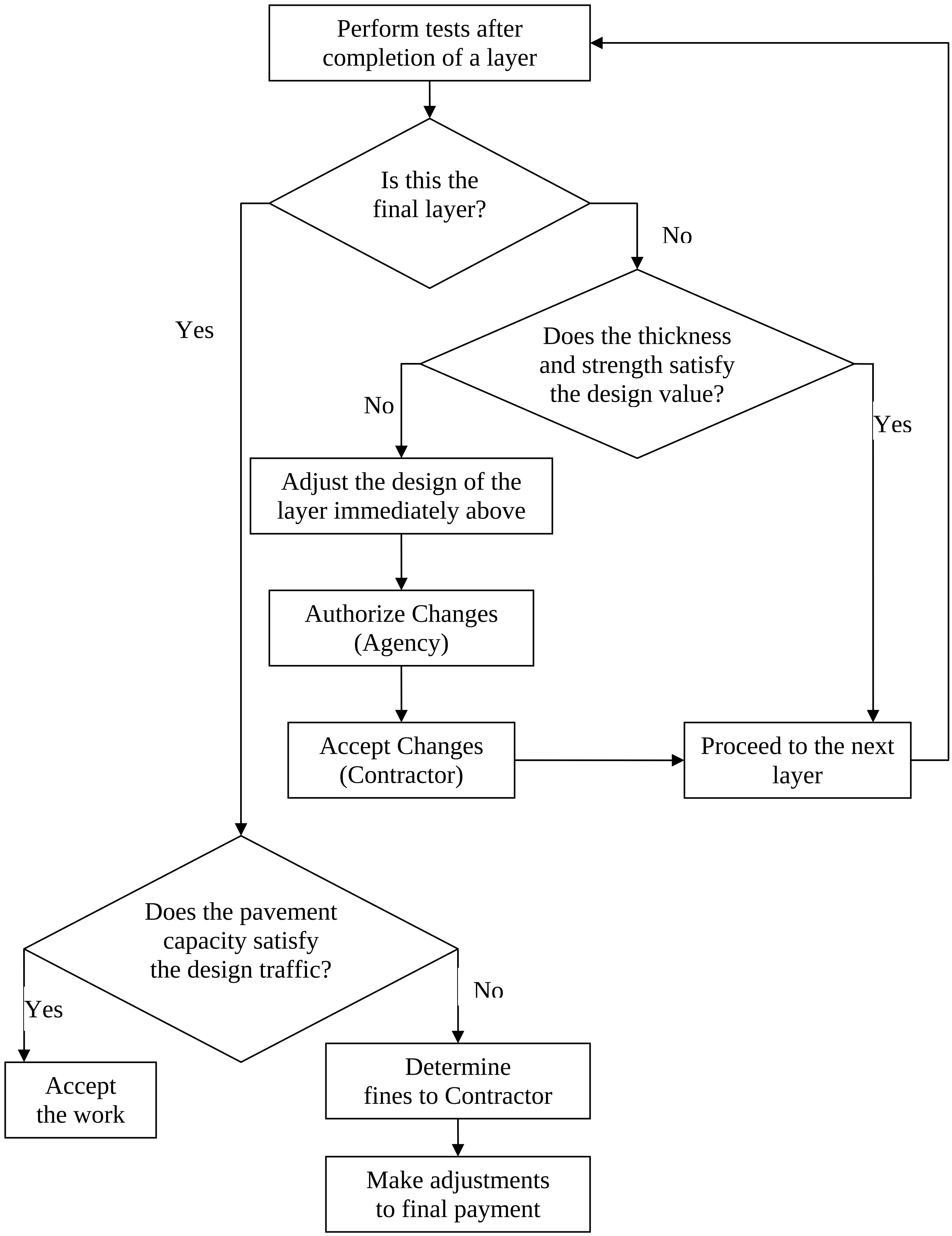


Figure 8.2: Flowchart for step-by-step quality control in pavement construction

7. The mean thickness and strength are usually determined from a small number of tests. Hence, the mean value should be adjusted to establish a confidence level, as discussed in section 8.5.1. If the level of significance i.e. the probability of occurring Type I error is selected as α and hence wants to assert a probability 1- α that the adjusted value is not less than the actual value, then the sample mean value is adjusted for a one sided confidence interval of mean in the following way:

$$\bar{x}_{adjusted} = \bar{x} - t_{\alpha} . \frac{S_N}{\sqrt{n}} \text{ -------------------------------- (8.9)}$$

where t_α represents the normal distribution co-efficient for the selected significance level and n represents the sample size.

8. Then the adjusted mean layer thickness and strength are compared with the design requirements. If satisfactory, then the constructed layer is approved and clearance is given to start the construction of the next layer.

9. However, if design value is not satisfied then this means that there is a deficiency in the completed layer. Then the design of immediate above layer should be modified in such a way that the overall pavement capacity remains the same. The layer thickness or layer strength of the layer immediately above could be modified in this regard. However, modifying the design by changing the layer thickness only should be tried first, since it is difficult to increase the strength of a layer in the field. The design is modified considering the requirements of the pavement in the following ways.

10. The pavement capacity (critical number of load repetitions before failure, $N_{critical}$) is evaluated by performing pavement structural analysis with the adjusted data on the mean layer thickness and strength of the completed layers and the modified design data of the above unconstructed layer. To this end, a pavement structural analysis model based on the method of equivalent thickness design is used in this research. However, any other structural analysis model can be used in this context. The evaluated pavement capacity is compared with the design value [$DV(N_{critical})$]. If it is not satisfactory the design of above unconstructed layer should be further modified. This process is continued until the design value of the pavement performance [$DV(N_{critical})$] is satisfied.

11. The same procedure is followed after the construction of the next layer. In this case also the adjusted mean layer thickness and strength are evaluated and compared with the design requirements. If a deficiency is found in this layer too then the design of the unconstructed layer immediately above also needs to be modified. This process is continued until the construction of the final layer.

12. After final layer is completed, the same procedure as proposed in the performance based quality control method (section 8.5.1) is followed, to evaluate the quality of the overall construction. The quality control tests are performed in the different locations and the adjusted mean pavement capacity [$\bar{x}_{adjusted}(N_{critical})$] is evaluated.

13. The adjusted mean pavement capacity [$\bar{x}_{adjusted}(N_{critical})$] is then compared with the acceptable design value of the pavement's performance [$ADV(N_{critical})$].

14. If it is not satisfied then the work is rejected. However, if the evaluated pavement capacity [$\bar{x}_{adjusted}(N_{critical})$] is slightly short of the acceptable design capacity [$ADV(N_{critical})$], then the work is accepted while acknowledging that the pavement is still under-design and a penalty is imposed on the contractor.

8.6 Summary

The chapter presented the complete development procedure of a quality control system for pavement construction in Bangladesh. In order to do so, the chapter first reviewed the existing methods and techniques and compared them with respect to some predefined criteria to identify a suitable technique for Bangladesh. The FWD or Benkelman beam was identified as a suitable test for evaluating pavement strength and the DCP for evaluating thickness. Stratified random sampling, a t-test, was also identified as suitable for Bangladesh. Then the detailed procedure was set out of the method which was developed in this research, with these identified components, taking pavement performance as the acceptance criterion. In the end the chapter presented the step-by-step quality control process during construction, which was also developed in this research to avoid disputes between the road agency and the contractor. The next chapter considers the development of a quality control procedure for embankment construction in Bangladesh, while examples of the above quality control processes in application are presented in Chapter 10.

Chapter 9 Embankment Construction Quality Control

9.1 Introduction

Embankment construction quality control is an important part of the overall success of the pavement embankment design system of Bangladesh. This is also an important objective of this paper. The basic concept, history and statistical measures of quality control were discussed in the literature review chapter. This chapter begins by reviewing the available methods of quality control in embankment construction in the literature and then compares them with respect to some criteria as defined in methodology chapter with a view to developing a suitable method for Bangladesh. Then the logical development leading to the proposed method is presented. Finally the procedure of the proposed method is described in detail.

9.2 Developed Methods

A very few methods are described in the literature for the quality control of embankment construction. A brief summary of them is presented below.

9.2.1 Quality control of embankment constructed with soft sedimentary rock

Nakamura et al. [1998] reported that soft rocks tend to exhibit slaking and weathering when they are excavated from an in-situ high confining pressure state to open-air stress-free conditions and this weathering weakens them and causes changes in compressibility in soft sedimentary earth fills which often lead to slope failure and/or extensive settlement. They proposed the following Talbot equation to express the grain size distribution of soft rock materials:

$$P = \left(d / D\right)^n \times 100\% \text{ ------------------------------------- (9.1)}$$

where P is the percentage of materials finer than grain size d, D = maximum grain size: n = power number. Nakamura et al. [1998] conducted laboratory experiments to investigate the influence of these factors on soft rock material and proposed a practical and useful method of quality control for embankments constructed with soft rock materials by using the degree of compaction (D_n) defined as the ratio of a compacted dry density (ρ_d) to the dry density of the intact rock particle (ρ_t). That is,

$$D_n = (\rho_d / \rho_t) \times 100\% \text{ ------------------------------------ } (9.2)$$

They reported that the value of maximum compacted dry density (ρ_{dmax}) itself in soft rock material changed greatly as construction progressed, due to particle breakage and was not a good reference for comparison. For this reason compacted dry density (ρ_d) was used in the above equation instead of the generally used ρ_{dmax}. They plotted their test results summary in the following way: first, the strength reduction due to weathering (expressed as the strength ratio of the deviator stresses after weathering process to that before them, R%) was plotted against parameter n for different combinations of degrees of compaction (D_n) and confining pressure (P_v) value (see Appendix F-1a). It was reported in interpreting this graph that the strength reduction could be restrained by increasing the finer content (decreasing n value), by compacting in higher density and by increasing overburden vertical confining pressure. Then the test results were redrawn for a relationship between n and P_v for different representative combinations of R and D_n (see Appendix F-1b). Nakamura et al. [1998] used the second graph to calculate the equivalent thickness of the overburden soil layer necessary to avoid strength reduction (R=100%) by reading the corresponding value of P_v for the determined values of n and D_n.

9.2.2 End-result based embankment construction quality control procedure

The Iowa Department of Transportation (Iowa DOT) proposed an end-result based procedure for embankment construction quality control and assurance, using a Dynamic Cone Penetrometer (DCP) and moisture and density tests [Larsen et al. 2007]. Their embankment quality control policy had three main components: personnel training, quality control testing and test strip construction. Moisture content, lift thickness, density, stability and uniformity were selected as the key criteria to evaluate embankment quality. Control limits were set for moisture content at +/-2.0% of standard proctor optimum moisture content and for density not less than 95% of standard proctor maximum dry density. To check the embankment construction quality with regard to stability and uniformity, a new parameter termed the average DCP index and variations in the DCP index were used. The weighted average method was used to calculate the average DCP index yielding the following equation:

$$\text{Average DCP Index } = \frac{1}{H}\sum_{i=1}^{n} d_i^{\ 2} \text{ ----------------------------- (9.3)}$$

where n represents the total number of blows, d_i represents penetration distance for the ith blow and H represents the depth of the desired test layer. The typical value of the DCP index was suggested for stiff soils 25mm/blow and for soft soils 100mm/blow.

The variation in the DCP index was calculated using the following equation:

$$\text{Variation in DCP Index} = \frac{1}{H}\sum_{i=2}^{n} \left|d_i - d_{i-1}\right| . d_{i-1} \text{ -------------------- (9.4)}$$

where n represents the total number of blows, d_i penetration distance for the ith blow and H depth of the desired test length. It was reported that, for well-compacted fill, a value of variation in the DCP index of around 5-20mm/blow is reasonable, although ideally it should be zero. It was suggested for constructing a test strip to establish proper rolling patterns, the number of roller passes and lift thickness (thickness of a stage of construction in a vertical direction) required to attain acceptable compaction. Random testing was suggested for moisture content, density, DCP index and variation in DCP index. Construction of a new test section was recommended if the type of soil or compaction methods/equipment needed to change.

9.2.3 AASHTO [1996] Guideline

The AASHTO [1996] quality assurance guide specification provided some guidelines for quality control and acceptance decisions in embankment construction. The guide suggested that the contractor is responsible for quality control and should submit a quality control plan for approval before work started. It was also suggested that all required field inspections, sampling and testing to determine the various properties as specified in the specification should be directed by the certified technician. Quality control tests were suggested to be statistically random and testing should be in accordance with the specification. The acceptance limits which were suggested for the evaluation of earthwork in an embankment are presented in Table 9.1.

Table 9.1: Acceptance limits for embankment construction QC [AASHTO, 1996]

Measured Characteristic	Lower Specification Limit	Upper Specification Limit
Moisture Content	Optimum – 2%	Optimum + 2%
Density	95% Reference density	None
Surface Tolerance	- 0.05 ft	+ 0.05 ft

9.3 Comparison of the developed methods

The developed methods were compared with respect to the criteria as described in the methodology chapter with a view to developing or selecting a suitable method for Bangladesh. A brief summary of this comparison is presented below:

Quality control of embankment constructed with soft sedimentary rock: This method reveals a way of quality control of an embankment constructed on soft sedimentary rocks. In Bangladesh, most of the embankments are constructed in these conditions, making the presented method highly appropriate. However, this method has no complete generalized procedure of quality control for embankment construction. Moreover, the method involves numerous laboratory experiments, which are time consuming and expensive. So, it is a matter of debate whether the above mentioned method of quality control is practically appropriate for the pavement-embankment design system of Bangladesh. A conditional incorporation of this method in the quality control system could be considered, letting this method act as a component which can be implemented when sufficient laboratory test facilities are available and when the embankment is constructed of soft sedimentary rock.

End-result based embankment construction quality control procedure: The Iowa DOT suggested a method for the quality control of embankment construction based on some research. The specification of this method is based on the end result; only the finished product must meet the specification and the contractor can use flexible methods and equipment during the construction. The quality control tests of this method required less laboratory work and involved DCP testing which is simple to perform. The test strip construction enables proper rolling patterns and the number of roller passes and thickness of the required lifts to be ascertained. However, this method lacks some essential components of a complete quality control procedure, such as the appropriate quality measures, establishment of a confidence interval of test results and a decision procedure regarding slightly deficient work. Moreover, no provision exists in this method for the quality control of embankments constructed with soft sedimentary rock.

AASHTO [1996] Guidelines: This quality assurance guide specification gives some suggestions and guidelines for quality control in embankment construction. However, the specification guide does not provide any quality control method or procedure for this purpose. Nevertheless, the suggestions of this guide could be used at times in the proposed quality control system for embankment construction in Bangladesh.

9.4 Logical development of the proposed method

According to the criteria defined in the methodology chapter, this method of quality control must be in accordance with the embankment design system, convenient to measure and quantify and easy to understand and implement. The existing literature with respect to these criteria provides no method perfectly suitable for Bangladesh, as discussed in section 9.3. However, some elements mentioned in the literature were identified as suitable (see section 9.3) and used in developing the proposed quality control system of Bangladesh. Nakamura et al. [1998] proposed a quality control procedure which is used as a component of the proposed quality control process and it is suggested that these should be implement when the embankment is constructed with soft sedimentary rock and sufficient laboratory test facilities exist. The Iowa DOT proposed an average DCP index and variation in DCP index, which are also used in the proposed procedure as a suitable measure of stability and uniformity of embankments. The Iowa DOT's recommended provision of test strip construction is also incorporated in the proposed procedure for Bangladesh. The AASHTO guide specifications as presented in Table 9.1 are suggested as useful in determining the quality of conformance for acceptance decisions. Stratified random sampling identified as suitable for highway construction (see section 8.3) is also used in tests sampling for embankment quality control. To find the population mean of quality characteristics, the mean and standard deviation of the sample mean are used since it was identified as a suitable quality measure in section 8.3. The *t*- test is used to establish the confidence interval of the mean.

Provision is also made for adjusting payment in the proposed procedure for reasonable deviation from the acceptance specification. A detailed description of the proposed method is presented in section 9.5.

9.5 Detailed description of the proposed method

The detailed description of the proposed method is given below with a flowchart, as presented in Figure 9.1.

Step 1: First it must be ascertained whether sufficient laboratory facilities exist, since the quality control of embankments constructed with soft sedimentary rock particle requires significant laboratory experiment. Sufficient laboratory facilities are usually provided in every large project. If they are, go to step 2; otherwise, go to step-4.

Step 2: The next step is to determine the characteristics of the fill material. If the embankment is to be constructed with soft sedimentary rock materials then a supplementary quality control programme should be incorporated in the overall quality control method of embankment construction. If embankment construction material is soft sedimentary rock, go to step 3; if not, go to step 4.

Step 3: The steps of the supplementary quality control programme are set out below:

- **i)** First the degree of compaction (D_n), defined as the ratio of a compacted dry density (ρ_d) to the dry density of the intact rock particle (ρ_t), should be determined. Here compacted dry density is used instead of maximum compacted dry density, since for soft sedimentary rock the value of maximum compacted dry density changes as the construction proceeds, as mentioned above.
- **ii)** Then laboratory tests must be conducted to determine the relation between initial gradations, field density and confining pressure with strength reduction. To this end, a cylindrical specimen of rock material is taken from the field and passed through five cycles of a wetting and drying weathering process under a specified constant vertical pressure which is then loaded vertically in a triaxial cell with a lateral confining pressure of 100~300kpa, as recommended by Nakamura et al [1998], to obtain the after weathering shear strength. The tests are to be performed for different test conditions (i.e., for various vertical loads, *n*-values and degrees of compaction). Then the tests results must be plotted as in Appendix F-1a with

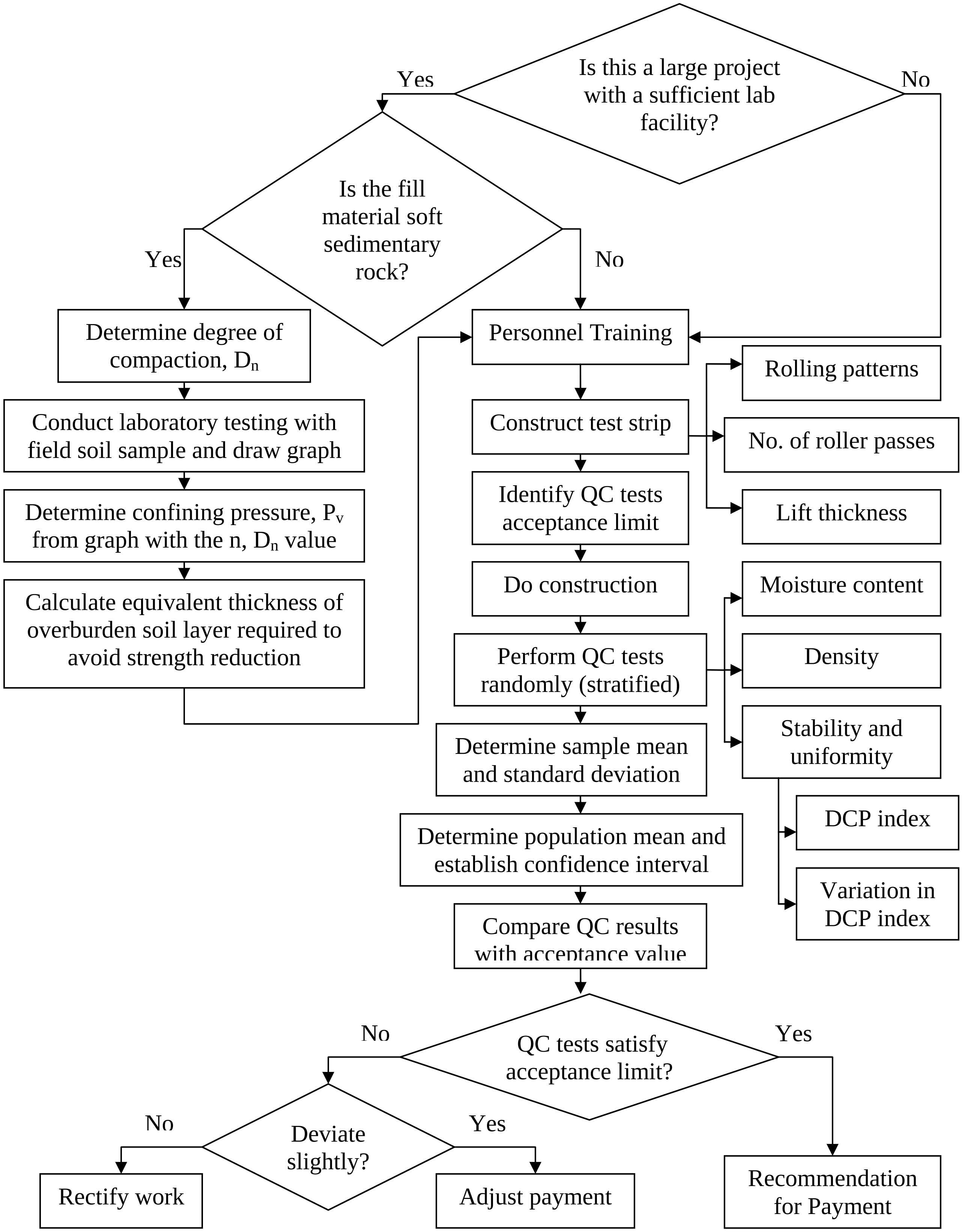


Figure 9.1: Embankment Construction Quality Control Flowchart

the strength ratio (the strength after weathering process to that before) on ordinates for every combination of the testing conditions.

iii) Then the tests results should be redrawn as in Appendix F-1b to establish the relationship between the *n*-value and confining pressure for different representative values of the strength ratio R (80%, 90% and 100%).

iv) Then the value of the confining pressure, $p_{v,}$ necessary to avoid strength reduction for specified conditions of field grain gradation (n) and degree of compaction (D_n) should be determined from the graph.

v) Then the thickness of the surface soil layer which should be overlaid to achieve the confining pressure needs to be determined.

vi) If sufficient soil material for the overburden load is limited then a slight strength reduction could be allowed, provided materials after excavation is kept at n ≤ 0.6 and the degree of compaction at 85-90%.

Step-4: The next step in the quality control process of embankment construction is the training of personnel involved in the supervision. Personnel training is the most important steps in the quality control process since all field inspection, sampling and testing will be carried out by these personnel.

Step-5: Then a test strip is constructed to determine the required rolling pattern, the required number of roller passes and the required lift thickness for achieving acceptable compaction.

Step-6: Then the acceptance limit of quality control tests is identified. The AASHTO quality assurance guide specification [1996] suggested (Table 9.1) acceptance limit for moisture content and density is recommended for this purpose. For DCP index typical value of 25mm/blow for stiff soil and 100mm/blow for soft soils could be used for the acceptance decision. For variation in DCP index, the value 5-20mm/blow could be taken as reasonably accurate for an acceptance decision.

Step-7: The quality control tests are performed after the completion of construction of the specified sections using stratified random sampling. These tests include moisture content, density, stability and uniformity. The average DCP index and variation in DCP index are evaluated to test the stability and uniformity of the constructed embankment.

Step-8: Then the mean and standard deviation of quality control tests results are determined. However, the mean value of the quality characteristics as determined in this process does not represent the actual quality characteristics of the embankment since it is based on a small number of tests.

Step-9: Hence, the mean value of the quality characteristics is adjusted to establish a confidence level so that the adjusted value of the quality characteristics is not less than the actual value. This is done in the following ways for a one sided confidence interval of mean (as discussed in section 8.5.1).

Adjusted sample mean pavement capacity,

$$\bar{x}_{adjusted}(characteristics) = \bar{x}(characteristics) - t_{\alpha}.\frac{S_N}{\sqrt{n}}$$

For a two sided confidence interval (moisture content), the value of confidence limits are determined by adjusting the sample mean in the following way:

$$\bar{x}_{adjusted}(characteristics) = \bar{x} - t_{\alpha/2}.\frac{S_N}{\sqrt{n}} \quad \text{and} \quad \bar{x}_{adjusted}(characteristics) = \bar{x} + t_{\alpha/2}.\frac{S_N}{\sqrt{n}}$$

Step-10: The adjusted mean values of quality characteristics are then compared with acceptance limit.

Step-11: If the quality control tests results (adjusted mean) satisfy the acceptance limit then the work is recommended for payment; otherwise the work is rejected and rectification suggested. However, if the tests results deviate slightly from the acceptance limit then the work could be accepted while adjusting the payment of contractor.

9.6 Summary

The chapter briefly reviewed first the different embankment quality control methods and guidelines available in the literature. Then it critically compared the existing methods to select or develop a suitable method for Bangladesh. Thereafter the logical development of the proposed method was discussed. Finally, the comprehensive quality control procedure which is proposed for embankment construction in Bangladesh was presented in detail with a flowchart. In the proposed procedure, provision is made for the quality control of embankment constructed of soft sedimentary rock material. Provision is also made for personnel training and test strip construction. The typical value and limiting value of quality control tests results are also specified in the proposed procedure. The adjusted payment provision is made in the proposed procedure to ease the acceptance decision when the work slightly deviates from specification. The next chapter will provide an example of applying the proposed quality control procedure of pavement construction using field data from Bangladesh.

Chapter 10 An example applying the quality control process

10.1 Introduction

This chapter considers an example of the proposed quality control procedure in practice, using the field testing data from one of the national highways (N302) of Bangladesh. To begin with, the performance based quality control process is explained with field data and next an example of step-by-step quality control process is presented.

10.2 Application example of the proposed method with field data

The proposed method is illustrated with the field testing data collected from road N302. The pavement capacity is evaluated with field testing data. In the illustrative examples a pavement structural analysis model based on the method of equivalent thickness design is considered to evaluate pavement performance. The field testing data of the same road is also used to illustrate the step-by-step quality control procedure.

10.2.1 Performance based quality control example

In performance based quality control, pavement performance is measured after construction is completed. The detailed steps of this method with field data are described below:

1. The design value of pavement capacity, that is, the critical numbers of load repetitions before failure, is selected as 0.25 million standard axles and tolerance is set as 10% of design value.

2. Then the project length of the road is divided into sections. Random testing data of only one section (the 7^{th} km.) of road N302 is considered in this example.

3. Six series of quality control tests were performed in the selected section. As an FWD is not available in Bangladesh, the Benkelman beam was used to measure the surface deflection. Pavement granular layer (base and sub-base) thickness is calculated from DCP tests data. Coring was done to measure the surface layer thickness. Pavement granular layer strength (CBR) and stiffness (resilient modulus, M_r) values are estimated from the DCP test data. Empirical correlations from the literature are used in this regard [Harison, 1987: Rada and Witczak, 1981]. The subgrade resilient moduli are determined from the subgrade soaked CBR values since it gives a more representative value. The data collected from field testing are shown below:

Location	Benkelman	Surface	Base		Sub-base	
Chainage	beam deflection	Thickness	DCP	Thickness	DCP	Thickness
Km+m	mm	mm	mm/b	mm	mm/b	mm
6+000	0.88	139.00	3.20	205.00	6.18	173.00
6+200	1.06	110.00	3.57	200.00	6.45	200.00
6+400	1.15	142.00	3.61	195.00	6.73	175.00
6+440	1.22	150.00	3.04	170.00	5.75	230.00
6+520	0.64	145.00	1.85	230.00	4.06	130.00
6+600	1.12	141.00	2.05	230.00	5.30	170.00

The pavement layer strength (CBR) and stiffness values as found in different locations of road N302 are presented below:

Location	CBR (DCP)		CBR	Resilient Modulus (M_r)		
	Base	Sub-base	Sub-grade	Base	Sub-base	Sub-grade
km.				MPa	MPa	MPa
Ch 6+000	90.00	38.00	4.49	336.34	103.36	44.90
Ch 6+200	80.00	37.00	3.84	298.97	100.64	38.40
Ch 6+400	78.00	35.00	2.50	291.50	95.20	25.00
Ch 6+440	98.00	42.00	2.68	366.24	114.24	26.80
Ch 6+520	100.00	70.00	5.34	373.72	190.41	53.40
Ch 6+600	100.00	50.00	2.92	373.72	136.01	29.20

4. Then a pavement structural analysis model as developed in this research based on the method of equivalent thickness is used to determine the pavement load carrying capacity with thickness and stiffness (resilient modulus) data measured at different locations of road N302. In determining the overall resilient modulus of the granular layer the proportional average of base and sub-base resilient modulus (according to thickness) is used. The surface deflection data are used to calibrate the model for an acceptable set of surface modulus data. In this calibration process, the surface modulus values continued to change until the model predicted deflection corresponded well with the measured surface deflection. Then the number of load repetitions that the pavement can sustain before cracking failure ($N_{cracking}$) and the number of load repetitions that it can sustain before rutting failure ($N_{rutting}$) are determined. The minimum of these two values, that is, the critical number of load repetitions that the pavement can sustain before failure ($N_{critical}$) is then determined. The data used in the analysis model and the pavement capacities found at different locations of N302 road are presented below:

Location	Thickness		Resilient Modulus			$N_{cracking}$	$N_{rutting}$	$N_{critical}$	Model predicted Deflection
	Surface	Granular	Sub-grade	Granular	Surface				
Km+m	Mm	mm	kpa	kpa	kpa	msa	msa	msa	mm
6+000	139.00	378.00	44.90	229.72	1100.0	0.2192	0.5360	0.2192	0.88
6+200	110.00	400.00	38.40	199.81	1150.0	0.1299	0.3110	0.1299	1.06
6+400	142.00	370.00	25.00	198.66	1700.0	0.3633	0.2466	0.2466	1.15
6+440	150.00	400.00	26.80	221.34	650.0	0.1294	0.2047	0.1294	1.22
6+520	145.00	360.00	53.40	307.52	1200.0	0.6310	1.0078	0.6310	0.64
6+600	141.00	400.00	29.20	272.69	650.0	0.2404	0.2716	0.2404	1.12

5. Then a statistical analysis of pavement performance data, as found in different locations of road N302, are made, to calculate the mean, standard deviation, co-efficient and variance of the variations. The result of this statistical analysis is presented below.

Quality control parameter	Mean $\bar{x}(N_{critical})$	Standard deviation $S_N(N_{critical})$	Co-efficient of variation $Cov(N_{critical})$	Variance $V(N_{critical})$
	msa	msa		
Pavement load carrying capacity ($N_{critical}$)	0.27	0.19	70.04	0.0347

6. Since the mean pavement capacity is determined from a small number of samples (*n* = 6), it should be adjusted, as discussed in section 8.5.1, to establish a confidence level. If the level of significance, that is, the probability of Type I error occurring, is selected as 5%, that is, if a 95% confidence level is wanted, then the sample mean pavement capacity for a one sided confidence interval of mean is adjusted in the following way:

$$\bar{x}_{adjusted}(N_{critical}) = \bar{x}(N_{critical}) - t_{0.05}.\frac{S_N(N_{critical})}{\sqrt{n}}$$

$$= 0.27 - 1.645 \times \frac{0.19}{\sqrt{6}} = 0.141 \text{ msa}$$

where $t_{0.05}$ = normal distribution co-efficient for the selected significance level = 1.645

7. Then the acceptable design value of pavement performance $ADV(N_{critical})$ is determined from the design value of pavement performance, $DV(N_{critical})$ and tolerance, as follows:

$$ADV(N_{critical}) = DV(N_{critical})\text{-Tolerance}$$
$$= 0.25 - (0.25 \times 10\%) = 0.225 \text{ msa}$$

8. The adjusted mean pavement capacity $\bar{x}_{adjusted}(N_{critical})$ is then compared with the acceptable design value of pavement performance $ADV(N_{critical})$, as follows:

Here, $\bar{x}_{adjusted}(N_{critical}) = 0.141$ msa $\quad ADV(N_{critical}) = 0.225$ msa

That is $\bar{x}_{adjusted}(N_{critical}) << ADV(N_{critical})$

Since the constructed pavement capacity is very much less than the acceptable design value, the work is rejected.

10.2.2 Step-by-step quality control example

In the step-by-step quality control method, quality control tests are performed after the construction of each layer. In this illustrative example a pavement is designed first for the required capacity and then a quality check is performed after the sub-base layer is constructed. The field testing data of subgrade and sub-base which were collected from six locations in road N302 is considered for this example. The detailed steps of this example are presented below:

1. The required capacity of pavement design is selected as 0.5 million standard axles and the required tolerance level is specified as 10% of design value. A structural analysis model based on the method of equivalent thickness design is used in this research to design a pavement for such a required capacity. The final design model is shown below:

Thickness				Strength (Resilient Modulus)				
Surface	Base	Sub-Base	Total Granular	Surface	Base	Sub-Base	Average Granular	Subgrade
mm	mm	mm	mm	MPa	MPa	MPa	MPa	MPa
150.00	150.00	200.00	350.00	1500.0	350.00	150.00	235.70	38.00

f_1	he_2	R	ε_t	f_2	he_3	ε_c	ε_t (std ld)	$N_{cracking}$	ε_c (std ld)	$N_{Rutting}$	$N_{critical}$
	m	m			m			MSA		MSA	MSA
1.09	0.303	278.0	0.00027	0.798	0.92	0.00061	0.00024	0.5013	0.00054	0.5061	0.5013

2. The project length of the road is next divided into sections. Random test data collected from only one section (7th km) of road N302 are considered in this example.

3. The quality control tests were performed randomly after the completion of sub-base layer. Pavement granular layer (sub-base) thicknesses are calculated from DCP tests data. Pavement granular layer strength (CBR) and stiffness (resilient modulus, M_r) values are estimated from the DCP test data. The subgrade resilient moduli are determined from the subgrade soaked CBR values. The layer thickness and stiffness data of the subgrade and sub-base layer of road N302 as determined from quality control tests are presented below:

Location Chainage	Sub-Base			Subgrade	
	Thickness	CBR	Modulus	CBR	Modulus
Km + m	mm		MPa		MPa
6+000	173.00	38.00	103.36	4.49	44.90
6+200	200.00	37.00	100.64	3.84	38.40
6+400	175.00	35.00	95.20	2.50	25.00
6+440	230.00	42.00	114.24	2.68	26.80
6+520	130.00	70.00	190.41	5.34	53.40
6+600	170.00	50.00	136.01	2.92	29.20

4. Then a statistical analysis was performed to determine the mean, standard deviation, co-efficient of variation and variance of thickness and strength data of the sub-base and subgrade layer. The analysis results are shown below:

Quality control parameter	unit	Mean $\bar{x}$	Standard deviation S_N	Co-efficient of variation *Cov*	Variance
Sub-base thickness	mm	179.67	33.39	18.58	1114.67
Sub-base strength	MPa	123.31	35.91	29.12	1289.39
Subgrade strength	MPa	36.28	11.31	31.16	127.83

5. The mean thickness and strength are determined from a small number of tests (n = 6). Hence, the mean value should be adjusted to establish a confidence level, as discussed in section 8.5.2. If the level of significance or the probability of Type I error occurring is selected as 5% and hence wants to establish a 95% confidence level that the adjusted value is not less than the actual value, then the sample mean values are adjusted for a one sided confidence interval of mean in the following way:

$$\bar{x}_{adjusted} = \bar{x} - t_{0.05} \cdot \frac{S_N}{\sqrt{n}}$$

where $t_{0.05}$, normal distribution co-efficient for the selected significance level = 1.645 (Appendix B-2) and sample size, n = 6

For sub-base thickness, $\bar{x}_{adjusted}\,(subbasethicknes) = 179.67 - 1.645 \times \dfrac{33.39}{\sqrt{6}}$ = 157.245 mm

For sub-base strength, $\bar{x}_{adjusted}\,(subbasestrength) = 123.31 - 1.645 \times \dfrac{35.91}{\sqrt{6}}$ = 99.197 MPa

For subgrade strength, $\bar{x}_{adjusted}\,(subgradestrength) = 36.28 - 1.645 \times \dfrac{11.31}{\sqrt{6}}$ = 29.936 MPa

6. Then the adjusted mean layer thickness and strength of the sub-base and subgrade layer are compared with design requirements. The corresponding field evaluated (adjusted mean) and design values of the sub-base and subgrade layer are shown below.

Quality control parameter	unit	Field evaluated value $\bar{x}_{adjusted}$	Design value
Sub-base thickness	mm	157.245	200.00
Sub-base strength	MPa	99.197	150.00
Subgrade strength	MPa	29.936	38.00

7. The field evaluated layer thickness and strength of the sub-base and subgrade layer are less than the design value. Hence, there is a deficiency in the sub-base and subgrade construction. The design of the base layer needs to be adjusted to overcome this deficiency. Base layer thickness or strength can be changed to ensure this. Since it is difficult to increase strength in the field, the first trial is made, increasing only the thickness of the base layer. First, the thickness of the base layer is increased by 75 mm. Hence,

New thickness of base layer = 225 mm (1st trial)

8. Then the pavement structural analysis is performed again with the data on the new base layer design thickness (225 mm) and the field test data (adjusted mean) of the sub-base and subgrade layer. The base strength and surface layer data are not changed. The structure analysis results and the pavement load carrying capacity, as found from the analysis, are presented below.

Thickness				Strength (Resilient Modulus)				
Surface	Base	Sub-Base	Total Granular	Surface	Base	Sub-Base	Average Granular	Subgrade
mm	mm	mm	mm	MPa	MPa	MPa	MPa	MPa
150.00	225.00	157.25	382.25	1500.0	350.00	99.20	246.83	29.94

f_1	he_2	R	ε_t	f_2	he_3	ε_c	ε_t (std ld)	$N_{cracking}$	ε_c (std ld)	$N_{Rutting}$	$N_{critical}$
	m	m			m			MSA		MSA	MSA
1.09	0.300	285.1	0.00026	0.789	1.05	0.00060	0.00023	0.5566	0.00054	0.5187	0.5187

9. Since the model evaluated pavement capacity (0.51 msa) with the new trial base layer thickness is slightly above the design value (0.5 msa), the first trial thickness of the base layer is acceptable for the new design thickness.

10. The new design of the pavement base layer thickness thus = 225.00 mm

11. After constructing the base layer with the modified design, the same quality control procedure is implemented again. If any deficiency is found in the base layer, the design of surface layer must be changed.

12. After final layer is completed, the performance based quality control procedure, as described in section 10.2.1, is followed to evaluate the quality of the overall construction.

13. If any deficiency is found in the final layer then the work is rejected. However, if the deviation is slight the work can be accepted with a penalty imposed on the contractors.

10.3 Summary

The chapter presented an example of applying the quality control process of Bangladesh for pavement construction. The field testing data of road N302 was considered in this regard. Both the performance based quality control process and the step-by-step quality control process were explained with data from the field. The next chapter will consider the overall discussion of this research, following which Chapter 12 will conclude this study.

Chapter 11 Discussion

11.1 Introduction

The aim of this research was to quantify the variability in design data and associated risk in the pavement embankment design system of Bangladesh. The study also aimed to incorporate a quality control system in this system, with a view to reducing the variability in the design data. To this end, pavement and embankment design parameters were identified first and data were collected for these parameters from field and laboratory testing at different locations on some of the country's representative roads. The data were found to be greatly variable. A procedure was developed for Bangladesh to quantify the risk in pavement and embankment design associated with variability in data. Since the pavement and embankment will act as an integral system, an overall risk quantification methodology was also developed. Then the overall risk of one of the roads of Bangladesh was quantified according to the proposed method. A methodology for pavement and embankment construction quality control was also developed in this research. All the developed methods were presented to some representative engineers of the roads and highways department of Bangladesh and their opinion was studied, since they are the ones who will ultimately use these developed methods. Some recommendations were also identified which would reduce the risk in design and performance of pavement and embankment. This chapter discusses the research progress and findings so far under the following headings:

- Variability in design data
- Quantification of pavement design risk
- Quantification of embankment design risk
- Quality control process for pavement and embankment construction
- Applicability of the proposed procedures
- Recommendations to reduce the risk in design and performance
- Further research recommendations

11.2 Variability in design data

To find the variability in design parameters, data were collected from different locations on four representative roads of Bangladesh. A database was developed with the collected data and statistical analysis was performed. The statistical parameter mean, median, standard deviation, co-efficient of variation, variance and range were considered in this regard. In traffic data, around 50% of annual daily traffic was found in the traffic category of trucks, of which 40%-60% are medium trucks. The axle load of the medium trucks was also found to play the greatest part in all traffic (average EALF = 7.205). The co-efficient of variation (COV) of axle load for large and medium trucks varied from 30% to 50%. Average variability was found in the pavement layer thickness data, with the COV of most layers ranging between 10% and 30%. However, significant variation was found in the pavement layer strength data with COV ranging from 0.63% to 89.34%. For the road section at hand the Dynamic Cone Penetration data were highly variable and it was felt necessary to consider the soaked CBR data to determine the pavement layer strength. The COV of the embankment slope stability design data varied from 20% to 50%. The embankment settlement data were found to be much too variable. The COV of the secondary compression index in most cases exceeded 100% and the COV of soil dry density in many layers exceeded 50%. The compression index, swell index, modulus, saturated density and initial void ratio data were also found highly variable with COV ranges between 30% and 50%. Since the variability in design data was found significant, a methodology was required for the design system of Bangladesh to quantify the risk associated with it.

11.3 Quantification of pavement design risk

Given this considerable variability, the consequent risk in pavement design had to be quantified. Some methods of risk quantification for pavement design were available in the literature but they were not suitable for Bangladesh on account of the design methods being proposed for the country and its prevailing conditions (the available data and the prevailing failure mode), as discussed in chapter 5. As a result, the present research developed a method of quantifying pavement risk more suitable for the design system of Bangladesh. Since the analytical design method was being proposed for Bangladesh, the variance in pavement performance was calculated using a pavement structural analysis model. Such a model was developed under a separate component of the Bangladesh Pavement Design Project. In determining the variance in pavement performance prediction a sensitivity analysis of the performance prediction model was performed for all performance prediction parameters and then the first order second moment theory was applied. The pavement design risk of road N302 was determined using the proposed method on the basis of field test data, as presented in Chapter 7. The reliability design factor for 75% design reliability level was found to be 1.86. This means that to get 75% reliability in the design, the pavement should be designed for a capacity of 1.86 times the predicted traffic. The results of the proposed method were compared with those of Noureldin et al. [1994] (a detailed calculation is given in Appendix B), which is based on AASHTO's empirical pavement performance model. To calculate the risk with this method, the pavement layer strength data were required to convert to a pavement layer co-efficient using AASHTO's presented graph and chart [1993] (see Appendix C-1 to C-3). The reliability design factor for a 75% design reliability level with Noureldin et al.'s proposed method [1994] was found to yield 1.78, which is slightly less than the value (1.86) found with the proposed method. The reason for this is the non consideration of systematic error (variance of mean due to the limited number of tests) and variance of some traffic parameters (such as axle per truck and growth rate) in the latter method. Moreover, Noureldin et al. [1994] considered the co-efficient of variation of Marshall Stability in determining the co-efficient of variation of the layer co-efficient a_1, whereas the co-efficient of variation of resilient modulus was considered in the present proposed method to determine the variation in surface layer strength. This was discussed more detailed in section 7.2.2.

The proposed method is suitable for incorporating desired reliability in a design. But it is sometimes necessary to determine the reliability of an existing pavement or a predesigned pavement. For this purpose, an alternate method was developed in the present research based on first order second moment theory, as presented in section 5.3.3. To compare the alternative method of this research with the proposed method a back analysis process was followed, where a pavement was designed for a capacity of 1.86 times the predicted traffic (i.e. a 75% reliability level) and then a risk analysis was performed with the alternate method (a detailed calculation is given in Appendix A). The reliability of the design was found to be 66.2%. The differences in results are due to differences in the concepts of reliability determination in the procedures of these two methods. However, good agreement was expected and found. From the two comparisons above it is clear that the proposed method gives reasonably good results.

With the proposed method, the reliability design factor of four representative roads of Bangladesh for different design reliability levels was determined and is presented in Table 11.1. A graphical representation of them is shown in Figure 11.1.

Table 11.1 Reliability design factor for different design reliability levels

Reliability Level %	Standard Normal Deviate	Reliability Factor, F_R			
		N4 (S_0=0.5626)	N302 (S_0=0.3996)	R301 (S_0=0.1692)	Z3024 (S_0=0.1375)
50	0.000	1.00	1.00	1.00	1.00
55	-0.126	1.18	1.12	1.05	1.04
60	-0.253	1.39	1.26	1.10	1.08
65	-0.386	1.65	1.43	1.16	1.13
70	-0.524	1.97	1.62	1.23	1.18
75	-0.674	2.39	1.86	1.30	1.24
80	-0.841	2.97	2.17	1.39	1.31
85	-1.037	3.83	2.60	1.50	1.39
90	-1.282	5.26	3.25	1.65	1.50

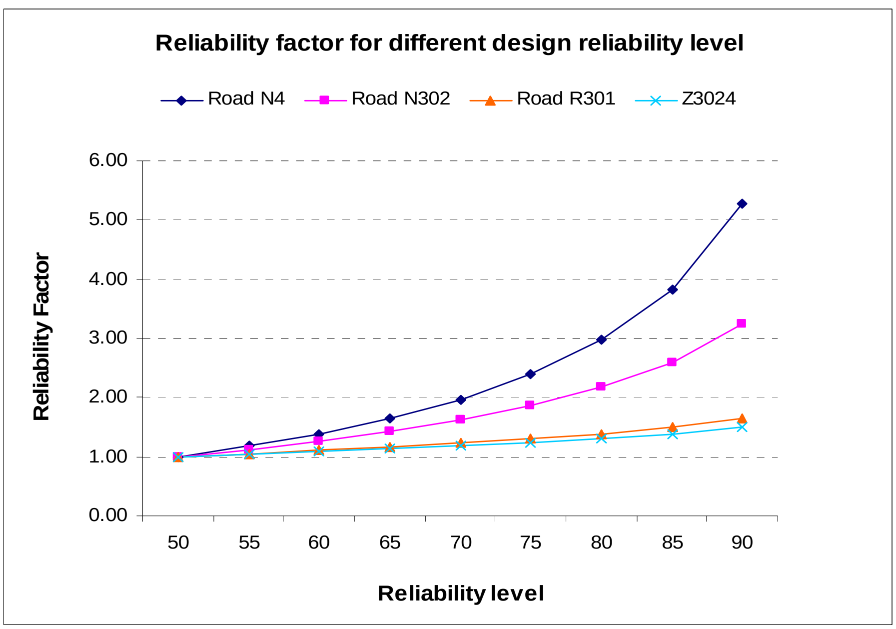


Figure 11.1: Variation of reliability design factor with design reliability level

The reliability factor as shown in Table 11.1 in different roads for different reliability level is significantly different from each other. This is because the pavement layer configuration in different types of road is different. Hence, the sensitivity of pavement performance in different roads is found different. In high standard roads such as N4 and N302 roads the overall variance is mostly influenced by variation in surface strength, base strength and surface thickness and the value of overall variance consequently the reliability factor is significant whereas in case of R301 and Z3024 roads the overall variance is mostly influenced by variation in traffic parameters and the value of overall variance and reliability factor is not so significant as high standard road. The higher value of reliability factor means that the pavement should be designed for more traffic capacity than predicted since, design traffic = reliability factor × predicted traffic.

11.4 Quantification of embankment design risk

Significant variability was also found in the embankment design data. Hence a methodology was required to quantify the risk in embankment design due to the variability in design data which took account of both slope stability and settlement. A number of methods were found in the literature for analysing embankment design risk against slope stability and these were compared with respect to some criteria, such as accuracy, simple computational procedure, capacity to address all the variability of the design data and suitability for the conditions in Bangladesh, so as to select or develop the most suitable method (discussed in some detail in Chapter 6). The first order second moment method was found to satisfy all the required criteria and the method was held to be widely used and well recognized. The detailed procedure for this method was discussed in section 6.2.2. The risk of N302 road embankment of Bangladesh was analyzed with this method, on the basis of field and laboratory test data, as presented in Chapter 7. The reliability of the N302 road embankment against slope stability was found to be 63%. A procedure was also suggested to incorporate the desired reliability in design against slope stability failure. The required value of the mean factor of safety for different desired reliability levels against slope stability was also determined for the N302 road embankment and presented in Table 7.3.

To find a way to quantify the embankment design risk against settlement the existing literature was reviewed, but no method was found to be perfectly suitable for Bangladesh. Hence, a method was developed in this research based on first order second moment theory to quantify the embankment design risk against settlement. The first order second moment theory was considered because it is simple, easy to understand, requires no computer program, capable of accommodating all the variability of settlement and suitable for the conditions in Bangladesh, not least because it uses data which are available there. This theory is widely used and well recognized. Moreover, the same theory is used for analyzing the risk in embankment slope stability. Hence, to maintain consistency the theory was also selected for settlement risk analysis. The detailed procedure of embankment settlement risk analysis with the proposed first order second moment theory was presented in section 6.3.2. The risk of N302 road embankment against settlement was analyzed according to this method and was found to be 85.5%. A procedure was also suggested for incorporating the desired reliability in design against settlement failure. The maximum allowable mean predicted settlement for various desired reliability levels against settlement was also determined for the N302 road embankment and presented in Table 7.4.

11.5 Quantification of overall design risk

In Bangladesh, pavements are usually built on embankments. The embankment acts as an integral part of the pavement. The unsatisfactory performance of embankment as a result of the variability in design data also influences the performance of pavement. Hence, the overall risk of pavement embankment design system of Bangladesh associated with data variability had to be quantified. A methodology was developed in this research to quantify the overall risk based on the special multiplication rules of probabilities, as discussed in some detail in Chapter 6. In the proposed methodology, the overall risk of embankment design was first quantified by multiplying the probabilities of failure of embankment design against slope stability by the probabilities of failure of embankment design against settlement. Then the overall risk of the pavement embankment design system was quantified, considering all possible outcomes which could occur when the pavement and embankment performance were assessed together. The overall risk of the design system of road N302 was quantified with this proposed procedure. The overall risk of N302 road embankment was found to be 90.86% having a 37% risk against slope stability and a 85.5% risk against settlement. Then, considering the risk of the pavement design (25%) and the risk of the embankment (90.86%), the overall risk of a pavement embankment design system for the worst possible outcome was found to be 93.15%.

11.6 Quality control process for Pavement and embankment construction

One of the objectives of this research is to incorporate a quality control system in the design system of Bangladesh to reduce the variability in design data. To this end, a quality control system for pavement design and a quality control procedure for embankment design were developed in this research.

11.6.1 Quality control system for pavement

To develop a suitable quality control procedure for the proposed design system of Bangladesh, the available methods and techniques were first investigated, but no complete procedure was found in the literature. However, significant research work was found on different components of quality control procedure, such as quality control tests, test frequency and sampling, quality measures and performance relationship, which were then compared to identify suitable components for Bangladesh. The Falling Weight Deflectometer (FWD) or Benkelman Beam was identified as suitable methods for evaluating pavement layer strength. Stratified random sampling, it was found, better represents the pavement data. Mean and standard deviation was identified as a suitable measure in determining the population mean of quality characteristics from the sample mean. The *t*-test was also identified as a suitable means for establishing a confidence interval of the sample mean. Since the proposed design for Bangladesh is an analytical pavement design, the evaluation of a critical number of load repetitions before failure by structural analysis was identified as the best measure of pavement performance. Having identified the suitable components, the study developed a complete performance based quality control procedure for Bangladesh; it is presented in section 8.5.1. The proposed procedure included provision for pay adjustment if the work slightly deviated from specification. The study also proposed a step-by-step quality control process for Bangladesh which made provision for a quality check after the construction of every layer to avoid disputes between the road agency and the contractor. The procedure was presented in section 8.5.2. All the developed procedures were verified with field data.

11.6.2 Quality control system for Embankments

The pavement in Bangladesh is usually constructed out of soft sedimentary rock. Hence a special provision was made in the proposed quality control procedure in this regard. The Dynamic Cone Penetrometer (DCP) was identified as simple, easy to use and maintain and cost effective. Consequently, it was suggested that the stability and uniformity of embankment be checked with DCP tests. Personnel training and test strip construction were identified as important components of embankment construction quality control and provision was made for them in the proposed procedure.

11.7 Applicability of the proposed procedure

All the proposed procedures are simple to use, data demanding, applicable with the proposed design system of Bangladesh and, most importantly, suitable for the prevailing conditions in Bangladesh; for instance they use the available data and distress mode. However, to judge the applicability of the proposed procedure, the developed methods were presented in a workshop which twenty-seven representative engineers (different ranks from different regions of Bangladesh) of the roads and highways department (RHD) of Bangladesh attended. After the presentation a questionnaire survey was conducted among them about the suitability and applicability of the proposed method since it is they who will ultimately use it. More than 80% of them responded that the procedure was clear and one which they liked. No one disliked the procedure. A good proportion (75%) of them agreed that the procedure covers all the fundamental design issues and 63% responded that the procedure is simple. Around 70% of the respondents agreed that the procedure is appropriate for Bangladesh and more than 50% disagreed with the view that the procedure is too theoretical. However, a considerable proportion of them (15%-26%) responded ‘neither agree nor disagree’ to some of the questions. This may be due to their not understanding the procedure since the reliability method is not used before in the design system of Bangladesh. After proper training, however, they will understand the procedure and it is hoped that they will respond positively. A summary of the survey results is presented in Table 11.2.

Table 11.2: Questionnaire survey results of the proposed procedure

Questions	**Agree**	**Neither Agree or Disagree**	**Disagree**
The procedure is simple	63%	26%	11%
The procedure is clear	82%	15%	3%
The procedure covers all the fundamental design issues	75%	15%	10%
The procedure is difficult to implement	36%	15%	49%
The procedure is too theoretical	29%	15%	56%
The procedure is inappropriate for Bangladesh	14%	19%	67%
I like the procedure	82%	18%	0%

11.8 Recommendations to reduce the risk in design and performance

1. The accuracy of a pavement design depends on how accurately the design data were collected. The data collection mechanism of Bangladesh is not good. There is no automated vehicle counting system, nor any system to measure the weight of vehicles. Traffic growth rate has never been monitored. The Falling Weight Deflectometer is still not used to evaluate pavement strength. Hence the data used in design do not represent the actual conditions. A proper data collection system considering the above factors is urgently needed to reduce the risk in design.

2. Traffic overloading is a serious problem for Bangladesh and consequently the main cause of failure of most of its pavements. No traffic monitoring system exists there. There is a load limit but it is never enforced. A questionnaire survey was conducted among some representative engineers of the roads and highways department of Bangladesh to elicit their views on traffic overloading and most of them echo the above opinions. The result of questionnaire survey is presented in Table 11.3.

Table 11.3: Questionnaire survey results about traffic overloading

Questions	**Agreed**	**Disagreed**	**Not Answered**
Traffic overloading is monitored	26%	70%	4%
There is a load limit	82%	15%	3%
The load limit is enforced	15%	78%	7%

Hence, a system is urgently needed to reduce the variability in axle load data through monitoring the levels of overloading and enforcing the load limit.

3. Medium trucks were found to have huge axle loads. This is due to the insufficient number of axles, in addition to overloading problem. In Bangladesh medium trucks have two axles. In an axle load survey in road R301 the rear axle load of a medium truck was found to be as high as 25.2 tonnes. Hence, multiple axle trucks are urgently needed in Bangladesh and their introduction is highly recommended.

4. High variability is found in the construction related data. Hence, a construction control mechanism is needed for Bangladesh. In the questionnaire survey 52% of the respondents also agreed that poor quality control mechanisms affect the quality of pavements in Bangladesh. The survey results are presented in Table 11.4.

Table 11.4: Quality control mechanism and the quality of pavement material

What affects the quality of pavement materials in Bangladesh?	Agreed	Disagreed	Not Answered
Quality control mechanisms	52%	7%	41%

5. However, the quality of pavement construction cannot be improved merely by introducing a good quality control procedure. There are other issues, such as the improvement of specifications, improvements in knowledge and technology, allocation of sufficient funds and incorporation of good quality materials. The questionnaire survey result as presented in Figure 11.2 also reveals that there are

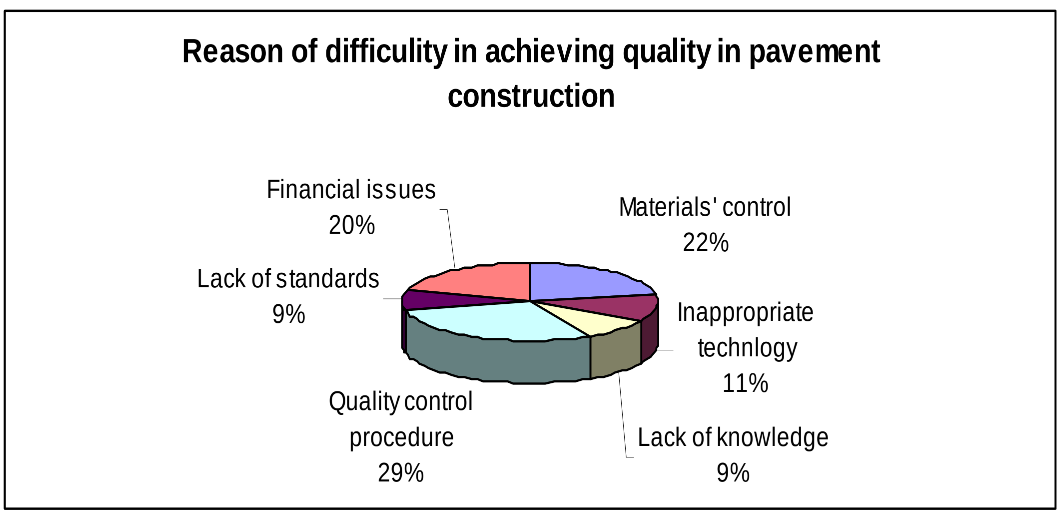


Figure 11.2: Survey results of reasons of difficulty in achieving quality in construction

other issues in addition to the quality control procedures which affect the quality of pavement construction of Bangladesh.

6. A database of all design parameters is needed. It will enhance the variability quantification and risk assessment system of Bangladesh.

7. The country requires a proper risk quantification methodology. The methodology which is developed in this research is strongly recommended for it.

8. The study recommended the training of field staff to obtain a good data.

11.9 Recommendations for further research

The study makes the following recommendations for further research into risk analysis:

- A long term performance study of representative roads of Bangladesh to develop a database on long term performance and then work to determine the design risk based on those long term performance data.
- Development of a performance model for Bangladesh by performing repeated tri-axial tests considering the differences between test conditions and in-service conditions
- A comparative study of as build pavement thickness with as designed of some representative roads of Bangladesh to determine the variability in the construction related data.
- A detailed questionnaire survey to assess the training needs of the personnel involved in the process of data collection and risk analysis.

The study makes the following recommendations for further research on quality control procedures:

- Development of a complete specification and implementation guide for the quality control of pavement and embankment construction in Bangladesh.
- A complete life cycle cost analysis of some of the country's representative pavements and embankments to allow it to develop its own pay adjustment factors.

11.10 Summary

The chapter has discussed the overall variability of the data found in Bangladesh, the procedure for quantifying pavement and embankment design risk and a procedure of construction quality control. These were developed or identified in this research for Bangladesh. It has also outlined the risk found in a typical road in Bangladesh and the general findings of the study. The acceptability of the proposed method among the prospective users was also discussed. The investigation carried out in this study and the consequent findings identified the topics which should be implemented to improve the reliability of design. Some areas were also identified where further investigation is needed to refine the risk quantification process of the design system and quality control procedures.

Chapter 12 Conclusions

This thesis presented a comprehensive methodology to quantify the pavement and embankment design risk associated with variability in data and a procedure for pavement and embankment construction quality control. The proposed methodology and procedures were developed with particular reference to the conditions in Bangladesh. The risk analysis methodology and quality control procedure will enable the design system of Bangladesh to incorporate reliability in design and to improve performance. The following conclusions may be drawn from this study:

- Variability of Data
    1. No comprehensive database was available in Bangladesh.
    2. A database of pavement and embankment design parameter was established by means of results from field and laboratory tests.
    3. The data were found to be extremely variable. The data on the COV of pavement layer thickness varied from 30%-50% and the COV of pavement layer strength data varied from 0.63%-89.34%. Significant variability was found in the data on embankment slope stability with COV ranging from 20% to 50%. At the same time, embankment settlement data showed tremendous variability with COV in some cases exceeding 100%.
    4. Insufficient axle and overloading problems were found in the traffic data
    5. Medium trucks were identified as contributing maximum damage with EALF = 7.205
    6. DCP tests data were found to give highly variable results and are not suitable for pavement layer strength evaluation.
- Quantification of pavement design risk
    1. The investigation of available methods revealed that no method was completely suitable for the pavement design system of Bangladesh to quantify the risk associated with the variability in design data.
    2. A risk quantification method suitable for pavement design system of Bangladesh was developed.

3. In calculating the variance in pavement performance prediction, a pavement structural analysis model based on the method of equivalent thickness was developed and used in the proposed procedure to make it compatible with the proposed pavement design system of Bangladesh.
4. The risk of a typical pavement in Bangladesh was quantified with the proposed method and the reliability factor was found to be 1.86 for a 75% reliability level, which means the pavement should be designed for a capacity of 1.86 times the predicted traffic to achieve 75% reliability in design.
5. The proposed method was verified by comparing with it other methods.
6. An alternative procedure was also proposed to determine the risk of an existing structure.

- Quantification of embankment design risk
  1. It was identified by investigating the developed methods that the first order second moment method is suitable for Bangladesh in calculating the embankment design risk against slope stability.
  2. A method of analysing embankment settlement risk was also proposed in this research, using first order second moment theory, having investigated the available methods.
  3. The risk of a typical road embankment of Bangladesh (N302) was found to be 37% for slope stability and 85.5% for settlement.
  4. A procedure was also presented to incorporate the desired reliability in embankment design for both slope stability and settlement.
- Quantification of overall risk
  1. A methodology was developed to calculate the overall risk of a pavement embankment design system using the special multiplication rules of probabilities.
  2. The overall risk of embankment design of a typical road N302 of Bangladesh was found to be 90.86% and the overall risk of a pavement embankment design system for the worst combination of failure was found to be 93.15%.
- Quality control of pavement construction
  1. A Falling Weight Deflectometer was identified as suitable for evaluating the strength of pavement layers in Bangladesh.

2. To evaluate the pavement layer thickness on a mass scale, DCP tests were also identified as suitable.
3. Stratified random sampling was identified as suitable for the quality control tests of pavement construction in Bangladesh.
4. Mean and standard deviation was identified as a suitable measure in determining the population mean of quality characteristics from the sample mean.
5. The *t*-test was selected as suitable for the quality control system of Bangladesh, to establish a confidence interval of the sample mean.
6. The evaluation of pavement capacity in terms of the critical number of load repetitions that a pavement can sustain before failure with the quality control test results was identified as the best way to establish performance relationship in the country's quality control process.
7. Pay adjustment provision was found to be suitable for Bangladesh
8. A performance based quality control process was developed in this research, using the identified suitable components of a quality control process for the country, as mentioned above.
9. A quality control process after the completion of every layer was also identified as suitable for Bangladesh to avoid disputes between the road agency and contractors.

- Quality control of embankment construction
  1. Moisture content, density, stability and uniformity tests were identified as suitable quality control tests for an embankment.
  2. DCP index and variation in the DCP index were selected as suitable measures for the stability and uniformity of an embankment
  3. The provision of test strip construction to determine the required rolling patterns, number of roller passes and lift thickness was identified as suitable for embankment construction quality control
  4. A form of pay adjustment provision was identified as suitable for the quality control system of Bangladesh, since it encourages a better product and helps in resolving disputes.

- Applicability of the proposed procedures
  1. All the proposed procedures are simple, data demanding, suitable for the conditions in Bangladesh and applicable to the proposed design system.
  2. The proposed procures was also liked by the prospective users of these procedures since almost 70%-80% of them responded positively in a questionnaire survey.
  3. A training programme will increase the applicability of the proposed procedures since a reasonable proportion (15%-26%) of the respondents avoided giving exact answers.
- Need for improvement in the following areas to reduce the risk
  1. Control mechanism for data collection
  2. Control mechanism for traffic
  3. Construction control mechanism
  4. Database and specifications
  5. Strategy for managing road maintenance
  6. Technology and skills in terms of training
- Need for work on the following projects:
  1. A long term pavement performance study of representative roads of Bangladesh.
  2. A detailed questionnaire survey to assess the training needs
  3. A complete life cycle cost analysis of different representative roads to develop Bangladesh's own pay adjustment factors.
  4. A comparative study of the specification guide and implementation manuals for quality control from different highway agencies, in order to develop a good standard for Bangladesh.

REFERENCES

AASHTO. [1993]. AASHTO Guide for Design of Pavement Structures. American Association of State Highway and Transportation Officials, Washington, D.C.

AASHTO. [1986]. AASHTO Guide for Design of Pavement Structures. American Association of State Highway and Transportation Officials, Volume-2, Washington, D.C.

AASHTO. [1996]. Implementation Manual for Quality Assurance - A Report of the AASHTO Highway Subcommittee on Construction, American Association of State Highway and Transportation Officials, Washington, D.C.

AASHTO. [1996]. Quality Assurance Guide Specification - A Report of the AASHTO Highway Subcommittee on Construction, American Association of State Highway and Transportation Officials, Washington, D.C.

ABU-FARSAKH, M. Y., NAZZAL, M. D., ALSHIBLI, K. AND. SEYMAN, E. [2005]. "Application of Dynamic Cone Penetrometer in Pavement Construction Control." Transportation Research Record 1913, Transportation Research Board, National Research Council, Washington, D.C., pp. 53-61.

AGRAWAL, G. AND. ALTSCHAEFFL, A.G. [1991]. "Effect of Water-Content Variability in Design of Clay Embankments." Journal of Geotechnical Engineering, Vol.117, N0.4, April, 1991, American Society of Civil Engineers, pp 673-683.

ALONSO, E. [1976]. "Risk Analysis of Slopes and Its Application to Slopes in Canadian Sensitive Clays." Geotechnique, Vol. 26, Issue.3, pp.453-72.

AL-QADI, I. L., LAHOUAR, S. AND. LOULIZI, AMARA. [2003]. "Successful Application of Ground-Penetrating Radar for Quality Assurance-Quality Control of New Pavements." Transportation Research Record 1861, Transportation Research Board, National Research Council, Washington, D.C., pp. 86-97.

AL-QADI, I. L., LAHOUAR, S., JIANG, K., MCGHEE, K. K. AND. MOKAREM, D. [2005]. "Accuracy of Ground-Penetrating Radar for Estimating Rigid and Flexible

Pavement Layer Thicknesses." Transportation Research Record 1940, Transportation Research Board, National Research Council, Washington, D.C., pp. 69-78.

ALSHERRI, A. AND. GEORGE, K. P. [1988]. "Reliability Model for Pavement Performance." Journal of Transportation Engineering, Vol.114, No.2, May, 1988, American Society of Civil Engineers, pp 294-307.

ASPHALT INSTITUTE [1981]. Thickness Design - Asphalt Pavements for Highways and Streets. Manual Series No. 1 (MS-1), The Asphalt Institute.

ATKINSON, J. [2007]. The Mechanics of Soils and Foundations, 2nd edition, Taylor & Francis, 2 Park Square, Milton Park, Abingdon, Oxon, UK.

ATTOH-OKINE, N. O. AND RODDIS, W. M. K. [1994]. "Pavement Thickness Variability and its Effect on Determination of Moduli and Remaining Life." Transportation Research Record 1449, Transportation Research Board, National Research Council, Washington, D.C., pp 39-45.

AUSTROADS. [2004]. Pavement Design-A Guide to the Structural Design of Road Pavements. ARRB Transport Research, Austroads Incorporated, Sydney, Australia.

AZZOUS, A. S., BALIGH, M. M. AND. LADD, C. C. [1983]. "Corrected Field Vane Strength for Embankment Design." Journal of Geotechnical Engineering, Vol.109, No. 5, American Society of Civil Engineers, pp 730-734.

BARABOSA, M., MORRIS, D., SARMA, S. [1989] "Factor of Safety and Probability of Failure of Rockfill Embankments." Geotechnique, Vol. 39, Issue3, pp 471-483.

BARKER, T. B. [1994]. Quality by Experimental Design, 2nd Edition. Marcel Dekker, Inc., New York.

BARNES, G. E. [1995]. Soil Mechanics-Principles and Practice. MacMillan Press Ltd, Houndmills, Basingstoke, Hampshire.

BHATTACHARYA, G., JANA, D., OJHA, S. AND. CHAKRABORTY, S [2003]. "Direct Search for Minimum Reliability Index of Earth Slopes." Computers and Geotechnics, Vol.30, pp 455-462.

BISHOP, A. W. [1955]. “The Use of the Slip Circle in the Stability Analysis of Slopes.” Geotechnique, London, 5(1), pp 7-17.

BROWN, J. L. [1994]. “Reliability in Pavement Design? Who’s Kidding Whom?” Transportation Research Record 1449, Transportation Research Board, National Research Council, Washington, D.C., pp 26-29.

BROWN, J. L. AND. BUTTLER, L. J. [1970]. “A Recommended Texas Highway Department Pavement Design System User’s Manual.” Texas Highway Department Research Report 123-2, Texas Highway Department.

BRUNTON, J. M., BROWN, S.F. AND. PELL, P.S. [1987]. “Development to the Nottingham Analytical Design Method for Asphalt Pavements.” Proceeding of the 6th International Conference on Structural Design of Asphalt Pavements, Vol. 1, ISAP, 366-377.

BURATI J. L., JR., AND WEED R. M. [2006] “Accuracy and Precision of Typical Quality Measures.” Transportation Research Record 1946, Transportation Research Board, National Research Council, Washington, D.C., pp 82-91.

BURR, I. W. [1976]. Statistical Quality Methods. Marcel Dekker, Inc., New York.

CASTELL, M.A. AND. PINTADO, P. [1999]. “Sensitivity Analysis for Estimation of Pavement Fatigue Life.” Journal of Transportation Engineering, Vol. 125, No. 2, March/April 1999, American Society of Civil Engineers, pp 114-122.

CHEN, D. H. AND. SCULLION, T. [2006]. “Using Nondestructive Testing Technologies to Assist in Selecting the Optimal Pavement Rehabilitation Strategy.” Journal of Testing and Evaluation, Vol. 35, No. 2.

CHEN, D. -H., WANG, J. -N. AND. BILYEU, J. [2001]. “Application of Dynamic Cone Penetrometer in Evaluation of Base and Subgrade layers” Transportation Research Record 1764, Transportation Research Board, National Research Council, Washington, D.C., pp. 1-10.

CHO, S. E. [2007]. “Effects of Spatial Variability of Soil Properties on Slope Stability.” Engineering Geology, Vol. 92, pp 97-109.

CHOWDHURY, R. N. AND. XU, D.W. [1995]. "Geotechnical System Reliability of Slopes." Reliability Engineering and System Safety, Issue 47, pp 141-151.

CHRISTIAN, J. T. [1996]. "Reliability Methods for Stability of Existing Slopes." In Uncertainty in the Geological Environment: From Theory to Practice, Proceedings of Uncertainty'96, American Society of Civil Engineers, Geotechnical Special Publication No. 58, C.D. Shackelford, P.P Nelson, and M.J.S. Roth, eds., pp. 409-418.

CHRISTIAN, J. T., LADD, C. C.AND. BAECHER, G. B. [1994]. "Reliability Applied to Slope Stability Analysis." Journal of Geotechnical Engineering, Vol.120, No. 12, December, 1994, American Society of Civil Engineers, pp 2180-2207.

CHRISTIAN, J. T. [2004]. "Geotechnical Engineering Reliability: How well do We Know What We Are Doing?" Journal of Geotechnical and Geoenvironmental Engineering, Vol.130, No. 10, October, 2004, American Society of Civil Engineers, pp 985-1003.

CHUA, K. H., KIUREGHIAN, A. D. AND. MONISMITH, C. L [1992]. "Stochastic Model for Pavement Design." Journal of Transportation Engineering, Vol.118, No.6, November/December, 1992, American Society of Civil Engineers, pp 769-785.

CRAIG, R. F. [2004]. Craig's Soil Mechanics. Spon Press, Taylor & Francis Group, London and New York.

DAMNJANOVIC, I. AND. ZHANG, Z. [2006]. "Determination of Required Falling Weight Deflectometer Testing Frequency for Pavement Structural Evaluation at the Network Level." Journal of Transportation Engineering, Vol.132, No.1, January, 2006, American Society of Civil Engineers, pp 76-85.

DAS, A. AND. PANDEY, B. B. [1999] "Mechanistic-Empirical Design of Bituminous Roads: An Indian Perspective." Journal of Transportation Engineering, Vol.125, No.5, September/October 1999, American Society of Civil Engineers, pp 463-471.

DAS, B. M. [1997]. Principles of Geotechnical Engineering, 4th Edition, PWS Publishing Company, 20 Park Plaza, Boston.

D' Andrea, R. [2001]. Discussion of "Search Algorithm for Minimum Reliability Index of Earth Slopes." By Hassan and Wolff, Journal of Geotechnical and Geoenvironmental Engineering, Vol.127, No. 2, American Society of Civil Engineers, pp 195-197.

DEMPSEY, B. J., HERLACHE, W. A., AND PATEL, A. J. [2006]. "Climate-Materials-Structural Pavement Analysis Program." Transportation Research Record 1095, Transportation Research Board, National Research Council, Washington, D.C., pp 111-118.

DOBROWOLSKI, J. AND. BRESSETTE, T. [1998]. "Development of Quality Control/ Quality Assurance Specifications by Using Statistical Quality Assurance for Asphalt Concrete Pavements in California." Transportation Research Record 1632, Transportation Research Board, National Research Council, Washington, D.C., pp. 13-21.

DOUGLAS, K. D., COPLANTZ, J., LEHMANN, R. AND. BRESSETTE, T. [1999] "Evaluation of Quality Control / Quality Assurance Implementation for Asphalt Concrete Specification in California." Transportation Research Record 1654, Transportation Research Board, National Research Council, Washington, D.C., pp. 95-101.

DUNCAN, A. J. [1974]. Quality Control and Industrial Statistics. 4th Edition. Richard D. Irwin, Inc., Homewood, IL.

DUNCAN, J. M., NAVIN, M. AND. WOLFF, T. F. [2003]. "Discussion of Probabilistic Slope Stability Analysis for Practice" Canadian Geotechnical Journal, Vol.40 pp 848-850.

EI-RAMLY, H., MORGENSTERN, N. R. AND. CRUDEN, D. M. [2003]. Reply to the Discussion by J.M. Duncan, M. Navin, and T.F. Wolff on 'Probabilistic Slope Stability Analysis for Practice' Canadian Geotechnical Journal, Vol.40 pp 851-855.

EI-RAMLY, H., MORGENSTERN, N. R. AND. CRUDEN, D. M. [2002]. "Probabilistic Slope Stability analysis for Practice." Canadian geotechnical Journal, Vol. 39, pp 665-683.

ELLIS, C. I. [1975]. “Risk and the Pavement Design Decision in Developing Countries.” TRRL Laboratory Report 667, Department of the Environment, Transport and Road Research Laboratory, Crowthorne, Berkshire.

EVDORIDES, H. [2007]. “Notes on Pavement Analysis and Evaluation.” Lecture Note, School of Engineering, University of Birmingham, March 2007.

FENTON, G. A. AND. GRIFFITHS, D. V. [2002]. “Probabilistic Foundation Settlement on a Spatially Random Soil.” Journal of Geotechnical and Geo-environmental Engineering, Vol.128, No. 5, American Society of Civil Engineers, pp 381-390.

FOOTT, R. AND. LADD, C. C. [1981]. “Undrained Settlement of Plastic and Organic Clays.” Journal of the Geotechnical Engineering Division, Proceedings of the American Society of Civil Engineers, ASCE, Vol. 107, No. GT8, August 1981.

FREUND, J. E. [1979]. Modern Elementary Statistics. Fifth Edition, Prentice-Hall, INC., Englewood Cliffs, New Jersey.

GRANT, E. L., AND LEAVENWORTH R. S. [1980]. Statistical Quality Control. 5^{th} Edition. McGraw-Hill, Inc., New York.

GRIFFITHS, D. V. AND. FENTON, G.A. [2004]. “Probabilistic Slope Stability Analysis by Finite Elements.” Journal of Geotechnical and Geo-environmental Engineering, Vol.130, No. 5, American Society of Civil Engineers, pp 507-518.

GRIFFITHS, D. V. AND. LANE, P.A. [1999]. “Slope Stability Analysis by Finite Elements.” Geotechnique, Vol. 48, No.3, pp 387-403.

HARISON, J. A. [1987]. “Correlation between California Bearing Ratio and Dynamic Cone Penetrometer Strength Measurement of Soils.” In Proceedings of Institution of Civil Engineers, Part 2, 1987, 83, Dec., 833-844, Technical note 463.

HARR, M. E. [1987]. Reliability-Based Design in Civil Engineering, McGraw-Hill, New York.

HASOFER, A. M. AND. LIND, N. [1974]. “Exact and Invariant Second-Moment Code Format.” Journal of Engineering Mechanics, Vol. 100, No.1, pp 111-121.

HOUSTON, W. N., MAMLOUCK, M. S. AND. PERERA, R. W.S. [1992]. “Laboratory versus Nondestructive Testing for Pavement Design.” Journal of Transportation Engineering, Vol.118, No.2, March/April, 1992, American Society of Civil Engineers, pp 207-222.

HUANG, Y. H. [1993]. Pavement Analysis and Design. Prentice-Hall, Englewood Cliffs, N.J.

HUDSON, S. B. [1971]. Handbook of Applications of Statistical Concepts to the Highway Industry, Part I – Initial Computations and Applications. Federal Highway Administration, Washington, D.C.

JAMBU, N. [1968]. “Slope Stability Computations.” Soil Mechanics and Foundation Engineering Report, The Technical University of Norway, Trondheim, Norway.

JIANG, Y. J., SELEZNEVA, O., MLADENOVIC, G., AREF, S. AND. DARTER, M. [2003]. “Estimation of Pavement Layer Thickness Variability for Reliability-Based Design.” Transportation Research Record 1849, Transportation Research Board, National Research Council, Washington, D.C., pp 156-165.

JUANG, C. H., JHI, Y. –Y.AND. LEE, D.-H [1998]. “Stability Analysis of Existing Slopes Considering Uncertainty.” Engineering Geology, Vol. 49, pp 111-122.

KHAN, H. R. [2005]. A Prototype Analytical Pavement Design Method for Bangladesh. M. Phil dissertation, School of Engineering, University of Birmingham.

KHER, R. K. AND. DARTER, M. I. [1973]. “Probabilistic Concepts and Their Applications to AASHO Interim Guide for Design of Rigid Pavements.” Highway Research Record No. 466, Highway Research Board.

KILLINGSWORTH, B. M. AND ZOLLINGER D. G. [1995] “Sensitivity Analysis of Input Parameters for Pavement Design and Reliability.” Transportation Research Record 1482, Transportation Research Board, National Research Council, Washington, D.C., pp 111-122.

KIM, H. B. [2006]. “Practical Probabilistic Design Format for Flexible Pavements.” Transportation Research Record 1947, Transportation Research Board, National Research Council, Washington, D.C., pp185-192.

KIM, J.R., KANG, H. B., KIM, D., PARK, D. S.AND KIM, W. J.[2007]. “Evaluation of In Situ Modulus of Compacted Subgrades Using Portable Falling Weight Deflectometer and Plate-Bearing Load Test” Journal of the Materials in Civil Engineering, Vol. 19, No.6, June, 2007, American Society of Civil Engineers, pp 492-499.

KSAIBATI, K., McNAMARA, R., MILEY, W. AND ARMAGHANI, J. [1999]. “Pavement Roughness Data Collection and Utilization.” Transportation Research Record 1655, Transportation Research Board, National Research Council, Washington, D.C., pp 86-92.

KULKARNI, R. B. [1994]. “Rational Approach in Applying Reliability Theory to Pavement Structural Design.” Transportation Research Record 1449, Transportation Research Board, National Research Council, Washington, D.C., pp 13-17.

LACASSE, S. AND. NADIM, F. [1996]. “Uncertainties in Characterizing Soil Properties.” In proceedings of Uncertainty ‘96. Geotechnical Special Publication 58, Vol. 1, pp. 49–75.

LARSEN, B.W., WHITE, D.J. AND. JAHREN, C.T. [2007]. “Embankment Construction QC/QA using DCP and Moisture Control: Iowa Case History for Unsuitable Soils”. Proceedings of the 2007 Mid-Continent Transportation Research Symposium, Ames, Iowa, August 2007.

LEMER, A. C. AND. NOAVENZADEH, F. [1971]. “Reliability of Highway Pavements.” Highway Research Record No. 362, Highway Research Board.

LIANG, R. Y., NUSIER, O. K. AND. MALKAWI, A. H. [1999]. “A Reliability Based Approach for Evaluating the Slope Stability of Embankment Dams.” Engineering Geology, Volume 54, Issue 3-4, October 1999, pp 271-285.

LI, K.AND. LUMB, P. [1987]. “Probabilistic Design of Slopes.” Canadian Geotechnical Journal, Vol.24, pp 520-535.

LUA, Y. J. AND. SUES, R. H. [1996]. “Probabilistic Finite-Element Analysis of Airfield Pavements.” Transportation Research Record 1540, Transportation Research Board, National Research Council, Washington, D.C., pp 29-38.

LIVNEH, M. [2002]. “On the Acceptance Criteria for Statistical Quality Control in Pavement Construction.” Presented for the 2002 Federal Aviation Administration Airport Technology Transfer Conference.

MAHERI, M. AND AKBARI, R. [1993]. “Comparison Between Iran and AASHTO Codes with Considering Overloaded Trucks.” Proceedings 6th International Conference of Civil Engineering, Vol.1, Isfehan University of Technology, Isfehan, Iran.

MALKAWI, A. I. H., HASSAN, W.F., ABDULLA, F.A. [2000]. “Uncertainty and Reliability Analysis Applied to Slope Stability.” Structural Safety, Vol. 22, pp 161-187.

MARSHALL, C., MEIER, R. AND. WELCH, M. [2001]. “Seasonal Temperature Effects on Flexible Pavements in Tennessee.” Transportation Research Record 1764, Transportation Research Board, National Research Council, Washington, D.C., pp. 89-96.

MEHTA, Y. AND. ROQUE, R. [2003]. “Evaluation of FWD Data for Determination of Layer Moduli of Pavements.” Journal of Materials in Civil Engineering, Vol. 15, No.1, February, 2003, American Society of Civil Engineers, pp 25-31.

MCELVANEY, J.AND. SNAITH, M. S. [2002]. Analytical Design of Flexible Pavement. Butterworth-Heinemann, Linacre House, Jordan Hill, Oxford.

MLADENOVIC, G., JIANG, Y. J., SELEZNEVA, O., AREF, S. AND. DARTER, M. [2003]. “Comparison of As-Constructed and As-Designed Flexible Pavement Layer Thicknesses.” Transportation Research Record 1853, Transportation Research Board, National Research Council, Washington, D.C., pp. 165-175.

NAKAMURA, Y., KOJIMA, J., NARITA, K. AND. OHNE, Y. [1998]. “Quality Control of High Embankments Constructed of Soft Sedimentary Rock.” International Symposium on Problematic Soils, 1998, Sendai, Japan, Problematic Soil, Vol 1, pp 701-704.

NATIONAL CO-OPERATIVE HIGHWAY RESEARCH PROJECT (NCHRP). [2004]. Guide for Mechanistic-Empirical Design of New and Rehabilitated Pavement Structures. Final Report, Project 1-37A. TRB, National Research Council, Washington, D.C., March 2004 [e] http//trb.org/mepdg/guide.htm. Last accessed june15, 2009.

NOURELDIN, A. S., ZHU, K., LI, S., HARRIS, D. [2003]. "Network Pavement Evaluation with Failing-Weight Deflectometer and Ground-Penetrating Radar." Transportation Research Record 1860, Transportation Research Board, National Research Council, Washington, D.C., pp. 90-99.

NOURELDIN, A. S., SHARAF, E., ARAFAH, A. AND. AL-SUGAIR, F. [1996]. "Rational Selection of Factors of Safety in Reliability-Based Design of Flexible Pavements in Saudi Arabia." Transportation Research Record 1540, Transportation Research Board, National Research Council, Washington, D.C., pp 39-47.

NOURELDIN, A. S., SHARAF, E., ARAFAH, A. AND. AL-SUGAIR, F. [1994]. "Estimation of Standard Deviation of Predicted Performance of Flexible Pavements Using AASHTO Model." Transportation Research Record 1449, Transportation Research Board, National Research Council, Washington, D.C., pp 46-56.

OH, J., RYU, D., FERNANDO, E. G. AND LYTTON, R. L. [2006]. "Estimation of Expected Moisture Contents for Pavements by Environmental and Soil Characteristics." Transportation Research Record 1967, Transportation Research Board, National Research Council, Washington, D.C., pp 135-147.

PROZZI, J. A. AND HONG, F. [2006]. "Seasonal Time Series Models to Support Traffic Input Data for Mechanistic-Empirical Design Guide." Transportation Research Record 1947, Transportation Research Board, National Research Council, Washington, D.C., pp175-184.

PAPAGIANNAKIS, A. T., BRACHER, M. AND. JACKSON, N.C [2006]. "Utilizing Clustering Techniques in Estimating Traffic Data Input for Pavement Design." Journal of Transportation Engineering, Vol.132, No.11, November 2006, American Society of Civil Engineers, pp 872-879.

PATEL, A. AND. THOMPSON, M. [1998]. “Consideration and Characterization of Pavement Construction Variability.” Transportation Research Record 1632, Transportation Research Board, National Research Council, Washington, D.C., pp. 40-50.

PATHOMVANICH, S. [2000]. Assessment of the Effectiveness of Quality Assurance Construction Specifications for Asphaltic Concrete Pavement. Ph.D. dissertation. University of Florida.

RADA, G. AND. WITCZAK, M.W. [1981]. “Comprehensive Evaluation of Laboratory Resilient Moduli Results for Granular Material.” Transportation Research Record 810, pp 23-33.

RHD [2005]. Benkelman Beam Deflection Survey Manual, Roads and Highways Department, Dhaka, Bangladesh, May 2005, pp 1-8.

RHD [2001]. Manual Classified Traffic Counts Instruction Guide, Roads and Highways Department, Dhaka, Bangladesh, October 2001, pp 14.

ROSENBLUETH, E. [1981]. “Two-Point Estimates in Probabilities.” Applied Mathematical Model, 5(2), pp329-335.

SAARENKETO, T.AND. SCULLION, T. [2000]. “Road Evaluation with Ground Penetrating Radar.” Journal of Applied Geophysics, Issue 43, pp119–138

SADEGHI, J.M. AND FATHALI, M. [2007]. “Deterioration Analysis of Flexible Pavements Under Overweight Vehicles.” Journal of Transportation Engineering, Vol.133, No.11, November 2007, American Society of Civil Engineers, pp 625-633.

SALEM, H. M., BAYOMY, F. M., AL-TAHER, M. G. AND. GENC, I. H. [2004]. “Using Long –Term Pavement Performance Data to Predict Seasonal Variation in Asphalt Concrete Modulus.” Transportation Research Record No. 1896, Transportation Research Board, National Research Council, Washington, D.C., pp. 119-128.

SCHMITT, R. L., RUSSELL J. S., HANNA A. S., BAHIA H. U. AND JUNG G. A. [1998]. “Summary of Current Quality Control/ Quality Assurance Practices for Hot-Mix

Asphalt Construction.” Transportation Research Record No. 1632, Transportation Research Board, National Research Council, Washington, D.C., pp. 22-31.

SCHWARTZ C. W. [2007] “Implementation of the NCHRP 1-37A Design Guide.” Final Report, Vol-1, Summary of Findings and Implementation Plan, Maryland State Highway Administration, 2007. [e]

SHAHJAHAN, M. [2009]. Materials Considered for Pavement and Embankment Design. A thesis submitted to the University of Birmingham for the degree of M. Phil.

SHELL PAVEMENT DESIGN MANUAL - ASPHALT PAVEMENTS AND OVERLAYS FOR ROAD TRAFFIC [1978]. Shell International Petroleum Ltd., London.

SNAITH, M.S., McMULLEN, D., FREER-HEWISH, R. J.AND. SHEIN, A. [1980]. “Flexible Pavement Analysis.” Final Technical Report, U.S. Army Research, Development & Standardization Group, United Kingdom.

SPENCER, E. [1967]. “A Method of Analysis of the Stability of Embankments Assuming Parallel Interslice Forces.” Geotechnique, London, 17(1), pp 11-26

TANG, W., YUCEMEN, M. AND. ANG A.-S. [1976]. “Probability-Based Short Term Design of Slopes.” Canadian Geotechnical Journal, Vol.13, Issue.3, pp 201-215.

TOMLINSON, M. J. [2001]. Foundation Design and Construction. Pearson Education Limited, Edinburgh Gate, Harlow, Essex, England.

TORRES-VERDIN, V. AND McCULLOUGH, B. F. [1991]. “Dynamic Quality Control of Flexible Pavement.” Journal of the Transportation Engineering, Volume 117, American Society of Civil Engineers, pp 23-33.

TRL [1993]. A Guide to the Structural Design of Bitumen-surfaced Roads in Tropical and Sub-tropical Countries, Overseas Centre, Transport Research Laboratory, Crowthorne, Berkshire, United Kingdom.

VARDEMAN, S. B., AND JOBE, J. M. [1999]. Statistical Quality Assurance Methods for Engineers. John Wiley & Sons, Inc., New York.

VENMARCKE, E. [1977]. “Reliability of Earth Slopes.” Journal of Geotechnical Engineering Division, American Society of Civil Engineers, Vol.103, Issue 11, pp 1247-1265.

WADSWORTH, H. M., KENNETH, S. S., AND GODFREY, A. B. [1986]. Modern Methods for Quality Control and Improvement. John Wiley & Sons, Inc., New York.

WEED, R. M. [2006]. “Mathematical Modelling Procedures for Performance- Related Specifications.” Transportation Research Record 1946, Transportation Research Board, National Research Council, Washington, D.C., pp 63-70.

WEED, R. M. [2003]. “Multicharacteristic Performance-Related Specification for Hot-Mix Asphalt Pavement: Complete Development Process.” Transportation Research Record 1861, Transportation Research Board, National Research Council, Washington, D.C., pp. 53-59.

WEED, R. M. [2001]. “Derivation of Equation for Cost of Premature Pavement Failure.” Transportation Research Record 1761, Transportation Research Board, National Research Council, Washington, D.C., pp. 93-96.

WEED, R. M. [1999]. “Practical Framework for Performance-Related Specification.” Transportation Research Record 1654, Transportation Research Board, National Research Council, Washington, D.C., pp. 81-87.

WEED, R. M. [1998]. “A Rational Method for Relating As-Built Quality to Pavement Performance and Value.” Transportation Research Record 1632, Transportation Research Board, National Research Council, Washington, D.C., pp. 32-39.

WOLFF, T. F., DEMSKY, E. C., SCHAUER, J. AND. PERRY, E. [1996]. “Reliability Assessment of Dam and Levee Embankments.” In Uncertainty in the Geologic Environment: From Theory to Practice, Proceedings of Uncertainty ’96, American Society of Civil Engineers Geotechnical Special Publication No. 58, C.D. Shackelford, P.P Nelson, and M.J.S. Roth, eds., pp. 636-650.

WONG, F. S. [1985]. “Slope Reliability and Response Surface Method.” Journal of Geotechnical Engineering, Vol.111, No. 1, American Society of Civil Engineers, pp 32-53.

WU, T. AND. KRAFT, L. [1970]. "Safety Analysis of Slopes." Journal of Soil Mechanics and Foundation Division, American Society of Civil Engineers, Vol.96, Issue 2, pp 609 -630.

XU, B. AND. LOW, B. K. [2006]. "Probabilistic Stability Analyses of Embankments Based on Finite-Element Method." Journal of Geotechnical and Geo-environmental Engineering, Vol.132, No. 11, November, 2006, American Society of Civil Engineers, pp 1444-1454.

YODER, E. J. AND. WITCZAK, M.W. [1975]. Principles of Pavement Design. 2nd Edition, John Wiley & Sons, Inc., New York.

ZAGHLOUL, S., SAEEED, N.A., JASSIM, A.A. AND. RAFI, A. M. [1998]. "End-result Specifications for Warranted Asphalt Pavements." Transportation Research Record 1632, Transportation Research Board, National Research Council, Washington, D.C., pp. 1-12.

ZHANG, Z. AND. DAMNJANOVIC, I. [2006]. "Applying Method of Moments to Model Reliability of Pavements Infrastructure." Journal of Transportation Engineering, Vol.132, No.5, May/2006, American Society of Civil Engineers, pp 416-424.

ZOU, J. –Z., WILLIAMS, D. J. AND. XIONG, W.-L.[1995]. "Search for Critical Slip Surfaces Based on Finite Element Method." Canadian Geotechnical Journal, Vol.32, No.2 pp 233-246.

ZUO, G., DRUMM, E. C. AND MEIER, R. W. [2007]. "Environmental Effects on the Predicted Service Life of Flexible Pavements." Journal of the Transportation Engineering, Volume 133, No. 1, American Society of Civil Engineers, pp 47-56.

APPENDIX A Proposed Alternative Method Example

A.1 Proposed alternative (FOSM) pavement design risk quantification example

The detail steps of the risk analysis procedure with the proposed alternative method with field tests data of road N302 are presented below.

Identification of variables and its tentative variance

1. At first the traffic prediction data and pavement performance prediction data as collected from field testing on road N302 are analyzed statistically for their variability. The detail analysis of data variability is discussed in chapter 4. The summary statistics of that analysis is presented below.

Traffic data

Design period = 10 years

The proportion of different types of vehicle and their equivalent axle load factor as found in road N302 is shown below.

Parameter	Large Truck			Medium Truck		Small Truck		Large Bus		Medium Bus	
	Front	Rear1	Rear2	Front	Rear	Front	Rear	Front	Rear	Front	Rear
Percent of Axles, p_i	1.62	1.62	1.62	19.43	19.43	4.87	4.87	11.12	11.12	12.14	12.14
Equivalent Factor, F_i	0.368	5.960	5.775	0.148	7.205	0.014	0.233	0.145	0.816	0.011	0.097

The statistical parameter of traffic data as found in that road is presented below.

Design factor	ADT_0	Growth Rate, r %	Percent of Truck, T %	Axle load factor, $\Sigma p_i F_i$	Axles per Truck, A
Mean	9303	8	51.44	1.76	2.03
Standard deviation	1395.45	0.008	0.06172	0.615	0.162
CV	15	10	12	35	8
Variance	1947281	0.000064	0.0038	0.3782	0.0264
Systematic error	324546.78	0.00001	0.00063	0.06303	0.00440

The AADT considers the traffic in both directions and the road has only one lane in the design direction. Hence, the direction distribution is taken as 50% and the lane distribution is taken as 100% with no variance. The co-efficient of variation of ADT, growth rate, percent of truck, axle per truck, and axle load factor are assumed since the traffic survey is conducted for a limited period of time. The systematic error is considered due to limited number of tests (as discussed in section 5.3.3).

Pavement performance data

The statistical analysis summary of pavement performance prediction data as collected from field testing of road N302 is shown below.

Design factor	Thickness				Strength (Resilient Modulus)				
	Surface	Base	Sub-Base	Granular	Surface	Base	Sub-Base	Granular	Sub-grade
	mm	mm	mm	mm	MPa	MPa	MPa	MPa	MPa
Mean	137.8	205.0	179.67	384.67	1679.12	289.07	93.72	197.83	27.65
Standard Deviation	14.16	22.80	33.39	57.13	1447.57	31.83	27.29	39.69	2.93
C-efficient of variation	10.27	11.12	18.58	14.85	86.21	11.01	29.12	20.07	10.60
Variance	200.5	520.0	1114.7	3264.4	2095457	1013.1	744.76	1575.7	8.58
Systematic error	33.43	86.67	185.78	544.07	349243	168.85	124.13	262.61	1.43

Systematic error is calculated here by dividing the variance V[*x*] of each parameter with the sample size (n = 6).

Formation of safety factor and its variance

2. The equation for safety factor, as discussed in section 5.3.3, is as follows:

$$Safety\quad Factor,\quad SF\quad = \frac{\mathrm{Pr}\,edicted\quad pavement\quad performance,\quad W_t}{\mathrm{Pr}\,edicted\quad Traffic,\quad w_T}$$

The predicted traffic (w_T) is calculated using the following equation

$$w_T = \left(\sum_{i=1}^{m} p_i F_i\right)(ADT_o)(T)(A)(G)(D)(L)(365)(Y)$$

Where, Traffic growth factor, $G = \frac{1}{2}\left[1+(1+r)^Y\right]$

Now using the mean value of traffic prediction parameter for road N302, the mean predicted traffic is calculated as follows:

$$w_T = \left(\sum_{i=1}^{m} p_i F_i \right) (ADT_o)(T)(A)(G)(D)(L)(365)(Y)$$

$= 1.76 \times 9303 \times 0.5144 \times 2.03 \times [0.5 \times \{1 + (1+0.08)^{10}\}] \times 0.5 \times 1.0 \times 365 \times 10$

$= 49.2$ Million ESA

A structural analysis model based on method of equivalent thickness design is used in this research to design a pavement for the desired performance. The reliability factor found in the proposed first procedure is 1.86 for a 75% reliability level. Hence, a pavement is designed first for a capacity of 1.86×49.2 = 91.5 Million ESA and then risk analysis is performed. The following design of pavement is ascertained for the capacity of 91.5 Million ESA.

Thickness		Strength (Resilient Modulus)			f_1	he_2	R
Surface	Granular	Surface	Granular	Subgrade			
mm	mm	MPa	MPa	MPa		m	m
280.00	600.00	2262.50	450.00	27.65	1.048	0.503	1814.75

ε_t	f_2	he_3	ε_c	ε_t (std ld)	$N_{cracking}$	ε_c (std ld)	$N_{Rutting}$	$N_{critical}$
		m			MSA		MSA	MSA
7.71E-05	0.775	2.12	0.00016	6.85E-05	91.54	0.000144	92.46	91.54

Consequently, the safety factor (SF) of design is:

$$Safety \quad Factor, \quad SF \quad = \frac{\Pr edicted \quad pavement \quad performance, \quad W_t}{\Pr edicted \quad Traffic, \quad w_T}$$

$= 1.86$

The reliability of this design is now calculated with the proposed alternative procedure.

3. To determine the variance in safety factor, sensitivity analysis of safety factor is done for each variable of traffic prediction parameter and pavement performance prediction parameter. This is done by changing each variable one standard deviation above and below while keeping the other variables unchanged and recording the changes in safety factor for each case. Then the ratio of the differences in safety factor and the difference in variable for each parameter is determined. The results of that analysis are presented below.

Parameter	Unit	Average	Standard deviation	Average +1 STD	Average - 1 STD	Safety factor	Safety factor	$\Delta SF/\Delta x_i$
				x_i^1	x_i^2	SF_1	SF_2	
Traffic, ADT_0	Nos.	9303	1395.45	10698.5	7907.6	1.6181	2.1892	0.0002
Growth rate, r		0.08	0.008	0.088	0.072	1.7683	1.9566	11.7737
Percent of truck, T		0.5144	0.0617	0.576	0.453	1.6614	2.1146	3.6707
Axle per truck, A	Nos.	2.03	0.162	2.192	1.868	1.7230	2.0226	0.9226
load factor, Σpi×Fi		1.76	0.615	2.372	1.142	1.3784	2.8628	1.2069
Surface thickness	mm	280.00	14.16	294.16	265.84	2.2374	1.3751	0.0304
Granular thickness	mm	600.00	57.13	657.13	542.87	1.8608	1.2270	0.0055
Surface strength	MPa	2262.5	1447.57	3710.07	814.93	3.4291	0.2173	0.0011
Granular strength	MPa	450.00	39.69	489.69	410.31	2.1011	1.6389	0.0058
Subgrade strength	MPa	27.65	2.93	30.58	24.72	1.8608	1.5931	0.0457

4. The results of this sensitivity analysis that is the divided difference ($\Delta SF/\Delta x_i$) is now squared up and multiplied with the spatial and systematic variance separately. The similar things is done for all parameters and summed up separately for spatial variance and systematic variance. The detail of this analysis is presented below.

Parameter	$\Delta SF/\Delta x_i$	Variance, V (x_i)		$(\Delta SF/\Delta x_i)^2.V(x_i)$	
		Spatial	Systematic	Spatial	Systematic
Traffic, ADT_0	0.0002	1947281	324546.78	0.0815374	0.013590
Growth rate, r	11.7737	0.000064	0.00001	0.0088716	0.001479
Percent of truck, T	3.6707	0.003810	0.00063	0.0513297	0.008555
Axle per truck, A	0.9226	0.026374	0.00440	0.0224473	0.003741
load factor, Σpi×Fi	1.2069	0.378199	0.06303	0.5508708	0.091812
Surface thickness	0.0304	200.567	33.42778	0.1859075	0.030985
Granular thickness	0.0055	3264.399	544.06658	0.1004215	0.016737
Surface strength	0.0011	2095457	349242.8	2.5788387	0.429806
Granular strength	0.0058	1575.657	262.60948	0.0534178	0.008903
Subgrade strength	0.0457	8.583000	1.43050	0.0179242	0.002987
	$\Sigma(\Delta w_t/\Delta x_i)^2.V(x_i) =$			3.6516	0.6086

5. The variance in safety factor (V [SF]) is now calculated in the following way:

$$V[SF] = \sum_{i=1}^{n}\left(\frac{\Delta SF}{\Delta x_i}\right)^2 V[x_i]_{spatial} + \sum_{i=1}^{n}\left(\frac{\Delta SF}{\Delta x_i}\right)^2 V[x_i]_{systematic}$$

= 3.6516 + 0.6086

= 4.2602

So, the variance in safety factor V [SF] = 4.2602

<u>Determination of reliability index and design reliability</u>

From the above calculation, the following is found:

Mean safety factor, E [SF] = 1.86

Variance in safety factor, V [SF] = 4.2602

Standard deviation of safety factor, σ [SF] = √ V [SF] = √4.2602 = 2.064

Now, the reliability index (β) is calculated using the equation 5.7 as follows:

$$\beta = \frac{E[SF] - 1.0}{\sigma[SF]}$$

$$= \frac{1.86 - 1.0}{2.064}$$

= 0.417

<u>Design Reliability</u>

The design reliability is determined from standard normal curve area tables for the value of reliability index.

In this case, the reliability of design is found = 66.2%

APPENDIX B Noureldin et al. [1994] Method Example

B.1 Pavement design risk quantification example: Noureldin et al. [1994] method

The detail steps of the risk analysis procedure with the Noureldin et al. [1994, 1996] method is presented below with Field tests data of road N302.

1. The following formula as derived by Noureldin et al. [1996] is used to estimate the variance in traffic prediction (S_w^2).

$$S_W{}^2 = \frac{\left[(C.O.V.ADT * D_d)^2 + (C.O.V.P)^2 + (C.O.V.L_d)^2 + (C.O.V.TF)^2\right]}{5.3}$$

The variance in traffic prediction is calculated using the same data as used in proposed method in the following ways.

$$S_w^2 = \frac{(0.15)^2 + (0.12)^2 + (0.35)^2}{5.3} = \frac{0.0225 + 0.0144 + 0.1225}{5.3} = 0.0301$$

2. The following formula as derived by Noureldin et al. [1994] using AASHTO's [1993] performance model is selected to estimate the variance in pavement performance prediction (S_N^2).

$$S_N^2 = \overline{COV(MR)}^2 + P_2 \quad \overline{SN}^2 \cdot \overline{COV(SN)}^2$$

Where,

$$P_2 = \text{var}\, iance \quad component \quad of \quad SN \cong \left[\frac{4.065}{\overline{SN}+1} - \frac{1135.57}{\left(0.4 + \dfrac{1094}{(\overline{SN}+1)^{5.19}}\right)^2 \quad (\overline{SN}+1)^{6.19}} \right]^2$$

And

$$\begin{aligned} \overline{SN}^2\,\overline{COV(SN)}^2 \cong\ & \bar{a}_1^2 \bar{d}_1^2 \left[\overline{COV(a_1)}^2 + \overline{COV(d_1)}^2\right] + \bar{a}_2^2 \bar{m}_2^2 \bar{d}_2^2 \left[\overline{COV(a_2)}^2 + \overline{COV(m_2)}^2 + \overline{COV(d_2)}^2\right] \\ & + \bar{a}_3^2 \bar{m}_3^2 \bar{d}_3^2 \left[\overline{COV(a_3)}^2 + \overline{COV(m_3)}^2 + \overline{COV(d_3)}^2\right] \end{aligned}$$

Here, MR = resilient modulus and SN = AASHTO structural number which is calculated using the following formula:

$$SN = a_1 D_1 + a_2 m_2 D_2 + a_3 m_3 D_3$$

3. The layer co-efficient value is read from AASHTO's [1993] graph and chart as presented in Appendix C-1 to C-3 for the corresponding value of field CBR (same CBR data as used in the proposed first method).

The following layer co-efficient value is found for the corresponding asphaltic concrete modulus and granular layer CBR value:

Layer	Field data	Corresponding Layer co-efficient
Asphaltic concrete	Modulus = 1679 MPa	$a_1 = 0.32$
Base	CBR = 77.35	$a_2 = 0.13$
Sub-base	CBR = 34.45	$a_3 = 0.11$

4. The following drainage coefficient values are selected from AASHTO recommended chart as presented in Appendix C-4 to modify the layer co-efficient of untreated base and subbase layer.

Layer	Quality of Drainage	Percent of time pavement structure is exposed to moisture levels approaching saturation	Value of Drainage co-efficient
Base	Fair	5-25%	$m_2 = 0.90$ Range 1.00-0.80
Subbase	Poor	5-25%	$m_3 = 0.70$ Range 0.80-0.60

5. The AASHTO structural number is now calculated using the same layer thickness data (in inch) as used in proposed first method as follows:

$$SN = a_1 D_1 + a_2 m_2 D_2 + a_3 m_3 D_3 = 0.32 \times 5.43 + 0.13 \times 8.07 \times 0.9 + 0.11 \times 7.07 \times 0.7 = 3.2254$$

6. Noureldin et al. [1994] used the following equations to estimate the COVs for AASHTO layer coefficients (a_i):

$$COV(a_1) \cong (0.33 - 0.5) COV \quad of \quad Marshall \quad Stability$$

$$COV(a_2) \cong (0.33 - 0.77) COV \quad of \quad CBR$$

$$COV(a_3) \cong (0.33 - 0.9) COV \quad of \quad CBR$$

To estimate the COVs for AASHTO drainage coefficients (m_i), Noureldin et al. [1994] used the ranges of drainage coefficient values recommended by AASHTO [1993] as presented in Appendix C-4 and the approach described by MS-17 of the Asphalt Institute as follows:

$$COV = \frac{range \cdot 0.3249}{range \quad midpoint} \cdot 100$$

Now, using these relations (midpoint for range) and the same field testing data as used in the proposed first method the following COV for layer co-efficient and drainage co-efficient is calculated (COV of Marshall stability was found = 27.7).

$$COV(a_1) \cong (0.33 - 0.5) COV \quad of \quad Marshall \quad Stability = 0.5 \times 27.70 = 13.85$$

$$COV(a_2) \cong (0.33 - 0.77) COV \quad of \quad CBR = 0.77 \times 11.01 = 8.48$$

$$COV(a_3) \cong (0.33 - 0.9) COV \quad of \quad CBR = 0.90 \times 29.12 = 26.21$$

$$COV(m_2) = \frac{range \cdot 0.3249}{range \quad midpoint} \cdot 100 = \frac{[(1.00 - 0.80) \times 0.3249]}{0.9} \times 100 = 7.22$$

$$COV(m_3) = \frac{range \cdot 0.3249}{range \quad midpoint} \cdot 100 = \frac{[(0.80 - 0.60) \times 0.3249]}{0.7} \times 100 = 9.28$$

Here, the co-efficient of variation of Marshall Stability of recovered bitumen is considered.

7. Now, using these values and the field testing (Road N302) data, the following parameter is calculated to estimate the variance in pavement performance prediction.

$$\overline{SN}^2\,\overline{COV(SN)}^2 \cong \bar{a}_1^2 \bar{d}_1^2 \left[\overline{COV(a_1)}^2 + \overline{COV(d_1)}^2\right] + \bar{a}_2^2 \bar{m}_2^2 \bar{d}_2^2 \left[\overline{COV(a_2)}^2 + \overline{COV(m_2)}^2 + \overline{COV(d_2)}^2\right]$$
$$+ \bar{a}_3^2 \bar{m}_3^2 \bar{d}_3^2 \left[\overline{COV(a_3)}^2 + \overline{COV(m_3)}^2 + \overline{COV(d_3)}^2\right]$$

$$= (0.32)^2 (5.43)^2 \left[(0.1385)^2 + (0.1027)^2\right] + (0.13)^2 (0.9)^2 (8.07)^2 \left[(0.0848)^2 + (0.0722)^2 + (0.1112)^2\right]$$
$$+ (0.11)^2 (0.7)^2 (7.07)^2 \left[(0.2621)^2 + (0.0928)^2 + (0.1858)^2\right]$$

= 0.14496

$$P_2 = \left[\frac{4.065}{\overline{SN}+1} - \frac{1135.57}{\left(0.4 + \frac{1094}{\left(\overline{SN}+1\right)^{5.19}}\right)^2 \quad \left(\overline{SN}+1\right)^{6.19}} \right]^2$$

$$= \left[\frac{4.065}{3.2254+1} - \frac{1135.57}{\left(0.4 + \frac{1094}{\left(3.2254+1\right)^{5.19}}\right)^2 \left(3.2254+1\right)^{6.19}} \right] = 0.66507$$

$COV\left(MR\right) = \left(0.106\right)^2 = 0.011236$

8. Then variance in pavement performance prediction (S_N^2) is calculated in the following way;

$$S_N^2 = \overline{COV(MR)}^2 + P_2 \quad \overline{SN}^2 \cdot \overline{COV(SN)}^2$$

$$= 0.011236 + 0.66507 \times 0.14496$$

$$= 0.10763$$

9. Now, with these values of variance in traffic prediction (S_w^2) and pavement performance prediction (S_N^2), the overall variance *(S_0^2)* is calculated as follows:

$$S_0^2 = S_w^2 + S_N^2 = 0.0301 + 0.1076 = 0.1377$$

And hence,

Overall Standard Deviation of variation (S_0) = √ *Overall variance* = √ (0.1377) = 0.3711

10. Now the reliability design factor for the same reliability level (75%) as used in the proposed first method is calculated as follows;

Reliability design factor, $F_R = 10^{-Z_R \times S_0} = 10^{0.674 \times 0.3711} = 1.78$

APPENDIX C Charts for Estimating Layer Co-efficient

C.1 Structural Layer Co-efficient of Dense-Graded Asphaltic Concrete (a_1)

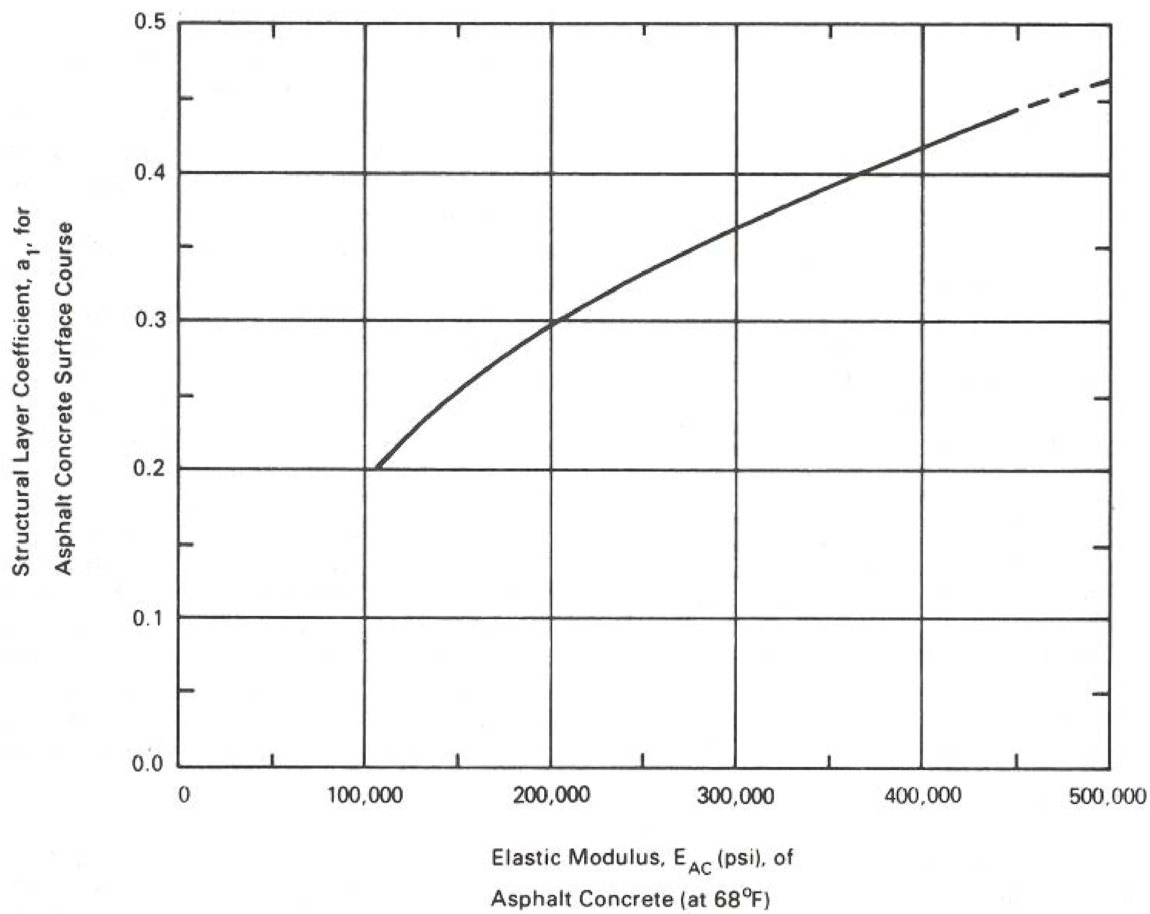


Figure C.1: Chart for Estimating Structural Layer Co-efficient of Dense-Graded Asphaltic Concrete Based on the Elastic (Resilient) Modulus (AASHTO, 1993)

C.2 Granular Base Layer Coefficient (a_2)

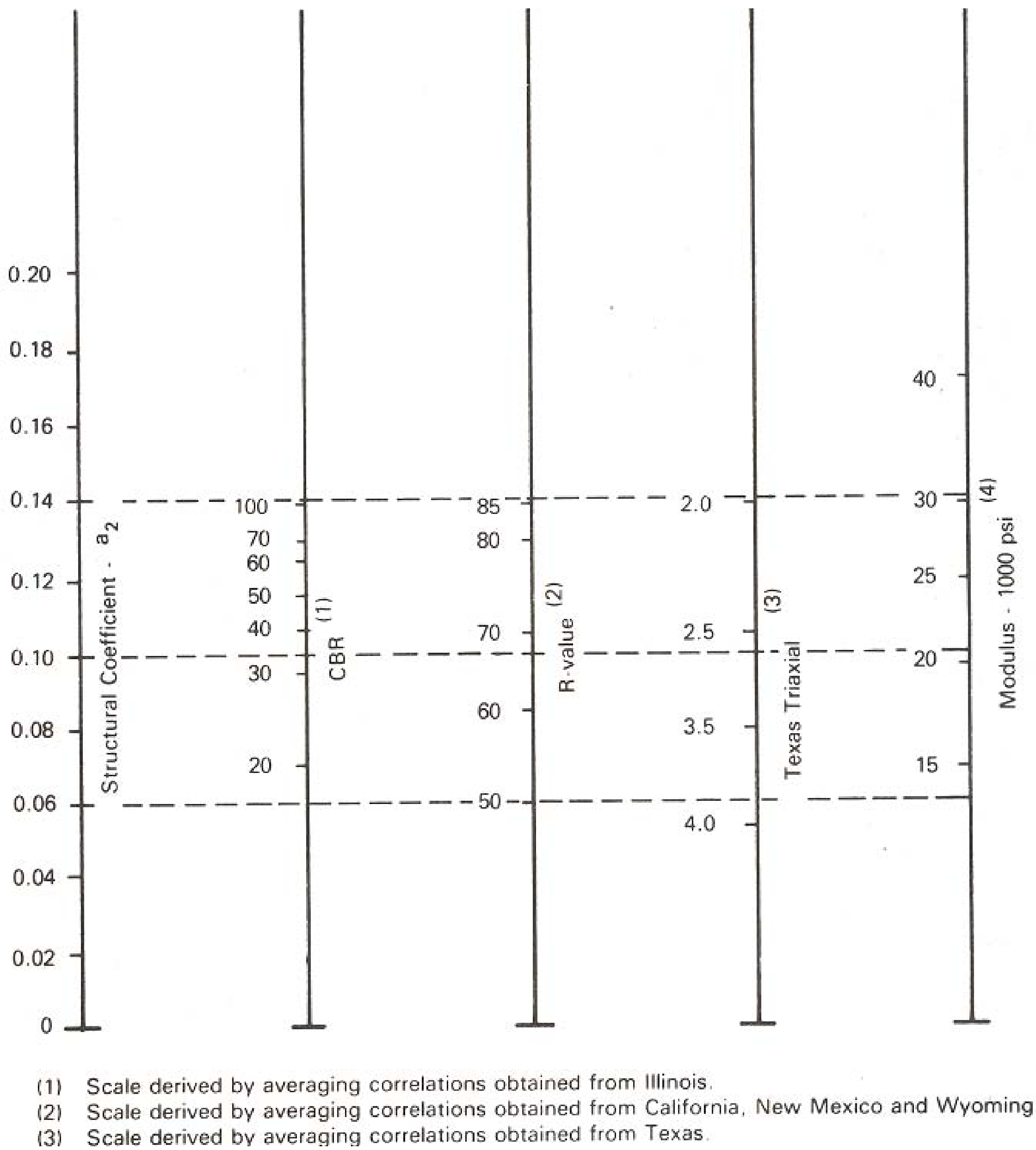
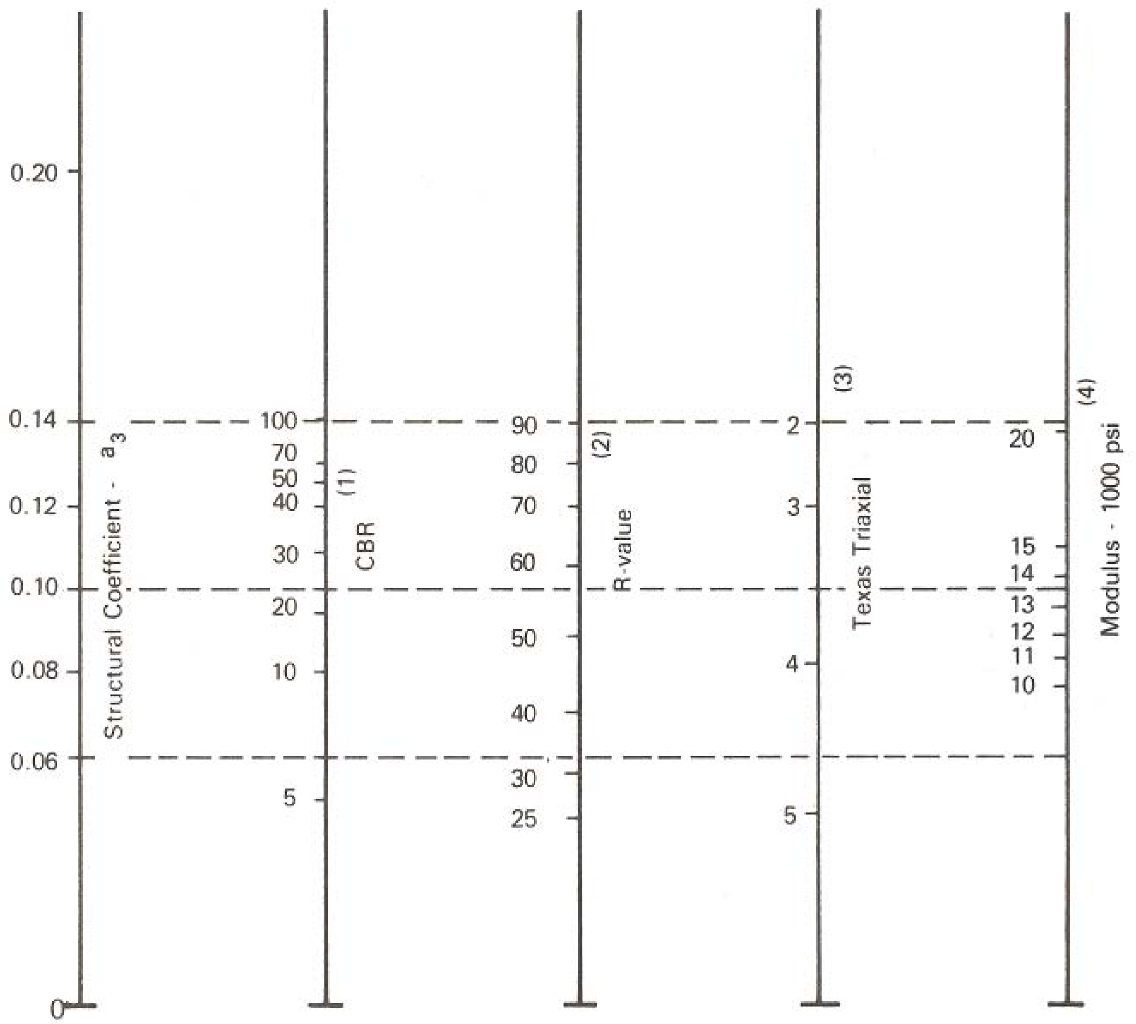


Figure C.2: Variation in Granular Base Layer Coefficient (a_2) with Various Base Strength Parameters (AASHTO, 1993)

C.3 Granular Subbase Layer Coefficient (a_3)



(1) Scale derived from correlations from Illinois.

(2) Scale derived from correlations obtained from The Asphalt Institute, California, New Mexico and Wyoming.

(3) Scale derived from correlations obtained from Texas.

Figure C.3: Variation in Granular Subbase Layer Coefficient (a_3) with Various Subbase Strength Parameters (AASHTO, 1993)

C.4: Drainage Co-efficient (m_i)

Table C.4: Recommended m_i Value for Modifying Structural Layer Coefficients of Untreated Base and Sub-base Materials in Flexible Pavements [AASHTO, 1993]

Quality of Drainage	Percent of Time Pavement Structure is Exposed to Moisture Levels Approaching Saturation			
	Less Than 1%	1-5%	5-25%	Greater Than 25%
Excellent	1.40-1.35	1.35-1.30	1.30-1.20	1.20
Good	1.35-1.25	1.25-1.15	1.15-1.00	1.00
Fair	1.25-1.15	1.15-1.05	1.00-0.80	0.80
Poor	1.15-1.05	1.05-0.80	0.80-0.60	0.60
Very Poor	1.05-0.95	0.95-0.75	0.75-0.40	0.40

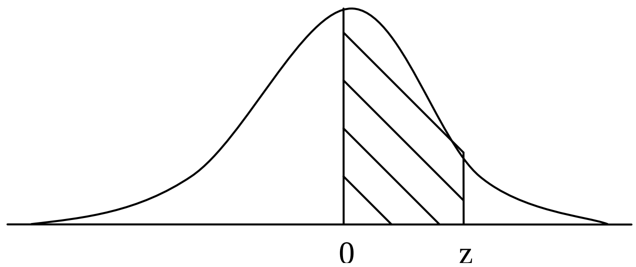
APPENDIX D Definition of Different Types of Vehicle

Definition of different types of Vehicle [RHD, 2001]

Vehicle Type	Definition
Heavy Truck	Three or more axles. Includes multi-axle tandem trucks, container carriers and other articulated vehicles.
Medium Truck	All 2-axle rigid trucks over three tones payload. Includes agricultural tractors and trailers.
Light Truck	Small Truck up to 3 tones payload.
Large Bus	More than 40 seats on 36 foot or longer chassis. Double Decker buses also included in this category.
Medium Bus (Minibus)	Between 16 and 39 seats.
Microbus	Up to 16 seats.
Utility	Pick-ups, jeeps and four wheels drive vehicles, such as Pajero’s and LandRover’s.
Car/Taxi	All types of car used either for personal or taxi services.
Auto Rickshaw	All three wheeled motorized vehicles. Includes Babytaxi, Mishuks, Auto-Tempo and Auto-Vans.
Motor Cycle	All two wheeled motorized vehicles.

APPENDIX E   Statistical Tables

E.1 Standard Normal Curve Area Table [Freund, 1979]

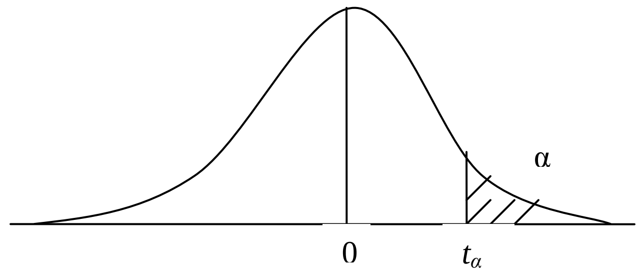


The entries in Table E.1 are the probabilities that a random variable having the standard normal distribution takes on a value between 0 and z; they are given by the area of the shaded region under the curve in the figure shown above[Freund, 1979].

z	.00	.01	.02	.03	.04	.05	.06	.07	.08	.09
0.0	.0000	.0040	.0080	.0120	.0160	.0199	.0239	.0279	.0319	.0359
0.1	.0398	.0438	.0478	.0517	.0557	.0596	.0636	.0675	.0714	.0753
0.2	.0793	.0832	.0871	.0910	.0948	.0987	.1026	.1064	.1103	.1141
0.3	.1179	.1217	.1255	.1293	.1331	.1368	.1406	.1443	.1480	.1517
0.4	.1554	.1591	.1628	.1664	.1700	.1736	.1772	.1808	.1844	.1879
0.5	.1915	.1950	.1985	.2019	.2054	.2088	.2123	.2157	.2190	.2224
0.6	.2257	.2291	.2324	.2357	.2389	.2422	.2454	.2486	.2517	.2549
0.7	.2580	.2611	.2642	.2673	.2704	.2734	.2764	.2794	.2823	.2852
0.8	.2881	.2910	.2939	.2967	.2995	.3023	.3051	.3078	.3106	.3133
0.9	.3159	.3186	.3212	.3238	.3264	.3289	.3315	.3340	.3365	.3389
1.0	.3413	.3438	.3461	.3485	.3508	.3531	.3554	.3577	.3599	.3621
1.1	.3643	.3665	.3686	.3708	.3729	.3749	.3770	.3790	.3810	.3830
1.2	.3849	.3869	.3888	.3907	.3925	.3944	.3962	.3980	.3997	.4015
1.3	.4032	.4049	.4066	.4082	.4099	.4115	.4131	.4147	.4162	.4177
1.4	.4192	.4207	.4222	.4236	.4251	.4265	.4279	.4292	.4306	.4319
1.5	.4332	.4345	.4357	.4370	.4382	.4394	.4406	.4418	.4429	.4441
1.6	.4452	.4463	.4474	.4484	.4495	.4505	.4515	.4525	.4535	.4545
1.7	.4554	.4564	.4573	.4582	.4591	.4599	.4608	.4616	.4625	.4633
1.8	.4641	.4649	.4656	.4664	.4671	.4678	.4686	.4693	.4699	.4706
1.9	.4713	.4719	.4726	.4732	.4738	.4744	.4750	.4756	.4761	.4767
2.0	.4772	.4778	.4783	.4788	.4793	.4798	.4803	.4808	.4812	.4817
2.1	.4821	.4826	.4830	.4834	.4838	.4842	.4846	.4850	.4854	.4857
2.2	.4861	.4864	.4868	.4871	.4875	.4878	.4881	.4884	.4887	.4890
2.3	.4893	.4896	.4898	.4901	.4904	.4906	.4909	.4911	.4913	.4916
2.4	.4918	.4920	.4922	.4925	.4927	.4929	.4931	.4932	.4934	.4936
2.5	.4938	.4940	.4941	.4943	.4945	.4946	.4948	.4949	.4951	.4952
2.6	.4953	.4955	.4956	.4957	.4959	.4960	.4961	.4962	.4963	.4964
2.7	.4965	.4966	.4967	.4968	.4969	.4970	.4971	.4972	.4973	.4974
2.8	.4974	.4975	.4976	.4977	.4977	.4978	.4979	.4979	.4980	.4981
2.9	.4981	.4982	.4982	.4983	.4984	.4984	.4985	.4985	.4986	.4986
3.0	.4987	.4987	.4987	.4988	.4988	.4989	.4989	.4989	.4990	.4990

Also, for $z = 4.0$, 5.0, and 6.0, the areas are 0.49997, 0.4999997, and 0.499999999.

E.2 Table for Values of *t* [Freund, 1979]



The entries in Table B.2 are values for which the area to the right under the t distribution with given degrees of freedom (the shaded area in the figure shown above) is equal to α. [Freund, 1979]

d.f.	$t_{.100}$	$t_{.050}$	$t_{.025}$	$t_{.010}$	$t_{.005}$	d.f.
1	3.078	6.314	12.706	31.821	63.657	1
2	1.886	2.920	4.303	6.965	9.925	2
3	1.638	2.353	3.182	4.541	5.841	3
4	1.533	2.132	2.776	3.747	4.604	4
5	1.476	2.015	2.571	3.365	4.032	5
6	1.440	1.943	2.447	3.143	3.707	6
7	1.415	1.895	2.365	2.998	3.499	7
8	1.397	1.860	2.306	2.896	3.355	8
9	1.383	1.833	2.262	2.821	3.250	9
10	1.372	1.812	2.228	2.764	3.169	10
11	1.363	1.796	2.201	2.718	3.106	11
12	1.356	1.782	2.179	2.681	3.055	12
13	1.350	1.771	2.160	2.650	3.012	13
14	1.345	1.761	2.145	2.624	2.977	14
15	1.341	1.753	2.131	2.602	2.947	15
16	1.337	1.746	2.120	2.583	2.921	16
17	1.333	1.740	2.110	2.567	2.898	17
18	1.330	1.734	2.101	2.552	2.878	18
19	1.328	1.729	2.093	2.539	2.861	19
20	1.325	1.725	2.086	2.528	2.845	20
21	1.323	1.721	2.080	2.518	2.831	21
22	1.321	1.717	2.074	2.508	2.819	22
23	1.319	1.714	2.069	2.500	2.807	23
24	1.318	1.711	2.064	2.492	2.797	24
25	1.316	1.708	2.060	2.485	2.787	25
26	1.315	1.706	2.056	2.479	2.779	26
27	1.314	1.703	2.052	2.473	2.771	27
28	1.313	1.701	2.048	2.467	2.763	28
29	1.311	1.699	2.045	2.462	2.756	29
inf.	1.282	1.645	1.960	2.326	2.576	inf.

E.3 Table for Standard Normal Deviate (Z_R) Values Corresponding to Various Reliability Levels [AASHTO, 1993]

Reliability, R (percent)	**Standard Normal Deviate, Z_R**
50	-0.000
60	-0.253
70	-0.524
75	-0.674
80	-0.841
85	-1.037
90	-1.282
91	-1.340
92	-1.405
93	-1.476
94	-1.555
95	-1.645
96	-1.751
97	-1.881
98	-2.054
99	-2.327
99.9	-3.090
99.99	-3.750

E.4 Table of Factors for Estimating Universe Standard Deviations

(After Burr, 1976)

Number of Observation in Subgroup	C_4
2	0.7979
3	0.8862
4	0.9213
5	0.9400
6	0.9515
7	0.9594
8	0.9650
9	0.9693
10	0.9727
11	0.9754
12	0.9776
13	0.9794
14	0.9810
15	0.9823
16	0.9835
17	0.9845
18	0.9854
19	0.9862
20	0.9869
21	0.9876
22	0.9882
23	0.9887
24	0.9892
25	0.9896
26	0.9901
27	0.9904
28	0.9908
29	0.9911
30	0.9914

Note: c_4 is the ratio of the mean of standard deviation to universe standard deviation

APPENDIX F Other Tables and Graphs

F.1 Strength Reduction due to Weathering and Determination of Design Values

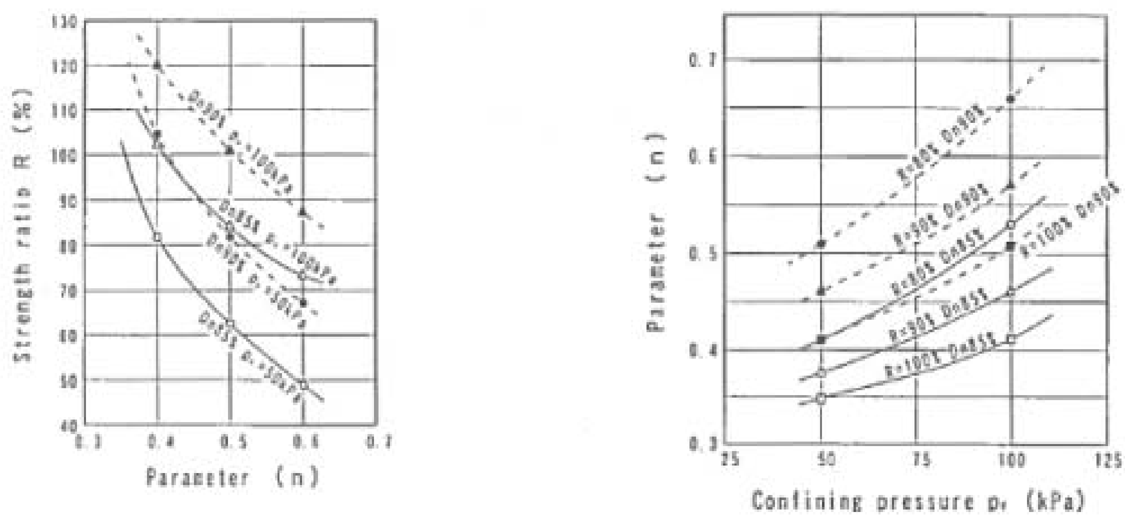


Figure F.1: **(a) Strength Reduction due to Weathering (b) Determination of Design Values**

[After Nakamura et al. 1998]

F.2 Table of Non-dimensional Influence Factors for Foundation*

		Non-dimensional influence factor, I_p		
	m_i	Flexible		Rigid
Shape		Centre	Corner	
Circular	-	1.00	0.64	0.79
Rectangular	1	1.12	0.56	0.88
	1.5	1.36	0.68	1.07
	2	1.53	0.77	1.21
	3	1.78	0.89	1.42
	5	2.10	1.05	1.70
	10	2.54	1.27	2.10
	20	2.99	1.49	2.46
	50	3.57	1.8	3.0
	100	4.01	2.0	3.43

m_i = length of the foundation/width of the foundation

* The above table is based on the following non dimensional influence factor equation expressed by Schleicher [1926; cited in Das, 1997] for the corner of a rectangular footing;

$$I_p = \frac{1}{\Pi}\left[m_1 \ln\left(\frac{1+\sqrt{m_1^2+1}}{m_1}\right) + \ln\left(m_1 + \sqrt{m_1^2+1}\right)\right]$$