EVALUATION OF FLEXIBLE PAVEMENT DESIGN METHODS FOR DEVELOPING COUNTRIES: A CASE STUDY IN BANGLADESH

By

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Approval

This is to certify that the thesis work entitled as " Evaluation of Flexible Pavement Design Methods for Developing Countries: A Case Study in Bangladesh" has been approved by the Board of Examiners for the partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering, Khulna University of Engineering & Technology, Khulna, Bangladesh in December 2015.

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-Md. Imrul Hassan

Abstract

The analysis of data and design of flexible pavement in Bangladesh by conventional flexible design procedures is mostly based on the basis of test and field condition prevailing in western countries. These procedures do not comply with the environmental and material conditions of the region. These conditions generally do not match the parameters for design used in Bangladesh and south Asia. The result is often over-designed being uneconomical or under-design resulting in heavy maintenance and rehabilitation costs. In this study climatic condition of the region is considered during the design. Coarse grained river sand shall be used as subgrade material. In this design mostly accepted conventional methods for flexible pavement design are compared to check their adaptability for design input condition prevailing in countries of Southeast Asia. For evaluation of flexible pavement design methods for Bangladesh, two currently used methods are selected, i.e., AASHTO 1993 Guide for Design of Pavements structures developed in the U.S.A. and Overseas Road Note 31 from Britain. Indian Road Congress (IRC 37) is also considered for this study for comparison. Pavement structures are designed by all the methods using same input parameters obtained from Bangladesh. The results indicate that both methods recommend nearly equal total thickness. However, AASHTO 1993 procedure suggests thicker concrete layer, whereas Road Note 31 and IRC 37 suggest thicker base and sub-base layers. Mechanistic responses in terms of stresses, strains and deflections at critical points in the pavement structure are calculated by computer program CIRCLY. The damaging factors are calculated using CIRCLY software. Mechanistic responses are usually related to failure modes in pavement. These failure modes generally are rutting, permanent deformation, fatigue and thermal cracking. The results of both methods show that structures designed by AASHTO 1993 procedure are safer against rutting, permanent deformation, fatigue and thermal crack failures. The thicker base and subbase layers suggested by Road Note 31 and IRC 37 to compensate for thinner asphalt concrete layer do not significantly contribute to lowering of stresses. The structures recommended by Road Note 31 and IRC 37 may cost less in the beginning due to lesser asphalt concrete layer thickness, but may fail permanently requiring higher maintenance and rehabilitation costs.

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CHAPTER 1

INTRODUCTION

1.1 Background

The design of flexible pavement basically is based on traffic loading, material properties and environmental conditions. Traffic loading is a heterogeneous mix of vehicles, axle types, and axle loads with distributions that vary with time throughout the day, from season to season, and over the pavement design life. Pavement materials respond to these loads in complex ways influenced by stress state and magnitude, temperature, moisture, time, loading rate, and other factors, exposure to harsh environmental conditions ranging from subzero cold to blistering heat and from parched to saturated moisture states adds further complications. It should not be wonder, then, that the profession has restored to largely empirical methods like the American Association of State Highway and Transportation Officials (AASHTO) guides for pavement design (AASHTO, 1993).

During the period of last two decades tremendous development has been offered for more rational and rigorous pavement design procedures. Advances in computational mechanics and in the computers available for performing the calculations have greatly improved the engineers' ability to predict response to load and climate effects. Improved material characterization and constitutive models make it possible to incorporate nonlinearities, rate effects, and other realistic features of material behavior. Large databases now exist for traffic characteristics, site climate conditions, pavement material properties, and historical performance of in-service pavement sections. These and other technical data have made possible the development of the mechanistic-empirical pavement design procedure.

The pavement design method practiced by Roads and Highways Department (RHD), Bangladesh based on two internationally recognized and reliable mechanistic-empirical methods, namely the AASHTO 1993 Design Procedure (U.S.), and the Road Note Design 31 (U.K.). In this study CIRCLY software is used for the mechanistic analysis and design of road pavements. CIRCLY uses the state-of-art material properties and performance models and is continuously being developed and extended when an integral component of the Pavement Design Guide that is widely used throughout the world. In this study the Indian Road Congress (IRC) method will also be considered to compare the flexible pavement design with AASHTO 1993 and Road Note 31.

1.2 Statement of the Problem

The funds and resources are usually insufficient to adequately research geological and climatic condition for the pavement design in Bangladesh. So, empirical methods used in developing countries are still being practiced, and implemented without properly considering local conditions.

The pavement designed by empirical methods reduces the pavement life to a great extent, and in turn, increase the maintenance and rehabilitation cost considerably. Pavement engineers are also restricted to apply these methods due to time, untrained personnel, and equipment and budget constraints.

It is necessary to compare the current pavement design methods with the developing countries so as to match the local conditions.

1.3 Objectives

The objectives of the study are as follows:

- To identify the current flexible pavement design methods practiced in Bangladesh.
- To compare conventional pavement design procedures based on design input parameters from Bangladesh.
- To evaluate mechanistic responses of designed pavement cases, hence to check the adaptability of design procedures for Bangladesh.

1.4 Project

The Government of Bangladesh (GOB) is emphasizing on improved connectivity between each part of the country. Accordingly, Government of Bangladesh announced its National Land Transport Policy in 2004 defining long term as 20 years. The Road Master Plan has identified many feasible and priority projects (RMP, 2008). One of the priority roads identified is the Joydevpur – Chandra – Tangail - Elenga (JCTE) road (National Highway No.4). This road has been included under Sub Regional Road Transport Project (SRTP) for design and construction which was financed by Asian Development Bank. The length of the road is 69 km. This road is a vital link in the national highway network and forms a part of the Asian Highway Network. The existing road has inadequate capacities and lack of safety. The project will upgrade the road to a four lane road with safety features, dedicated SMVT lane, flyovers at business junctions, overpasses, and queening lanes at intersections.

1.5 Organization of the thesis

This thesis consists of six chapters briefly stated as follows:

Chapter 1: Introduction to the title of research, definition of the problem statement, objectives of the study and scope of research are discussed.

Chapter 2: Extensive literature review of the title is presented. Discussion of the past and present techniques of pavement design, advantages and shortcomings of different methods are discussed. Scope of other researchers on the title is provided, and direction of this research is fixed.

Chapter 3: Methodology for the research is formed. Detail about data collection is discussed. Methods of analysis for the data are introduced.

Chapter 4: Pavement cases are designed by three selected pavement design procedures. Range of data input variables is fixed. Formation of Pavement Design Input Parameters is carried out.

Chapter 5: Mechanistic responses of designed sections are calculated by the computer program. Based on the responses, the design methods are reviewed. Recommendations are given for better design procedure for developing countries.

Chapter 6: Conclusions and findings of the research are discussed. Recommendations for adopting suitable design procedure for developing countries are provided. Scope of this research, for improvement by future researchers is also discussed.

CHAPTER 2

LITERATURE REVIEW

2.1 General

Economic and efficient pavement system designed through mechanistic behavioral analysis can increase design life and reduce maintenance cost. Flexible pavements are constructed of bituminous and granular materials. The pavement design procedures has evolved from preliminary design methods based on experience and soil tests, like group index method, to highly sophisticated mechanistic design approach. A brief history of this evaluation is provided in this chapter. In general, mechanistic-empirical flexible pavement design process, involves:

1. Estimating the amount of traffic and the cumulative number of equivalent standard axles that will use the pavement over the selected design period.
2. Assessing the strength of the Subgrade soil over which the pavement is to be built.
3. Selecting the most economical combination of pavement materials and layer thickness that will provide satisfactory service over the design life of the pavement

The purpose of structural design, by mechanistic-empirical design approach is to limit the stresses induced in the subgrade by traffic to safe level at which subgrade deformation is insignificant while at the same time ensuring that the pavement layers themselves do not deteriorate to any serious extend within a specified period of time (Overseas Road Note 31, 1993).

2.1.1 Structural Component of a Pavement

There are four layers/components in a Flexible Pavement:

a) Subgrade

b) Sub base Course

c) Base Course

d) Surface Course

Subgrade: This is the upper layer of the natural soil which may be undisturbed local material or may be soil excavated elsewhere and placed as fill. In either case it is compacted during construction to give added strength.

Improved subgrade (Capping layer): Where very weak soils encountered a capping layer is sometimes necessary. This may better quality subgrade material imported from elsewhere or existing subgrade material improved by mechanical or chemical stabilization.

Sub-base: This is the secondary load-spreading layer underlying the roadbase. It will normally consist of a material of lower quality than that used in the roadbase such as unprocessed natural gravel, gravel-sand, or gravel-sand-clay. This layer also serves as a separating layer preventing contamination of the roadbase by the subgrade material and, under wet conditions, it has an important role to play in protecting the subgrade from damage by construction traffic.

Base/Roadbase: This is the main load-spreading layer of the pavement. It will normally consist of crushed stone or gravel, or of gravelly soils, decomposed rock, sands and sand-clays stabilized with cement, lime or bitumen.

Surfacing: This is the upper most layer of the pavement and will normally consist of a bituminous surface dressing or a layer of premixed bituminous material. Where premixed bituminous materials are laid in two layers, these are known as the wearing course and the base course (or binder course) as shown in Figure 2.1.

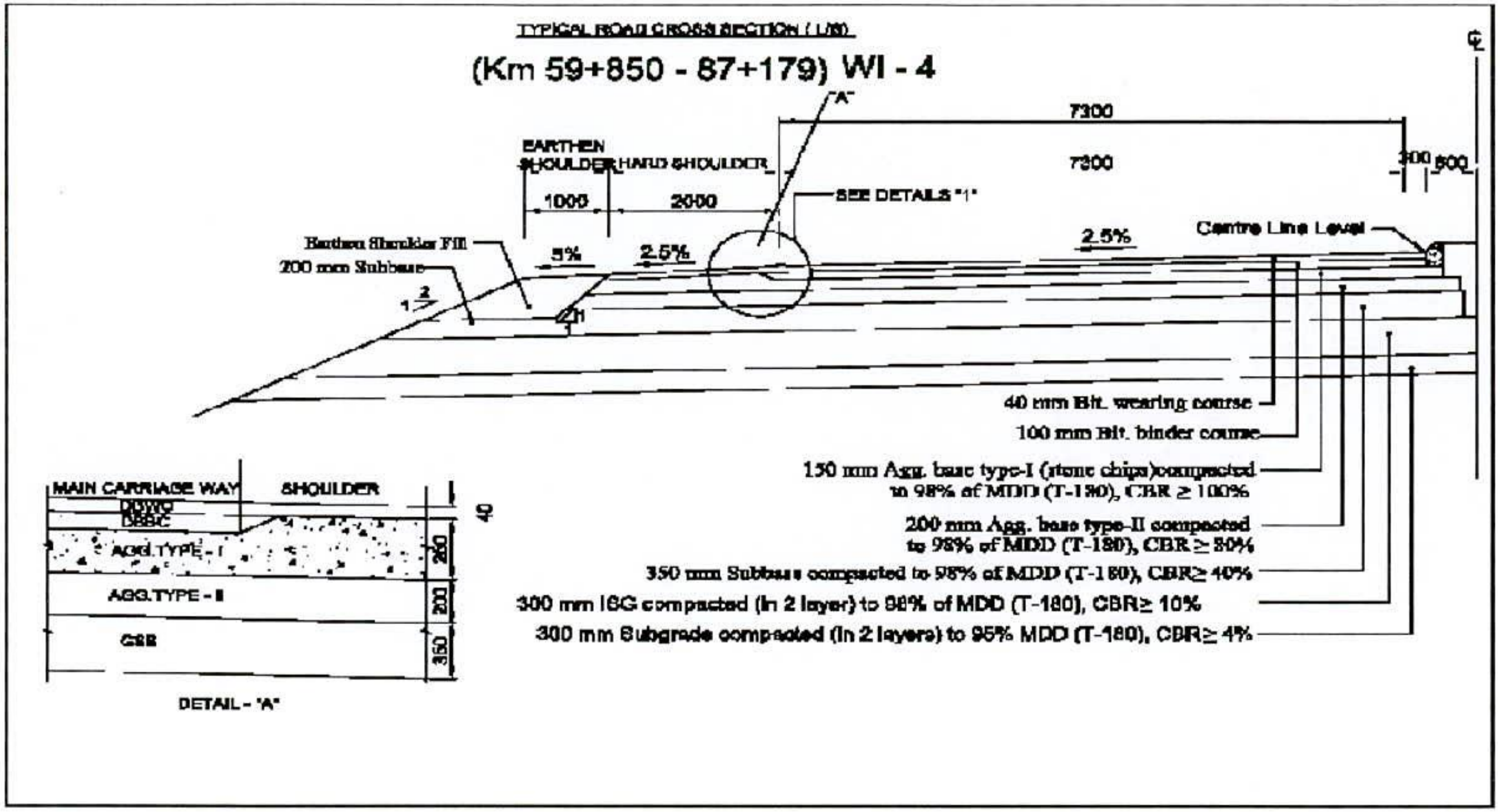


Figure 2.1 Typical Section of Flexible Pavement

2.2 Review of Flexible Pavement Design Principles

Before the 1990s, pavement design consisted basically of defining thickness of layered materials that would provide strength and protection to a soft, weak subgrade. Pavements were designed against subgrade failure. As experienced evolved, several pavement design methods based on subgrade shear strength were developed.

Since then, traffic volume has increased and the design criteria have changed. As important as providing subgrade support, it was equally important to evaluate pavement performance through ride quality and other surface distresses that increase the rate of deterioration of pavement structures. Performance became the focus point of pavement designs. Methods based on serviceability (an index of the pavement service quality) were developed based on test track experiments. The AASHTO Road Test in 1960s was a seminal experiment from which the AASHTO design guide was developed.

Methods developed from laboratory test data or test track experiments in which model curves are fitted to data are typical examples of empirical methods. Although they may exhibit good accuracy, empirical methods are valid only for the materials and climate conditions for which they were developed.

Meanwhile, new materials started to be used in pavement structures that provided better subgrade protection, but with their own failure modes. New design criteria were required to incorporate such failure mechanisms (e.g., fatigue cracking and permanent deformation in the case of asphalt concrete). The Asphalt Institute method (Asphalt Institute, 1982, 1991) and the Shell method (Claussen *et al.*, 1977; Shook *et al.*, 1982) are examples of procedures based on asphalt concrete's fatigue cracking and permanent deformation failure modes. These methods were the first to use linear-elastic theory of mechanics to compute structural responses (in this case strains) in combination with empirical models to predict number of loads to failure for flexible pavements. The dilemma is that pavement materials do not exhibit the simple behavior assumed in isotropic linear-elastic theory. Nonlinearities, time and temperature dependency, and anisotropy are some examples of complicated features often observed in pavement materials. In this case, advanced modeling is required to predict performance mechanistically. The mechanistic design approach is based on the theories of mechanics and relates pavement structural behavior and performance to traffic loading and environmental influences. Progress has been made in recent years on isolated pieces of the mechanistic performance prediction problem, but the reality is that fully mechanistic methods are not yet available for practical pavement design.

The mechanistic-empirical approach is a hybrid approach. Empirical models are used to fill in the gaps that exist between the theory of mechanics and the performance of pavement structures. Simple mechanistic responses are easy to compute with assumptions and simplifications (i.e., homogeneous material, small strain analysis, static loading as typically assumed in linear elastic theory), but they by themselves cannot be used to predict performance directly; some type of empirical model is required to make the appropriate correlation. Mechanistic-empirical methods are considered an intermediate step between empirical and fully mechanistic methods.

The objective of this section is to review briefly some of these advancements in pavement design, focusing on flexible pavement.

2.2.1 Mechanistic-Empirical Methods

An empirical design approach is one that is based solely on the results of experiments or experience. A Mechanistic-empirical (M-E) method is the modified empirical method of pavement design. The induced state of stress and strain in a pavement structure due to traffic loading and environmental conditions is predicted using theory of mechanics. Empirical models link these structural responses to distress predictions. Kerkhoven and Dormon (1953) first suggested the use of vertical compressive strain on the top of subgrade as a failure criterion to reduce permanent deformation. Saal and Pell (1960) published the use of horizontal tensile strain at the bottom of asphalt layer to minimize fatigue cracking. Dormon and Metcalf (1965) first used this concept for pavement design. The shell method (Claussen *et al.*, 1977 and the Asphalt Institute method (Shook *et al.*, 1982) incorporated strain-based criteria in their mechanistic-empirical procedures. Several studies over the past fifteen years have advanced mechanistic-empirical techniques. Most of the work, however, was based on variants of the same two strain-based criteria developed by Shell and the Asphalt Institute. The Departments of Transportation of the Washington State (WSDOT), North Carolina (NCDOT) and Minnesota (MNDOT), to name a few, developed their own M-E procedures. The National Cooperative Highway

Research Program (NCHRP) 1-26 project report, Calibrated Mechanistic Structural Analysis Procedures for Pavements (1990), provided the basic frame work for most of the efforts attempted by state DOTs. WSDOT (Pierce, 2007; WSDOT, 1995) and NCDOT (Corley-Lay, 1996) developed similar M-E frameworks incorporating environmental variables (e.g., asphalt concrete temperature to determine stiffness) and cumulative damage model using Miner's Law with the fatigue cracking criterion. MNDOT (Timm, 1998) adopted a variant of the Shell's fatigue cracking model developed in Illinois and the Asphalt Institute's rutting model.

The NCHRP 1-37A project (NCHRP, 2004) delivered the most recent M-E based method that incorporates nationally calibrated models to predict distinct distresses induced by traffic load and environmental conditions. The NCHRP 1-37A methodology also incorporates vehicle class and load distributions in the design, a step forward from the Equivalent Single Axle Load (ESAL) approach used in the AASHTO design equation and other methods. The performance computation is done on a seasonal basis to incorporate the effect of climate conditions on the behavior of materials.

The Shell Petroleum International and the Asphalt Institute apply the above mentioned concepts for their mechanistic empirical methods of design. Advantages of this design approach are improvement in reliability of design, ability to predict type of distress, feasibility to extrapolate from limited field and laboratory data (Huang, 1993).

The comparison of the empirical AASHTO 1993 and Road Note 31 based on mechanistic structural responses (stresses, strains and deflections) have been described in Chapter 4 and 5 which is one of the objectives of this study.

2.3 Pavement Distress Criteria

Generally fatigue, permanent deformation, rutting and thermal cracking are four major modes of distress that lead to a reduction in the serviceability of flexible pavements. These distress modes are defined by the critical stress conditions within the pavement structure.

Fatigue is defined as the initialization of cracks at the bottom side of the asphalt concrete layer. This is associated with increasing number of the repetitive loading. Rutting is the longitudinal depressions formed along the wheel paths, with small upheavals on the sides, associated with increasing number of load applications. Thermal cracking generally occurs at temperatures below zero degree centigrade. At low temperatures, asphalt layer when subjected to heavy loads of traffic, exhibit high tensile stress at its bottom. Table 2.1 shows the major structural responses (critical conditions) and the associated modes of distress (Monismith, 1992).

Table 2.1 Major Structural Responses

Types of Response	Associated With
Horizontal tensile strain at bottom of AC layer	Fatigue
Horizontal tensile stress at bottom of AC layer	Thermal cracking
Vertical strain at top of subgrade	Rutting
Vertical compressive stress at top of subgrade	Permanent deformation
Deflection at top of subgrade	Rutting

In short the mechanistic-empirical design approach accounts for structural analysis of the pavement, i.e., calculation of stresses, strains and deflections developed in a pavement due to traffic loads, temperature, and/or moisture.

2.3.1 Multilayer Elastic Computer Programs

A number of general solutions for determination of stresses and deformations in multilayer elastic solids were presented in 1962. These general solutions foster the development of the current generation of multilayer elastic and viscoelastic computer programs. One of the most commonly used programs is CIRCLY.

2.3.1.1 CIRCLY Software

CIRCLY software is used for the mechanistic analysis and design of road pavements. The first mainframe version of CIRCLY was released in 1977 and the current Windows version is Version 5. It is an integral component of the Austroads Pavement Design Guide (Austroads, 2010). The system calculates the cumulative damage induced by a traffic spectrum consisting of any combination of user-specified vehicle types and load configurations. As well as using the usual 'equivalent' single wheel and axle load approximations, optionally the contribution of each vehicle/load configuration can be explicitly analyzed. Other geotechnical applications, such as foundation engineering and settlement analysis, can also be analyzed using CIRCLY. The program computes the various components of stresses, strains and displacement along with principal values at locations specified by the user, within the layered system.

Cumulative Damage Concept

The system explicitly accumulates the contribution from each loading in the traffic spectrum at each analysis point by using Miner's hypothesis. The damage factor is defined as the number of repetitions (n_i) of a given response parameter divided by the 'allowable' repetitions (N_i) of the response parameter that would cause failure. The Cumulative Damage Factor (CDF) for the parameter is given by summing the damage factors over all the loadings in the traffic spectrum:

$$\text{Cumulative Damage} = \sum \frac{n_i}{N_i}$$

The system is presumed to have reached its design life when the cumulative damage reaches 1.0. If the cumulative damage is less than 1.0 the system has excess capacity and the cumulative damage represents the proportion of life consumed. If the cumulative damage is greater than 1.0 the system is predicted to 'fail' before all of the design traffic has been applied.

The procedure takes account of:

- The design repetitions of each vehicle/load condition; and
- The material performance properties used in the design model.

CIRCLY has many other powerful features, including selection of:

- Cross-anisotropic and isotropic material properties;
- Fully continuous (rough) or fully frictionless (smooth) layer interfaces;
- A comprehensive range of load types, including, horizontal, torsional, etc.;
- Non-uniform surface contact stress distributions; and
- Automatic sub-layering of unbound granular materials.

The objective of the computer analysis using multilayer elastic program CIRCLY is to find out this Cumulative Damage Factor and also structural responses (stress, strain and deflection) for mechanistic-empirical flexible pavement design.

2.3.1.2 Maximum Stress, Strain and Deflection used in CIRCLY

The general relationship between the maximum tensile strain in asphalt produced by a specific load and the allowable number of repetitions of that load is:

$$N = RF\left[\frac{6918(0.856V_b + 1.08}{S_{mix}^{0.36}\mu\varepsilon}\right]^5 \text{--} (2.1)$$

Where:

N = allowable number of repetitions of the load;

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

V_b = percentage by volume of bitumen in the asphalt (%); and

S_{mix} = asphalt modulus (MPa)

RF= reliability factor for asphalt fatigue

The calibrated tolerable deflection with cumulative design traffic (DESA) expressed in Equivalent Standard axles as shown by Equation 2.2.

$$D_0 = 3.1833 * DESA^{-0.107} \text{--} (2.2)$$

Where:

D_0 = maximum surface deflection in mm

DESA = design traffic in Equivalent Standard Axles

AUSTROADS (2004a) has criteria for the allowable number of ESA loadings. The allowable number of repetition of load is as shown in Equation 2.3.

$$N = \left[\frac{9300}{\mu\varepsilon}\right]^7 \text{---} (2.3)$$

Where:

$\mu\varepsilon$ = maximum compressive strain on top of subgrade in units of micro strain;

Equation 2.3 can be rearranged as Equation 2.4 (AUSROADS, 2004a).

$$\varepsilon = 9.3x10^{-3} xN^{-1/7} \text{ ---------- } (2.4)$$

In this context ε (units of strain), can be regarded as the limiting design strain for a total design traffic loading of N. As per Equation 2.4, the maximum allowable subgrade strains $\varepsilon_{SG,}$ are 0.0018, 0.0013 and 0.0009 for traffic levels of 10^5, 10^6 and 10^7 ESA, respectively.

By controlling the compressive strain at top of the subgrade, the maximum surface deflection will be controlled. The deflection on top of subgrade can be estimated by the Equation 2.5.

$$D_0 = 1275.1 * \varepsilon_{ZZ} \text{ ---------- } (2.5)$$

ε_{ZZ} = compressive strain on top of subgrade;

D_0 = maximum surface deflection under the standard axle load

$$N_f = \left[\frac{3065}{\varepsilon_{ZZ}}\right]^{9.3469} \text{ ---------- } (2.6)$$

Where,

N_f = design traffic in standard axle repetitions

Cumulative damage factor, $CDF = \dfrac{6.8 * DESA}{N_f}$ ---------- (2.7)

Where, DESA=cumulative design traffic

CDF is always ≤ 1.0

2.4 Methods Based on Mechanistic-Empirical approach

The mechanistic empirical approach developed over the years has fostered many agencies to develop design charts for satisfying their local conditions. The examples include methods developed by the Shell International Petroleum Co., Ltd in London, the Asphalt Institute in Lexington, National Institute for Transport and Road Research (NITRR) in

South Africa, University of Nottingham in Great Britain, National Cooperative Highway Research program (NCHRP) AASHTO, and Transport and Road Research Laboratory Road Note 31 design, U.K. (Monismith,1992).

2.4.1 AASHTO Pavement Design Guide

The guide was based on the results of the AASHO Road Test supplemented by existing design procedure. After the guide had been used for several years, the AASHTO Design Committee prepared and AASHTO published the current 'AASHTO Interim Guide for Design of Pavement Structures- 1972.' Revisions were made in the year 1986.

Although the new guide is still based on data obtained at the AASHO Road Test, the scope of the guide has been largely extended to cover many areas and applications in pavement design. Some problems exist beyond the ability of current pavement literature. In many cases, it may be necessary to adjust the design based on a combination of traffic factors, environmental factors and experience (AASHTO 1993).

Recent research provided some guidelines to simplify the AASHTO 1993 Guide and reduce its scope to match local conditions. This design guide gives conservative designs, and even if some parameters are modified to suit local conditions, the design obtained considerably fulfills the requirements.

2.4.2 Road Note 31 Pavement Design Guide

For flexible pavements the main criteria used to assess performance are deformation and cracking. In a number of experimental flexible pavements designed by stress, strains, deformation and temperature measurements were recorded. The recommended minimum thickness of each of the pavements layers was obtained from figures derived from evidence gained from full scale road experiments. Following are the main features of the procedure.

- Pavement life: twenty to forty years
- Subgrade: assessed in terms of CBR value
- Drainage and Weather protection
- Sub-base
- Base and Surfacing

This method was developed to keep due regards for the conditions in tropical and sub-tropical countries. The latest form of this procedure is available as the Overseas Road Note 31, 1993.

2.4.3 Indian Road Congress 37 Pavement Design Guide

The design of flexible pavement involves the interplay of several variables, such as, the wheel loads, traffic, climate, terrain and sub-grade conditions. The IRC was first brought out in 1970 and were based on California Bearing Ratio (CBR) method. In this approach the pavement thickness was related to the cumulative number of standard axles to be carried out for different sub-grade strengths up to 30 million. With the rapid growth of traffic, the pavements are required to be designed for heavy volume of traffic of the order of 150 million standard axles.

Based on the performance of existing design and using analytical approach, simple design charts (Figure 1 and Figure 2) and a catalogue of pavement designs (Plate 1 and Plate 2) have been added for thickness of each layer (Appendix E). The pavement designs are given for subgrade CBR values ranging from 2 per cent to 10 per cent and design traffic ranging from 1 msa to 150 msa for an annual average pavement temperature of $35^{\circ}C$. The layer thickness obtained from the analysis has been slightly modified to adapt the designs to stage construction. Using the following parameters, appropriate designs could be chosen for the given traffic and soil strength:

(i) Design traffic in terms of cumulative number of standard axles; and

(ii) CBR value of sub-grade

IRC method is widely used in India and in neighbouring countries for the flexible pavement design.

2.4.4 RHD Bangladesh Pavement Design Guide

For flexible pavements, the RHD Design Guideline has been developed based on AASHTO Pavement Design Guide 1993 and Road Note 31 Pavement Design Guide.

Road Note 31 design guide is considered for local and feeder road that operates small volume of traffic with maximum ESALs of 30 million. AASHTO 1993 design guide is considered for national highways, regional road and feeder road Type A that operates large volume of traffic and high load. Design ESALs value depends on survey report on traffic volume carried out by RHD Bangladesh.

AASHTO 1993 and IRC: 37-2001 design guide are followed for design of feeder road Type B and rural roads of Local Government Engineering Department (LGED) that operate small to medium volume of traffic. Design ESALs value depends on survey report on traffic volume carried out by LGED, Bangladesh.

The Vehicle Equivalency Factors according to RHD Guideline are shown in Table 2.2.

Table 2.2 Vehicle Equivalency Factors

Source/Classification	Truck [H]	Truck [M]	Truck [S]	Bus [L]	Bus [M]	Minibus
RHD Guideline	4.80	4.62	1.00	1.00	0.50	0.50

2.5 Materials Characterization

An integral part of the development of analytically based methodologies has been the evolution of the procedures to define requisite materials characteristics. To define pavement materials response characteristics properly, one must consider the following service conditions:

- Stress State (associated with loading)
- Environmental conditions (moisture and temperature)
- Construction conditions (including water content and dry density for untreated material)

Subgrade resilient modulus selection by AASHTO 1993 method involves conducting resilient modulus tests at different moisture contents on subgrade soil and using the average monthly relative damage factor to select the effective roadbed soil resilient modulus. In the guide suitable relations are provided which fairly correlate CBR and R-value to resilient modulus of respective soil.

2.6 Traffic Volume

The deterioration of flexible pavement is caused by traffic results from both the magnitude of the individual wheel loads and the number of times these loads that is applied. The AASHTO Road Test results showed that the damaging effect of the passage of an axle of any load can be represented by a number of 18 kip equivalent single axle loads (ESALs).

2.7 Other Material Variables

Other materials for mechanistic-empirical design approach include elastic modulus of asphalt concrete, base and sub-base layers, drainage coefficient, and loss of serviceability, reliability level, overall standard deviation, performance period, analysis period, and economic factors. The scope of these factors is documented in the guide.

2.8 Material Properties

2.8.1 Empirical design

AASHTO 1993 is an empirical design method, primarily based upon the results determined through actual vehicle and pavement testing undertaken in the AASHO road tests conducted in the 1950s in the USA. The AASHTO pavement design method relies upon the specification of a number of material design parameters:

- □ Layer Strength Coefficients, values are dependent upon the material type; and
- □ Roadbed soil (subgrade) resilient modulus

2.8.1.1 Subgrade Properties

The California Bearing Ratio (CBR) test is the most commonly used method to establish material properties and to specify indices of shear strength for unbound materials. The CBR value can be translated into resilience moduli for the purposes of design.

Table 2.3 gives the M_R values derived from equations published by a number of authorities. The last column of the table shows the value adopted for design purposes.

Table 2.3: Subgrade Properties

Factor	CBR Value [%]	Corresponding Resilience Modulus M_R [psi]					Design Value Adopted (psi)
Source of Correlation	-	AASHTO 1993	Ausroads	US Army Corps of Engineers	CSIR [South Africa]	TRL [UK]	-
Formula	-	1,500 x CBR	1450 x CBR	5,409 x $CBR^{0.71}$	3,000 x $CBR^{0.65}$	2,555 x $CBR^{0.64}$	-
Improved subgrade fill	8	12,000	11,603	23,670	11,600	9,670	11,600

2.8.1.2 Pavement Material Coefficients

The AASHTO 1993 design process requires selection of coefficients upon which to base an assessment of the different support strengths contributed by the individual pavement layers. These vary with the nature of the composite materials involved as well as the compaction effort applied during their placement.

For the project roads, pavement layer material coefficients have been produced based upon the results of testing of samples. These values were taken from 'critical' values appropriate for Bangladesh road construction and are commonly applicable to both locally available and imported materials according to RHD specifications. They are summarized in Table 2.4. Figure 2.2 to 2.4 show the layer coefficients for asphalt, base and sub-base layers.

Table 2.4: Design Layer Coefficients

Contributing Layer	AC		Base Course		Sub-base Course	
	Wearing Course	Base Course	Upper	Lower	Upper	Lower
RHD Standard Elastic Modulus Values [psi]	400,000	400,000	50,800	43,500	29,000	22,000
Adopted Elastic Modulus Values [psi]	210,000	250,000	50,800	43,500	29,000	22,000
Corresponding Layer Coefficients	$a_1 = 0.31$	$a_2 = 0.33$	$a_3 = 0.15$	$a_4 = 0.14$	$a_5 = 0.12$	$a_6 = 0.11$

2.8.2 Mechanistic – Elastic Design

A critical input for mechanistic-elastic design models is the engineering properties of the various materials to be used in the pavement construction. It is necessary to ascribe the values of these properties for all layers in the pavement including asphalt, base aggregate base course, sub-base and improved and natural subgrade materials. Table 2.5 shows the input data used for pavement design.

Table 2.5: Engineering Properties for Granular Pavement Layers

Layer	Resilient Modulus (psi)	Resilient Modulus (MPa)
Aggregate Base Type I	50,800	350
Aggregate Base Type II	43,500	300
Sub-base	29,000	200
Improved Subgrade	11,600	80

2.8.2.1 Asphalt

The in-service performance of asphalt layers is highly dependent upon the temperature regime at the project location. Asphalt stiffness is usually determined at the weighted mean annual pavement temperature (WMAPT) for the project site. RHD's standard design procedure adopts a value of 400,000 psi for the asphalt resilient modulus, but as can be seen in the following figure, this is for asphalt at a temperature of 20°C. Figure 2.2 shows the estimation of structural layer coefficients for Dense Graded Asphalt (AASHTO, 1993).

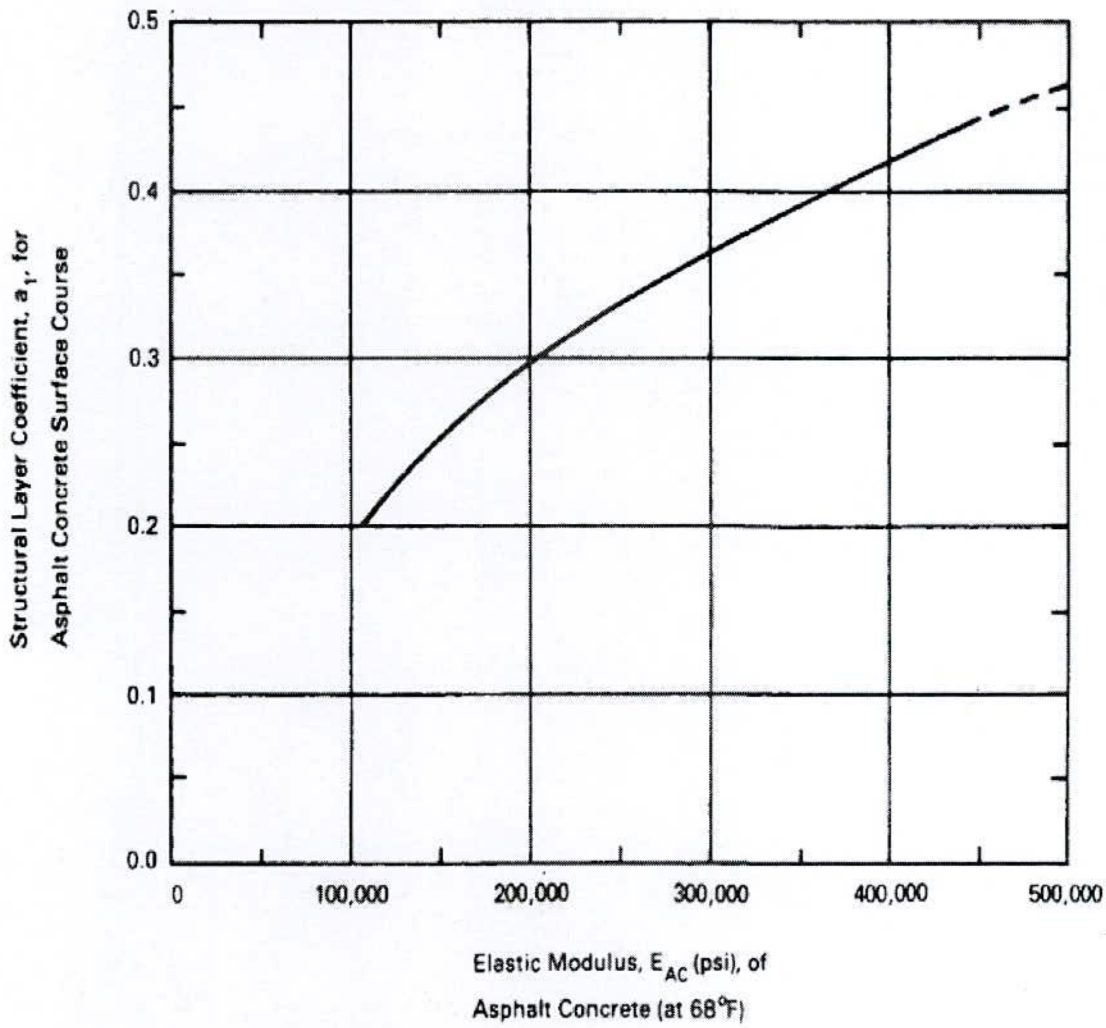


Figure 2.2 Estimation of Structural Layer Coefficients for Dense Graded Asphalt Concrete (AASHTO, 1993)

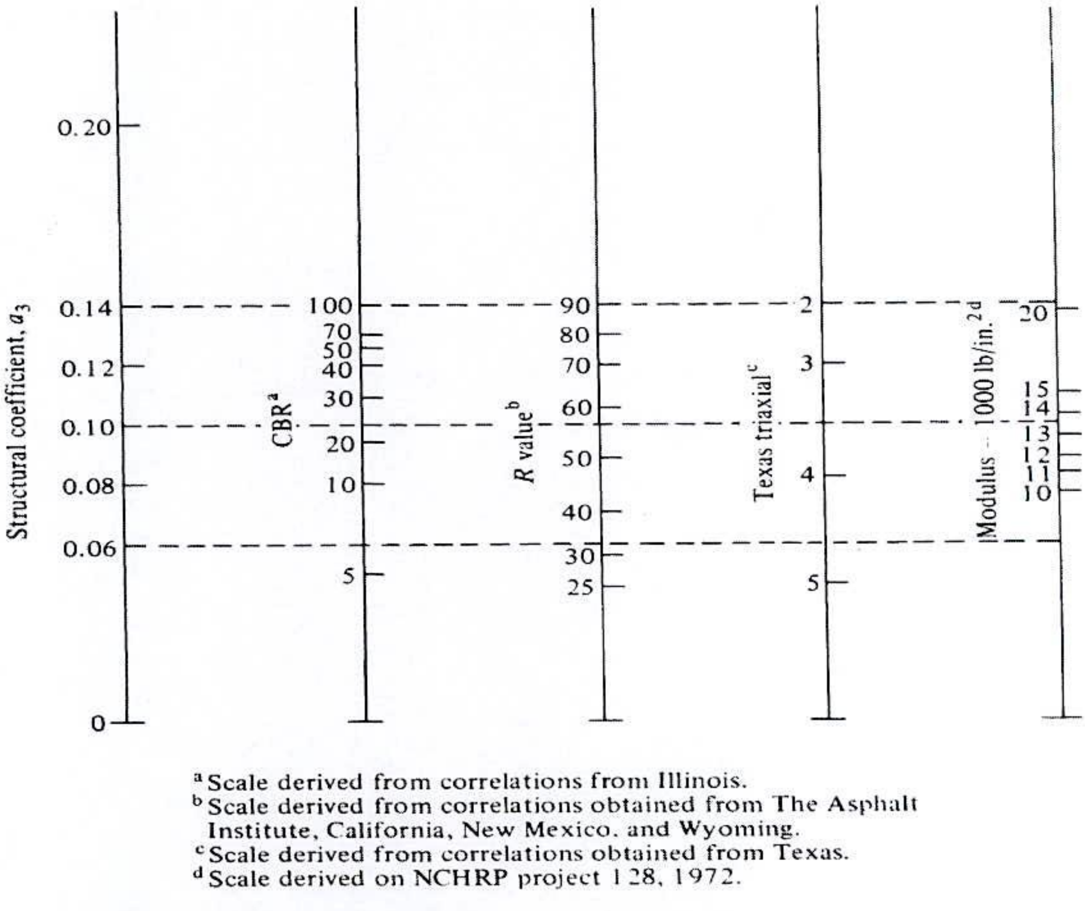
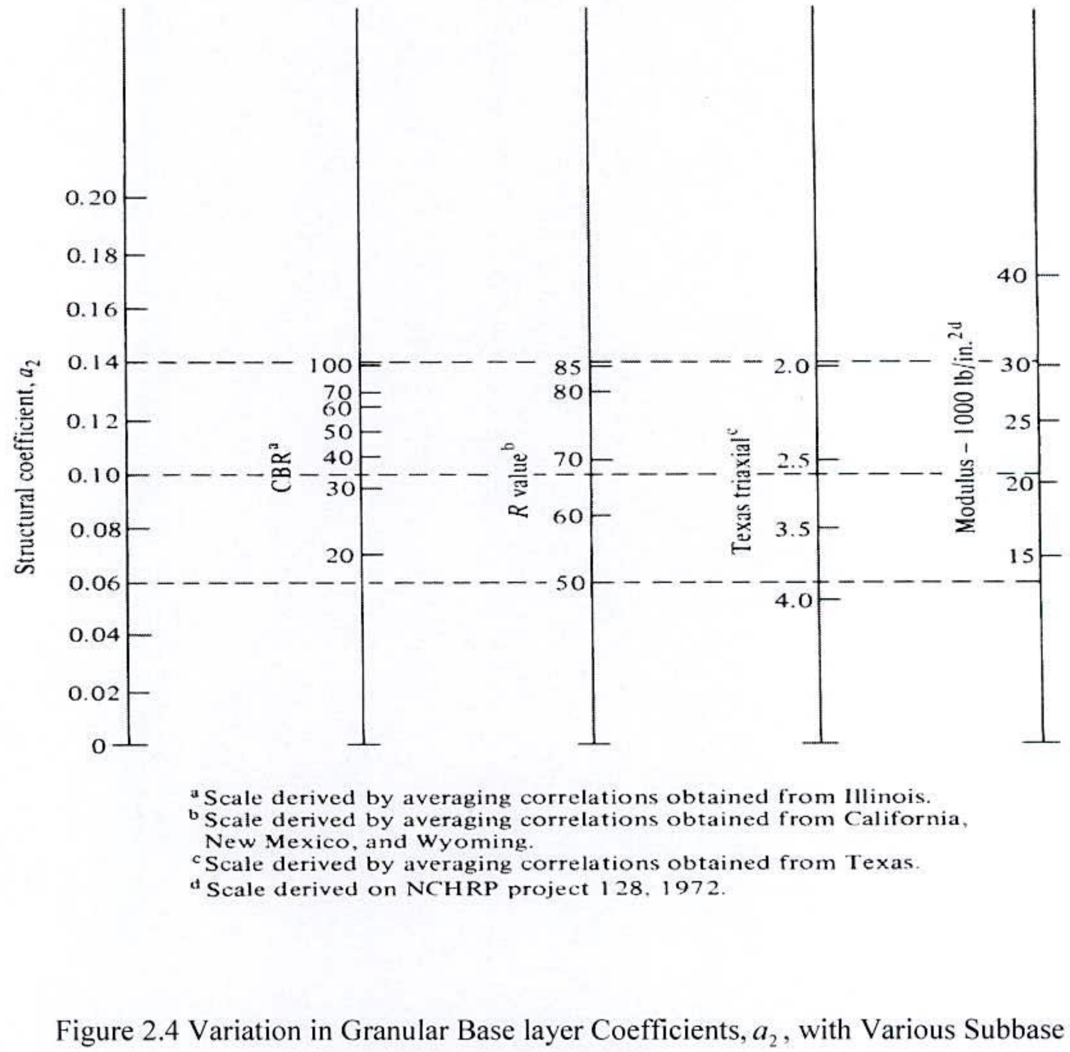


Figure 2.3 Variation in Granular Sub-base layer Coefficients, a_3, with Various Subbase Strength Parameters (AASHTO, 1993)



[a] Scale derived by averaging correlations obtained from Illinois.
[b] Scale derived by averaging correlations obtained from California, New Mexico, and Wyoming.
[c] Scale derived by averaging correlations obtained from Texas.
[d] Scale derived on NCHRP project 128, 1972.

Figure 2.4 Variation in Granular Base layer Coefficients, a_2, with Various Subbase Strength Parameters (AASHTO, 1993)

A check on the stiffness values for the asphalt layers has been undertaken using the procedure for the determination of WMAPT. The WMAPT value is determined from the average monthly climate and rainfall data for the project location. Average climate data for Dhaka and Tangail have been taken from the Environmental Impact Assessment Report being prepared for this project.

Table 2.6: Average Ambient Temperature and Rainfall Data (Dhaka) in 2013

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Max. Temp (°C)	28.3	31.7	36	36.9	36.4	35.4	34.5	34.9	35	34.8	32.4	29.1
Min. Temp (°C)	7.7	9.9	13.8	18.7	20.5	22.6	24.2	24.2	23.8	19.8	14.2	10.0
Rainfall (mm)	8	21	54	112	279	331	392	324	330	187	25	12

Using this data leads to a calculated weighted mean annual air temperature (WMAAT) of 25.58°C and weighted mean annual pavement temperature WMAPT of 37.47°C for Dhaka.

Table 2.7: Average Ambient Temperature and Rainfall Data (Tangail) in 2013

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Max. Temp (°C)	27.6	31.5	35.9	37.5	37.2	36	34.7	35.1	35.3	34.9	32.4	29
Min. Temp (°C)	7.7	10.0	13.7	18.8	20.5	22.7	24.1	24.2	23.8	19.8	14.3	10.0
Rainfall (mm)	7	20	48	96	262	320	324	282	294	173	21	10

Using this data leads to a calculated weighted mean annual air temperature (WMAAT) of 25.66°C and weighted mean annual pavement temperature WMAPT of 37.57°C for Tangail. Based on the WMAPT values calculated above, the pavement temperatures experienced around the project location are similar to those for areas of north Queensland in Australia.

Queensland Main Roads provides presumptive values for asphalt stiffness for a range of mixes, aggregate sizes and binder types. These presumptive values are shown in Table 2.8.

Table 2.8: Asphalt Design Moduli at WMAPT of 32°C

Asphalt Mix Type	Binder Type	Volume Binder (%)	AC Modulus at heavy operating speed (MPa at 32°C)			
			10 km/hr	30 km/hr	50 km/hr	80 km/hr
DG10 (320)	C320	11	1000	1250	1500	1800
DG14 (320)	C320	10	1000	1550	1850	2200
DG14 (600)	C600	10	1250	1900	2250	2700
DG20 (320)	C320	10	1100	1700	2000	2400
DG20 (600)	C600	10	1350	2050	2450	2900
DG28 (320)	C320	9	1200	1800	2200	2600
DG28 (600)	C600	9	1450	2150	2600	3100

The presumptive values shown in Table 2.8 can be adjusted to any given WMAPT by the following formula:

$$E_{WMAPT} = max\left(1000,\ E_{32^{\circ}C} \times e^{(-0.08 \times [WMAPT-32])}\right)$$

where EWMAPT = asphalt modulus at the desired WMAPT (MPa)

E32°C = asphalt modulus at 32°C (MPa)

WMAPT = WMAPT in °C

The mix types considered representative of those that will be used on this project are the 14 and 28 mm dense graded asphalts with Class 320 bitumen binders (equivalent to a 60/70 penetration grade binder).

The calculated asphalt stiffness values at WMAPT of 37.47°C are shown in Table 2.9. As the WMAPT results are so close, the one value that for 37.47°C has been used for the design.

Table 2.9: Asphalt Design Moduli at WMAPT of 37.47°C

Layer	**Asphalt Mix Type**	**Binder Type**	**Volume Binder (%)**	**AC Modulus at heavy vehicle operating speed (MPa at 32°C)**			
				10 km/hr	**30 km/hr**	**50 km/hr**	**80 km/hr**
DBS - Wearing Course	DG14 (320)	C320	10	1,000	1,001	1,194	1,420
DBS - Base Course	DG28 (320)	C320	9	1,000	1,162	1,420	1,678

Selection of the appropriate asphalt stiffness values is also dependent upon the heavy vehicle operating speed anticipated for the project. Bitumen is a time-load-temperature dependent material, i.e. it's response to an applied load varies with the rate of loading and the material temperature, and, its stiffness reduces not only with temperature but also as the vehicle speed (rate of loading) reduces. For SRTP project, a heavy vehicle operating speed of 80 km/hr is considered appropriate, reflecting the higher operating speeds that will be achieved when the road is upgraded to four lanes.

A comparison of the asphalt stiffness values assumed in the design and those values calculated is shown in Table 2.10.

Table 2.10: Comparison of Asphalt Resilient Modulus Values

Layer	RHD's Standard Value		Corrected Value*		Adopted Value (psi)
	MPa	psi	MPa	psi	
DBS - Wearing Course	2,758	400,000	1,420	205,970	210,000
DBS - Base Course	2,758	400,000	1,678	243,419	250,000

It is recommended that the asphalt stiffness values used in the design be reduced to the values of 210,000 psi for the wearing course and 250,000 psi for the base course.

CHAPTER 3

METHODOLOGY

3.1 General

Appropriate design of the pavement is crucial for the long term performance of the road as well as from the cost perspective.

The following design methodologies were reviewed for application on the Sub Regional Transport Project (SRTP).

Method I: Bangladesh Pavement Design Guide published by Roads and Highways Department (RHD) (MoC, 2005). This is based on both the AASHTO design guidelines from USA and the corresponding Overseas Road Note 31 publication by the UK's Transport Research Laboratory;

Method II: CIRCLY multi-layer mechanistic elastic pavement design software is a powerful package uses state-of-the-art material properties and performance models for the convenient mechanistic analysis and design of pavement. It is an integral component of the Austroads Pavement Design Guide (Austroads, 2010) that is widely used throughout the world. The system calculates the cumulative damage induced by a traffic spectrum consisting of any combination of user- specified vehicle types and load configurations. The program also calculates the various component stresses, strains and displacement.

Method III: Overseas Road Note 31; A Guide to the Structural Design of Bitumen Surface Roads in Tropical and Sub-tropical Countries, UK Transport Research Laboratory, 1993. This contains a number of catalogs from which different pavement layer configurations can be selected to meet particular sub-grade strengths and anticipated cumulative traffic loadings.

Method IV: Indian Road Congress: 37-2001; A Guide to the Flexible Pavement Design in India and neighbouring Countries. This contains a number of catalogues from which different pavement layer configurations can be selected to meet particular sub-grade strengths and anticipated cumulative traffic loadings.

3.2 Selection of Highway/Project

In this study one of the national highways of Bangladesh (N4) was considered. This road is the Joydevpur – Chandra - Tangail - Elenga road.

3.3 Data Collection

Various input parameters are required for the selected design procedures to get the outputs in the form of layer thickness. The inputs can be classified as follows:

- Primary materials variables: These include the resilient moduli or layer coefficients for asphalt concrete, base and sub-base layers, the resilient modulus or the CBR value for roadbed soil, and the traffic volume in terms of 18 kip single axle load.
- Secondary materials variables: These include drainage coefficients and loss of serviceability because of the environmental factors.
- Other variables: Among other variables are reliability level, the overall standard deviation, performance period, analysis period, economic factors, initial serviceability index and terminal serviceability index.

The impacts of secondary and other variables can be addressed after analyzing the effects of the primary materials variables.

3.3.1 Material Properties

Resilient Modulus of Roadbed

Roadbed soil strength parameter is obtained for the selected existing pavements in terms of California Bearing Ratio (CBR). Since for Road Note 31 design procedure the strength input for subgrade is in the form of CBR value. For the AASHTO 93 design Procedure the CBR value can be converted to resilient modulus by the relationships provided in the guide.

Resilient Modulus for Asphalt

Raw data for this design is obtained in the form of elastic modulus of Asphalt Concrete (AC) is in psi.

Resilient Modulus for Base and Sub-base

The resilient modulus for base and sub-base is obtained in the form of their respective elastic moduli.

3.3.2 Traffic Analysis

The AASHTO design procedure for highways is based on cumulative 18 Kip (8.2 MT) Equivalent Single Axle Loads (ESAL) during the design period. Equivalency factors were derived from 1993 AASHTO Road Test Method. The recommended equation to estimate the value of ESAL is given in Equation (3.1).

$$ESAL = D_D \times D_L \times \left[365 \times \sum \left((VEF \times AADT) \times \left(\frac{(1+i)^n - 1}{i} \right) \right) \right] \text{----- (3.1)}$$

Where, ESAL= Equivalent Single Axle Load;

AADT= Annual Average Daily Traffic used in the design for the year 2011;

D_D = Directional Distribution Factor;

D_L = Lane Distribution Factor;

VEF = Vehicle Equivalency Factor; and

i = Annual Growth factor;

n = Design Period.

Vehicle equivalency factor (VEF)/ESAL per vehicle (EV) has been defined with the Equation (3.2).

$$EV = \sum \left(\frac{P_{Measured}}{P_{Standard}} \right)^n \text{------------------------------------ (3.2)}$$

Where,

$P_{Measured}$ = Measured Axle Load (Specific Type of Axle), MT

$P_{Standard}$ = Standard Axle Load (Specific Type of Axle), MT

EV = ESAL per vehicle.

The design Traffic was calculated from the RHD survey report of 2011.

3.3.3 Determination of Secondary and Other Input Variables

Drainage

The AASHTO flexible design procedure provides means to adjust layer coefficients to take into account the effects of varying drainage conditions on pavement performance. Specific guidelines are provided in the guide to select the drainage coefficients under particular conditions. The effect drainage on the performance of flexible pavements is considered in the AASHTO 1993 guide with respect to the effect water has on the strength of the base material and roadbed soil. The approach used is to provide for the rapid drainage of the free water (noncapillary) from the pavement structure by providing a suitable drainage layer and by modifying the structural layer coefficient. The modification is carried out by incorporating a factor m_i for the base and sub-base layer coefficients (a_2 and a_3). The m_i factors are based both on the percentage of time during which the pavements structure will be nearly saturated and on the quality of drainage, which is independent on the time it takes to drain the base layer to 50 percent of saturation.

The recommended drainage coefficients for unbound bases and subbases used in flexible pavement designing in Bangladesh are provided in Table 3.1 (Huang, 1993).

Table 3.1 Drainage coefficient

Quality of drainage		Percentage of time pavement structure exposed to moisture levels approaching saturation			
Rating	Water removed within	Less than 1%	1-5%	5-25%	Greater than 25%
Excellent	2 hours	1.40-1.35	1.35-1.30	1.30-1.20	1.20
Good	1 day	1.35-1.25	1.25-1.15	1.15-1.00	1.00
Fair	1 week	1.25-1.15	1.15-1.00	1.00-0.80	0.80
Poor	1 month	1.15-1.05	1.05-0.80	0.80-0.60	0.60
Very Poor	Never drain	1.05-0.95	0.85-0.75	0.75-0.40	0.40

Design Serviceability Loss, ΔPSI

The serviceability of a pavement is expressed in terms of the present serviceability index (PSI), defined as the pavements ability to serve the type of traffic for which it is designed. The initial and terminal serviceability index must be established to compute the total change in serviceability that is used in the Equation 4.1. Recommended values in AASHTO Guide for flexible pavement, initial serviceability is 4.2 and terminal serviceability is 2 and 2.5 for less traffic and major highways respectively. For relatively minor highways, where economic considerations dictate that initial expenditure be kept low, at P_t of 1.5 may be used. Such a low value of P_t should only be used in special cases on selected classes of highways.

Reliability level

The reliability level used in designing parameters in Bangladesh is based on guidelines provided in Table 3.2. Reliability is defined as the probability that the design pavement will achieve its design life with serviceability higher than or equal to the specified terminal serviceability. Although the reliability factor is applied directly to traffic in the design equation, it does not imply that traffic is the only source of uncertainty. There are many other source of uncertainties, e.g., traffic prediction, material characterization and behavior modeling, environmental conditions, etc. as well as variability during construction and maintenance.

Table 3.2 Reliability level

Functional Classification	Urban	Rural
Interstate and other freeways	85-99.9	80-99.9
Principal arterial	80-99	75-95
Collectors	80-95	75-95
Locals	50-80	50-80

SOURCE: Adapted with permission from *AASHTO Guide for Design of Pavement Structures,* American Association of State Highway and Transportation Officials, 1993.

Overall Standard deviation

Overall standard deviation ranges from 0.40 to 0.50 for flexible pavement. An average value of 0.45 has been used in the analysis for this roadway.

Performance Period

Recommended values for performance period are shown in Table 3.3. Actual performance period data were collected for the pavements samples which are considered in Bangladesh. In case of pavement design in Bangladesh a performance period of twenty years is used.

Table 3.3 Performance Period

Highway Conditions	Analysis Period (Years)
High volume urban	30-50
High volume rural	20-50
High volume paved	15-25

Source: AASHTO Guide for Design of Pavement Structure, 1993

3.4 Data Source

The Roads and Highway Department (RHD) was established for major upgrading and maintenance of National and Regional highways of Bangladesh. RHD is responsible for the development of database regarding design, performance and rehabilitation of national and Regional highways. The establishment of database involves a carefully planned program of data collection, field surveys, site investigations and data analysis.

3.5 Establishing of deign input parameters

After obtaining preliminary data for pavement sections, the next step is to redesign the pavements according to specified design procedures for both the selected methods. The selection of typical values for every input in this design is discussed in more details in Chapter 4.

3.6 Evaluation of Pavement Design

Mechanistic analysis in terms of displacement, stresses, strains and cumulative damage factor (CDF) of selected cases is conducted using the linear, non-linear elastic solutions given by computer program CIRCLY. Poisson's ratios of 0.40, 0.40, 0.35, 0.35, 0.35 and 0.45 for the AC wearing course, base course, base type I, base type II, sub-base and subgrade soils are used. For pavement structural analysis, permanent deformation, rutting, fatigue and thermal crack failure are major distress modes. Critical stress locations within the pavement structure, for analyzing the distress modes for design methods employed in this research are described in Chapter 5.

3.7 Mechanistic Responses

For pavement structural analysis permanent deformation, rutting, fatigue and thermal crack failure are major distress modes. Critical stress locations within the pavement structure, for analyzing the distress modes for design methods employed in this research are described in Chapter 5.

3.8 Comparison of Mechanistic Responses with Pavement Performance

Based on the data and its analysis, the evaluation of selected pavement design procedures can be addressed. The total thickness obtained from AASHTO 1993 Pavement Design Procedure can be compared with the design thickness provided in Road Note 31 for developing countries. According to mechanistic design approach the magnitude of mechanistic responses (stresses, strains and deflections), delivered to pavement section can be used as a measure of pavement damage. Likewise, higher mechanistic responses indicate a lower protection level from damage because of traffic loading. In this study an attempt has been made to relate performance prediction with the mechanistic responses.

CHAPTER 4

PAVEMENT DESIGN

4.1 General

The pavement design of the selected road was conducted in accordance with the AASHTO Guide for Design of Pavement Structures 1993 and Road Note 31.

The pavement design input is also prepared to compare the design with Road Note 31 and multilayer elastic computer software CIRCLY. The following design procedures are applied in pavement analysis and design.

1. AASHTO 1993 Guide for design of Pavement Structures
2. Road Note 31 Pavement Design Procedure
3. IRC: 37-2001 Pavement Design Procedure
4. Application of CIRCLY multilayer elastic computer software

4.2 Data Types

Typical data required for design procedures, used in this research are subgrade strength, traffic, asphalt concrete (AC) and unbound granular material characteristics, reliability and standard deviations, performance criteria and performance period.

4.2.1 Subgrade Strength Evaluation

This is one of the major input requirements for both the design procedures. The roadbed soil should be able to carry the traffic load reaching on its surface within safe limits in terms of deflection and rutting. The determination of roadbed soil strength in turn determines the thickness of the layers overlaying it. Therefore any misjudgment in evaluation of roadbed soil strength may lead to a under or over design for the pavement.

4.2.1.1 Road note 31

CBR is dependent on the type of soil, its density and its moisture content. For designing the thickness of a road pavement, the strength of subgrade should be taken at wettest moisture condition likely to occur in subgrade after road is opened to traffic (Road Note 31, 1993).In Bangladesh in situ CBR values are obtained in the field by Dynamic Cone Penetrometer. A CBR value of 8% typically the Subgrade strength on route N4, is normally fix for design in Bangladesh.

4.2.1.2 IRC: 37-2001

The subgrade whether in cut or fill should be well compacted to utilize its full strength. The current MORT & H Specification for Road & Bridge Works (Third Revision 1995) recommended that the subgrade shall be compacted to 97 per cent of dry density achieved with heavy compaction (modified proctor density) as per IS: 2720 (Part 8). This density requirement is recommended for subgrade compaction for Expressways, National Highways, State Highways, Major District Roads and other heavily trafficked roads. For design, the subgrade strength is assessed in terms of the CBR of the subgrade soil in both fill and cut sections at the most critical moisture conditions likely to occur in-situ.

For determining the CBR value, the standard test procedure should be strictly adhered to as per IS: 2720 (Part 16) "Methods of Test for Soils; Laboratory Determination of CBR".

4.2.1.3 AASHTO 1993

The Effective Roadbed Resilient Modulus (M_R) is used to characterize roadbed soil strength under stress and moisture conditions for pavement design in AASHTO 1993 design guide. Alternatively, the seasonal resilient modulus values may be determined by correlations with soil properties, i.e., clay content, moisture, plasticity index (PI), etc. The purpose of identifying seasonal moduli is to quantify the relative damage of a pavement is subjected to during each season of the year and treat it as part of the overall design. An effective roadbed soil resilient modulus is then established which is equivalent to the combined effect of all the seasonal modulus values.

The first step of this process is to enter the seasonal moduli in their respective time periods. If the smallest season is half month, then all seasons must be in terms of half months and each of the boxes must be filled. If the smallest season is one month, then all seasons must be in terms of whole months and only one box per month may be filled in. The next step is to estimate the relative damage (u_r) values corresponding to each seasonal modulus.

The relative damage u_r is described by the Equation (4.1)

$$u_r = 1.18x10^8 M_R^{-232} \quad \text{------------------------------------} \quad (4.1)$$

The average relative damage (u_f) is computed by taking the average of u_r of all seasons. The effective subgrade resilient modulus is then given by Equation (4.2)

$$M_R = 3015.u_f^{-0.431} \text{------------------------------------} \quad (4.2)$$

The modulus, M_R, is a function of CBR

$$M_R = K \times CBR \text{ (MPa/psi)}$$

Where 'K' factor depends upon soil types as well as on the CBR value

AASHTO 1993 Pavement Design Guide has listed the relationship by using dynamic compaction in situ soil modulus as follows:

$$M_R = 1500 \times CBR \text{ (psi)}$$

$$M_R = 10 \times CBR \text{ (MPa)}$$

With the reservation, data from which this correlation is developed ranged from 750-3000 times CBR. This relationship has been used extensively by design agencies and researchers and is considered reasonable for fine-grained soils with a soaked CBR of 10 or less.

Table 4.1 Monthly Effective Roadbed Soil Modulus Data for N4

Month	Monthly M_R (Psi)	Relative Damage u_r
January	20000	0.012
February	20000	0.012
March	2800	1.187
April	4500	0.394
May	6500	0.168
June	7200	0.132
July	7600	0.117
August	8000	0.103
September	8000	0.103
October	7500	0.120
November	10000	0.061
December	18000	0.015
		$\sum u_r$=2.424

Average u_r=2.424/12=0.202. From Table 4.1 and using Equation, $u_r = 1.18x10^8 M_R^{-232}$, it was calculated that the effective roadbed soil resilient modulus, M_R = 6000 psi, where u_r=0.202.

From AASHTO source of correlation, resilient modulus of roadbed soil for N4 highway is determined from the seasonal resilient modulus values i.e. for CBR value of 8, M_R = 1500 x 8 = 12000 psi; design value is 11600 psi (From Table 2.3).

4.2.2 Traffic

Traffic is recorded as average annual daily traffic (AADT) in National Housing Authority's (NHA) database based on route-kilometer post method of referring sections. In route-kilometer post method, each route is given a unique name or number and the starting point of route is defined. Then, the kilometer posts are sequentially numbered along the whole length of the route. AADT is defined as the total annual traffic in the both directions and divided by 365. It is usually obtained by recording actual traffic flows over a shorter period from which the AADT is then estimated.

4.2.2.1 Traffic Data Selection

Traffic Data in terms of AADT collected for N-4 roads by Roads & Highway Department (RHD) is tabulated in Table 4.2. AADT for the year 2011 along the entire N4 highway length is shown in Figure 4.1 to Figure 4.3. In this research, collected traffic data for the year 2011 were used.

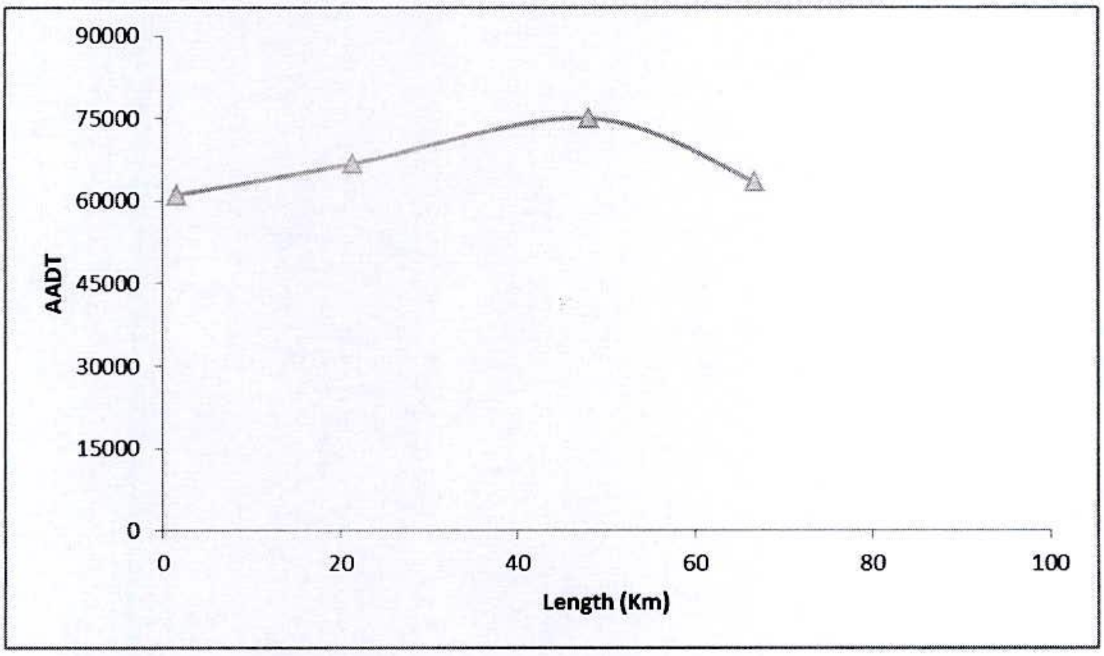


Figure 4.1 AADT (20th year) Vs Length (km) for year 2011

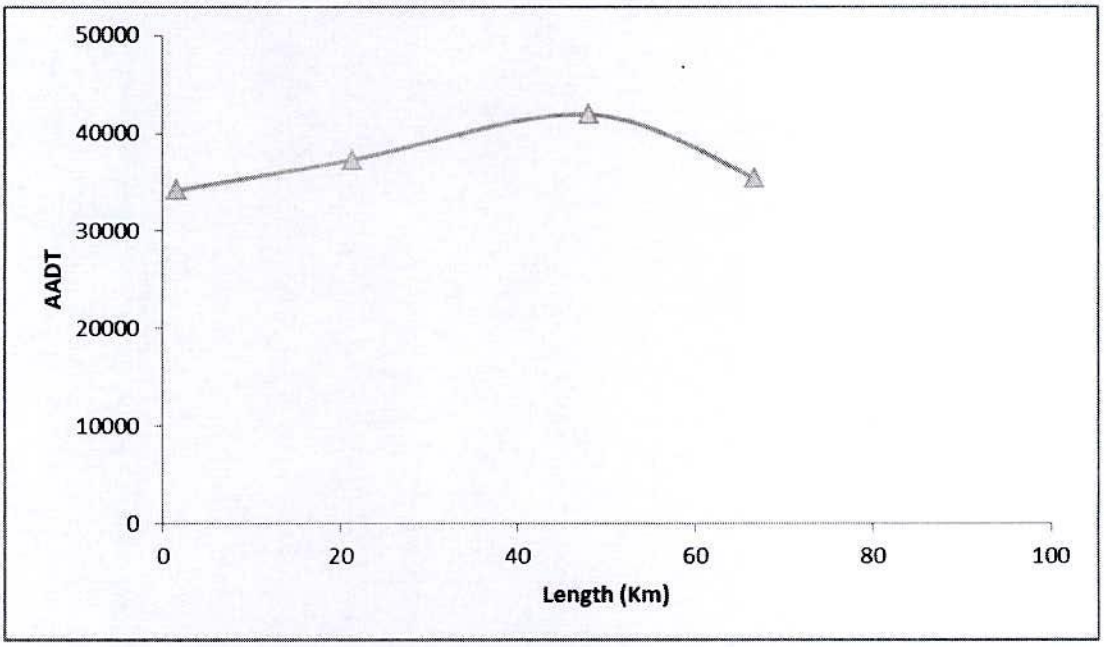


Figure 4.2 AADT (10th year) Vs Length (km) for year 2011

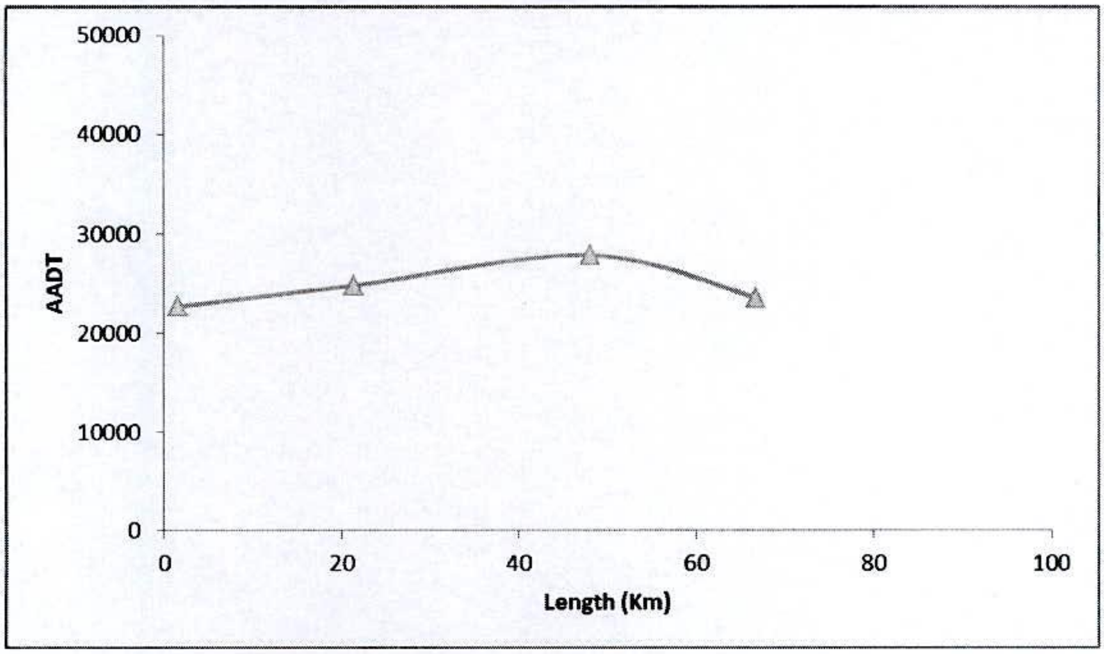


Figure 4.3 AADT (3rd year) Vs Length (km) for year 2011

Table 4.2 Traffic Volume Data , N4: Joydevpur – Chandr - Tangail - Elenga Road, Source: RHD Traffic Volume Data, Survey Year: 2011.

Link Name	Average Annual Daily Traffic (AADT)														
	Motorized										Non-motorized			Total	
	Truck			Bus			Utility	Car	Auto-rickshaw	Motor-cycle	Bicycle	Cycle-rickshaw	Cart		
	Heavy	Medium	Small	Large	Medium	Micro								MT	NMT
Kadda - Chandra	96	4481	1702	761	1443	1164	1484	1293	1927	1144	325	1597	0	15495	1922
Chandra – Kaliakoir Bus Stand	100	5435	1101	2599	319	1211	1391	1421	475	455	78	297	0	14507	375
Kaliakoir Bus Stand - Gorai	166	5447	1882	2386	1025	1446	1287	1526	1249	974	619	1359	0	17388	1978
Tangail - Elenga	50	5150	654	3042	402	855	1311	798	1862	500	102	97	0	14624	199

4.2.2.2 Design ESALs

Detail about obtaining equivalent single axle load (ESAL) for design is discussed in chapter 3(Equation 3.1 and 3.2). The following data has been used to determine the design ESAL.

$$ESAL = D_D \times D_L \times \left[365 \times \sum\left((VEF \times AADT) \times \left(\frac{(1+i)^n - 1}{i}\right)\right)\right]$$

According to RHD Pavement Design Guideline 2005, the vehicle equivalency factor (VEF) is given in Table 2.2.

D_D = Expected Directional Distribution = 0.8,

D_L = Design Lane Factor = 0.5,

VEF = Vehicle Equivalency Factor,

Traffic Growth rate (Truck) = 8%,

Traffic Growth rate (Bus) = 7%, and

Design life = 20 years

This procedure was followed in calculating design ESAL for N4 highway.

4.2.2.3 Structural Number required carrying future Traffic (SN)

The objective of the design using the AASHTO method is to determine a flexible pavement structural number (SN) adequate to carry the design ESAL. This design procedure is used for ESALs greater than 50,000 for the performance period. The design for ESALs less than this is usually considered under low-volume roads.

The current basic design equation given in the AASHTO 1993 Guide is as given in Equation (4.3).

$$\log_{10} W_{18} = Z_R S_o + 9.36 \log_{10}(SN+1) - 0.20 + \frac{\log_{10}[\Delta PSI/(p_i - p_t)]}{0.40 + [1094/(SN+1)^{5.19}]} + 2.32 \log_{10} M_r - 8.07$$

---------- (4.3)

Where,

W_{18} = Estimated total number of 18- kip (8.2-ton) ESAL applications;

Z_R = Standard normal deviation for the given Reliability (%);

S_0 = Overall standard deviation;

M_R = Effective roadbed soil resilient modulus;

ΔPSI = Design serviceability loss; and

SN = Design structural number

Knowing SN it is possible to determine the thickness of the different layers of the pavement structure using the following expression:

$$SN \leq a_1 * D_1 + a_2 * D_2 * m_2 + a_3 * D_3 * m_3 \text{ --------------------------- (4.4)}$$

Where:

SN = Required AASHTO Structural Number;

D_1, D_2, D_3 = Pavement Layers Thickness (AC, Granular Base and Sub-base);

a_1, a_2, a_3 = Structural Coefficient representatives of each pavement layer; and

m_2, m_3 = Drainage Coefficient of each granular layer

4.2.2.4 Selection of Representative ESAL Values

ESAL values along the entire N4 route length are shown in from Figure 4.4 to Figure 4.6. This shows that the cumulative ESAL value is 30 million for 3rd year, ESAL value is 133.7 million for 10th year and 375 million for 20th year, respectively.

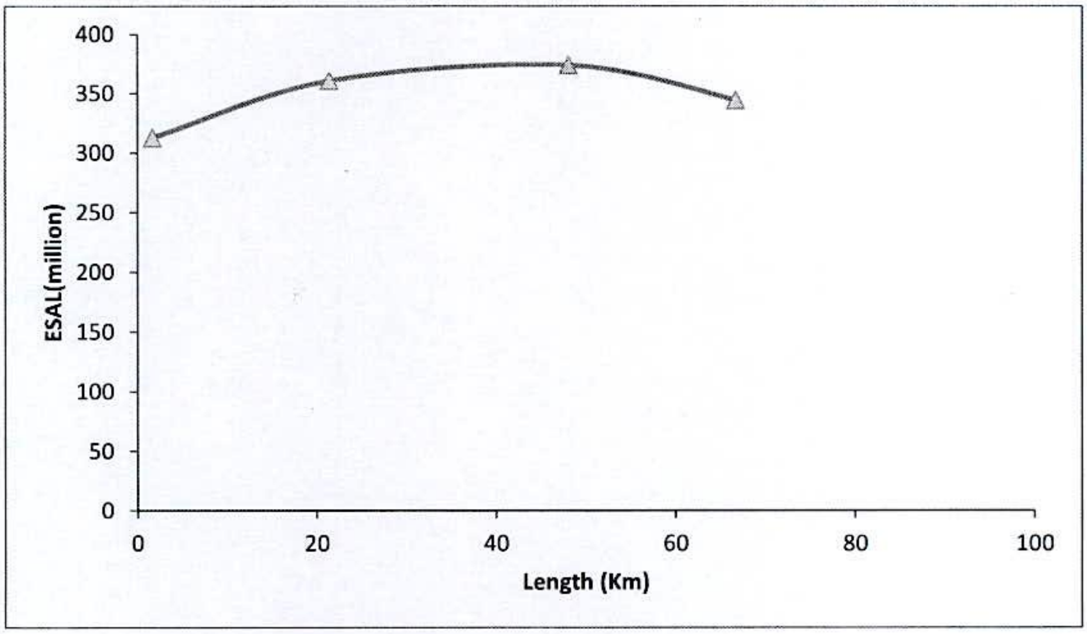


Figure 4.4 ESALs (20^{th} year) Vs Length (km) for the year 2011

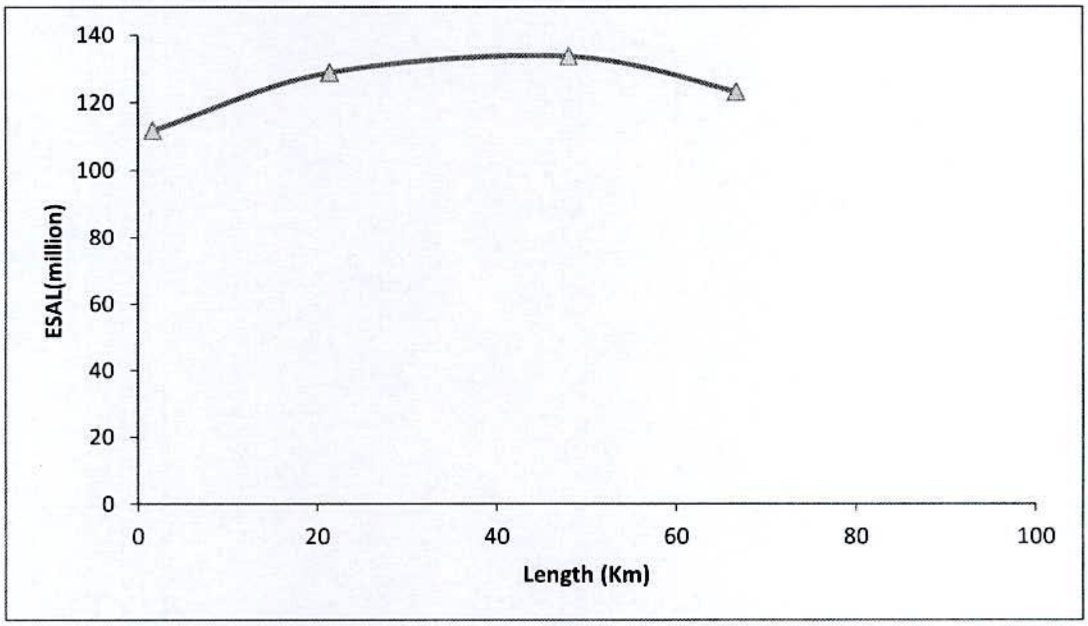


Figure 4.5 ESALs (10^{th} year) Vs Length (km) for the year 2011

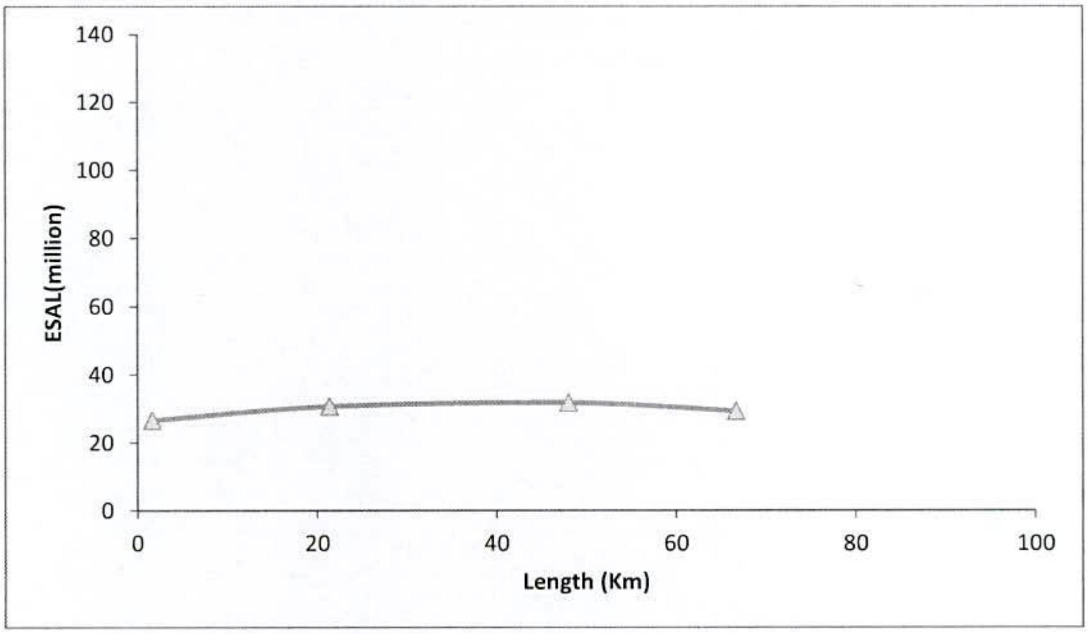


Figure 4.6 ESALs (3^{rd} year) Vs Length (km) for the year 2011

4.2.3 Asphalt concrete (AC) and unbound granular material characteristics

4.2.3.1 Asphalt Concrete layer

According to the AASHTO 1993 design guide, elastic (resilient) modulus of dense graded asphalt concrete represents the asphalt material characterization for pavement design ranges from 100 ksi to 500 ksi. Typical values of elastic modulus of AC for Bangladesh are 300 ksi to 450 ksi. The layer coefficients for wearing surfaces have been obtained from the AASHTO 1993 pavement design guide.

The viscosity of bituminous binders and its variation with temperature is of great practical importance. The standard test for viscosity is known as Penetration Test, which indicates that greater the penetration distance, the softer is the bitumen and the lower its viscosity. According to Road Note 31 guidelines, grade/Temperature of 60/70 or 80/100 range are suitable and a typical grade of 80/100 is used in Bangladesh.

4.2.3.2 Unbound Granular Material Characterization

The Road Note 31 pavement design procedure assigns granular material based on their mechanical properties such as particle size distribution and particle shape providing high mechanical stability and a dense material when compacted. The AASHTO 1993 pavement design procedure considers material characterization in terms of resilient modulus which accounts for stress/strain behavior of the material. The typical values recommended by AASHTO pavement design guide 1986 and Interim pavement design guide 1972 are:

Sub-base M_R= 15,000 psi (for CBR=30, a_3=0.11)

Crushed Base Course M_R= 30,000psi (for CBR=90, a_2=0.14)

The value of granular material increases as the quality of material increases. Therefore the values of 15000 psi for sub-base and 30000 psi for granular base course are reasonable for work and material quality used in Bangladesh, and hence is selected for design by Roads & Highway Department. These material properties relate to the following design classes in Road Note 31 pavement design procedure.

Base = GB1 - GB3 CBR>80

Sub-base = GS CBR>30

Where, GB1 = Granular road base type 1 (Graded crushed stone)

GB2 = Granular road base type 2 (Dry/water bound macadam)

GB3 = Granular road base type 3 (Natural coarsely graded granular material)

GS = Granular sub-base (Natural gravel)

Granular sub-base materials confirm the Clause 401 of MORT and H specifications of Road and Bridge Works in India (IRC 37, 2001). These specifications suggest three grading for coarse graded granular sub-base materials and specify that the materials passing 425 micron sieve when tested in accordance with IS:2720 (Part 5) should have liquid limit and plasticity index of not more than 25 and 6, respectively. These requirements and the specified grain size distribution of sub-base material should be strictly enforced in order to meet stability and drainage requirements of the granular sub-base layer.

4.2.4 Reliability and Standard Deviations

The cumulative ESAL is an important input to any pavement design method. However, the determination of this input usually is based on assumed growths, which may not be accurate. The AASHTO 1993 guide has proposed the use of a reliability factor that considers the possible uncertainties in traffic prediction. Suitable ranges for reliability level against functional classification of roads are provided in the guide. For structural design of N4 highway a reliability of 85% and a standard deviation (S_o) of 0.45 is recommended, based on guidelines provided by AASHTO 1993 pavement design guide.

4.2.5 Performance Criteria for Functional Classification

The serviceability of a pavement is defined as its ability to serve the type of traffic (automobiles and trucks) which use the facility. The primary measure of serviceability is the Present Serviceability Index (PSI), which ranges from 0 (impassable road) to 5 (perfect road). The basic philosophy of AASHTO 1993 guide is the serviceability – performance concept which provides a means of designing a pavement based on a specific total traffic volume and a minimum level of serviceability desired at the end of the performance period.

An index of 2.5 or higher is suggested for design of major highways and 2.0 for highways with lesser traffic volumes. For relatively minor highways where economic dictate that the initial capital outlay be kept at a minimum, it is suggested that this be accomplished by reducing the design period or the total traffic volume, rather than by designing for a terminal serviceability less than 2.0.

Since the time at which a given pavement structure reaches its terminal serviceability depends on traffic volume and the original or initial serviceability (p_o), AASHTO 1993 recommends the value of P_o is 4.2 for flexible pavement design. The soil types found on N4 route are not expected to cause serviceability losses of 0.25, due to swelling. Therefore the values adopted for design are: $\Delta PSI=1.7$, $P_0 = 4.2$ and $P_t= 2.5$

4.2.6 Performance Period

Recommendations of AASHTO 1993 pavement design guide about performance period are discussed in preceding chapters. For pavement design of N4 highway, a performance period of 20 years is assumed before any major rehabilitation or reconstruction is carried out.

4.2.7 Drainage facilities

A good drainage system should ensure proper side drains and culverts and their proper functioning (Road Note 31). Recommendations of AASHTO 1993 pavement design guide about drainage facilities are described earlier. For N4 highway design, adopted values of drainage coefficient for base (m_2), and for sub-base (m_3) are 1.0 and 0.9, respectively. This corresponds to good quality of drainage, in which water will be drained out within a day.

4.3 Establishment of Pavement Design input parameters

Until now the ranges have been fixed for all the input parameters that are required for the pavement design by AASHTO 1993 Design of Pavement Structures, and Road Note 31 Pavement Design Procedure for tropical and subtropical countries. The next step is the formation of the tables of design input parameters. Table 4.3 and Table 4.4 show the design input parameters for this research. The cumulative ESAL for different four cases are shown in Table 4.3.

There are four cases considered in National Highway N4 on Joydevpur – Chandra - Tangail - Elenga road:

Case 1: Kadda – Chandra;

Case:2: Chandra – Kaliakoir Bus Stand;

Case 3: Kaliakoir Bus Stand – Gorai; and

Case 4: Tangail – Elenga.

Table 4.3: Traffic input parameters

Case No.	AADT			Cumulative ESAL(million)		
	3rd year	10th year	20th year	3rd year	10th year	20th year
1	22690	34118	61100	26.65	111.80	312.90
2	24803	37294	66789	30.74	128.90	360.80
3	27859	41889	75017	31.87	133.70	374.10
4	23552	35414	63421	29.34	123.10	344.40

Table 4.4: Material properties input parameters

Component Layers	CBR (%)	Resilient Modulus (M_R) Psi	Resilient Modulus (M_R) MPa
Subgrade	8	11600	80
Sub-base	30	29000	200
Base Type - II	55	43500	300
Base Type - I	90	50800	350
Base Course	100	250000	1678
Wearing Course	-	210000	1420

4.4 Design Output for 10th year Cumulative ESAL Load by AASHTO 1993 and IRC 37 Design Guide for Design of Pavement Structures

The design output of four cases is shown in Table 4.5 and Table 4.6 in English and Metric units respectively:

Table 4.5 Cases designed by AASHTO 1993 Pavement Design Procedure (Metric Units)

Case No.	SN	Total Thickness (inch)	Layer Thickness (inch)					Layer Modulus (Ksi)				
			WC	Base Course	Base Type -I	Base Type-II	Sub base	WC	Base Course	Base Type-I	Base Type-II	Sub base
1	5.82	845	50	120	225	225	225	1420	1678	350	300	200
2	5.93	875	50	150	225	225	225	1420	1678	350	300	200
3	5.95	875	50	150	225	225	225	1420	1678	350	300	200
4	5.89	875	50	150	225	225	225	1420	1678	350	300	200

Design of Flexible Pavement as per AASHTO 1993 Pavement Design Guide and Chart [Appendix E]

Table 4.6 Cases designed by IRC 37 Pavement Design Procedure (Metric Units)

Case No.	CBR (%)	Total Thickness (mm)	Layer Thickness (mm)				Layer Modulus (MPa)			
			WC	Base Course	Base	Sub-base	WC	Base Course	Base	Sub-base
1	8.00	652.50	50.00	152.50	250.00	200.00	2800	2800	350	200
2	8.00	652.50	50.00	152.50	250.00	200.00	2800	2800	350	200
3	8.00	652.50	50.00	152.50	250.00	200.00	2800	2800	350	200
4	8.00	652.50	50.00	152.50	250.00	200.00	2800	2800	350	200

Design of Flexible Pavement as per IRC 37 Pavement Design Catalogue, Plate-2 [Appendix E]

4.5 Comparison of Design Output for 3rd year Cumulative ESALs Load by AASHTO 1993 Guide with Road Note 31 and IRC 37 for Design of Pavement Structures

As Road Note 31 design guide has been prepared to carry up maximum 30.00 million equivalent standard single axles load, the comparison will be made by this loads. The design output of four cases is shown in Table 4.7 to Table 4.12 in English and Metric units respectively

Table 4.7 Cases designed by AASHTO 1993 Pavement Design Procedure (English Units)

Case No.	Structural No.	Total Thickness (inch)	Layer Thickness (inch)			Layer Modulus (ksi)		
			AC	Base	Sub-base	AC	Base	Sub-base
1	4.76	19.00	7.60	6.40	5.00	400	50.8	29
2	4.85	19.00	7.60	6.40	5.00	400	50.8	29
3	4.86	19.00	7.60	6.40	5.00	400	50.8	29
4	4.85	19.00	7.60	6.40	5.00	400	50.8	29

Table 4.8 Cases designed by AASHTO 1993 Pavement Design Procedure (Metric Units)

Case No.	Structural No.	Total Thickness (mm)	Layer Thickness (mm)			Layer Modulus (MPa)		
			AC	Base	Sub-base	AC	Base	Sub-base
1	4.75	475.00	190.00	160.00	125.00	2800	350	200
2	4.83	475.00	190.00	160.00	125.00	2800	350	200
3	4.84	475.00	190.00	160.00	125.00	2800	350	200
4	4.84	475.00	190.00	160.00	125.00	2800	350	200

Design of Flexible Pavement as per AASHTO 1993 Pavement Design Chart [Appendix E]

Table 4.9 Cases designed by Road Note 31 Pavement Design Procedure (English Units)

Case No.	Subgrade Strength Classes	Traffic Classes	Total Thickness (inch)	Layer Thickness (inch)		
				AC	Base	Sub-base
1	S4	T8	23.0	6.0	10.0	7.0
2	S4	T8	23.0	6.0	10.0	7.0
3	S4	T8	23.0	6.0	10.0	7.0
4	S4	T8	23.0	6.0	10.0	7.0

Table 4.10 Cases designed by Road Note 31 Pavement Design Procedure (Metric Units)

Case No.	Subgrade Strength Classes	Traffic Classes	Total Thickness (mm)	Layer Thickness (mm)		
				AC	Base	Sub-base
1	S4	T8	575.00	150.00	250.00	175.00
2	S4	T8	575.00	150.00	250.00	175.00
3	S4	T8	575.00	150.00	250.00	175.00
4	S4	T8	575.00	150.00	250.00	175.00

Here, S4 = Subgrade Strength Class (CBR% = 8 to 14) and

T8 = Traffic Class (ESAL= (17 to 30) x 10^6 [Appendix E]

Table 4.11 Cases designed by IRC 37 Pavement Design Procedure (English Units)

Case No.	Subgrade CBR (%)	Cumulative Traffic (msa)	Total Thickness (inch)	Layer Thickness (inch)		
				AC	Base	Sub-base
1	8	30	23.6	5.6	10.0	8.0
2	8	30	23.6	5.6	10.0	8.0
3	8	30	23.6	5.6	10.0	8.0
4	8	30	23.6	5.6	10.0	8.0

Table 4.12 Cases designed by IRC 37 Pavement Design Procedure (Metric Units)

Case No.	Subgrade CBR (%)	Cumulative Traffic (msa)	Total Thickness (mm)	Layer Thickness (mm)		
				AC	Base	Sub-base
1	8	30	590.00	140.00	250.00	200.00
2	8	30	590.00	140.00	250.00	200.00
3	8	30	590.00	140.00	250.00	200.00
4	8	30	590.00	140.00	250.00	200.00

Design of Flexible Pavement as per IRC 37 Pavement Design Catalogue, Plate-2 is as shown in Appendix E.

4.6 Comparison of results obtained by the Three Design Procedures

Several important points are inferred from the eight cases designed by the AASHTO 1993 Pavement Design Procedure and Road Note 31 Pavement Design Procedure. Following discussed aspects are the most significant.

4.6.1 Asphalt Concrete Layer

The following Figure 4.7 shows the comparison of asphalt concrete layer recommended by the two design procedures.

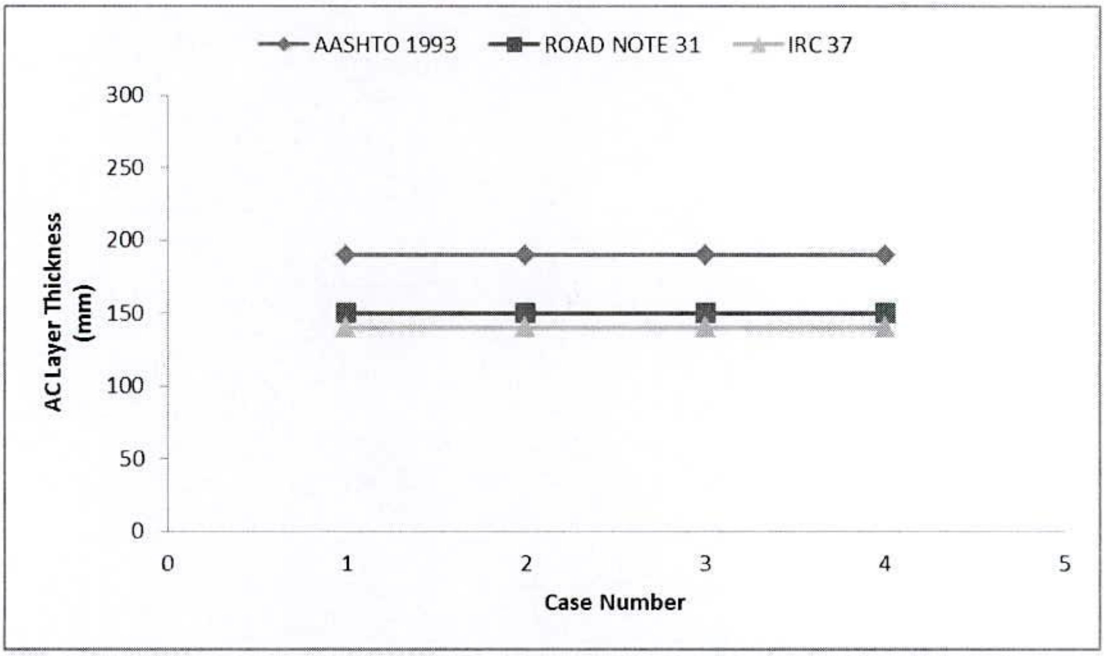


Figure 4.7 AC layer thickness obtained by AASHTO 1993, Road Note 31 and IRC 37 Pavement design Procedures

It is observed in Figure 4.7 that AASHTO 1993 design procedure gives much higher thickness for the asphalt concrete layer than is recommended by Road Note 31 and IRC 37 design procedures. The less thickness of asphalt concrete recommended by Road Note 31and IRC 37 design procedures may be economical in the construction stage, but may decrease the structural capacity of the pavement.

4.6.2 Base Layer Thickness

The Figure 4.8 shows the comparison of base thickness obtained for all cases designed by the selected two pavement design procedures.

It is seen that the higher base thickness is recommended by Road Note 31 and IRC 37 Pavement Design Procedures. This thicker value may be provided to offset the less asphalt concrete layer given by the Road Note 31 and IRC 37 design methods. The analysis of mechanistic response obtained in Chapter 5 supports the above statement.

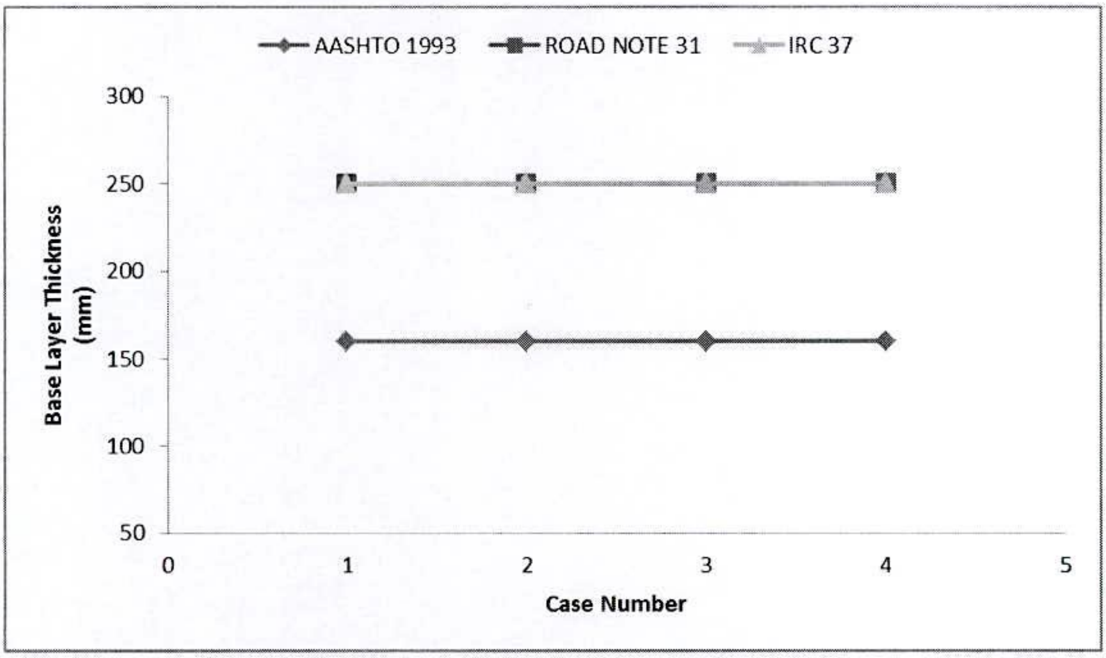


Figure 4.8 Base layer thickness obtained by AASHTO 1993, Road Note 31 and IRC 37 Pavement design Procedures

4.6.3 Sub-base Layer Thickness

The Figure 4.9 is obtained by comparing the sub-base thickness recommended by the two pavement design procedures.

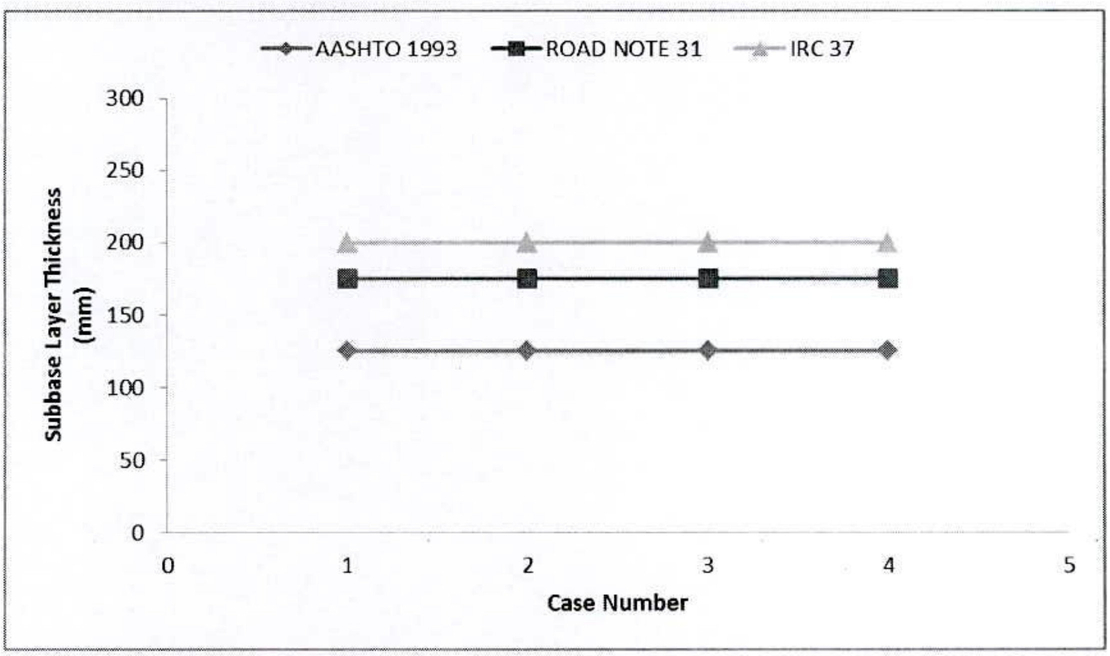


Figure 4.9 Sub-base layer thickness obtained by AASHTO 1993, Road Note 31 and IRC 37

The same trend is shown in case of sub-base also as is the case for base, i.e., Road Note 31 and IRC 37 Pavement Design Procedures recommend higher thickness for sub-base layers. Until now it is seen that the lower asphalt layer thickness obtained in case of Road Note 31 and IRC 37 design procedures is compensated by thicker base and sub-base. The evaluation of these approaches were carried out by the analysis of the three design procedures in terms of their mechanistic responses.

4.7 Total Design Thickness

Figure 4.10 shows the total design thickness for all cases designed by the three selected design procedures. It is seen that the total design thickness obtained by the three design procedures is approximately same, with AASHTO 1993 Procedure giving a slightly lower thickness in most of the cases than the Road Note 31 and IRC 37 Pavement Design Procedures. This lower value can be explained by the presence of thick asphalt layer recommended by the AASHTO 1993 design which consequently results in less base and sub-base layers as shown previously. With nearly equal thickness for the pavement structure, Road Note 31 and IRC 37 Pavement Design Procedures tend to provide an economical design by suggesting lower thickness for asphalt concrete layer.

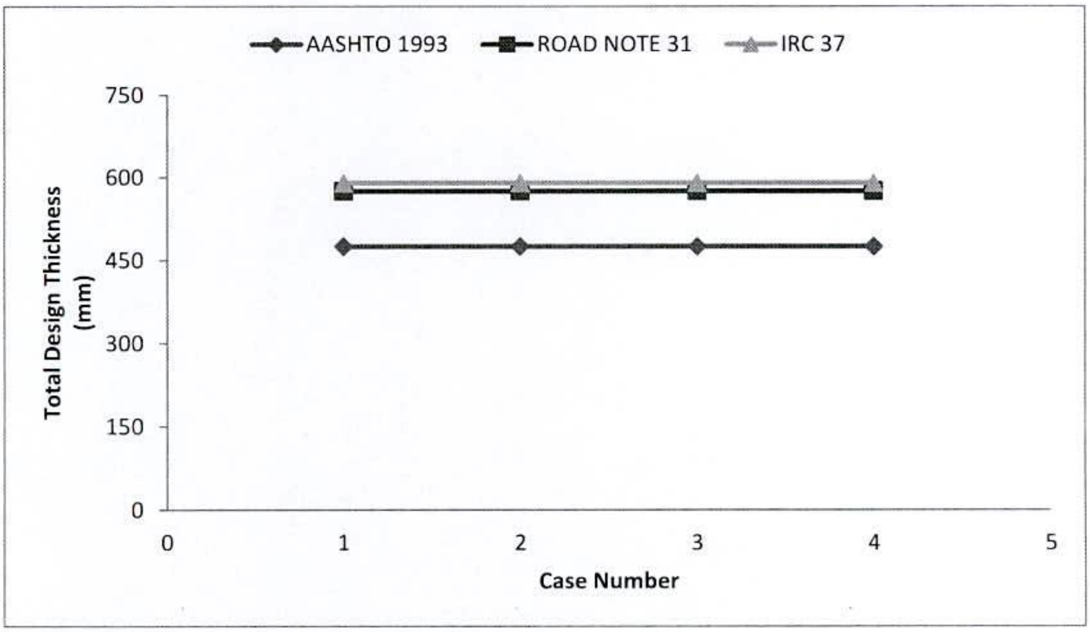


Figure 4.10 Total design thicknesses obtained by AASHTO 1993, Road Note 31 and IRC 37 Pavement design Procedures

4.8 Summary

The objective of this chapter is to design the pavement according to design input parameters from Bangladesh. Reasonable ranges of variables are fixed. Four representative design cases as described in Clause 4.3 for N4 highway were designed by AASHTO 1993 Guide for Design of Pavement Structures with Road Note 31 and IRC 37 Pavement Design Procedures for Tropical and Sub-Tropical countries. The differences in design obtained by all the methods are compared. This comparison helps in providing a general idea about the design approach carried out by the design procedures. Thus evaluation of methods is carried out based on mechanistic approach of pavement design.

CHAPTER 5

EVALUATION OF PAVEMENT DESIGN METHODS

5.1 General

The empirical methods of flexible pavement design do not consider the load spreading abilities of the pavement layers, and are such restricted to comply with the varying traffic loads without expensive field performance results. The stresses and strains developed by traffic in these structures are obtained by the solution of general elastic equations representing the behavior of elastic layered systems. This theory calls for fixing the thickness of layers so that the intensity of the stresses in different layers is within the permissible limits for the material used in respective layers of the pavement structure. Application of elastic layer theory requires the determination of the elastic properties of layers such as Poisson's ratio (v,) and elastic modulus (E) of the materials in each layer. Methods for their determination are explained in preceding chapters. Typical values of these properties used in this research (which are the material properties used for road construction in Bangladesh) were obtained from Roads and Highway Department, Bangladesh.

5.2 Modes of Distress

Generally fatigue, permanent deformation and rutting and thermal cracking are the three major modes of asphalt pavement distress, and these are analyzed for the evaluation of selected design procedures for this research, i.e.,

- AASHTO Guide for Design of Pavement Structures 1993
- Overseas Road Note 31 (A Guide to the Structural Design of Bitumen-Surface Roads in Tropical and Sub-tropical Countries) 1993
- Indian Road Congress 37 (A Guide to the Structural Design of Flexible Pavement) 2001

5.2.1 Permanent deformation and Rutting

The underlying soil should be protected from excessive stresses produced by traffic loads otherwise; excessive deformation of the subgrade can cause cracking of the overlaying layers. To check permanent deformation the vertical compressive stress reaching at the top of each layer should be kept within the permissible limits of the material. Rutting is the longitudinal depressions formed along the wheel paths, with small upheavals on the sides, after increasing number of load applications.

5.2.2 Fatigue Cracking

Fatigue cracking at the bottom of asphalt layer is dependent on the cumulative action of repetitive traffic loads. The passage of a wheel load causes the pavement to deflect. Monismith (1992) described that the larger this deflection is and the higher the frequency of its occurrence, the greater the propensity for fatigue cracking. The fatigue effect associated with this horizontal tensile strain at bottom of asphalt concrete layer.

5.2.3 Thermal Cracking

Thermal cracking is associated with occasional very heavy load particularly at low temperatures, initiating cracks at the bottom of asphalt layer. At low temperatures, initiating cracks at the bottom of asphalt layer is very stiff, and a heavy load may produce a large tensile force at bottom of asphalt layer. This is checked by controlling the horizontal tensile stress at bottom of asphalt concrete layer. Figure 5.1 shows the critical stress location within the pavement structure.

Critical stress locations within the pavement structure, for analyzing the distress modes for design methods employed in this research are:

- Deflection at top of asphalt concrete layer, base, sub-base and roadbed. (Rutting)
- Vertical Compressive stress at top of AC, base, sub-base and roadbed. (Permanent deformation)
- Vertical micro strain at top/bottom of layers. (Rutting)
- Tensile micro strain at bottom of AC layer. (Fatigue cracking)
- Tensile stress at bottom of AC layer. (Thermal cracking)

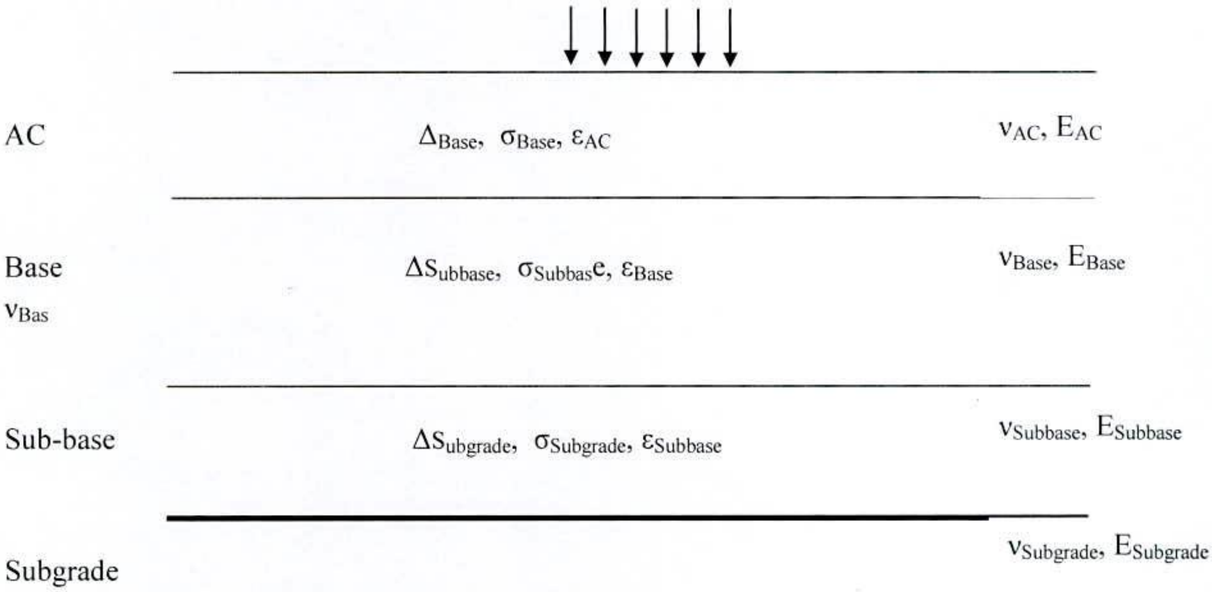


Figure 5.1 Critical Stress Locations within the Pavement Structure

Where, ΔBase, ΔSubbase, ΔSubgrade, = Deflection at top of base, sub-base and asphalt concrete

σBase, σSubbase, σSubgrade = Vertcal stresses at top of base, sub-base and asphalt concrete

εAC, εBase , εSubbase = Horizontal strains at top of base, sub-base and asphalt concrete

5.3 Traffic Loading

For calculating stresses and strains in layered systems, the load was considered to be applied to a circular contact area as a uniformly distributed stress. A single wheel was considered as the application point of load because it was suggested by Dormon and Metcalf (1965) that consideration of other wheel loads such as tandem axles, will not have any adverse influence on the stress conditions except at considerable depth. Even in these circumstances the effect is very small and can be ignored.

A typical load of 9000 lb (half of 18 kip ESAL on one tire) was selected for this research. (RHD Pavement Design Guide, April 2005, AASHTO Pavement Design Guide 1993) Representative tire pressure for heavy vehicles in Bangladesh is 0.75 MPa. This gives radius of circular contact area as 92.1 mm. Stress values were considered directly below the point of application of the load, since maximum stress results at a radial distance of zero.

5.3.1 Load Condition as per AASHTO 1993 Guide

In the AASHTO 1993 design method, the traffic load is determined in terms of the number of repetitions of an 18000 lb (80 KN) single axle load applied to the pavement on two sets of dual tires. This is usually referred to as the equivalent single axle load (ESAL).The dual tires are represented as two circular plates, each 4.51 in. radius, spaced 13.57 in apart. This representation corresponds to a contract pressure of 70 lb/in.

The equivalency factors used in this case are based on the terminal serviceability index to be used in the design and the structural number (SN). The total ESAL applied on the highway during its design period can be determined only after the design period and traffic growth factors are known. Flexible pavements are usually designed for a 20-year period.

The growth factors (G_m) for different growth rates and design periods can be obtained from Equation 5.1.

$$G_m = \left[\frac{(1+r)^n - 1}{r}\right] \text{--} (5.1)$$

Where, r = i/100, i = growth rate and n = design life, years

The total Equivalent Single Axle Load (ESAL) over the design period can be determined using Equation 5.2 and Equation 3.1.

$$ESAL_i = f_d x G_m x AADT_i x 365 x N_i x F_{Ei} \text{------------------------------} (5.2)$$

Where

$ESAL_i$ = equivalent accumulated 18000-lb (80 KN) single axle load for the axle category I;

f_d = design lane factor;

G_m = growth factor for a given growth rate r and design period n;

$AADT_i$ = first year annual average daily traffic for axle category I;

N_i = number of axles on each vehicle in category I; and

F_{Ei} = load equivalency factor for axle category i

AASHTO 1993 code is considered for national and regional highways that operates large volume of traffic.

5.3.2 Load Condition as per Road Note 31 Procedure

For pavement design purposes, it is necessary to consider not only the total number of vehicles that will use the road but also the wheel loads of this vehicles. The total number and the axle loading of the heavy vehicles that will use the road during its design life need to be considered. In this context, heavy vehicles are defined as those having an unladen weight of 3000 kg or more.

For pavement design purposes, the damaging power of axles is related to a standard axle of 8.16 tones using equivalency factors which have been derived from empirical studies is shown in Equation 5.3. It is necessary to express the total number of vehicles that will use the road over the design period in terms of the cumulative number of equivalent standard axles (esa).

$$\text{Equivalence factor} = \left(\frac{Axleload\,(kg)}{8160}\right)^{45} \text{--} (5.3)$$

It is also noted that Road Note 31 procedure is considered for small volume of traffic (maximum 30 million standard axles).

5.3.3 Load Condition as per IRC 37 Code

In Standard Axles method traffic is defined in terms of the cumulative number of standard axles (8160 kg) to be carried out during the design life of the road. Its computation involves the estimation of the initial volume of commercial vehicles per day, lateral distribution of traffic, the growth rate, the design life and the vehicle damage factor to convert commercial vehicles to standard axles.

Equation 5.4 can be used to make the required calculation.

$$N_s = \frac{365xA(1+r)^n - 1}{r} xF \text{--} (5.4)$$

Where,

N_s = the cumulative number of standard axles to be catered for in the design;

A = initial traffic, in the year of completion of construction, in terms of the number of commercial vehicles per day;

r = annual growth rate of commercial traffic;

x = design life in years; and

F = vehicle damage factor

IRC 37 code is considered for heavy volume of traffic of the order of maximum 150 million standard axles..

5.3.4 Loading Condition for RHD Guideline of Bangladesh based on AASHTO 1993 and Road Note 31 Guide

For pavement design purposes all heavy commercial vehicles are expressed in terms of the equivalent number of standard axles. A Standard Axle is taken to be 8160 Kg. Based on axle load studies in Bangladesh, the equivalence factors have been determined which are shown in Table 2.2. Using the estimated AADT for the vehicle categories together with their equivalence factors shown in Table 2.2, estimates should be made of the current daily ESAs for the road. This should be multiplied by 365 to obtain the annual ESAs for the road. The cumulative ESALs over the design period can be determined using Equation 3.1.

To obtain the cumulative ESA loading over the design life of the road, the current annual ESA loading should be multiplied by one of the following factors:

For National Road: 57.3; For Regional Road 41.0

These factors have been derived from the following formula

Cumulative Growth Factor $= \frac{(1+r)^n - 1}{r}$ where r=annual traffic growth rate and n= design life in years.

For National Roads r=10% and n=20years; For Regional Roads r=7% and n=20 years.

In the RHD Guideline, it assumes that the minimum CBR value of sub-grade is 5%. In Bangladesh, apart from higher ground within the Chittagong Hill Tracts where in situ CBRs will be higher, most roads are constructed on embankment that will have a CBR values less than 5%. Under these circumstances an improved sub-grade layer should be provided. In all cases, sub-grade material with a CBR value less than 2% should be removed and replaced with fill material.

5.4 Multilayer Elastic Analysis Computer Program

The general solutions for stresses and strains are coupled with elastic and viscoelastic computer programs. A brief introduction of different computer programs is discussed in Chapter 2. For the purpose of comparison of the results, mechanistic responses for pavement sections were analyzed by the computer programs, namely CIRCLY.

5.5 Evaluation of Design Procedures by Distress Modes Considered

Distress modes considered in this research are already defined in previous sections. The graphs plotted below for each distress mode from the values obtained by the computer program can provide a clear idea of a safe and better design.

5.5.1 Rutting

Rutting is characterized by the deflection and vertical stress at top of Subgrade. By examining the Figure 5.2 it is indicated that, for the same material properties, deflection at top of subgrade in the cases designed by Road Note 31 Pavement Design Procedure is higher as compared to the cases designed by AASHTO 1993 Pavement Design Procedure. The reason for this can be explained by examining the cases designed by the two pavement design procedures. It is obvious from the design that in case of AASHTO 1993 a higher thickness of asphalt concrete layer is obtained. It results in less stresses transferred to the base and sub-base layers, and consequently less stress is reaching at the subgrade level. Although Road Note 31 pavement design approach tries to compensate this by providing greater thickness for base and sub-base layers, but still the subgrade surface experiences a larger stress value. In case of vertical strain at top of Subgrade, pavements designed by Road Note 31 bear higher values than AASHTO 1993 designed cases. Figure 5.3 shows that vertical strain at the top of subgrade in the cases designed by Road Note 31 Pavement Design Procedure is also higher than the cases designed by AASHTO 1993 Pavement Design Procedure.

The case no. 3 out of four cases governs because this road generates maximum traffic and heavy traffic load as well. Therefore mechanistic analysis for pavement design of case no. 3 has been considered in this study. From Table 5.1 to Table 5.4 show the mechanistic responses i.e. stress, strain, deflection and cumulative damage factor of pavement case no. 3.

To compare the results from mechanistic analysis by CIRCLY software designed by AASHTO 1993, Road Note 31 and IRC 37 design procedure, 3^{rd} year ESALs which is 31.87×10^6 for case no. 3 (Table 4.3). Because by Road Note 31, the analysis can be done for a limited value of 30×10^6 ESALs. (Road Note 31, Fourth Edition).

Table 5.1 Mechanistic responses of pavement case no. 3 (10^{th} year ESALs) by using CIRCLY (Metric Units)

10^{th} year ESALs for case no. 3 = $133.7x10^6$

Case no.	Layer	Thickness (mm)	Layer Modulus (MPa)	Critical strain at bottom of layers	Cumulative Damage Factor(CDF)	Remarks
3	AC-Wearing Course	50.00	2800.00	-0.0000562	0.00649	As CDF is less than 1.0, the system has excess capacity and the cumulative damage represents the proportion of life consumed
	AC-Base Course	150.00	2800.00	-0.000152	0.980	
	Granular Base Type I	225.00	350.00	n/a	n/a	
	Granular Base Type II	225.00	300.00	n/a	n/a	
	Granular Sub- base	225.00	200.00	n/a	n/a	
	Subgrade	-	80.00	0.000195	.000527	

The most commonly used model form to predict the number of load repetitions to fatigue cracking is a function of the tensile strain and mix stiffness (modulus). The critical locations of the tensile strains may either be at the surface resulting in top-down cracking or at the bottom of the asphaltic layer resulting in bottom-up cracking. It is seen from Table 5.1 that the critical stains obtained by CIRCLY analysis are within the tolerable limit and the cumulative damage factor s are less than 1 that mean the system has excess capacity and the cumulative damage represents the portion of life consumed as per Equation 2.1 to 2.7 as described in Chapter 2.

Table 5.2 Mechanistic responses of pavement case no. 3 (3^{rd} year ESALs) Designed by AASHTO 1993 design procedure by using CIRCLY

3^{rd} year ESALs for case no. 3 = $31.87x10^6$, Say $30.00x10^6$

Case no.	Layer	Thickness (mm)	Deflection at top of layers (mm)	Vertical stress at top of layers (MPa)	Vertical strain at top of layers	Critical strain at bottom of layers	Cumulative Damage Factor
	AC-Wearing Course	50.00	-0.46320	0.7900	-0.00048	-0.00000143	0.0000000000138
	AC-Base Course	140.00	-0.47060	0.5000	-0.00068	-0.000158	0.228
3	Granular Base Type I	160.00	-0.46234	0.0714	0.00178	n/a	n/a
	Granular Sub- base	125.00	-0.42248	0.0360	0.00246	n/a	n/a
	Subgrade	-	-0.36430	0.0254	0.000315	0.000341	0.00427

The results obtained from CIRCLY analysis shown in Table 5.2 by AASHTO 1993 design method for deflection, vertical stress and strain are within the tolerable limit. It is also seen that the critical stains obtained by CIRCLY analysis are within the tolerable limit and the cumulative damage factors are less than 1.00 that mean the system has excess capacity and the cumulative damage represents the portion of life consumed as per Equation 2.1 to 2.7 described in Chapter 2.

Table 5.3 Mechanistic responses of pavement case no. 3 (3rd year ESALs) Designed by Road Note 31 design procedure by using CIRCLY

3rd year ESALs for case no. 3 = $31.87x10^6$, Say $30.00x10^6$

Case no.	Layer	Thickness (mm)	Deflection at top of layers (mm)	Vertical stress at top of layers (MPa)	Vertical strain at top of layers	Critical strain at bottom of layers	Cumulative Damage Factor
3	AC-Wearing Course	50.00	-0.48840	0.7800	-0.00078	-0.00000631	0.00000446
	AC-Base Course	100.00	-0.49440	0.6300	-0.00116	-0.000200	0.538
	Granular Base Type	250.00	-0.49001	0.1170	0.00236	n/a	n/a
	Granular Sub- base	175.00	-0.42160	0.0368	0.00280	n/a	n/a
	Subgrade	-	-0.38000	0.0265	0.000326	0.000364	0.00549

The results obtained from CIRCLY analysis shown in Table 5.3 by Road Note 31 design method for deflection, vertical stress and strain are within the tolerable limit. It is also seen that the critical stains obtained by CIRCLY analysis are within the tolerable limit and the cumulative damage factors are less than 1.00 that mean the system has excess capacity and the cumulative damage represents the portion of life consumed as per Equation 2.1 to 2.7 described in Chapter 2. It is observed that the values are higher than those of AASHTO 1993 design methods. This is because the AASHTO method suggests higher asphalt layer thickness than that of than that of Road Note 31 design method.

Table 5.4 Mechanistic responses of pavement case no. 3 (3rd year ESALs) Designed by Indian Roads Congress 37 design procedure by using CIRCLY

3rd year ESALs for case no. 3 = $31.87x10^6$, Say $30.00x10^6$

Case no.	Layer	Thickness (mm)	Deflection at top of layers (mm)	Vertical stress at top of layers (MPa)	Vertical strain at top of layers	Critical strain at bottom of layers	Cumulative Damage Factor
3	AC-Wearing Course	40.00	-0.4960	0.7600	-0.00088	-0.00000631	0.0000000234
	AC-Base Course	100.00	-0.5024	0.7200	-0.00128	-0.000168	0.745
	Granular Base Type	250.00	-0.4992	0.1390	0.00260	n/a	n/a
	Granular Sub- base	200.00	-0.4330	0.0429	0.00268	n/a	n/a
	Subgrade	-	-0.3920	0.0276	0.00336	0.000352	0.0679

The results obtained from CIRCLY analysis shown in Table 5.4 by IRC 37 design method for deflection, vertical stress and strain are within the tolerable limit. It is also seen that the critical stains obtained by CIRCLY analysis are within the maximum limit and the cumulative damage factors are less than 1.00 that mean the system has excess capacity and the cumulative damage represents the portion of life consumed as per Equation 2.1 to 2.7 described in Chapter 2. It is observed that the values are higher than those of AASHTO 1993 design methods and almost equal to those of Road Note 31 method. This is because the AASHTO method suggests higher asphalt layer thickness than that of IRC 37 design method.

Figure 5.2 to 5.7 show the stress, strain and deflection of different layers of the pavement.

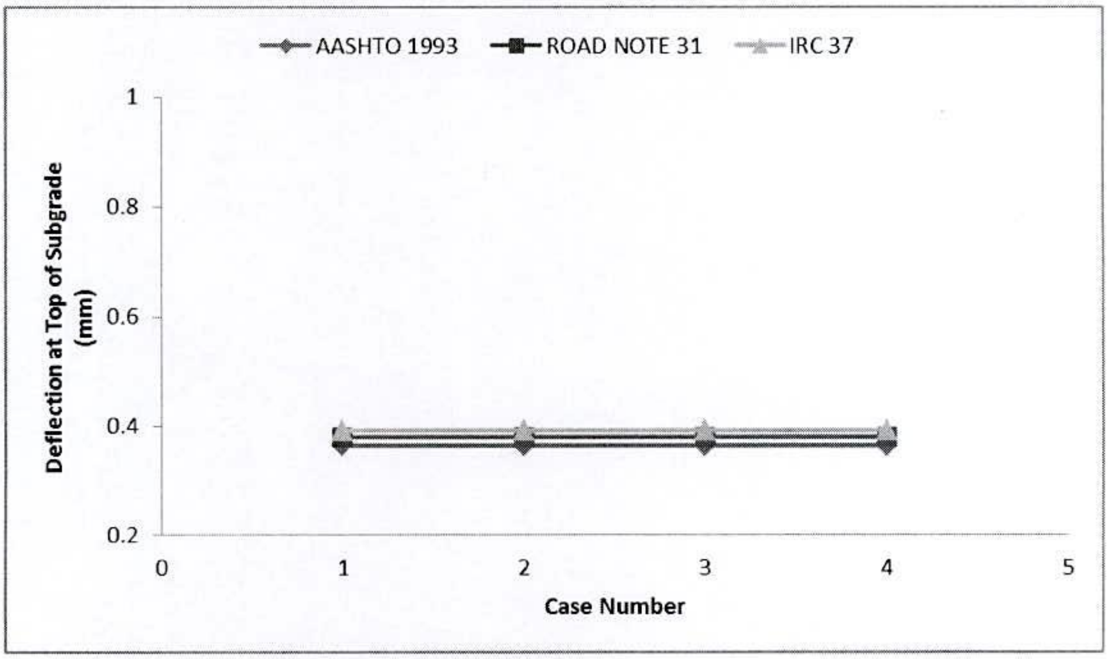


Figure 5.2 Deflection at top of Subgrade (3rd year)

It is seen from Figure 5.2 that in case of deflection the AASHTO 1993 recommends lesser value of deflection than that of Road Note 31 and IRC 37 Pavement Design Procedures at top of subgrade layer. It is because the lesser asphalt concrete layer thickness is recommended by Road Note 31 and IRC 37 than that of AASHTO 1993 Pavement Design Procedures.

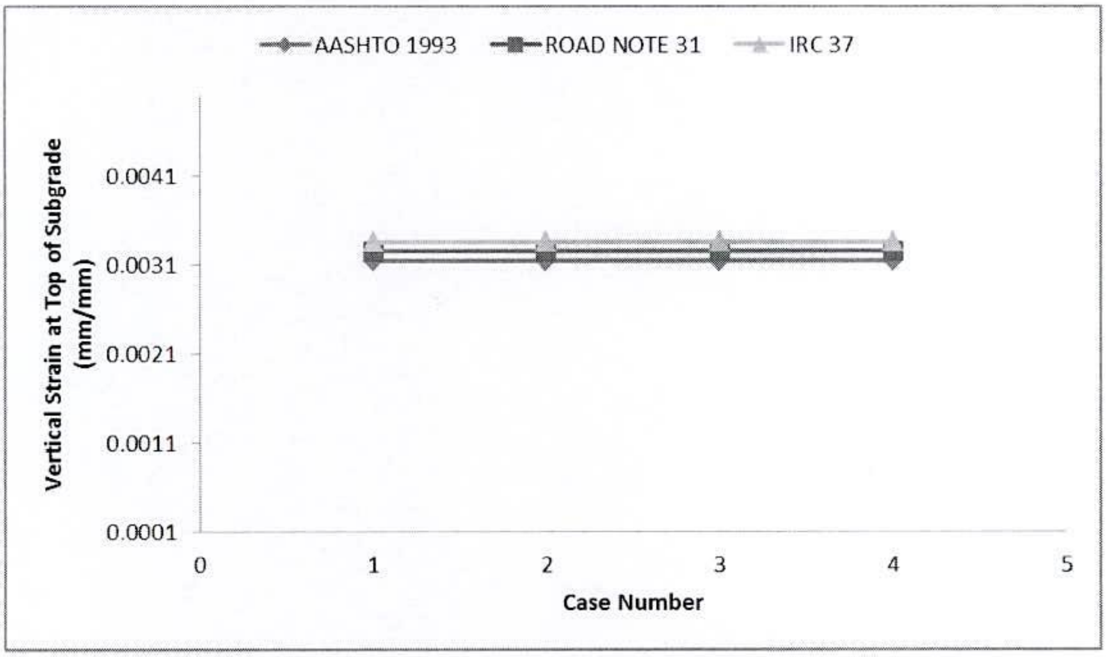


Figure 5.3 Vertical Strain at top of Subgrade (3rd year)

It is seen from Figure 5.3 that in case of vertical strain the AASHTO 1993 recommends lesser value of vertical strain than that of Road Note 31 and IRC 37 Pavement Design Procedures at top of subgrade layer. Road Note 31 and IRC 37 recommend higher value of thickness for base and sub-base layers than that of AASHTO 1993 Pavement Design Procedure. On the other hand the lesser asphalt concrete layer thickness is recommended by Road Note 31 and IRC 37 than that of AASHTO 1993 Pavement Design Procedures.

5.5.2 Permanent Deformation

The mechanistic responses of the designed sections obtained by the computer program shows the following trends in case of vertical compressive stress coming to road bed soil of the pavement structures. From Figure 5.4, it is observed that vertical compressive stress at top of road bed soil is higher in case of Road Note 31 design as compared with the AASHTO 1993 design. The allowable vertical stress on surface of subgrade depends on the strength or modulus of the subgrade. This indicates that for same subgrade strength or modulus value, the subgrade of pavement designed by Road Note 31 procedure will have to bear more stresses, and thus will be prone to deformation and subsequently pavement failure.

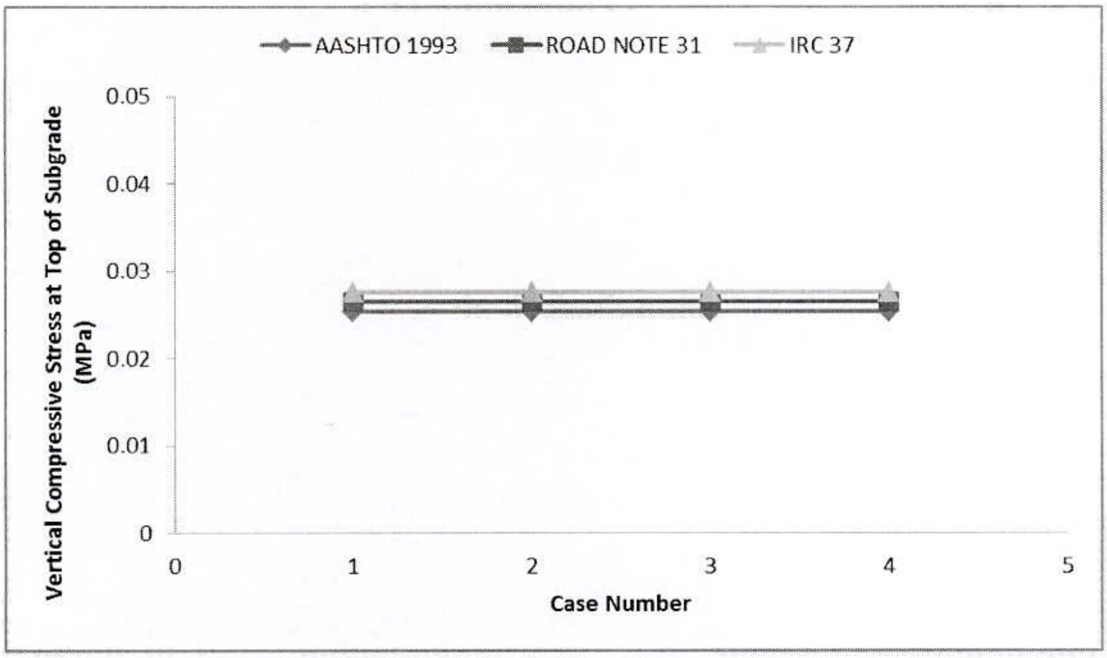


Figure 5.4 Vertical compressive stresses at Top of Subgrade (3^{rd} year)

It is seen from Figure 5.4 that in case of vertical compressive stress the Road Note 31 and IRC 37 recommend higher value of vertical compressive stress at top of subgrade because of higher thickness of base and sub-base layers than that of AASHTO 1993 Pavement Design Procedure. This thicker value may be provided to offset the less asphalt concrete layer given by the Road Note 31 and IRC 37 design methods. The analysis of mechanistic response obtained in Chapter 5 supports the above statement.

5.5.3 Fatigue Cracking

The comparison for fatigue cracking between the two pavement designs methods selected in this research (for evaluating their compatibility with conditions encountered in developing countries) is made in the following graph obtained by the mechanistic analysis.

In case of fatigue cracking in Figure 5.5, similar trend as shown previously is observed, i.e., the tensile strain at bottom of AC layer is more pronounced in pavement structures designed by Road Note 31 Design Procedure than the pavement structures designed by AASHTO 1993 Design Procedure. This also shows that the AASHTO 1993 design procedure gives more conservative and safe design as opposed to Road Note 31 design.

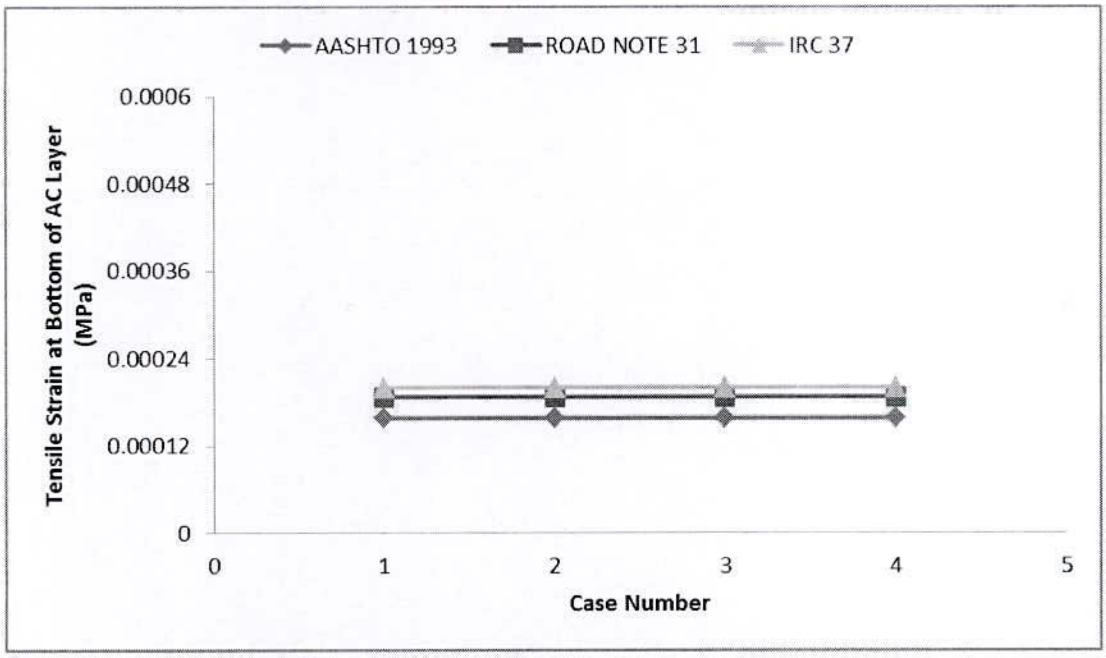


Figure 5.5 Tensile strains at Bottom of AC Layer (3rd year)

It is seen from Figure 5.5 that in case of tensile strain the AASHTO 1993 recommends lesser value of tensile strain than that of Road Note 31 and IRC 37 Pavement Design Procedures at top of AC layer. It is because the higher asphalt concrete layer thickness is recommended by AASHTO 1993 than that of Road Note 31 and IRC 37 Pavement Design Procedures.

5.5.4 Thermal Cracking

The fourth distress mode analyzed by the mechanistic responses in the thermal cracking in the asphalt layer. The tensile stress at the bottom of AC layer is believed to be a significant parameter related to thermal cracking. The limitation of both design methods against thermal cracking is presented by the following graphs.

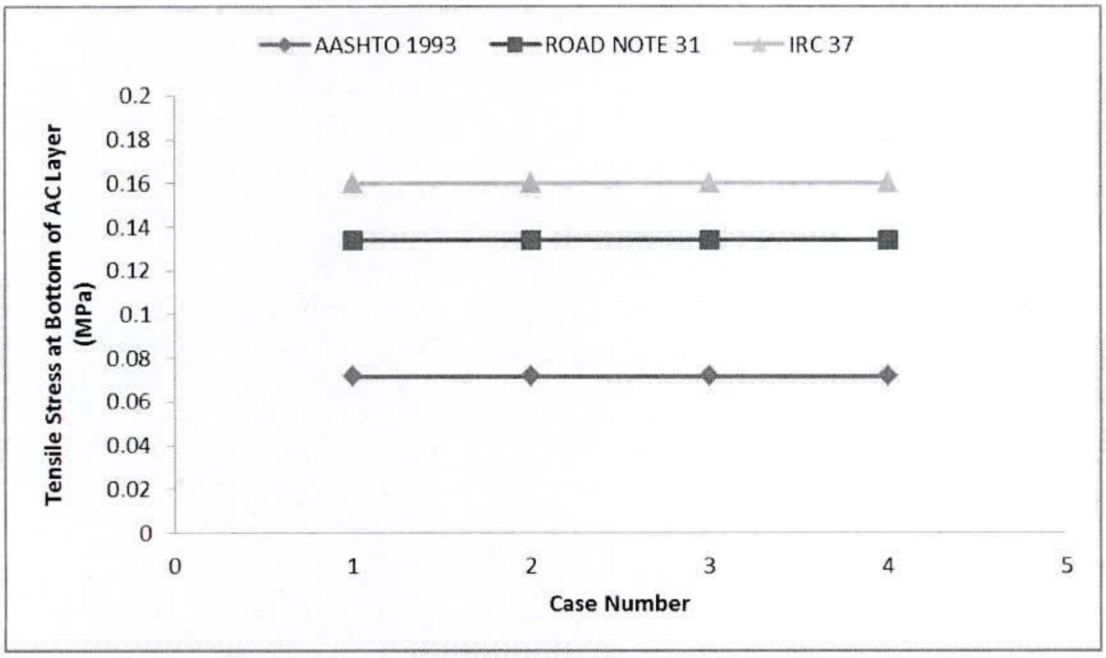


Figure 5.6 Tensile stresses at Bottom of AC Layer (3^{rd} year)

By studying Figure 5.6, it is shown that the tensile stress at the bottom of AC layer is higher than in pavement structure designed by Road Note 31 and IRC 37 than AASHTO 1993 Procedure. This high tensile value at bottom of AC layer cannot be compensated by increasing the thickness of underlying layer. This fact makes AASHTO 1993 Design Procedure more realistic, as higher AC layer thicknesses are recommended in this approach, resulting in much less stress values. However, it may be remembered that thermal cracking distress generally occurs at much lower temperature than zero degree centigrade. The lowest temperature on N-4 highway rarely reach zero degree centigrade, therefore this stress may not be much pronounced in tropical countries like Bangladesh, as opposed to colder regions.

5.5.5 Stresses in the Base and Sub-base Layers

The vertical compressive stresses at the surface of a base and sub-base courses should be limited to prevent excessive deformation of the respective layers. The base layers are also subjected to flexure and it is thought that this may sometimes cause a failure of the structure. Figure 5.7 shows the amount of vertical stress coming on base layers of the sections designed by two selected design methods.

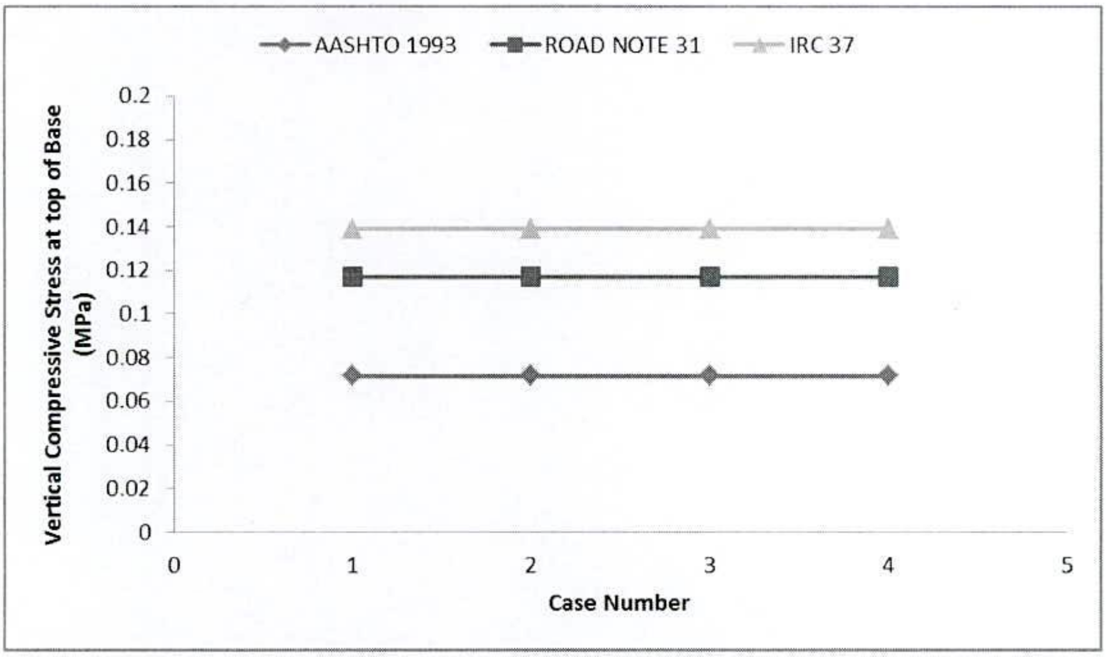


Figure 5.7 Vertical Compressive stresses at Top of Base (3rd year)

From Figure 5.7, it is seen that the vertical stress at the top of base as designed by Road Note 31 and IRC 37 deign methods is more than AASHTO 1993 design method. Similar trend is true for the sub-base layer also. This higher stress on base and sub-base layers will lead to a more total deflection in these layers, which subsequently will lead to a more total deflection of the pavement structure.

5.5.6 The effect of temperature changes on the elastic modulus

The strength of the flexible pavement is determined by the quality of the materials used and is usually measured as elastic modulus (E). The stiffness of flexible pavement is also affected by changes in the weather and season in the area. Changes in temperature will affect the modulus of the asphalt concrete layer (Bohn *et al*., 1977 and Charles & Regis, 2007)).

An empirical equation to describe the relationship between temperature and the elastic modulus of the asphalt has been developed by AASHTO 1993. A relationship between temperature and the elastic modulus of the asphalt mixtures of the motorways can be calculated using Equation 5.5.

$$E_l(t) = 15000 - 7900x\log(t) \quad (5.5)$$

Where,

$E_t(t) =$ The elastic modulus of asphalt mixture layer at temperature t ($\geq 1^{\circ}C$)

The stiffness moduli calculated using the SHELL method is generally in good agreement with the results obtained from the flexural tests (Ullidtz, 1987; Benedetto & Olard, 2009). At high temperatures, however, the flexural tests tend to give a lower stiffness modulus. In contrast, the AASHTO (1993) equation shows reliable asphalt mixture modulus results at high temperatures. Temperatures below $20^{\circ}C$ for the AASHTO show results , with the modulus being greater than that obtained from the SHELL method. This is because the low-temperature flexural test in the laboratory (UTM) gives lower tensile values compare to those obtained from the FWD test in the field. Asphalt is a viscoelastic linear material and its stretch levels decrease at a certain increase in the modulus Bohn *et al.* (1977). However, the difference in modulus values at low temperatures is vital for pavement performance. For temperatures above $20^{\circ}C$, the modulus of the mixture obtained by the SHELL method falls rapidly compared to the UTM and AASHTO methods. At temperature above $40^{\circ}C$, the SHELL method shows lower modulus values compared to the modulus of aggregates. At temperature above $30^{\circ}C$, the asphalt modulus values are medium for both the SHELL and AASHTO methods.

In general, the higher the temperature applied to the flexible pavement layer the lower the elastic modulus value will be. In contrast, the elastic modulus of the pavement layer decreases as the temperature increases.

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

The design of National Highway N4 (Joydevpur – Chandra – Tangail – Elenga) under the Sub Regional Road Transport Project (SRTP) was carried out to evaluate the conventional flexible pavement design procedures adopted in Bangladesh as well as in developing countries. The highway carries a large amount of truck and bus traffic of Bangladesh. Data for design input parameters including subgrade strength, traffic, and asphalt concrete (AC) and unbound granular material characteristics, reliability, standard deviations and drainage facilities were obtained from Roads and Highways Department, Bangladesh for the year 2011. Based on the analysis of data, the evaluation of conventional flexible pavement design procedures implemented in developing countries was carried out. A review of the evaluation made in this study reveals the following important conclusions.

- Currently the flexible pavement is designed facilitating heavy traffic following RHD, Bangladesh guideline based on AASHTO Flexible Pavement Design Procedure 1993, and Road Note 31 Pavement Design Procedure. Therefore these two pavement design procedures along with IRC 37 procedures were selected for checking their compatibility with traffic and environmental conditions prevailing in Bangladesh.
- It was found that for the same input parameters for design, all the methods produced approximately equal total thickness for the designed pavement. It is seen that AASHTO 1993 recommends higher AC layer thickness, while Road Note 31and IRC 37 Pavement Design Procedure suggests lesser AC layer thickness with recommending higher base and sub-base layer thickness to compensate for the relatively thin AC layer.
- Mechanistic responses in terms of stresses, strains and deflections at critical points in the pavement structure were estimated by the computer program CIRCLY, based on the multi-layered elastic system. From analysis result, it is observed that the cumulative damage factor (CDF) for AC layers and subgrade is less than 1.00

that indicates that the system has excess capacity and the cumulative damage represents the proportion of life consumed

- Four major distress modes, i.e., rutting, permanent deformation, fatigue and thermal cracking are associated with the stresses, strains and deflections at critical locations in a pavement. These distress modes contribute to the failure of flexible pavement. Therefore, these distress modes were used for the evaluation of selected design procedures.
- It was found that lesser values of stresses, strains and deflections were indicated in the pavement structures designed by AASHTO 1993 Pavement Design Procedure. While much higher values of mechanistic responses were obtained on pavement structure designed by Road Note 31 Design Procedure. But the values designed by IRC 37 were found lesser than those of Road Note 31.This concluded that pavement structures designed by AASHTO 1993 design procedure are relatively safer against rutting, fatigue and thermal crack failures, than the structures designed by the Road Note 31 and IRC 37 design procedures. Pavement structures designed by Road Note 31 Pavement Design Procedure may be cost effective in construction due to lesser AC layer thickness but they may fail prematurely requiring high maintenance and rehabilitation costs.

It is concluded that the pavement structures supporting heavy traffic in Bangladesh and other developing countries may be designed by AASHTO 1993 Pavement Design Procedure and must be checked by multilayer elastic computer software for mechanistic pavement design.

6.2 Recommendations

6.2.1 Recommendations for RHD

- As the weighted mean annual pavement temperature (WMAPT) is 37.57°C for Tangail based on average ambient temperature and rainfall data, it is recommended that the asphalt stiffness values used in the design be reduced to the values of 210,000 psi for the wearing course and 250,000 psi for the base course. RHD's standard design procedure adopts a value of 400,000 psi for the asphalt resilient modulus at a temperature of 20°C.

- 150 mm of asphalt concrete (AC) base is needed to prevent bottom-up cracking arising from the high loading of 133.7 million ESALs which will lead to tensile strain in the underside of the dense bituminous surface (DBS) asphalt concrete base course.
- Estimating of equivalent axle load per vehicle (EV) followed in this design in accordance with RHD recommendation which could be accurately assessed if Weigh in Motion facility to weigh and count vehicles is available in the country. Available Weigh Bridge will be used to check EV of vehicles.
- AASHTO 1993 pavement design procedure results in more conservative design for design input conditions of pavements in Bangladesh
- The soil strength parameter CBR used in Bangladesh for pavement design should be replaced by effective roadbed soil resilient modulus as suggested by AASHTO Pavement Design Guide 1993.
- In case of traffic data analysis, passenger car unit (PCU) value must be specified for small, medium and heavy truck and bus traffic separately.
- Structural analysis in terms of mechanistic responses should design for new pavement.

6.2.2 Recommendations for further study

This study has provided a first step towards proper implementation of the mechanistic-empirical design procedures in developing countries. The following efforts can be carried out to further enhance the evaluation of flexible pavement design methods in developing countries.

- Compatibility of other design methods such as Asphalt Institute, NAASRA, SHELL pavement design methods etc., can also be evaluated for their implementation in developing countries.
- Evaluation of design methods used in this study can be improved if other important highways of Bangladesh are also included in the analysis. This way validity of this evaluation can be checked further.

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APPENDIX A

DESIGN OF NATIONAL HIGHWAY N4

A1. Results of National Highway N4 (Joydevpur – Chandra - Tangail – Elenga) by using AASHTO Pavement Design Guide 1993.

A1.1 AASHTO Method

As described above the AASHTO method requires the input of a number of design parameters. The design parameters are as follows:

A1.1.1 Design Equivalent Standard Axle Load

The design lane ESAL has been calculated using the following basic equation:

$$ESAL = D_D \times D_L \times \left[365 \times \sum \left((VEF \times AADT) \times \left(\frac{(1+i)^n - 1}{i} \right) \right) \right]$$

The Calculated Cumulative ESALs of 10th year = 133,700,000

A1.1.2 Structural Number

The Structural Number was calculated by using the following equation:

$$\log_{10} W_{18} = Z_R S_o + 9.36 \log_{10}(SN+1) - 0.20 + \frac{\log_{10}[\Delta PSI/(p_i - p_t)]}{0.40 + [1094/(SN+1)^{5.19}]} + 2.32 \log_{10} M_r - 8.07$$

A1.1.3 Reliability [%]

For this highway, the reliability factor was taken as **85%** for design purposes.

A1.1.4 Standard Normal Deviation (Z_R)

Standard Normal Deviation (Z_R) was taken as -1.037

A1.1.5 Serviceability Index [PSI]

The AASHTO recommended values for new major highways with flexible surfaces are P_i = 4.2 and P_t = 2.5 giving serviceability loss value = (4.2 – 2.5) = 1.7. This value was used in the design of this study.

A1.1.6 Overall Standard Deviation (S_O)

An average value of 0.45 was used in the analysis for this study.

A1.1.7 Drainage Coefficient (m_i)

Drainage Coefficient value of 1.0 has been adopted for this project representing granular base and asphalt layer. Drainage Coefficient value of 0.9 was used for granular sub-base.

A1.1.8 Subgrade Value from Table 2.3

Factor	CBR Value (%)	Corresponding Resilience Modulus, M_R (psi)	Design Value Adopted of Resilience Modulus, M_R (psi)
Source of Correlation	-	AASHTO	
Formula	-	1,500 x CBR	
Improved Subgrade Fill	8	12,000	11,600

A1.1.9 Vehicle Equivalency Factors from Table 2.2

Source/Classification	Truck (H)	Truck (M)	Truck (S)	Bus (L)	Bus (M)	Minibus (S)
RHD Guideline	4.80	4.62	1.00	1.00	0.50	0.50

A1.1.10 Design Calculation

Table: Estimated Thickness of Different Layers by the AASHTO Method

Layer & Roadway Section	N4 Main Road	N4 Service Road
10-year Design Loading (ESAL x 10^6)	133.7	2.2
DBS – Wearing Course (mm)	50	50
DBS – Base Course (mm)	150	-
Aggregate Base Type I (mm)	225	200
Aggregate Base Type II (mm)	225	-
Sub-base (mm)	225	275
Improved Subgrade (mm)	300	300

A1.1.11 Design Calculation

Table: Estimated Thickness of Different Layers by the IRC 37 Method

Layer & Roadway Section	N4 Main Road	N4 Service Road
10-year Design Loading (ESAL x 10^6)	133.7	2.2
DBS – Wearing Course (mm)	50	50
DBS – Base Course (mm)	152.50	-
Aggregate Base (mm)	250	200
Sub-base (mm)	200	275
Improved Subgrade (mm)	300	300

A2.1 Results of National Highway N4 (3^{rd} year ESALs) using AASHTO 1993 Guide.

Total Cumulative ESALs (3^{rd} year) = 30.74×10^6

Say, ESALs (3^{rd} year) = 30.00×10^6

Now, using the AASHTO chart of structural number and figures (Figure 2.1 to 2.3), the following SN and Layer Coefficients are obtained:

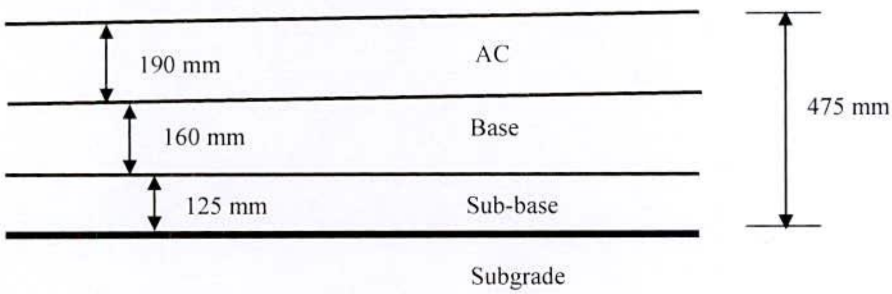
$SN= 4.84$ and $a_1= a_{AC}= 0.44$, $a_2= a_{Base}= 0.15$ & $a_3= a_{Subbase}= 0.12$

Drainage qualities for base (m_2), and sub-base (m_3) are good and taken as 1.0 & 0.9 respectively as described earlier.

So, $SN= a_1D_1+ a_2\, m_2D_2 + a_3\, m_3D_3$

$=0.44 \times 7.60 + 0.15 \times 6.40 \times 1.00 + 0.12 \times 5 \times 0.90 = 4.844 > 4.84$ Ok.

The final design is as follows:



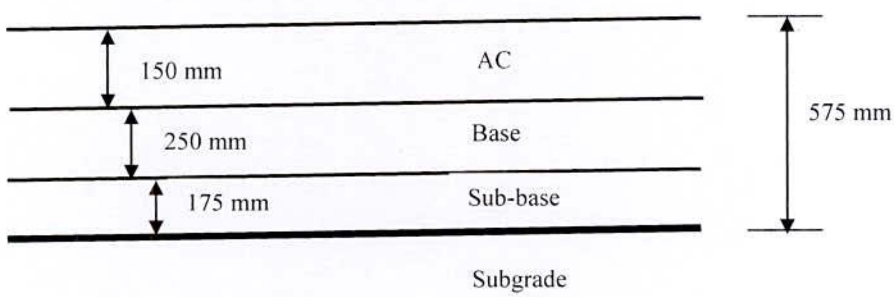
A2.2 Results of National Highway N4 (3^{rd} year ESALs) by using Road Note 31 Guide for Design of Pavement Structures

CBR= 8.0 (Subgrade strength class S4)

ESAL= 30.00 million (Traffic class T8), flexible bituminous surface

Base= GB1-GB3 CBR>80, Sub-base= GS CBR>30

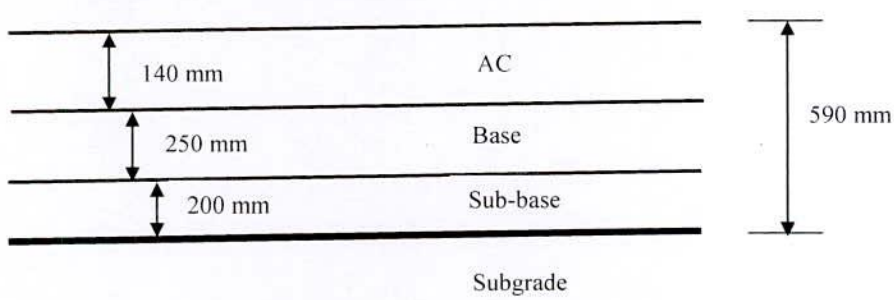
Following design chart number 5 (Appendix E), provided in the guide, the resulting layer thickness are shown as



A2.3 Results of National Highway N4 (3^{rd} year ESALs) by using IRC 37 Guide for Design of Pavement Structures

CBR= 8.0 and ESAL= 30.00 million

Following pavement design catalogue Plate 2 (Appendix E), provided in the guide, the resulting layer thickness are shown as



APPENDIX B

DESIGN ESAL CALCULATION

Traffic [Survey year:2011 (RHD) Kaliakoir - Gorai, N4-Joidebpur-Tangail-Jamalpur Road]
ESAL FOR PAVEMENT DESIGN

	Small buses	Medium buses	Large buses	small trucks	Medium trucks	Heavy trucks
Annual Traffic Growth Rate (2012-2020)	7%	7%	7%	8%	8%	8%
Annual Traffic Growth Rate (2021-2037)	6%	6%	6%	6%	6%	6%
ESAL PER VEHICLE (2011-2017) [only for Bangladesh Traffic]	1.00	1.00	2.80	1.00	11.00	10.28
ESAL PER VEHICLE (2018-2037) [Both SAARC & Bangladesh Traffic]	0.50	0.50	1.00	1.00	4.62	4.80
ESAL PER VEHICLE (2016 -2017) [only for SAARC Traffic]	0.50	0.50	1.00	1.00	4.62	4.80

EQUIVALENT SINGLE AXLE LOAD 8.2 MT (ESAL)

DD= DIRECTIONAL DISTRIBUTION FACTOR (0.3,0.5,0.7):

DL= LANE DISTRIBUTION FACTOR (NUMBER OF LANES IN ONE DIRECTION:1=1, 2=0.8 A 1, 3=0.6 A 0.8):

$D_D = 0.5$

$D_L = 0.8$

Year	Year	Small buses	Medium buses	Large buses	Small trucks	Medium trucks	Heavy trucks	AADT	ESAL	CUMULATIVE EASL	REMARKS	DESIGN ESAL
AADT RHD 2011 Trafifc Count	1	1446	1025	2386	1882	5447	166	12352	10,607,963		Base Year	
2012	2	1547	1097	2553	2033	5883	179	13292	11,443,238			
2013	3	1656	1174	2732	2195	6353	194	14303	12,344,400			
2014	4	1771	1256	2923	2371	6862	209	15392	13,316,655			
2015	5	1895	1344	3128	2560	7411	226	16563	14,365,619			
2016	6	2028	1438	3346	2765	8003	244	17825	15,497,354			
2016 (only Bhutan & Nepal Traffic)	6	-	-	-	-	-	184	184	-			
2016 (Total ESAL)	6	2028	1438	3346	2765	8003	428	18009	15,773,516			
2017	7	2170	1538	3581	2986	8644	462	19381	17,016,656			
2018	8	2322	1646	3831	3225	9335	499	20859	7,966,508			
2018 (Including Indian Traffic)	8	2322	1646	3831	3225	9335	1443	21803	8,628,063			
2018 (Including additional medium Traffic for permissible wt. limit + generated traffic)	8	2485	1761	4099	3225	11024	1443	24038	9,826,749	9,826,749	Traffic Opening	
2019	9	2659	1884	4386	3483	11906	1559	25878	10,603,805	20,430,554		
2020	10	2845	2016	4693	3762	12859	1683	27859	11,442,388	31,872,942	3rd year	

Year	Year	Small buses	Medium buses	Large buses	Small trucks	Medium trucks	Heavy trucks	AADT	ESAL	CUMULATIVE EASL	REMARKS	DESIGN ESAL
2021	11	3016	2137	4975	3988	13630	1784	29530	12,128,932	44,001,874		
2022	12	3197	2265	5274	4227	14448	1891	31302	12,856,668	56,858,542	5th year	56,900,000
2023	13	3388	2401	5590	4481	15315	2005	33180	13,628,068	70,486,610		
2024	14	3592	2545	5925	4750	16234	2125	35171	14,445,752	84,932,362		
2025	15	3807	2698	6281	5035	17208	2253	37281	15,312,497	100,244,859		
2026	16	4036	2860	6658	5337	18240	2388	39518	16,231,247	116,476,105		
2027	17	4278	3031	7057	5657	19335	2531	41889	17,205,122	133,681,227	10th year	133,700,000
2028	18	4535	3213	7481	5996	20495	2683	44402	18,237,429	151,918,656		
2029	19	4807	3406	7929	6356	21724	2844	47066	19,331,675	171,250,330		
2030	20	5095	3610	8405	6737	23028	3014	49890	20,491,575	191,741,905		
2031	21	5401	3827	8909	7142	24410	3195	52884	21,721,070	213,462,975		
2032	22	5725	4057	9444	7570	25874	3387	56057	23,024,334	236,487,309	15th year	236,500,000
2033	23	6068	4300	10011	8024	27427	3590	59420	24,405,794	260,893,103		
2034	24	6432	4558	10611	8506	29072	3806	62985	25,870,141	286,763,244		
2035	25	6818	4832	11248	9016	30816	4034	66765	27,422,350	314,185,594		
2036	26	7227	5122	11923	9557	32665	4276	70770	29,067,691	343,253,285		
2037	27	7661	5429	12638	10131	34625	4533	75017	30,811,752	374,065,037	20th year	374,100,000
ESAL PER VEHICLE (2011-2017) [only for Bangladesh Traffic]		1.00	1.00	2.80	1.00	11.00	10.28					
ESAL PER VEHICLE (2018-2037) [Both SAARC & Bangladesh Traffic]		0.50	0.50	1.00	1.00	4.62	4.80					

APPENDIX C

PAVEMENT DESIGN CALCULATION

AASHTO FLEXIBLE PAVEMENT DESIGN, N4: (Kaliakoir - Gorai)

Layer Thickness Determination Using Layered Analysis Approach

Layer No.	Description	Layer Coefficient, a_i	Drainage Coefficient, m_i	Elastic Modulus, psi	SN Using E of next lower layer in inputs box below	Min. Layer Thickness, D, inches	Practical Layer Thickness, D, inches	Associated SN	Design Pavement mm
Layer 1	AC Wearing Course	0.31	1.00	210,000	2.00	6.45	1.97	0.61	50
Layer 2	AC Base Course	0.33	1.00	250,000	3.66	9.24	5.91	1.95	150
Layer 3	Gran Base Type I	0.15	1.00	50,800	3.87	8.73	8.86	1.33	225
Layer 4	Gran Base Type II	0.14	1.00	43,500	4.48	4.22	8.86	1.24	225
Layer 5	Gran Sub Base	0.12	0.90	29,000	5.64	4.72	7.87	0.85	225
Layer 6	Improved Subgrade					0.00		0.00	0
Subgrade	Subgrade	N/A	N/A	11,600	N/A	N/A	N/A	N/A	N/A
					Total Pavement Thickness, inches,	33.36	33.47	5.98	Calculated SN
								5.95	SN Required

1 MN/mm2 = 1 Mpa = 10.2 kg/cm2 = 145 psi

Design is sufficient

Inputs Box

W18 =	133,700,000		*ESALs Applications Over Design Period*
R =	85	%	*Reliability*
So =	0.45		*Standard Deviation*
MR =	11,600	psi	*Subgrade Resilient Modulus*
Pi =	4.2		*Initial Serviceability*
Pt =	2.5		*Terminal Serviceability*
SN on top of layer =	5.95		

Total Thickness 875

APPENDIX D

INPUT AND OUTPUT FORMAT FOR CIRCLY

FOR 3rd & 10th YEAR

(AASHTO 1993)

(ESAL = $30.00x10^{6}$ for 3rd year)

CIRCLY Version 5.1b (5 November 2015)

Job Title: MSc Thesis

Damage Factor Calculation

Assumed number of damage pulses per movement:
One pulse per axle (i.e. use NROWS)

Traffic Spectrum Details:

ID: RHD Title: 30^6

Load No.	Load ID	Movements
1	ESA75-Full	3.00E+07

Details of Load Groups:

Load No.	Load ID	Load Category	Load Type	Radius	Pressure/ Ref. stress	Exponent
1	ESA75-Full	SA750-Full	Vertical Force	92.1	0.75	0.00

Load Locations:

Location No.	Load ID	Gear No.	X	Y	Scaling Factor	Theta
1	ESA75-Full	1	-165.0	0.0	1.00E+00	0.00
2	ESA75-Full	1	165.0	0.0	1.00E+00	0.00
3	ESA75-Full	1	1635.0	0.0	1.00E+00	0.00
4	ESA75-Full	1	1965.0	0.0	1.00E+00	0.00

Layout of result points on horizontal plane:
Xmin: 0 Xmax: 165 Xdel: 165
Y: 0

Details of Layered System:

ID: 1201506 Title: MSc Thesis

Layer No.	Lower i/face	Material ID	Isotropy	Modulus (or Ev)	P.Ratio (or vvh)	F	Eh	vh
1	rough	Asph2800	Iso.	2.80E+03	0.40			
2	rough	Asph2800	Iso.	2.80E+03	0.40			
3	rough	Gran_350	Aniso.	3.50E+02	0.35	2.60E+02	1.75E+02	0.35
4	rough	Gran_200	Aniso.	2.00E+02	0.35	1.50E+02	1.00E+02	0.35
5	rough	Sub_CBR8	Aniso.	8.00E+01	0.45	5.52E+01	4.00E+01	0.45

Performance Relationships:

Layer No.	Location	Performance ID	Component	Perform. Constant	Perform. Exponent	Traffic Multiplier
1	bottom	ShellA12.9	ETH	0.005889	5.000	1.100
2	bottom	ShellA12.9	ETH	0.005889	5.000	1.100
5	top	Sub_2004	EZZ	0.009300	7.000	1.600

Reliability Factors:
Project Reliability: Austroads 85%

Layer No.	Reliability Factor	Material Type
1	2.00	Asphalt
2	2.00	Asphalt
5	1.00	Subgrade (Austroads 2004)

Details of Layers to be sublayered:
Layer no. 3: Austroads (2004) sublayering
Layer no. 4: Austroads (2004) sublayering

Results:

Layer No.	Thickness	Material ID	Load ID	Critical Strain	CDF
1	50.00	Asph2800	ESA75-Full	-1.43E-06	1.38E-11
2	140.00	Asph2800	ESA75-Full	-1.58E-04	2.28E-01
3	160.00	Gran_350		n/a	n/a
4	125.00	Gran_200		n/a	n/a
5	0.00	Sub_CBR8	ESA75-Full	3.41E-04	4.27E-03

INPUT AND OUTPUT FORMAT FOR CIRCLY

(ROAD NOTE 31)

(ESAL = $30.00x10^6$ for 3^{rd} year)

CIRCLY Version 5.1b (5 November 2015)

Job Title: MSc Thesis

Damage Factor Calculation

Assumed number of damage pulses per movement:
One pulse per axle (i.e. use NROWS)

Traffic Spectrum Details:

ID: RHD Title: 30^6

Load No.	Load ID	Movements
1	ESA75-Full	3.00E+07

Details of Load Groups:

Load No.	Load ID	Load Category	Load Type	Radius	Pressure/ Ref. stress	Exponent
1	ESA75-Full	SA750-Full	Vertical Force	92.1	0.75	0.00

Load Locations:

Location No.	Load ID	Gear No.	X	Y	Scaling Factor	Theta
1	ESA75-Full	1	-165.0	0.0	1.00E+00	0.00
2	ESA75-Full	1	165.0	0.0	1.00E+00	0.00
3	ESA75-Full	1	1635.0	0.0	1.00E+00	0.00
4	ESA75-Full	1	1965.0	0.0	1.00E+00	0.00

Layout of result points on horizontal plane:
Xmin: 0 Xmax: 165 Xdel: 165
Y: 0

Details of Layered System:

ID: 1201506 Title: MSc Thesis

Layer No.	Lower i/face	Material ID	Isotropy	Modulus (or Ev)	P.Ratio (or vvh)	F	Eh	vh
1	rough	Asph2800	Iso.	2.80E+03	0.40			
2	rough	Asph2800	Iso.	2.80E+03	0.40			
3	rough	Gran_350	Aniso.	3.50E+02	0.35	2.60E+02	1.75E+02	0.35
4	rough	Gran_200	Aniso.	2.00E+02	0.35	1.50E+02	1.00E+02	0.35
5	rough	Sub_CBR8	Aniso.	8.00E+01	0.45	5.52E+01	4.00E+01	0.45

Performance Relationships:

Layer No.	Location	Performance ID	Component	Perform. Constant	Perform. Exponent	Traffic Multiplier
1	bottom	ShellA12.9	ETH	0.005889	5.000	1.100
2	bottom	ShellA12.9	ETH	0.005889	5.000	1.100
5	top	Sub_2004	EZZ	0.009300	7.000	1.600

Reliability Factors:
Project Reliability: Austroads 85%

Layer No.	Reliability Factor	Material Type
1	2.00	Asphalt
2	2.00	Asphalt
5	1.00	Subgrade (Austroads 2004)

Details of Layers to be sublayered:
Layer no. 3: Austroads (2004) sublayering
Layer no. 4: Austroads (2004) sublayering

Results:

Layer No.	Thickness	Material ID	Load ID	Critical Strain	CDF
1	50.00	Asph2800	ESA75-Full	-1.80E-05	4.46E-06
2	100.00	Asph2800	ESA75-Full	-1.88E-04	5.38E-01
3	250.00	Gran_350		n/a	n/a
4	175.00	Gran_200		n/a	n/a
5	0.00	Sub_CBR8	ESA75-Full	3.54E-04	5.49E-03

INPUT AND OUTPUT FORMAT FOR CIRCLY

(IRC 37)

(ESAL = $30.00x10^6$ for 3^{rd} year)

CIRCLY Version 5.1b (5 January 2015)

Job Title: MSc Thesis

Damage Factor Calculation

Assumed number of damage pulses per movement:
One pulse per axle (i.e. use NROWS)

Traffic Spectrum Details:

ID: RHD Title: 30^6

Load No.	Load ID	Movements
1	ESA75-Full	3.00E+07

Details of Load Groups:

Load No.	Load ID	Load Category	Load Type	Radius	Pressure/ Ref. stress	Exponent
1	ESA75-Full	SA750-Full	Vertical Force	92.1	0.75	0.00

Load Locations:

Location No.	Load ID	Gear No.	X	Y	Scaling Factor	Theta
1	ESA75-Full	1	-165.0	0.0	1.00E+00	0.00
2	ESA75-Full	1	165.0	0.0	1.00E+00	0.00
3	ESA75-Full	1	1635.0	0.0	1.00E+00	0.00
4	ESA75-Full	1	1965.0	0.0	1.00E+00	0.00

Layout of result points on horizontal plane:
Xmin: 0 Xmax: 165 Xdel: 165
Y: 0

Details of Layered System:

ID: 1201506 Title: MSc Thesis

Layer No.	Lower i/face	Material ID	Isotropy	Modulus (or Ev)	P.Ratio (or vvh)	F	Eh	vh
1	rough	Asph2800	Iso.	2.80E+03	0.40			
2	rough	Asph2800	Iso.	2.80E+03	0.40			
3	rough	Gran_350	Aniso.	3.50E+02	0.35	2.60E+02	1.75E+02	0.35
4	rough	Gran_200	Aniso.	2.00E+02	0.35	1.50E+02	1.00E+02	0.35
5	rough	Sub_CBR8	Aniso.	8.00E+01	0.45	5.52E+01	4.00E+01	0.45

Performance Relationships:

Layer No.	Location	Performance ID	Component	Perform. Constant	Perform. Exponent	Traffic Multiplier
1	bottom	ShellA12.9	ETH	0.005889	5.000	1.100
2	bottom	ShellA12.9	ETH	0.005889	5.000	1.100
5	top	Sub_2004	EZZ	0.009300	7.000	1.600

Reliability Factors:
Project Reliability: Austroads 85%

Layer No.	Reliability Factor	Material Type
1	2.00	Asphalt
2	2.00	Asphalt
5	1.00	Subgrade (Austroads 2004)

Details of Layers to be sublayered:
Layer no. 3: Austroads (2004) sublayering
Layer no. 4: Austroads (2004) sublayering

Results:

Layer No.	Thickness	Material ID	Load ID	Critical Strain	CDF
1	40.00	Asph2800	ESA75-Full	-6.31E-06	2.34E-06
2	100.00	Asph2800	ESA75-Full	-2.00E-04	7.45E-01
3	250.00	Gran_350		n/a	n/a
4	200.00	Gran_200		n/a	n/a
5	0.00	Sub_CBR8	ESA75-Full	3.64E-04	6.79E-03

INPUT AND OUTPUT FORMAT FOR CIRCLY

(AASHTO 1993)

(ESAL = $133.70x10^6$ for 10^{th} year)

CIRCLY Version 5.0t (1 November 2015)

Job Title: RCI

Damage Factor Calculation

Assumed number of damage pulses per movement:
One pulse per axle (i.e. use NROWS)

Traffic Spectrum Details:

ID: RHD Title: 1.33E+8

Load No.	Load ID	Movements
1	ESA75-Full	1.33E+08

Details of Load Groups:

Load No.	Load ID	Load Category	Load Type	Radius	Pressure/ Ref. stress	Exponent
1	ESA75-Full	SA750-Full	Vertical Force	92.1	0.75	0.00

Load Locations:

Location No.	Load ID	Gear No.	X	Y	Scaling Factor	Theta
1	ESA75-Full	1	-165.0	0.0	1.00E+00	0.00
2	ESA75-Full	1	165.0	0.0	1.00E+00	0.00
3	ESA75-Full	1	1635.0	0.0	1.00E+00	0.00
4	ESA75-Full	1	1965.0	0.0	1.00E+00	0.00

Layout of result points on horizontal plane:
Xmin: 0 Xmax: 165 Xdel: 165
Y: 0

Details of Layered System:

ID: RHD Title: RCI

Layer No.	Lower i/face	Material ID	Isotropy	Modulus (or Ev)	P.Ratio (or vvh)	F	Eh	vh
1	rough	Asph2800	Iso.	2.80E+03	0.40			
2	rough	Asph2800	Iso.	2.80E+03	0.40			
3	rough	Gran_350	Aniso.	3.50E+02	0.35	2.60E+02	1.75E+02	0.35
4	rough	Gran_300	Aniso.	3.00E+02	0.35	2.20E+02	1.50E+02	0.35
5	rough	Gran_200	Aniso.	2.00E+02	0.35	1.50E+02	1.00E+02	0.35
6	rough	Sub_CBR8	Aniso.	8.00E+01	0.45	5.52E+01	4.00E+01	0.45

Performance Relationships:

Layer No.	Location	Performance ID	Component	Perform. Constant	Perform. Exponent	Traffic Multiplier
1	bottom	ShellA12.9	ETH	0.005889	5.000	1.100
2	bottom	ShellA12.9	ETH	0.005889	5.000	1.100
6	top	Sub_2004	EZZ	0.009300	7.000	1.600

Reliability Factors:
Project Reliability: Austroads 85%

Layer No.	Reliability Factor	Material Type
1	2.00	Asphalt
2	2.00	Asphalt
6	1.00	Subgrade (Austroads 2004)

Details of Layers to be sublayered:
Layer no. 3: Austroads (2004) sublayering
Layer no. 4: Austroads (2004) sublayering
Layer no. 5: Austroads (2004) sublayering

Results:

Layer No.	Thickness	Material ID	Load ID	Critical Strain	CDF
1	50.00	Asph2800	ESA75-Full	-5.62E-05	6.49E-03
2	150.00	Asph2800	ESA75-Full	-1.52E-04	9.80E-01
3	225.00	Gran_350		n/a	n/a
4	225.00	Gran_300		n/a	n/a
5	225.00	Gran_200		n/a	n/a
6	0.00	Sub_CBR8	ESA75-Full	1.95E-04	5.27E-04

APPENDIX E

AASHTO 1993, ROAD NOTE 31 AND IRC 37 CHARTS

(STRUCTURAL NUMBER, CATALOGUE & GRANULAR ROAD BASE)

DESIGN CHART FOR FLEXIBLE PAVEMENT DESIGN BASED ON AASHTO 1993 PAVEMENT DESIGN GUIDE

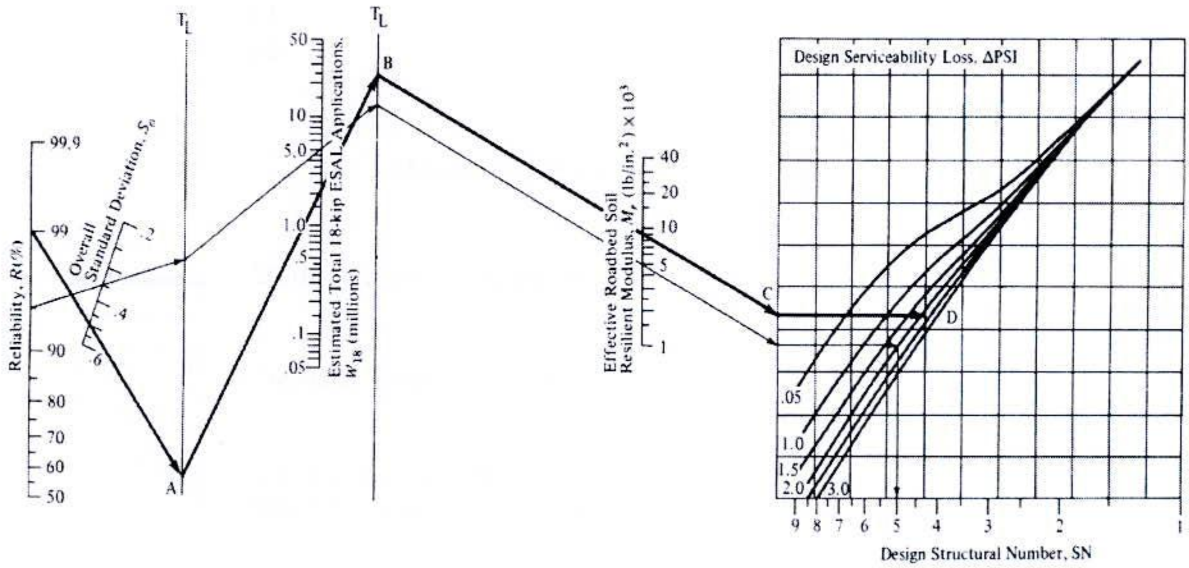


CHART 5 GRANULAR ROADBASE / STRUCTURAL SURFACE

	T1	T2	T3	T4	T5	T6	T7	T8
S1						100 200 225 * 350	125 225 225 350	150 250 250 350
S2						100 200 225 * 200	125 225 225 200	150 250 250 200
S3						100 200 250	125 225 250	150 250 275
S4						100 200 175	125 225 175	150 250 175
S5						100 200 100	125 225 100	150 250 100
S6						100 200	125 225	150 250

Note: 1 * Up to 100mm of sub-base may be substituted with selected fill provided the sub-base is not reduced to less than the roadbase thickness or 200mm whichever is the greater. The substitution ratio of sub-base to selected fill is 25mm : 32mm.

2 A cement or lime-stabilised sub-base may also be used.

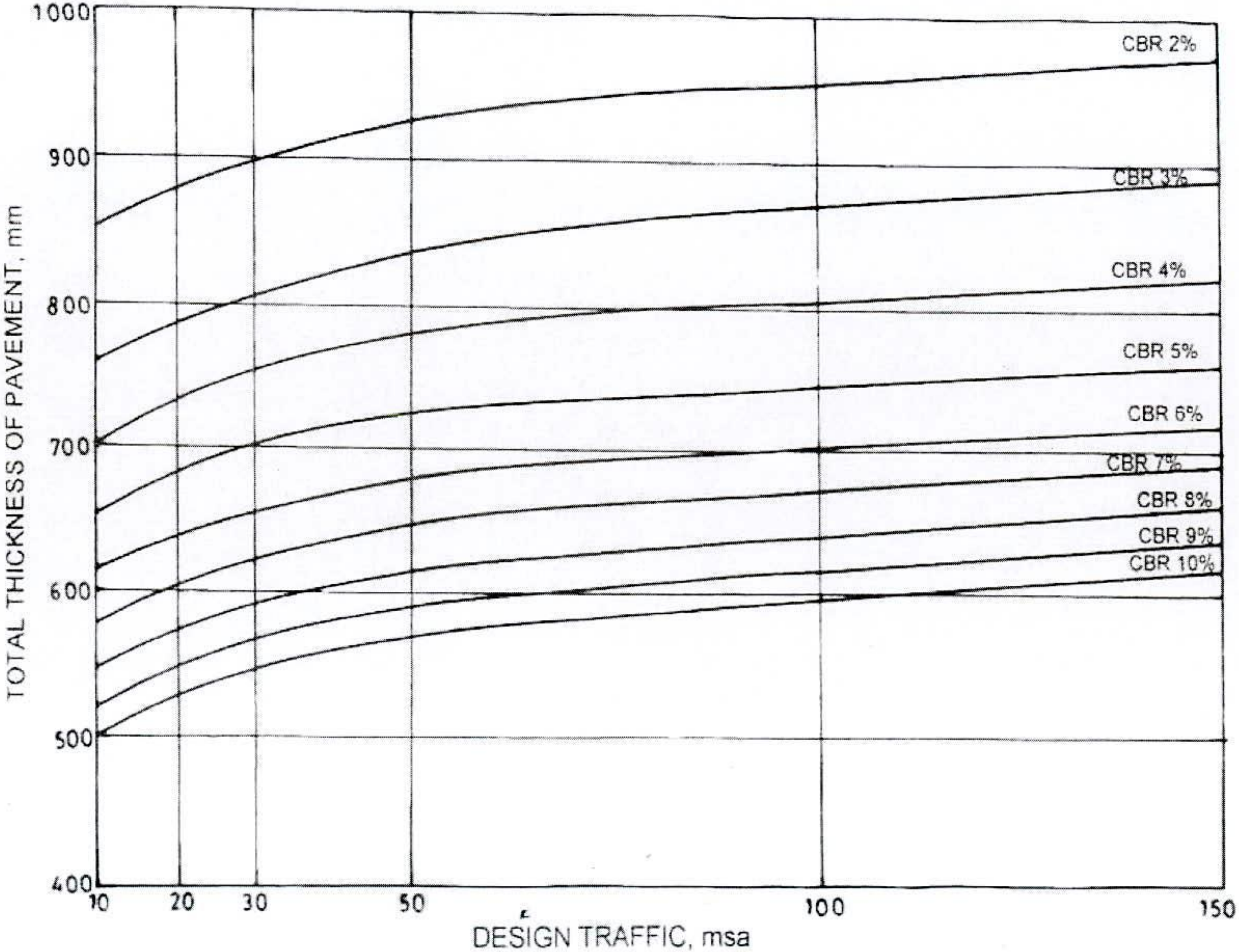
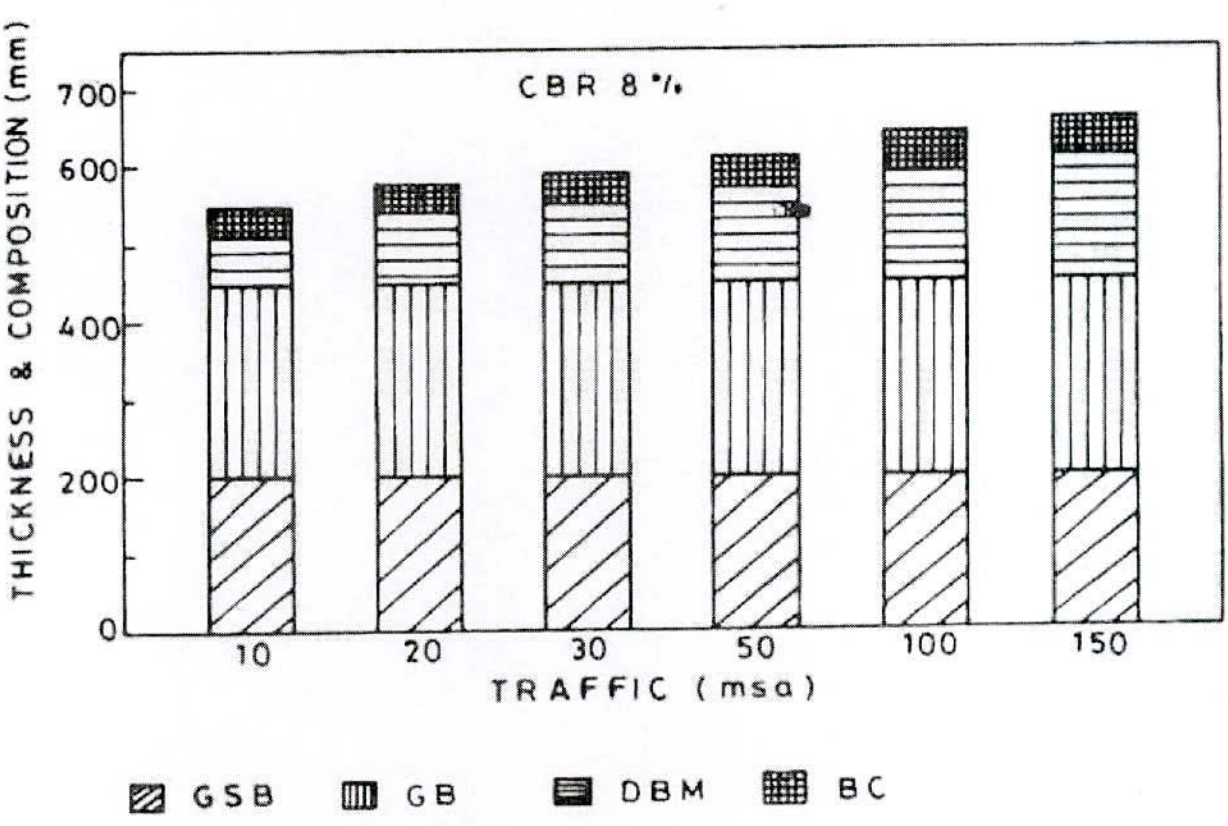


Fig. 2. Pavement Thickness Design Chart for Traffic 10-150 msa

PAVEMENT DESIGN CATALOGUE

PLATE 2 – RECOMMENDED DESIGNS FOR TRAFFIC RANGE 10-150 msa

CBR 8%				
Cumulative Traffic (msa)	Total Pavement Thickness (mm)	PAVEMENT COMPOSITION		
		Bituminous Surfacing		Granular Base & Sub-base (mm)
		BC (mm)	DBM (mm)	
10	550	40	60	Base = 250 Sub-base = 200
20	575	40	85	
30	590	40	100	
50	610	40	120	
100	640	50	140	
150	660	50	160	



Contd.