パキスタンにおける経験的理論的舗装設計システムの提案

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Proposals for Mechanistic-Empirical Pavement Design System in Pakistan

By

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Abstract

A reliable and efficient road transport system is absolutely crucial to the economic as well as social uplift of developing countries, including Pakistan. Road transport not only plays a pivotal role in cost effective transportation of freight and passengers, it provide equal opportunities of access to jobs and trade to all segments of society, thus promoting national cohesion and alleviation of poverty; which are the issues shared by almost all the developing nations around the world. Realizing the importance of road transport, a major portion of the national income of Pakistan is spent on construction and maintenance of roads. However, the economic and social benefits of an efficient road transport system are seldom realized due to accelerated deterioration and premature failure of road pavement structures. Some of the primary causes of premature failure of flexible pavement structures in Pakistan can be attributed to excessive axle loading and over inflation of truck tires, inappropriate design inputs for materials and climatic conditions, and the use of empirical pavement design method which is not truly representative of the geo-environmental conditions prevailing in Pakistan.

Flexible pavements with hot mix asphalt concrete (AC) surfacing constitutes majority of road pavements around the world, including Pakistan; predominantly due to its low initial/maintenance cost, easy and quick construction, superior riding quality and skid resistance, etc. However, the design and performance of flexible pavements includes a multitude of potentially influencing variables including the complex behavior of ***pavement geomaterials*** in each layer; resting on a variety of ***subgrade soils***, under dynamic ***traffic loads*** and in an array of ***climatic conditions***. Deficient knowledge or inadequate assembling and/or inappropriate handling of these variables in the design as well as construction stages may adversely affect the performance of flexible pavement structures.

The recent advancement in support technologies; particularly more sophisticated laboratory and in-situ instrumentation, and much efficient computers now available for general use; have indeed facilitated the development of analytical or Mechanistic-Empirical (M-E) design systems which are capable of analyzing variabilities in all the design input.

Capitalizing on these advancements, this research explores the current situation of flexible pavements in Pakistan with a view to suggesting practical proposals towards developing Mechanistic-Empirical pavement design system in Pakistan.

Conclusion of this research may be summarized as follows:

1) The Mechanistic-Empirical (M-E) Pavement design approach, realistically capture the variations in all factors that influence the pavement performance, resulting in rational pavement design.
2) The performance of flexible pavement is sensitive to not only axle loading but also significantly to tire inflation pressure. The damaging influence of increase in tire pressure keeps on magnifying with each axle load increment.
3) Time-dependency has considerable effect on the deformation properties of uniformly graded crushed gravel, constituting drainable base courses. Moreover, the effect is more evident in materials with cyclic loading history.
4) A relationship between $K_{P.FWD}$ and K_{30} for base course materials (well-graded gravels) and subgrade materials (clayey soils), was found to be 2:1 and 1:1, respectively.
5) Laboratory test results and in-service performance reveal that the HS asphalt mixture has better characteristics compared to semi-flexible pavement mixtures.

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Chapter 1 Introduction

1.1 General

Road transport plays a pivotal role in cost effective transportation of freight and passengers. it provides equal opportunities of access to jobs and trade to all segments of society, thus promoting national cohesion and alleviation of poverty; which are the issues shared by almost all the developing nations around the world, including Pakistan. Realizing the importance of road transport, a major portion of the national income of such countries is spent on construction and maintenance of roads. Consequently, there has been a phenomenal shift towards road transportation options in the past few decades. For example in Pakistan, the ratio of road verses rail transportation was 20:80 in 1950s which has drastically changed over time and now stands at 88:12.

Flexible pavements with hot mix asphalt concrete (AC) surfacing constitutes majority of road pavements around the world predominantly due to its low initial/maintenance cost, easy and quick construction, superior riding quality and skid resistance, etc.

Flexible pavements are considered to be the most complex among the civil engineering structures. The design and performance considerations of flexible pavements includes a multitude of potentially influencing variables including the complex behavior of pavement geomaterials in each layer; resting on a variety of subgrade soils, under dynamic traffic loads and in an array of climatic conditions. Deficient knowledge or inadequate assembling and/or inappropriate handling of these variables in the design as well as construction stages may adversely affect the performance of flexible pavement structures.

More so due to the phenomenal demand shift towards road transportation options; the enhanced pavement serviceability and longevity demand by the society and the worldwide tendency of funds depletion for construction of new and maintenance of large portfolios of existing ageing road networks, pavement technologists and researchers are concentrating more towards coherent design systems to respond to those demands and for optimal use of resources. In the process significant development has been achieved since the era of road tests in the USA (e.g., Maryland road test, 1950; WASHO road test, 1953-54; AASHO road test, 1958-1960). This was the time when the road engineers started moving forward from the rules of thumb to developing performance based empirical pavement design methodologies (Carl L. Monismith, 2004). Pavement design systems thus developed and further refined over time with improved characterization of the input variables (traffic, materials and climate) have served well to this time.

However, the quest towards rational and economical pavement design, construction and maintenance techniques remained on.

The recent advancement in support technologies; particularly more sophisticated laboratory and in-situ instrumentation, and nearly super computers now available for general use; have indeed facilitated the development of analytical or Mechanistic-Empirical (M-E) design systems which are capable of analyzing variabilities in all the design input parameters/conditions (Huang. Y. H., 2004).

Capitalizing on these advancements, this research explores the current situation of flexible pavements in Pakistan with a view to suggesting practical proposals towards Mechanistic-Empirical pavement design system in Pakistan.

1.2 Layout of the thesis

The thesis has been organized in to 9 chapters as outlined in Figure 1.1. Chapter 1 is the introduction and broadly highlights the research objectives and outline of the thesis.

Chapter 2 presents an overview of the flexible pavement design and performance. Basic components of flexible pavement structures have been defined in this chapter. State of the practice and state of the art pavement design approaches have been discussed followed by few examples of design systems based on these approaches. Some important facts about asphalt concrete mixture design and performance have also been highlighted towards the end of this chapter.

Current situation of pavement network, and design and construction practices in Pakistan are discussed in chapter 3 with a view to broadly identifying potential weaknesses in the current road pavement system in Pakistan.

Measures which need immediate attention towards reliable and economical flexible pavement system in Pakistan are outlined in chapter 4.

Chapters 5 elucidate detailed analysis of flexible pavement performance under various loading, climate, and geomaterials conditions. The damaging influence due to excessive axle loading, tire inflation pressure, seasonal variation in climate and changes in stiffness of unbound pavement material has been analyzed using M-E design approach with conditions prevailing in Pakistan as case study.

Investigation of the time dependent strength and deformation properties of UGM are reported in Chapter 6. Test method and results of triaxial compression tests with monotonic loading on granular material under various loading conditions has been discussed in this chapter.

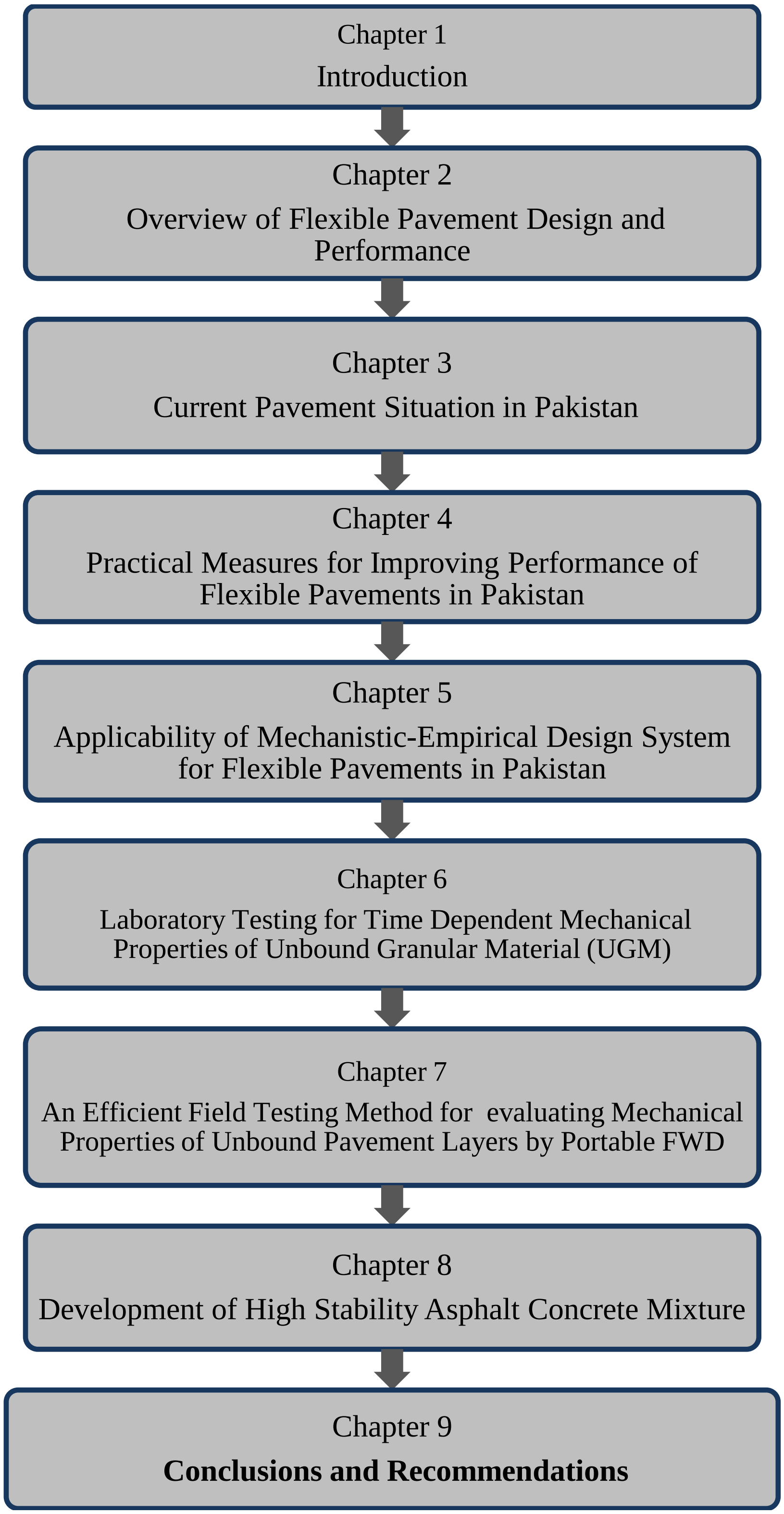

Figure 1.1 Layout of the thesis

Chapter 7 includes discussion on quality control management during execution of unbound base course and subgrade. Evaluation of mechanical properties (stiffness) of unbound base, subbase and subgrade; based on portable FWD tests has been discussed. A relationship between $K_{P.FWD}$ and K_{30} (based on conventional plate loading test), established as a result of comprehensive field testing and available data, has been reported in this chapter.

Development of a High Stability (HS) AC mixture and its in-service performance based on tests results of laboratory compacted mixture and core specimens of a test pavement section have been discussed in chapter 8.

Finally, chapter 9 summarizes the conclusions of this research and recommendations for future research.

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Chapter 2 Overview of Flexible Pavement Design and Performance

2.1 Pavement

Pavement can be defined as a structure build with carefully selected, blended, processed and arranged (in layers) geomaterials over the natural soil foundation (Sub-grade); so as to withstand the combined effect of vehicular loading, climate and ageing; for the ultimate goal of rapid, safe, convenient and economical transportation of freight and passengers.

2.2 Types of pavement

Pavements are generally categorized into three types known as; Flexible, Rigid and Composite.

Typically flexible pavement consists of asphalt concrete (AC) surface layer/s placed over granular base/subbase layer/s supported by compacted soil foundation; referred to as subgrade. Rigid pavement consists of Portland cement concrete (PCC) surface placed over the subgrade with an optional granular base layer (depending on the bearing capacity of the subgrade). The nomenclature flexible and rigid relates to the way the surface layer; AC and PCC respectively, transmit stress and deflection due to the traffic wheel loads to the underlying layers and so to the subgrade. Practically the flexible and rigid pavements depend on the relative stiffness of the AC and PCC layers compared to the underlying granular layers. The relative stiffness of AC is much lower than the PCC and thus termed as flexible and rigid, respectively. The third type which has emerged more recently as one of the pavement rehabilitation treatment known as composite pavement is typically a combination of the flexible and rigid pavements with AC used to cover the damaged PCC or vice versa.

Since the focus of this study is the design and performance of flexible pavements, therefore the discussion hereinafter is concentrated on this type only.

2.3 Components of Flexible Pavements System

The flexible pavement structure is a combination of sub-base, base course, and surface course placed on a subgrade to support the traffic load and distribute it to the roadbed.

Figure 2.1 shows a typical structural section of a flexible pavement system, identifying the basic components, discussed here bellow:

2.3.1 Subgrade

The subgrade is the top surface of a roadbed upon which the pavement structure and shoulders are constructed. The purpose of the subgrade is to provide a platform for construction of the pavement and to support the pavement without undue deflection that would impact the pavement's performance. For pavements constructed on-grade or in cuts, the subgrade is the natural in-situ soil at the site. The upper layer of this natural soil may be compacted or stabilized to increase its strength, stiffness, and/or stability.

For pavements constructed on embankment fills, the subgrade is a compacted borrow material. Other geotechnical aspects of the subgrade of interest in pavement design include the depth to rock and the depth to the groundwater table, especially if either of these is close to the surface. The actual thickness of the subgrade is somewhat nebulous, and the depth of consideration will depend on the design method.

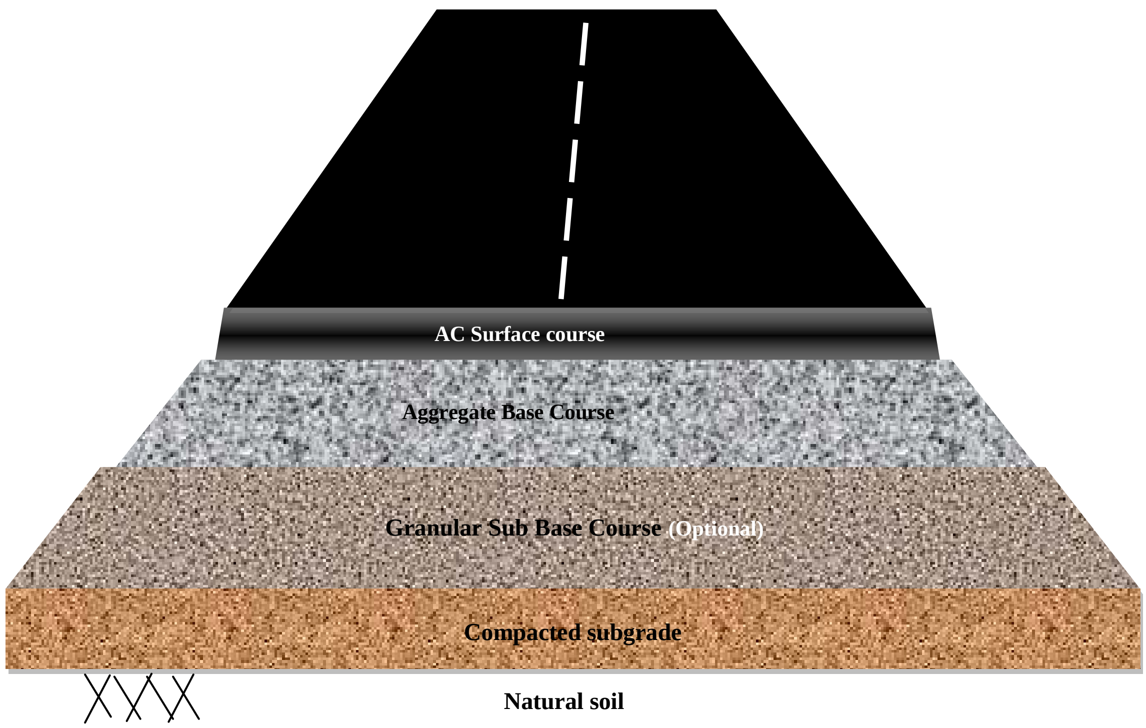

Figure 2.1 Typical structural components of Flexible Pavement

2.3.2 Subbase Course

The sub-base is a layer or layers of specified or selected materials of designed thickness placed on a subgrade to support a base course. The subbase layer is usually of somewhat lower quality than the base layer. In some cases, the sub-base may be treated with Portland cement, asphalt, lime, fly ash, or combinations of these admixtures to increase its strength and stiffness. A sub-base layer is not always included and is typically considered when the subgrade soils are of very poor quality and/or suitable material for the base layer is not available locally, and is, therefore, expensive. Inclusion of a sub-base layer is primarily an economic issue and alternative pavement sections with and without a subbase layer should be evaluated during the design process.

In addition to contributing to the structural capacity of flexible pavement systems, sub-base layers have additional secondary functions:

- Preventing the intrusion of fine-grained subgrade soils into the base layer. Gradation characteristics of the sub-base relative to those of the subgrade and base materials are critical here.
- Minimizing the damaging effects of frost action. A subbase layer provides insulation to frost-susceptible subgrades and, in some instances, can be used to increase the height of the pavement surface above the groundwater table.
- Providing drainage for free water that may enter the pavement system. The sub-base material must be free draining for this application, and suitable features must be included in the pavement design for collecting and removing any accumulated water from the subbase.
- Providing a working platform for construction operations in cases where the subgrade soil is very weak and cannot provide the necessary support.

2.3.3 Base Course

The base is a layer or layers of specified or select material of designed thickness placed on a subbase or subgrade to provide a uniform and stable support for binder and surface courses. The base layer typically provides a significant portion of the structural capacity in a flexible pavement system. The base layer also serves the same secondary functions as the subbase layer, including a gradation requirement that prevents subgrade migration into the base layer in the absence of a subbase layer. It usually consists of high quality aggregates, such as crushed stone, crushed slag, gravel and sand, or combinations of these materials. The specifications for base materials are usually more stringent than those for the lower-quality sub-base materials.

High quality aggregates are typically compacted unbound (*i.e.*, without any stabilizing treatments) to form the base layer. Materials unsuitable for unbound base courses can provide satisfactory performance when treated with stabilizing admixtures, such as Portland cement, asphalt, lime, fly ash, or a combination of these treatments, to increase their strength and stiffness. These stabilizing admixtures are particularly attractive when suitable untreated materials are not available locally. Base layer stabilization may also reduce the total thickness of the pavement structure, resulting in a more economical overall design.

2.3.4 Surface course

The surface course is one or more layers of properly designed, produced and laid asphalt concrete mixture to withstand the traffic load. The top layer also resists skidding, traffic abrasion, and the disintegrating effects of climate in addition to repetitive load impact. Being the most cost intensive structural component, surface course/s must be designed carefully taking into account all the influencing variables e.g. traffic, climate, materials (aggregates and binders), the mixture production and laying processes and the maintenance strategies.

2.4 Flexible pavement structural design

Structural design is the process of determining the thicknesses and vertical positioning of layers of selected geomaterials to provide a serviceable pavement for the predicted traffic over the design life. Each layer is selected and positioned according to the particular physical properties of the material in that layer, the layer underneath and the supporting subgrade. The purpose of the pavement design is to select the most economical arrangement and minimum thickness of each layer to protect the underlying layers and the subgrade from distresses caused by the traffic and environmental loads.

There are two basic approaches for the design of flexible pavements which are vigorously being developed and refined with new knowledge and technology over the past few decades. These are *Empirical approaches* and *Mechanistic-Empirical (M-E) approaches.* The two approaches are briefly outlined below.

2.4.1 Empirical design approaches

Empirical design procedures are derived from experience or observations, often without detailed consideration of system behavior or fundamental pavement theory. Empirically derived regression equations which relate the pavement performance to loads and pavement thickness for a given geographic location and climatic condition are the basis of many existing design methods. These empirical relationships are generally used to determine the required pavement thickness based on the number of load application required to cause failure due to the observed material properties, subgrade type, climate, and traffic conditions.

The advantage of empirical methods is its simplicity and ease of use; often requiring limited input data. The disadvantage of these methods is that it can be applied to a given set of loading, material and environmental conditions. They may be invalid if the conditions are changed and new methods must be developed through trial and error for the new conditions.

2.4.2 Mechanistic-Empirical (M-E) design approaches

Mechanistic-Empirical design methods are based on the mechanics of pavement materials under cyclic wheel loads. The design system involve computing the pavement structural responses (such as stresses, strains or deflections) to wheel load, translating them into damage (such as fatigue or plastic deformation in one or more pavement components), and accumulating the damage into distresses (such as alligator cracking or rutting in the wheel path), which reduces the pavement performance over time. The laboratory established *mechanistic* distress prediction models are calibrated to the actual field performance through *empirical* transfer function from the observed in-service pavement performance. The adequacy of the pavement structural design for specific site conditions (traffic characteristics, material properties and climatic conditions) is achieved iteratively by adjusting the thicknesses of component layers and corresponding material properties to obtain the responses (stress/strain) that allow the distresses to remain at acceptable level of performance at the end of the design/analysis period. Figure 2.2 illustrates the basic process of M-E design approach.

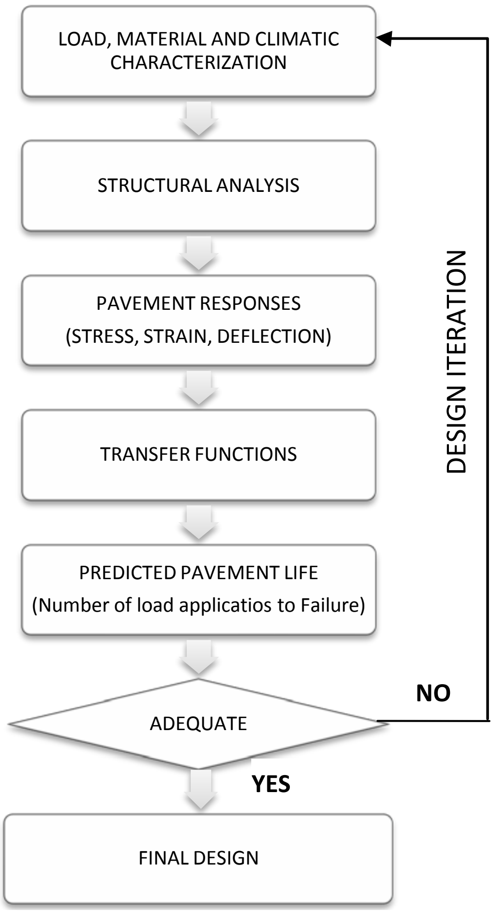

Figure 2.2 Basic process of M-E pavement design system

Advantages of M-E design procedures far outweigh a few disadvantages including more comprehensive and sophisticated data requirement, increased computation time etc. some of the practical advantages are highlighted as below (AASHTO, 2002):

- Estimates of the consequences of new loading conditions can be evaluated. For example, the damaging effects of increased loads, high tire pressures, multiple axles, and other factors can be modeled using M-E procedures.
- Better utilization of available materials can be considered. For example, the use of stabilized materials in flexible pavement can be simulated to predict future performance.

- Improved procedures to evaluate premature failures can be developed, and better diagnostic techniques can be utilized.
- Aging can be included in estimates of performance (e.g., asphalt hardens with time, which, in turn, affects low temperature thermal cracking).
- Seasonal effects such as temperature dependency of AC stiffness, thaw-weakening etc can be included in estimates of performance.
- Methods can be developed to better evaluate the long-term benefits of providing improved drainage in the roadway section.

2.5 Design Factors

Following are major factors considered in the design of flexible pavements:

- Traffic volume and loading
- Materials
- Climate
- Failure criteria

2.5.1 Traffic volume and loading

Pavements are constructed to carry traffic safely and efficiently. Traffic loads particularly from heavy commercial vehicles (trucks) induces stresses and strains in the pavement structure. The repetitive application of these loads causes the pavement to deteriorate over time. Therefore truck traffic load is one of the basic input factors in any pavement design methodology. The impact of truck traffic loads are quantified in terms of number of truck axles/wheels, configuration of these axles, load magnitude over each axle, number of repetition of axles and tire inflation pressure. In most design methods, the cumulative effect of the entire volume of mixed traffic over the design/analysis period is transformed into a single traffic input of; either equivalent single axle load (ESAL) or Equivalent single wheel load (ESWL). However, the recently introduced M-E design system by American Association of State Transportation Officials (AASHTO) consider the axle load spectrum, which is believed to be a realistic representation of traffic factor in the design of pavement structure (AASHTO, 2002).

2.5.1.1 Equivalent single axle load (ESAL)

The total projected magnitude and repetition of various types of vehicles (mix traffic) are converted in to a single variable i.e. the total number of repetition of a standard single axle; usually 80 kN (18 kips) with dual tires inflated to 550 kPa, using the equivalent axle load factor (EALF) for each particular vehicle. EALF defines the damage per repetition caused to a pavement by the axle type in question relative to the damage per repetition by a standard (80 kN) single axle load. The number of repetition of each axle load groups (single, tandem, Tridem etc) are multiplied by the corresponding EALF and summed up over the design period to estimate design ESALs as below:

$$ESAL = \sum_{i=1}^{m} Fi\, ni \qquad (2.1)$$

Where; m is the number of axle load groups, Fi is EALF for ith axle load group and ni is repetition of ith load group during the design period.

In the context of flexible pavement the EALF depend on the thickness and structural capacity (stiffness) of each component layer, and the failure criteria. The EALFs can be determined either empirically based on experience or mechanistically based on theory. The empirical relationship developed by AASHTO based on road test (AASHO, 1962) is the most commonly used and expressed here below as:

$$EALF = \frac{W_{tx}}{W_{t18}} \qquad (2.2)$$

$$log\left(\frac{W_{tx}}{W_{t18}}\right) = 4.79\ log(18+1) - 4.79\ log(L_x + L_2) + 4.33\ log\ L_2 + \frac{G_t}{\beta_x} - \frac{G_t}{\beta_{18}} \qquad (2.3a)$$

$$G_t = log\left(\frac{4.2-P_t}{4.2-1.5}\right) \qquad (2.3b)$$

$$\beta_x = 0.40 + \frac{0.081\,(L_x+L_2)^{3.23}}{(SN+1)^{5.19}\,{L_2}^{3.23}} \qquad (2.3c)$$

In the above equations, W_{tx} is the number of repetition of x-axle load at the end of time t; W_{t18} is the number of repetition of 18-kips (80 kN) single axle load to time t; L_x is the load (in kips) on axle group in question (one single axle, one set of tandem axles, or one set of tridem axles; L_2 is the axle code (1, 2, 3 for single, tandem and tridem axles, respectively); SN is the structural number, which is a function of the thickness and stiffness modulus of each layer and drainage condition of base and subbase (as explained in section 2.6); P_t is the terminal serviceability which indicate the failure condition of the pavement; G_t is the function of P_t and β_{18} is the value of β_x when L_x is equal to 18 and L_2 is equal to 1.

Determination of EALF using M-E analysis approach is demonstrated in chapter 5; wherein load factors have been identified by relative damage factors (RDFs).

2.5.1.2 Axle load spectrum

The concept of axle load spectrum is a recent and significant improvement in consideration of traffic factor in the pavement design which supports state of the art M-E pavement design methods. In this approach, instead of single traffic input e.g ESAL all the potential traffic factors including volume; vehicle classes by axle configuration and axle load distribution are directly considered in the design process. Pavement structural responses due to each individual vehicle class are translated into damages which in turn are accumulated into distresses over the design life. Though this approach represents traffic load effects more rationally, the extensive data collection and management make it a cost intensive option, particularly for under developed countries.

2.5.2 Material Properties

The basic materials used in construction of flexible pavements are asphalt concrete (AC), granular bases and subbases, and subgrade soil (NHI, 2006).

Traditionally empirical static load tests; indicative of mainly strength properties of the materials; have been used in the pavement design practices including the famous California bearing ratio (CBR) test, R-value test, plate loading tests, marshal stability test (for AC mix design) etc. whilst these empirical tests are simple and easy to handle, it have many drawbacks including their inability to extrapolate historical knowledge to changing conditions, new materials and increased traffic loadings. Realizing these shortcomings, much of the research during the last few decades has been focused towards defining the mechanical properties of the pavement geomaterials under cyclic loads with a view to characterizing not only the strength but deformation properties as well.

Mechanistic characterization of pavement materials require testing procedure that determine stress-strain responses under test conditions as closely representative of the actual field conditions as possible (Papagiannakis et. al., 2008). Repeated load triaxial compression tests are generally performed for pavement material characterization. Geomaterials may display a wide range of response to loading from elastic to plastic and viscous; depending on the rate of loading, the temperature and/or moisture conditions during testing and the time dependent properties i.e. effect of material ageing etc. The effect of loading rate and ageing on the deformation properties of untreated granular material (UGM) has been discussed in more detail in chapter 6.

In the M-E design approaches, pavement is considered as layered elastic system and each layer is characterized by its resilient modulus (M_r); more commonly known as stiffness; and the poison`s ratio (μ). The resilient modulus can be defined as the elastic modulus based on recoverable strain under repeated loads and is expressed as:

$$M_r = \frac{\sigma_d}{\varepsilon_r} \tag{2.4}$$

Where; σ_d is the deviator stress (which is the axial stress less the confining pressure in triaxial compression test) and ε_r is the recoverable strain.

It is desirable to use the laboratory determined M_r values for the actual material to be utilized in the pavement structure. However, various correlations relating the traditional strength based parameters e.g. CBR, R-value, layer coefficient etc. to M_r are available in the literature e.g. AASHTO (1986/1993), the Asphalt Institute (1982/1999); which can be used for the network level pavement analysis and design.

The effects of stiffness of pavement geomaterials on the performance of flexible pavements under various loading, tire inflation pressures and climatic conditions has been investigated and reported in chapter 5 of this report.

2.5.3 Climatic conditions

Climate has profound effects on material properties and thus on the performance of pavement structures. The major climatic conditions that affect the pavements are moisture (rainfall/ground water), temperature and solar radiations.

Moisture in pavement reduces the strength and stability of base, subbase and subgrade. Similarly both low and high temperatures poses damage threat to pavements. Low temperatures in winter freeze the moisture in the pavement structure. As the temperature rises during spring, the pavement thaws. The ice in the lower layers of the pavement traps the water in the structure, thus reducing the strength and making it susceptible to further deterioration by the traffic loads. This phenomenon; known as freeze-thaw; is one of the major causes of pavement damage in moderate temperature zones where seasonal temperature cycles occur annually.

High temperature also causes softening of the asphalt binder thereby reducing the viscosity and thus the stability and stiffness of the AC layer, causing rutting in the pavement surface. Whereas low temperature make the AC rigid/brittle and susceptible to fatigue fracture or cracking of the surface layer under repeated traffic loading.

The effects of seasonal variation in ambient temperature coupled with excessive axle loading on the performance of flexible pavement were analyzed for the typical conditions in Pakistan. Results are discussed in chapter 5. A new high stability AC mixture which can withstand the detrimental effects of high temperature and excessive loading was developed and evaluated from the results of a test pavement section. Results of the laboratory and field performance of the said mixture are presented in chapter 8 of this report.

2.5.4 Failure criteria

Establishing a failure or terminal condition at which the pavement ceases to perform its intended *structural* or *functional* purpose and require reconstruction or rehabilitation is a prerequisite for the design system; to assess whether a design alternative is accepted.

According to Yoder and witczak (1975), *structural failure* occurs when the pavement is incapable of sustaining the loads imposed on its surface due to the breakdown of one or more of its components. Whereas, *functional failure* occurs when the pavement is unable to perform its intended function without causing undue discomfort to pavement users or imposing high stresses on the vehicles.

Major causes of these failure conditions may be excessive loads, climate and environmental condition, poor drainage affecting the subgrade condition and disintegration of the component materials. Excessive loads, inordinate load application and increased tire inflation pressure can cause either structural or functional premature failure. In the context of flexible pavements, properties and failure modes of asphalt concrete (AC) must also be considered in the pavement design process. Adequate knowledge and practice of AC mix design procedures are therefore essentially required.

In the empirical design procedures, pavement serviceability assessed in term of initial and terminal 'serviceability index' is considered as performance criteria. The serviceability of a pavement is a measure of its fitness to carry traffic comfortably, safely and economically. The level of serviceability of a pavement declines gradually and continuously over time/number of traffic application to a state where it can no longer carry the traffic acceptably and will have to be withdrawn from service for major rehabilitation. The concept is shown in Figure 2.3.

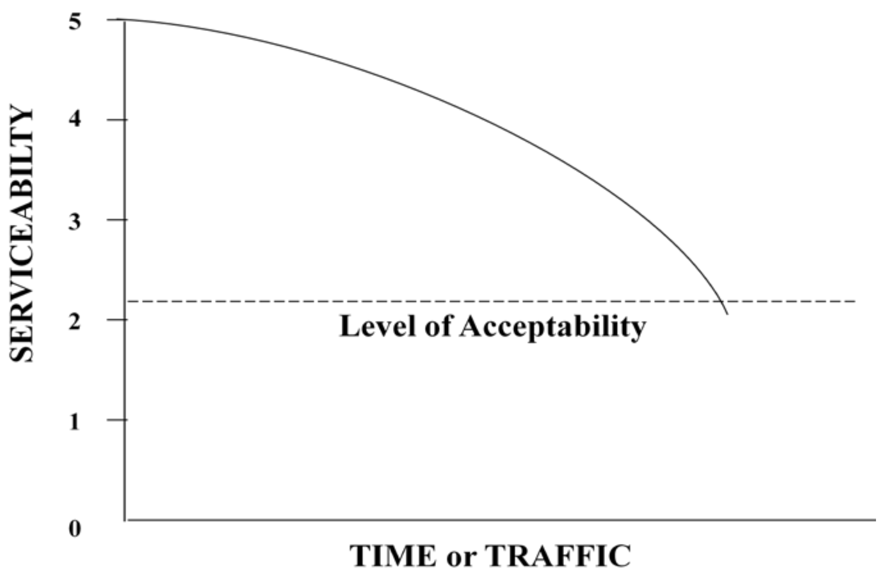

Figure 2.3 Pavement Serviceability history

The serviceability-performance concept in terms of “Present Serviceability Index” (PSI) was developed in the AASHO road test and is based upon a rating scale which defines the condition of the pavement at any instant of time. A rating of 5.0 indicates a “perfect” pavement; whereas 0 rating indicate an “impassable” or “failed” pavement. The subjectively determined rating by a panel of drivers was correlated with objectively measured pavement distresses such as cracking, rutting and roughness by the following relation.

$$PSI = 5.03 - 1.91 \log(1 + \overline{SV}) - 1.38(\overline{RD})^2 - 0.01(C + P)^{1/2} \quad (2.5)$$

Where; $\overline{SV}$ is the mean slope variance in both wheel paths (a measure of roughness/riding quality), $C + P$ is area of cracking and patching (ft^2 per 100 ft^{2}of surface area), and $\overline{RD}$ mean rut depth in wheel paths (a measure of permanent deformation).

Failure criterion in terms of terminal serviceability ranging from 1.5 – 2.5 is usually selected; depending upon the importance of the road, road user satisfaction and availability of resources. Design alternative is assessed in terms of allowable number of ESALs to the predetermined terminal serviceability.

In the M-E design practices a number of failure criteria; depending on the dominating distress; is determined. Flexible pavements are generally assessed based on the extent of AC fatigue damage (cracking), permanent deformation (rutting) and low temperature cracking.

Fatigue cracking results from horizontal tensile strain at the bottom of AC layer. The allowable number of load repetitions is related to the tensile strain through laboratory fatigue tests on AC specimens. Due to the inherent differences in laboratory and actual field conditions, transfer functions are usually determined based on long term observations of in-service pavements. Fatigue cracking models of the following form are generally used.

$$N_f = k_3 \left(\frac{1}{\varepsilon_t}\right)^{k_1} \left(\frac{1}{E}\right)^{K_2} \quad (2.6)$$

Where; N_f is the allowable number of repetition of load to cause cracking to a predetermined extent (usually10% to 20% of the surface area), ε_t is the tensile strain at the bottom of AC layer/s, E is elastc modulus of AC layer/s, k_1 and K_2 are material constants, and k_3 is the transfer function.

In most of the current M-E flexible pavement design practices, rutting is controlled to limit the vertical compressive strain on the top of the subgrade. The following general model forms are used to determine the allowable number of load repetition to cause rutting.

$$N_R = k_5 \left(\frac{1}{\varepsilon_c}\right)^{k_4} \tag{2.7}$$

Where; N_R is the allowable number of repetition of load to cause rutting (mean rut depth of 12mm – 15mm, measured on the pavement surface), ε_c is vertical compressive strain on the top of the subgrade, k_4 and k_5 are calibration parameters.

Miner`s (1945) principal of cumulative damage is generally adopted in order to account for varying conditions of climate (e.g. monthly or seasonal variation in ambient temperature) and traffic (e.g. various axle load groups) using the following relation.

$$D_R = \sum_{ij} \frac{n_{ij}}{N_{ij}} \tag{2.8}$$

Where; D_R is the cumulative damage ratio, n_{ij} is predicted number of repetition of axle group j for season i and N_{ij} is the allowable number repetition of axle group j for season i. The cumulative damage ratio should be less than 1 for all axle groups and seasonal interval during the design period.

2.6 Flexible pavement design methods

Some of the methods for structural design of flexible pavement are discussed here below.

2.6.1 Empirical design methods

2.6.1.1 AASHTO design method (1986/1993)

The design guide introduced by the American association of state highway and transportation official (AASHTO) is widely used; not only in the USA but also in many other countries for structural design of flexible pavements. The AASHTO design equations are based on regression analysis of extensive data; obtained from systematic observations during the famous AASHO road test; conducted from late 1950 to early 1960 in Ottawa; Illinois (AASHO, 1962). First published in the form of an interim design guide in 1961 and later revised in 1972 and 1981, a complete guide was issued in 1986 with some minor changes in 1993. The revisions were primarily necessitated by the need to make the design guide applicable to other regions in the USA, since the original design equations were developed under a given climatic condition with specific set of pavement geomaterials and subgrade soil.

The empirical relationship that computes the loss in serviceability (ΔPSI) in terms of number of cumulative ESAL repetitions during the design/performance period is given as:

$$\log(W_{18}) = Z_R\, S_0 + 9.36\log(SN+1) - 0.20 + \frac{log\left[\frac{\Delta PSI}{4.2-1.5}\right]}{0.4+\frac{1094}{(SN+1)^{5.19}}} + 2.32\log(M_r) - 8.07 \qquad (2.9)$$

Where; W_{18}: the number of ESALs that will result in a change in serviceability of ΔPSI

M_r : Resilient modulus of the subgrade soil

Z_R : the standard normal deviate for a given reliability "R"

S_0 : Standard deviation (to account for combined errors in prediction of traffic and performance)

SN : the structural number of pavement expressed as:

$$SN = a_1 D_1 + a_2 D_2 m_2 + a_3 D_3 m_3 \qquad (2.10)$$

Where; a_1, a_2, a_3 are layer coefficients for surface, base and subbase, respectively which represent the relative ability of a unit thickness of a given material to function as a structural component of the pavement. Layer coefficients are usually determined from the correlations with material properties (e.g. M_r, CBR, etc). $D_1,\ D_2,\ D_3$ are the thicknesses of the surface, base and subbase (inches), respectively. m_2 and m_3 are drainage coefficients of the base and subbase, respectively.

2.6.1.2 The Japan Road Association (JRA) T_A design method

The JRA manual for design and construction of asphalt pavement is generally used in Japan for design of public roads (JRA, 1980). The design method is based on the basic principles of the AASHO road test and the CBR design curves. Subgrade CBR is the basic design input which is used to determine the equivalent thickness `$T_A$`.

T_A represents the thickness of the pavement which would be required when the entire pavement structure is to be constructed with asphalt concrete, with no intermediate layers in between. Equivalent thickness is estimated as:

$$T_A = R \ \frac{N^{0.16}}{CBR^{0.3}} \tag{2.11}$$

Where; N is the total number of predicted traffic applications measured in terms of equivalent 49 kN wheel loads during the design period, R is the reliability factor valuing 3.07, 3.43 and 3.84 for the design reliability of 50%, 75% and 90%, respectively.

Thickness of each individual layer for a conventional flexible pavement structure including intermediate layers of base and subbase is worked out using the following relation:

$$T_A = a_1T_1 + a_2T_2 + \cdots + a_nT_n \tag{2.12}$$

Where T_1, T_2, $\cdots$, T_n are thicknesses of individual component layer, cm; and a_1, a_2, $\cdots$, a_n are strength coefficient of each layer relative to the strength of unit thickness of AC. Coefficients of relative strength for various pavement materials as recommended by JRA based on observations in Japan are shown in Table 2.1 below.

2.6.2 M-E design methods

2.6.2.1 The Asphalt Institute (AI) Method

The basic process of M-E design method introduced by the Asphalt Institute is the same as outlined in fig. 2.2. Traffic input is considered in terms of 18-kips (80kN) ESALs as discussed earlier. Component materials are characterized in term of resilient modulus (M_r) in psi and Poisson's ratio (μ).

Climate is considered in terms of mean annual air temperature (MAAT). The AI design method is based on the following two criteria:

Table 2.1 Coefficients of Relative Strength for Calculating T_A (after JRA 1980)

Pavement Course	**Method and material of construction used**	**Conditions**	**Coefficient**
Binder and Surface	Hot mixed asphalt	-	1.00
Base	Bituminous stabilized	Hot mixed, Marshal stability: 350kg or more	0.80
		Cold mixed, Marshal stability: 250kg or more	0.55
	Cement stabilized	Unconfined compressive strength (7 days) : 30kg/m^2	0.55
	Lime stabilized	Unconfined compressive strength (10 days) : 10kg/cm^2	0.45
	Mechanically stabilized Gravel and Slag	Modified CBR : 80 or more	0.35
	Hydraulic Mechanically stabilized Slag	Modified CBR : 80 or more	0.55
	Penetration Macadam	Unconfined compressive strength (14 days) : 12kg/cm^2	0.55
Subbase	Crusher-run, Slag, Sand etc.	Modified CBR : 30 or more	0.25
		Modified CBR : 20 - 30	0.20
	Cement stabilized	Unconfined compressive strength (7 days) : 10kg/cm^2	0.25
	Lime stabilized	Unconfined compressive strength (10 days) : 7kg/cm^2	0.25

- Limiting the tensile strain at the bottom of the AC layer/s to prevent fatigue cracking distress.
- Limiting the compressive strain at the top of the subgrade to prevent permanent deformation of subgrade that result into rutting distress.

The total number of load repetition (N_f) to fatigue failure (defined by fatigue cracking to the extent of 10% of the wheel path area) is expressed as:

$$N_f = 0.00432\, C \left(\frac{1}{\varepsilon_t}\right)^{3.291} \left(\frac{1}{E}\right)^{0.854} \tag{2.13}$$

Where ε_t: the tensile strain at the bottom of the AC layer/s, E : the elastic modulus of AC (in psi) and C : the correction factor for AC mix properties defined as:

$$C = 10^{M} \tag{2.13a}$$

In which $$M = 4.84\left(\frac{V_b}{V_a + V_b} - 0.69\right) \tag{2.13b}$$

Where V_a : the percentage volume of air voids in the mix, and V_b: percentage volume of asphalt binder in the mix.

The number of load repetition(N_R) to rutting failure (defined by mean rut depth of 12.5mm) is expressed as:

$$N_R = 1.365 \times 10^{-9}\left(\frac{1}{\varepsilon_c}\right)^{4.447} \tag{2.14}$$

Where ε_c: the compressive strain at the top of the subgrade.

To simplify the design process, various nomographs were produced by performing series of layered elastic analysis runs for various combinations of material properties and layer thicknesses; using a computer program known as DAMA. The nomographs allow determining the thickness of AC layer given the design traffic in terms of ESALs and the M_r of the subgrade with a wide choice of base layer thickness and material type (treated/untreated). The AI manual (MS-1, 9th edition, 1981) contains nomographs for MAAT of 7°C, 15.5°C and 24.4°C. An example nomograph is shown in Figure 2.4.

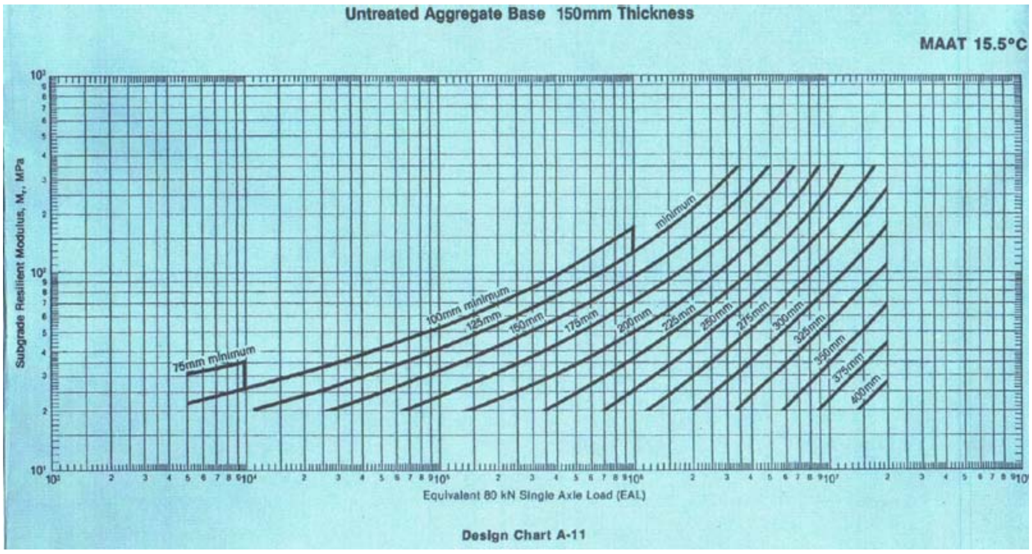

Figure 2.4 The AI thickness design nomograph for 150 mm untreated aggregate base; MAAT 15.5°C (after AI; 1981)

2.6.2.2 The JRA M-E design method

The Japan Road Association (JRA) M-E pavement design system is a modified version of the AI design procedure with the distress transfer function calibrated to the local conditions and a support structural analysis tool known as "General Analysis of Multi-layered Elastic System" (GAMES), developed specifically for the propose. Traffic input is considered in terms of 49 kN equivalent wheel (dual tires 320mm c/c) load. The general flow of the design process is shown in Figure 2.5 below.

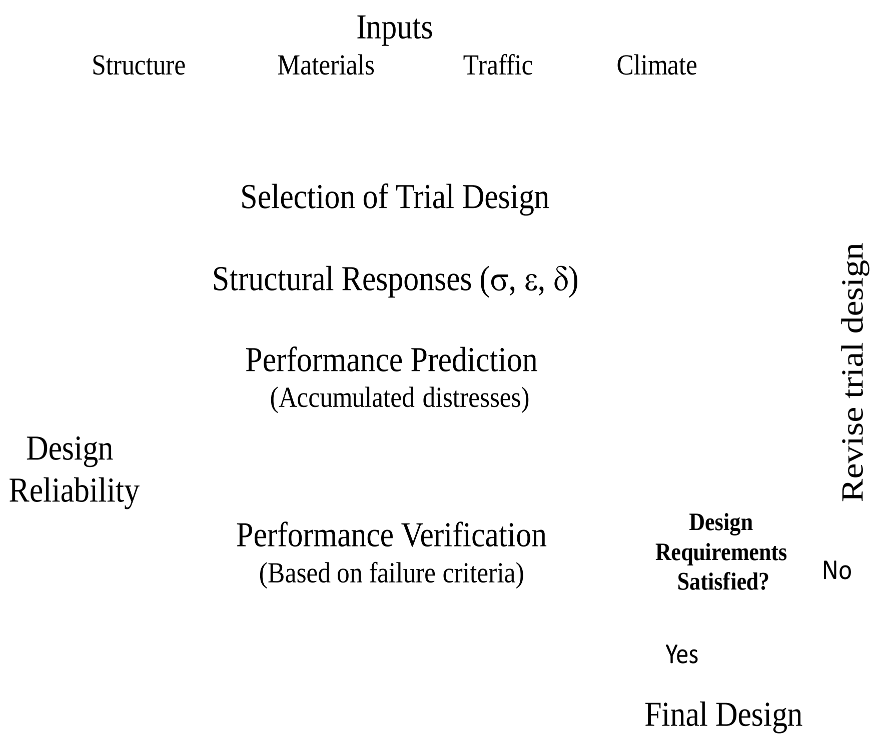

Figure 2.5 General process of the JRA Mechanistic-Empirical design sys-

In the M-E design system proposed by JRA, a pavement section is considered to be failed when either 20% of lane area is cracked and/or the pavement structure exhibit 15 mm of rut depth. The model to predict the allowable number of load repetition to fatigue cracking (N_{ff}) is a function of tensile strain ($\boldsymbol{\varepsilon_t}$) and Asphalt concrete stiffness (elastic modulus) "E" and is presented here bellow (JRA, 2001):

$$\boldsymbol{N_{ff}} = \beta a1 \cdot C \cdot 6.617 \cdot 10^{-5} \cdot \left(\frac{1}{\varepsilon_t}\right)^{3.291 \cdot \beta a2} \left(\frac{1}{E}\right)^{0.854 \cdot \beta a3} \qquad (2.15)$$

Where; $$C = 10^{M} \tag{2.15a}$$

and $$M = 4.84\left[\frac{VFA}{100} - 0.69\right] \tag{2.15b}$$

Where; *VFA*: Volume of voids filled with Asphalt binder (%). $\beta a1$, $\beta a2$ and $\beta a3$ are calibration constant in which:

$$\beta a1 = Ka \cdot \beta a1' \tag{2.15c}$$

"*Ka*" is a correction coefficient for Asphalt concrete layer thicknesses (hac) and is expressed as:

$$Ka = \frac{1}{8.27\text{x}10^{-0.11} + 7.83 \cdot e^{-0.11(hac)}} \tag{2.15d}$$

The values of $\beta a1'$, $\beta a2$ and $\beta a3$ are given as $5.229\text{x}10^{4}$, 1.344 and 3.018, respectively.

The model to predict the allowable number of load repetition to permanent deformation or rutting (N_{fd}) is a function of vertical compressive strain (ε_c) at the top of the subgrade and is expressed as:

$$N_{fd} = \beta s1 \cdot \left(1.365\text{x}10^{-9} \cdot \epsilon_c^{-4.477 * \beta s2}\right) \tag{2.16}$$

Where; $\beta s1$ and $\beta s2$ are calibration constants, its values are 2.134 x 10^{4} and 0.819, respectively.

The climatic effects are addressed by incremental damage analysis with 12 (monthly) increments in each year over the design period. Mean monthly air temperature and corresponding AC modulus values are used in the analysis. Material properties of base, subbase and subgrade and traffic are assumed to be constant during each increment. fatigue cracking and rutting damages are accumulated over the entire design period using the Miner`s principle and the adequacy of the design alternative (trial design structure) is assessed.

A practical application of this design system is demonstrated in chapter 5 of this report.

2.7 Asphalt Concrete Mixture

Asphalt concrete (AC) is a paving material produced by blending together aggregates, mineral filler and asphalt binder (bitumen) in precise proportion and at specified temperatures. The relative proportions of these materials determine the physical properties of the mixture and thus its performance as the dominant component of flexible pavement structures under all conceivable loading and environmental conditions.

The AC mixtures are used in surface, binder or base courses in flexible pavement structures. Stresses from the traffic loads are transmitted mainly through the stone to stone contact of the aggregates cemented together by the asphalt binder. Therefore the characteristics of both the elements are important. AC is a preferred paving material throughout the world due to its ease and efficiency of construction and cost effectiveness compared to more costly PCC pavements which require long construction time (due to formwork, steel reinforcement and curing etc) during which the pavement remained closed to traffic. Conversely, AC pavements can be opened to traffic almost immediately after construction and is therefore also a preferred choice for rehabilitation/overlays of the existing pavements. Other advantages include its potential for staged construction and complete recyclability. It is for these reasons that nearly 96% of the entire road network in the USA alone is paved with AC (MAPA, 2006).

2.7.1 Characteristics and Behavior of AC mixture

The AC mixture is analyzed in the laboratory prior to construction; to determine its probable in-service performance in a pavement structure. The following four characteristics are considered in the analysis (also known as volumetric analysis) and their influence on the behavior of the mixture are analyzed (AI, 2001):

- Mix density
- Air voids
- Voids in the mineral aggregate
- Asphalt content

2.7.1.1 Mix density

Density of a compacted mixture is the weight of a specific volume of mix. High density of a finished pavement is essentially required for the lasting performance of the pavement. In the testing and analysis of the mixture, density of compacted specimen is expressed in kilogram/ cubic meter (kg/ m^3) and is calculated by multiplying the bulk specific gravity of the mix by density of water (1000 kg/m^3). The maximum theoretical density o specimen determined in the laboratory is used as standard to assess whether the density of the finished pavement meets the specification requirement.

2.7.1.2 Air Voids (VA)

Air voids are small pockets of air between the asphalt coated aggregates in the final compacted AC. A certain percentage of air voids is necessary in the finished AC to allow for a small amount of compaction under traffic and a slight expansion of the asphalt binder due to hot temperature. Depending on the specific design for surface course and base course etc, the allowable air voids in the laboratory specimen is 3-5 percent. Air void content in the finished AC pavement determines its density and thus durability. Lower air voids makes the mixture less permeable. A mixture with too high air void contents is susceptible to ingress of water and air; both of which have degenerating influence on the mixture. Job specifications usually require air void content of 3 - 8% in the finished AC pavements.

2.7.1.3 Voids in Mineral Aggregate (VMA)

VMA are the void spaces that exist between the particle grains of the aggregate in a compacted mixture. VMA is expressed as a percentage of total bulk volume of the compacted mixture and includes spaces filled with asphalt binder and the remaining air voids. As shown in Figure 2.6, VMA is the space within the compacted mixture that is available to the effective volume of asphalt binder (i.e., all the binder less the portion lost by absorption into the aggregate). The more VMA in the dry aggregate, the thicker the binder film around the aggregate particles and the more the mixture is durable. Therefore; depending on the aggregate gradation; percentage of VMA is specified for the mixture.

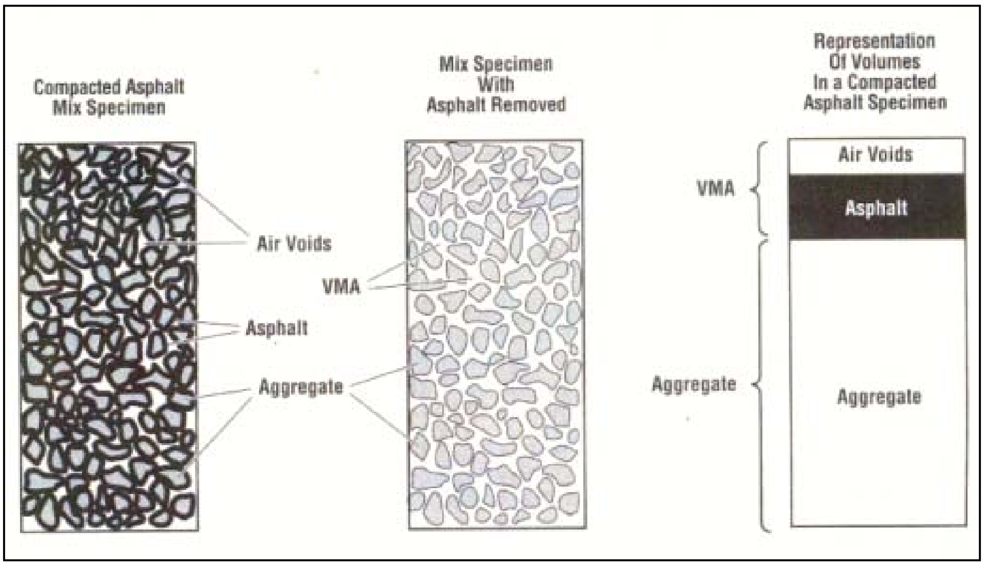

Figure 2.6 Voids in Mineral Aggregate (VMA) in a compacted mix specimen (after AI, 1997)

2.7.1.4 Voids Filled with Asphalt (VFA)

Voids filled with asphalt binder (VFA), is the percentage of VMA that is filled with asphalt binder. Amount of VFA content in the mixture determine its durability and stability. Too little VFA leave the mixture dry and hence less durable giving rise to chances of premature cracking and raveling. Increasing the VFA may result into unstable mixture, susceptible to plastic deformation. The acceptable range of VFA in the mixture depends upon the anticipated design traffic. Higher traffic requires a lower VFA to ensure the strength and stability of the mixture. Whereas, lower traffic do not pose any threat to the stability of the mixture and therefore a higher range of VFA is specified to ensure durability.

2.7.1.5 Asphalt Content

The amount of asphalt content is critical to all the desired physical properties of AC mixture and must be precisely determined in the laboratory and then controlled in the field. The optimum asphalt content is a function of the aggregate characteristics such as gradation and absorption. The finer the aggregate gradation in a mix, the larger total surface area of the aggregate and the greater the amount of asphalt binder required to uniformly coat the particles. Conversely, coarser aggregate have less total surface area and require less asphalt. Similarly, the extent of aggregate absorption is critical to ascertain the asphalt being lost (due to absorption) and the expected asphalt bonding film in the mixture.

2.7.2 Purpose of AC Mixture Design

Design of AC mixture is the process of proportioning and blending of well graded (continuously graded) aggregate and weather resistant asphalt binder in such a way as to ensure certain desirable properties in the finished pavement. These include stability, durability, impermeability, workability, flexibility, resistance to fatigue and skid in all weather and traffic conditions. These properties are more critical in adverse service conditions such as extreme climate (extreme higher or lower temperature ranges), excessive axle loads and increased tire inflation pressures. In such condition the mix design process becomes more complex; particularly, the balancing of somewhat conflicting requirements of good durability and high resistance to deformation. The durability of the mixture is associated with resistance of the asphalt binder to age hardening. One way of reducing the age hardening is to increase the binder content in the mix which reduces the air voids and increases the thickness of the binder film coating the aggregate particles. However, increasing the binder content reduces the stiffness and thus resistance to plastic deformation (rutting).

Table 2.2 shows some of the causes of deficiencies of in the AC mix properties and their corresponding effects on the pavement performance. The key to AC mix design is to produce a mixture that possesses an acceptable balance of these properties.

The following three standardized procedures are generally used for AC mixture design and construction quality control.

- Marshal method (ASTM-1559)
- Hveem method (ASTM D-1560)
- Superpave method (AASHTO M-320)

Table 2.2 Causes and effects of deficiencies in AC Mix Properties (after AI, 2001)

Description	Causes	Effects
Pavement Instability	Excess asphalt binder in mix	Rutting, shoving and bleeding
	Excess medium size sand in mix	Tenderness during rolling and for period after construction, difficult to compact
	Rounded aggregate, little or no crushed surfaces	Rutting and channeling
Lack of durability	Low asphalt content	Dryness or raveling
	High voids by design or due to lack of compaction	Early hardening of asphalt followed by cracking or disintegration
	Water susceptible aggregate in mix	Asphalt film strips from aggregate followed by abrasion and raveling
Permeability	low asphalt content	Thin asphalt film will cause early ageing and raveling
	High voids content design mix	Easy ingress of water and air followed by oxidation and disintegration
	Inadequate compaction	Will result in high voids in pavement, leading to water infiltration and low strength
poor workability	Large maximum size particle	Rough surface, difficult to place
	Excessive course aggregate	Hard to compact
	Too low mix temperature	Uncoated aggregate, not durable, rough surface, hard to compact
	Too much medium size sand	Mix shoves under roller, remains tender
	Low mineral filler content	Tender mix, highly permeable
	High mineral filler content	Mix may be dry, hard to handle, not durable
Poor Fatigue resistance	Low asphalt content	Fatigue cracking
	High design voids	Early ageing of asphalt followed by Fatigue cracking
	Lack of compaction	Early ageing of asphalt followed by Fatigue cracking
	Inadequate pavement thickness	Excessive bending, followed by fatigue cracking
Poor skid resistance	Excess asphalt content	Low skid resistance, bleeding
	Poorly textured or graded aggregate	Smooth pavement surface, potential for hydroplaning
	Polishing aggregate in mixture	Low skid resistance

Historically asphalt mix design has been accomplished using either the Marshall or the Hveem design method due to its simplicity and inexpensive test equipments. The most common has been the Marshall method. The Superpave mix design procedure was introduced in 1995. It builds on the knowledge from Marshall and Hveem procedures. The primary differences between the three procedures are the machine used to compact the specimens and performance tests used to evaluate the mixes. Details of these tests procedures are amply available in the literature (e.g. AI-1997, 2001)

2.7.3 Modified Asphalt Binders

Modification of asphalt binders to enhance its visco-elastic properties has been necessitated by the increase in traffic loadings, extreme climatic conditions (extreme hot or cold or seasonal variations in temperatures), new asphalt refining and polymer technologies, and the increasing trends for using recycled materials in pavement construction.

Depending upon the refining process, each asphalt binder exhibits the right combination of viscous and elastic properties in a certain ideal range of temperatures as shown in Figure 2.7. Modifiers (polymers/additives) are added to asphalt binders to alter their natural visco-elastic behavior thus enhancing the ideal temperature ranges. For example binder-III in Figure 2.7 has a narrow ideal range of temperature and may perform adequately in moderate climatic condition with consistent year round temperature. However, the same binder is not suitable in extreme seasonal temperature conditions. By introducing certain modifiers to binder-III, its characteristics can be enhanced to perform like binder-I; which has a wide ideal range of temperature and better suited to seasonal temperature variations.

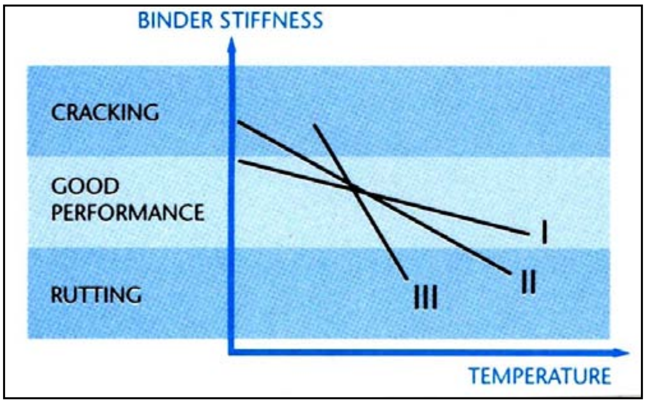

Figure 2.7 Ideal temperature ranges for good AC pavement performance (after PTA-D5, 2005)

Table 2.3 outlines some of commonly used modifiers, their generic classification and effects on pavement distresses.

Modified asphalt binders are typically more viscous (thicker) compared to unmodified binders resulting into improved adhesion and thicker coating of aggregates in the mixture and thus more resistant pavement disintegration due to age hardening and oxidation. The durability of pavement sections which are exposed to excessive loading and/or slow moving heavy trucks can be improved by using modified asphalt mixtures. The increased viscosity coupled with improved aggregate bonding help resist plastic deformation (*rutting*) under heavy loads, while the enhanced elastic features improve resistance against fatigue fracture (*cracking*) due to repetitive traffic loading over the life span of the pavements.

Table 2.3 Types of Asphalt binder modifiers (after Bhaia et.al. 2001)

Modifier Type	Generic class	Effect on Distresses				
		PD[a]	FC[b]	LTC[c]	MD[d]	AG[e]
Fillers	Carbon black	x	x			x
	Hydrated Lime	x				x
	Fly ash	x				
	Portland cement	x				
Extenders	Sulfur	x	x	x		
Elastomeric polymers	Styrene-butadiene-styrene(SBS)	x	x	x		
	Styrene-butadiene-rubber latex(SBR)	x		x		
	Natural rubber	x				
Thermoplastic polymers	Ethylene vinyl acetate(EVA)	x	x			
	Ethylene propylene diene monomers(EPDM)	x				
	Polyethylene (PE)	x		x		
Crumb rubber	Different size, treatment and process	x	x	x		
Oxidants	Manganese compounds	x				
Anti strip	Amines (Amidoamines/Polyamines)				x	
	Polyamides				x	
Fibers	Polypropylene	x	x	x		
	Polyester	x		x		
	Natural Cellulose	x				
[a] Permanent Deformation	[b] Fatigue cracking	[c] Low-temperature cracking				
[d] Moisture damage	[e] Oxidative ageing					

2.8 Summary

An overview of the flexible pavement design and performance has been presented in this chapter. Basic components of flexible pavement structures have been defined. State of the practice and state of the art pavement design approaches have been discussed followed by few examples of design systems based on these approaches. Some important facts about asphalt concrete mixture design and performance have also been highlighted towards the end of this chapter.

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Chapter 3 Current Pavement Situation in Pakistan

3.1 Introduction

Transportation network of any country is of vital importance to its development and affects all sectors through economic linkages. It ensures safe and timely travelling, encourages business activities and cuts down transportation costs while granting access to producers for marketing their goods and products. Pakistan is the ninth most populous country of the world with an estimated population of 167 million. It has a vast area of 803,940 **S**q. Km (about 3 times that of UK). Pakistan's economic development also depends on improvement and modernization of its transport sector accounting for 11 percent to GDP & 16 percent to fixed investment.

3.2 Road Network System in Pakistan

Road transportation network in Pakistan is not only the lifeline for the country's own economic and social development, it is also an integral part of the greater Asia road network and thus open up enormous prospects for trade, cultural exchange and peace in the entire region. Pakistan's road transport system caters to around 92 percent and 96 percent of nationwide passenger and freight traffic, respectively (NHA, 2009). The road network comprises of 259,618 kilometers (kms.), including 179,290 kms. of paved and 80,323 kms. of unpaved roads. An increase of 13 percent in road length has occurred in the last decade alone (MOF, 2010). The road density in Pakistan is currently 0.32 km/km^2. However, keeping in view the future demand and the increasing trend towards road transportation, the road density is planned to be increased to 0.45 km/km^2 by constructing 80,000 km of roads under the medium term development strategy (JICA, 2006).

The road network of Pakistan is closely knitted with the Highway systems of its neighboring countries i.e. India, China, Iran and Afghanistan. Furthermore our network provides an easy and inexpensive transit route to the Land Locked Afghanistan and a number of Central Asian States e.g. Tajikistan, Kyrgyzstan, Turkmenistan, Kazakhstan and Uzbekistan. For these very reasons, the Government of Pakistan has accorded highest priority to the development and upkeep of the country's road network. Tens of mega projects of further expansions and rehabilitation of existing network are under execution.

Out of the total road network about 11,500 kms are termed as National Highways as shown in Table 3.1 and Figure 3.1, and are being administered by an independent federal road agency, the "National Highway Authority (NHA)". The asset value of these roads is about Rupees 2.5 Trillion (US $ 3.5 Billion). These roads comprise only around 4 percent of Pakistan's total road network; however, they connect all the major cities and industrial centers around the country. The National highways carry more than 80 percent of the country's gross commercial traffic. Therefore the analysis parameters in this study primarily relates to the National Highway system of Pakistan.

3.3 Traffic Situation

In one of the latest study (JICA, 2006) the traffic volume on the country`s busiest 1800 kms intercity road from the port city of Karachi in the south to Peshawar in the north-west; bordering Afghanistan and on to the central Asian republics [historically known as grand trunk (GT) road] was found to be ranging between 7000 – 23000 vehicles per day with an annual growth rate of around 4%. Composition of cars, buses and trucks was found to be 37%, 24% and 39%, respectively. The truck classification study in Pakistan indicated that 2-axle trucks (with single rear axle on dual tires assembly) dominate the truck types with a share of about 70% in the entire truck fleet in the country (Kamal M. A. et al., 2009). More than 90% of trucks carry the pay loads far in excess of standard axle/legal loads. This is causing pre-mature failure of our roads and thus loss of precious public money.

In order to check and prevent premature failures, the Government of Pakistan has fixed a legal load limit of 118 kN on a single axle with dual tires. A limit of 828 kPa tire inflation pressure has also been fixed accordingly. However, it has been observed in various studies that even these upper limits beyond the standard axle load of 80 kN and 551 kPa tire pressure; are rarely being followed (e.g., Kamal M. A. et al., 2009, JICA, 2006, NTRC, 1995). These studies reveal that in practice, the load on this axle ranges from 98 kN to as high as 195 kN. In one of the recent study the mean axle load has been observed to be 145 kN (Kamal M. A. et al., 2009). Similarly, the mean tire inflation pressure has been observed to be 896 kPa (NTRC. 1995).

3.4 Current Condition of Highway Network

In recent years the newly constructed as well as rehabilitated pavements have shown accelerated deterioration and premature failure causing not only waste of public money but also safety hazard and inconvenience to the road users. The recent road condition survey conducted by NHA (2009), reveals that more than 60 % of the network exhibit pavement surface cracking, including more than 23 % high severity cracks (crack opening > 10 mm). The survey also indicates that more than 30 % of the network has developed permanent deformation (rutting) of various magnitudes. The Remaining Service Life (RSL) data indicate that nearly 46 % of the entire network has RSL of not more than 2 years [Refer to Figures 3.2 through 3.5].

Table 3.1 Road Administration in Pakistan

Serial No.	Type of Road	Level of Government	Road Works Organization	Controlling Ministry	Length (Kms)
1	Motorways/ Expressways	Federal	National Highway Authority	Ministry of Communication	2314
2	National Highways including Toll Roads	Federal	National Highway Authority	Ministry of Communication	9213
3	Provincial Roads	Provincial or District	Communication and Works Works and Services	Ministry of Communication Ministry of Works and Services	110000
4	District/Municipal roads, Farm to Market Roads, Cantonment Roads, Others	District	Local Govt & Rural Dev Deptt. Various City Development Authorities	Ministry of Local Government and Rural Development Department, Ministry of Urban Development	~130000

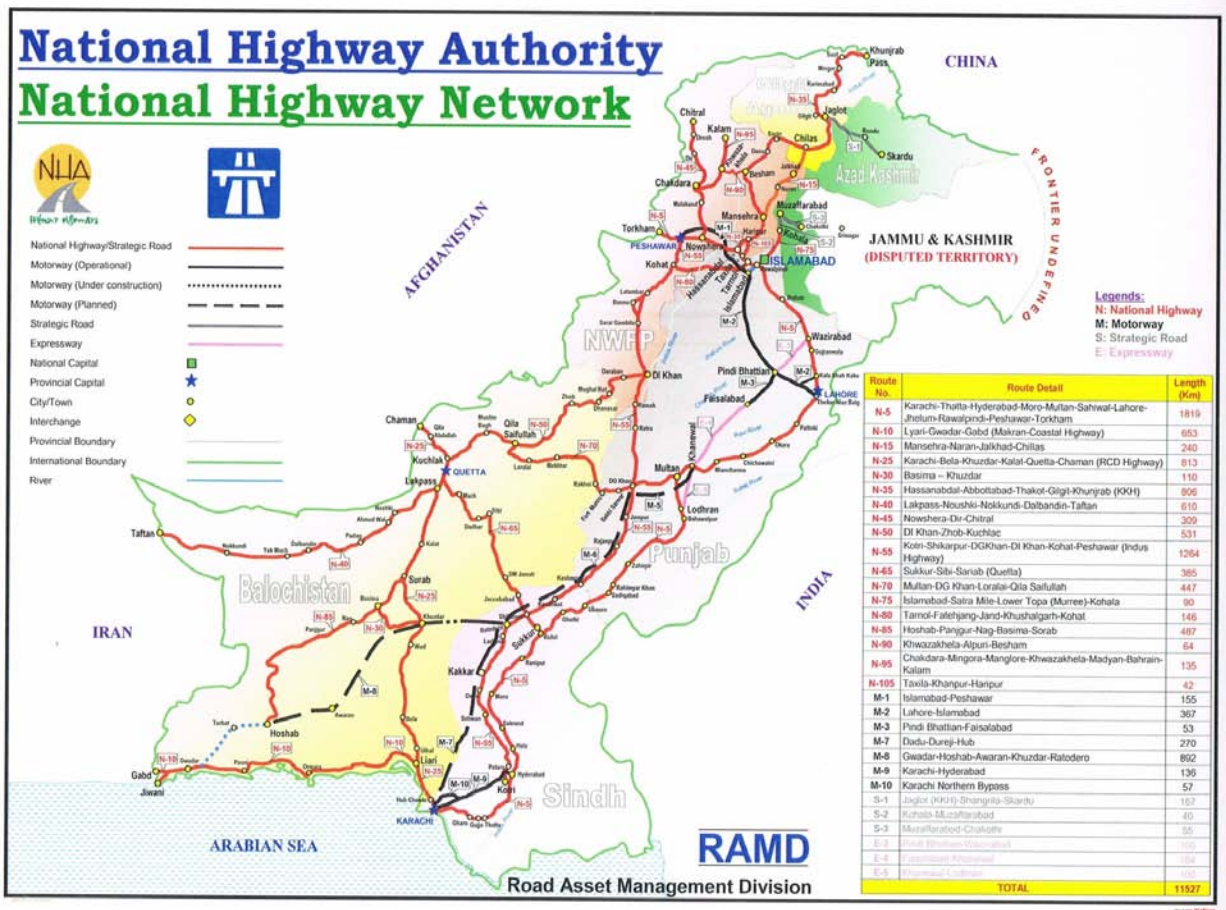

Figure 3.1 Map of Pakistan showing National Highways and Motorways

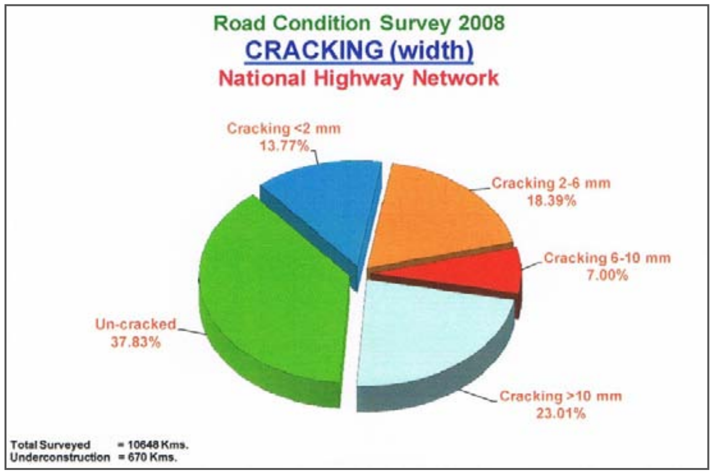

Figure 3.2 Extent of Cracking on National Highways in Pakistan (after NHA, 2009)

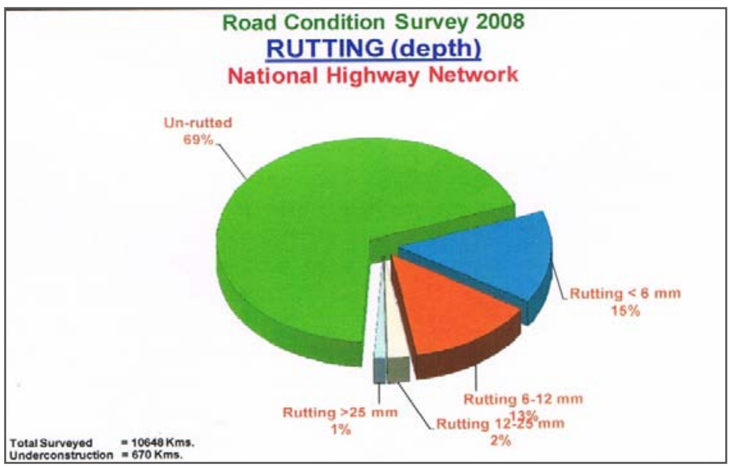

Figure 3.3 Extent of Rutting on National Highways in Pakistan (after NHA, 2009)

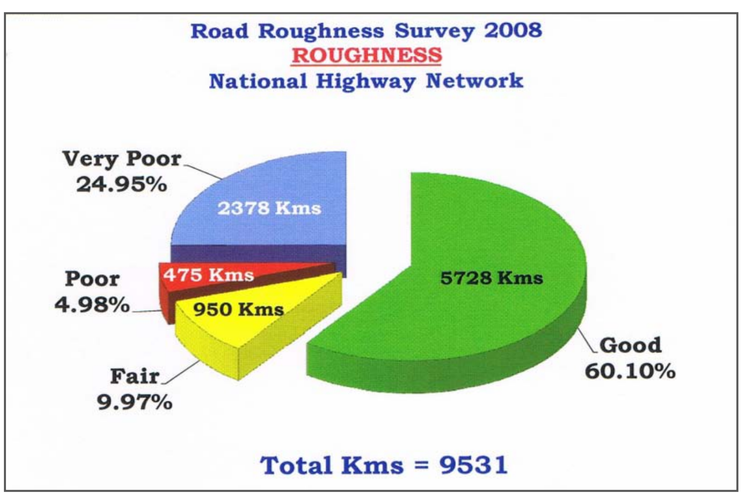

Figure 3.4 Extent of Roughness on National Highways in Pakistan (after NHA, 2009)

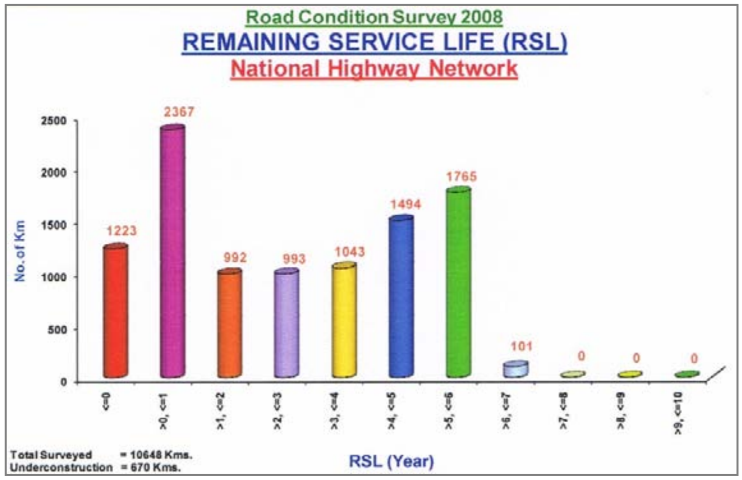

Figure 3.5 Remaining Service Life of National Highways in Pakistan (after NHA, 2009)

3.5 Current Pavement Design Practice in Pakistan

Like many other developing countries, in Pakistan the American Association of State Highway and Transportation Official (AASHTO) design guide is being followed for design of new and rehabilitated flexible pavements structures. The AASHTO design equation estimates pavement life in terms of the numbers of equivalent standard axle loads (80 kN on single axle with dual tires), (AASHTO, 1993). The AASHTO Design method is based on the empirical analysis of data from an accelerated pavement test (AASHO Road Test) conducted in Ottawa, Illinois during the late 1950s (HRB, 1962). Since the said test was conducted in a specific geo-environment with limited loading conditions and local material properties, the design equation may not produce reliable results when either the load (Traffic & environment) or the materials are changed from those used in the pavement test.

Figure 3.6 below demonstrate the comparison of the actual conditions of the AASHO road test and those which are generally experienced in the current situation. In the road test (HRB, 1962), limited pavement structural section were experimented, whereas there can be number of options for both new and rehabilitated pavement sections. The load repetitions employed during the entire two years experiment were only 1.1 million ESALs, whereas; in today’s traffic situation the number of ESALs during the design period (generally 10 years) can be well over 50 million. Similarly, the test was carried out in one specific climate (Ottawa, Illinois) and with one set of geomaterials and subgrade conditions where as in practice these factors vary from place to place. Due to these practical variabilities, the pavement are usually under designed in the current perspective of design factors (traffic, materials and environments) resulting into premature failure and thus irreparable loss to the economy.

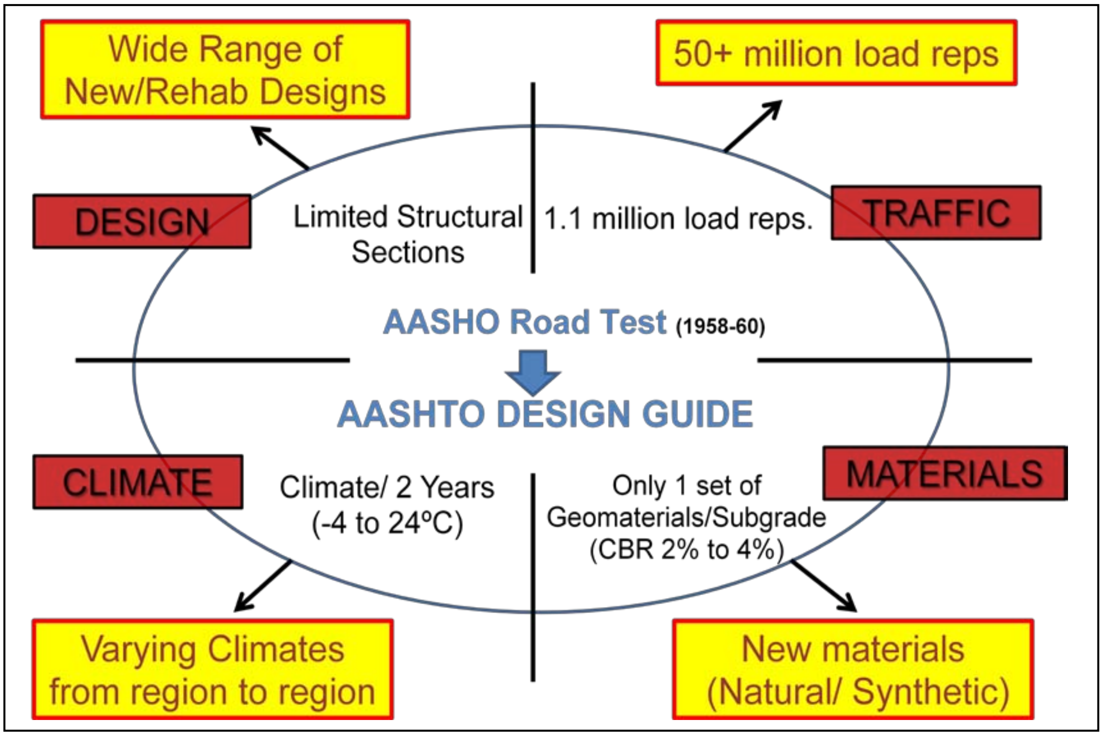

Figure 3.6 Comparison of AASHO test and Current conditions

3.6 Pavement Construction Practices in Pakistan

The construction industry in Pakistan is still in its infancy with inadequate/insufficient construction and quality control equipments coupled with lower level of technical skills. Due to poor economic conditions, adopting current technology in pavement construction is sometimes unaffordable. The pavement construction as well as maintenance is usually undertaken through competitive contracting between the executing agencies and pre-qualified local/foreign contractors. The pre-qualification and contract conditions usually require a minimum number of construction equipments and skilled manpower for road works. However, practically there are only a few local road construction contracting companies in Pakistan with appropriate equipment and skilled/unskilled manpower, and up to date quality control technologies. More so, due to the increased infrastructure development works; even these few contractors are usually over committed. To make up the deficiency, these contractors generally sublet some of the works to smaller contractors with a further lower level of technical skill base and equipment inventory. A large bulk of the construction and quality control equipment available within the country is generally second hand (used) and outdated, mostly procured from disposed off lots from completed mega projects, in and outside the country. All these factors are ultimately reflected in the overall low quality finished road pavements, susceptible to early deterioration.

3.6.1 Execution quality control of unbound pavement layers

As stated earlier, in Pakistan; the pavement designs are based on AASHTO design guide which assume certain physical properties of unbound pavement layers (including subbase and base layers), and are required to be ensured during the pavement construction by making certain quality control tests a mandatory part of the standard specifications for pavement works. The design equation estimates the structural number of pavement as a function of stiffness or elastic modulus (in psi) of each pavement layer.

However, during the construction execution the physical properties are evaluated indirectly, using the CBR test values of each layer; after being compacted to required field density. The minimum CBR of 50% and 80% at field density of 98% and 100% for unbound subbase and base layers, respectively are generally specified for execution quality control (NHA, 1998). For the field density, the sand cone or sand replacement method is generally used in Pakistan. A sample is removed by hand excavation of a pit in the finished layer. The in-situ volume of the sample is determined by measuring the volume of dry free-flowing sand in to the pit necessary to fill the pit, through a special cone. The dry density of sample is then determined in the field laboratories. This method is destructive in nature and is also time consuming thereby restricting the frequency of test points and increasing the chances of localized discrepancies in the finished layer which are reflected in localized failure of pavement; immediately after opening to traffic.

3.7 Climate of Pakistan

Due to its sub-tropical location, Pakistan has two main seasons i.e. summer and winter. The summer season of the country lasts for seven months in plain and for four months in highland, while the winter season varies for five months in plain and seven months in highland. These two main seasons of Pakistan are further sub-divided into four sub-seasons i.e. cold, hot, monsoon, and warm. The cold season varies from mid-November to mid-April, hot season from mid-April to June, and monsoon season from July to mid-September and warm season from mid-September to mid-November. The mean monthly air temperatures data derived from the 60 years` (1931-1990) historical climate data of

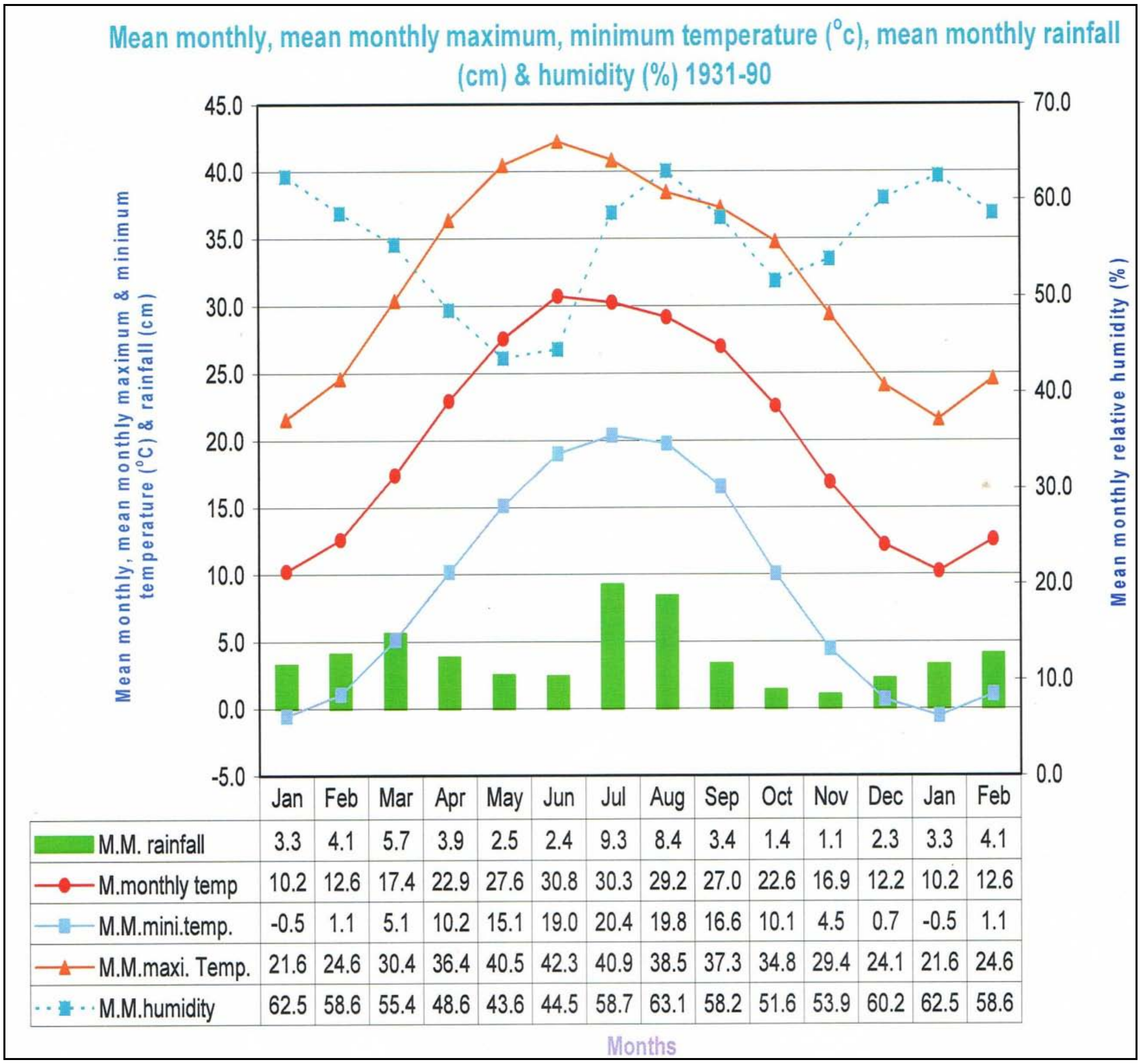

Figure 3.7 Climate classification of Pakistan (after Khan. S. U. 2010)

Pakistan; derived in a recent study by Khan. S. U. (2010) is shown in Figure 3.7.

The apparent variation in ambient temperature in Pakistan over the year and even during each month (maximum and minimum) is also one of the reasons for pavement deterioration in Pakistan; particularly the visco-elastic properties of asphalt concrete are highly temperature dependent. The empirical pavement design systems do not appropriately address seasonal temperature variations.

Summary

The road network system in Pakistan was discussed and the current situation of flexible pavements was highlighted in this chapter. Some of the reasons for premature failure of flexible pavements in Pakistan were identified to be excessive axle loading, over inflation of truck tires, the use of purely empirical methods for the pavement design which are not adapted to the local geo-environmental and loading conditions, and inadequate pavement construction and execution quality control practices.

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Chapter 4 Practical Measures for Improving Performance of Flexible Pavements in Pakistan

4.1 Introduction

The extent of public investment in the road infrastructure and its importance in the nation`s socio-economic development require engineering road pavements with ultimate care and use of state of the art technology, both for designing/constructing new pavements and maintaining/rehabilitating existing pavements. The technology includes characterization of the materials involved and the structural design of layers selected to withstand prevailing as well as future traffic and environmental conditions. It also necessitates the evaluation of in-service performance with time and the consequential economic implication to the road agency as well as the users (Papagiannakis A. T and Masad E. A., 2008). Foregoing in view, this research explores some of the practical measures which need to be adopted for improving the current situation (as highlighted in chapter 3) of flexible road pavements in Pakistan and are briefly outlined here below, followed by detailed discussion on each in the succeeding chapters.

4.2 Development of Rational Pavement Design System

As mentioned in chapter 3, in Pakistan the AASHTO design guide for design of flexible pavements is used as a standard guide. The empirical relationships suggested in the guide for estimating the expected traffic in terms of ESALs are based on the AASHO road test conducted about 50 years ago in a specific geo-environment with limited loading conditions and fixed tire pressure of 551 kPa. The equivalent load factors derived in the said test are still being used for traffic estimation in terms of design ESALs; without due consideration of the current axle loading, tire pressure, climatic conditions, geomaterials and subgrade soils which are much different from those encountered in the road test. The combined effect of non compliant empirical pavement design system and the severe loading conditions (load and environment) often result into premature deterioration of flexible pavements in Pakistan.

However, with the tremendous research and development effort in the last few decades towards the Mechanistic-Empirical (M-E) pavement design systems with better pavement material characterization, response analysis and damage prediction models; coupled with much efficient and faster computing technologies, it is now possible to investigate not only the impact of changing loading conditions, but also variabilities in the materials` properties and environmental conditions can also be investigated with much precision and greater reliability (Huang Y. H., 2004), resulting into rational and economical pavement design. Such an analysis has been conducted for the typical conditions (traffic, materials and climate) prevailing in Pakistan using the M-E pavement design model introduced by Japan Road Association (JRA). The analysis procedure and results are presented in Chapter 5 and it was shown that the results closely correspond to the actual pavement condition in Pakistan. Therefore it is considered that development of M-E design system for project level pavement designs in Pakistan is essentially required.

4.3 Characterization of unbound pavement layers

The properties of unbound pavement layers i.e. base and subbase play a significant role in the overall structural integrity and performance of flexible pavements. As discussed in chapter 2, the base and subbase are the major structural components that need to provide sufficient strength while limiting the stresses to such level which can be sustained by the foundation soil (subgrade). While the research on pavement geomaterials has largely been focused on their physical properties, the time dependent properties (i.e. the effect of ageing) on the mechanical behavior of these materials has not been thoroughly investigated. Therefore it was considered to examine the time dependent strength and deformation characteristics of unbound pavement material. As a starting point, series of triaxial compression tests on uniformly graded crushed gravel (generally used in free draining base layer) under various loading conditions were carried out. The test conditions and results are presented in chapter 6.

4.4 Development of Efficient Method for execution Quality Control Unbound Pavement Layers

Referring to section 3.6.1, the current test procedures for evaluation of stiffness characteristics and on field execution quality control of unbound pavement layers are either destructive in nature or/and time consuming. The procedures are also labour intensive and need the technical skill as well as motivation of the technicians performing the field tests, giving further rise to chances of errors. To address these issues the use of Portable FWD (P.FWD) for evaluation of stiffness of unbound pavement layers was examined by conducting physical tests on under construction pavement sites and scrutiny of earlier research on the subject. Analysis of results is presented in chapter 7 of this report. It is considered that the use of P.FWD being handy, economical and efficient equipment, can greatly enhance the construction quality control of pavement layers and the chances of localized discrepancies in the finished layers can be significantly minimized.

4.5 Development of long life Asphalt Concrete Mixture

Asphalt concrete is the most important and cost intensive component of flexible pavement system. Furthermore, since it is directly exposed to the traffic loading and climatic effects therefore utmost care is required in the design, production and construction of asphalt concrete layer/s. The finished asphalt concrete layer must exhibit the following desirable properties (Richard Robinson and Bent Thagesen, 2004):

- High stiffness to reduce the stresses transmitted to the underlying layer/s
- High resistance to deformation(Stability)
- High resistance to Fatigue
- High resistance to weathering (Durability)
- Low permeability to prevent ingress of air and water
- Sufficient surface texture to provide adequate skid resistance in wet weather

Under normal/controlled traffic loading and moderate climatic conditions, the above characteristics are fairly easily achieved; even with the conventional asphalt binders. However, the mix requirements become more critical in conditions such as prevailing in Pakistan; with extreme climate (higher temperature ranges) and severe axle loading/tire inflation pressure, as observed in chapter 3 and 5. In order to respond to the mentioned severe conditions, the development of a robust and highly stable asphalt concrete mixture was attempted for this study. The mixture properties were evaluated both in the laboratory and in the field (test pavement section). Results and recommendations are reported in chapter 8.

4.6 Summary

In response to the current pavement situation in Pakistan; as highlighted in the preceding chapter, including the current condition of pavement network; the axle loading and climatic condition; the current design and construction practices etc, a few practical measure have been suggested in this chapter. Systematic evaluations of these measures are reported in the following chapters.

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Chapter 5 Applicability of Mechanistic-Empirical Design System for Flexible Pavement in Pakistan

5.1 Introduction

A good road transport system is absolutely crucial to the economic as well as social uplift of developing nations. Road transport not only plays a pivotal role in cost effective transportation of freight and passengers, it provide equal opportunities of access to jobs and trade to all segments of society, thus promoting national cohesion and alleviation of poverty; which are the issues faced by almost all the developing nations around the world. Realizing the importance of road transport, a major portion of the national income of such countries is spent on construction and maintenance of roads (R. Robinson and B. Thagesen, 2004). Consequently, there has been a phenomenal shift towards road transportation options in the past few decades. For example in Pakistan the ratio of road verses rail transportation was 20:80 in 1950s which has drastically changed over time and now stands at 88:12. However, in Pakistan, the economic and social benefits of an efficient road transport system are seldom realized due to accelerated deterioration and premature pavement failure. Some of the primary causes of premature failure can be attributed to excessive axle loading and over inflation of truck tires coupled with inappropriate design inputs; including climatic consideration, in empirical pavement design methods being used by pavement designers in Pakistan.

The study presented in this chapter attempts to capture the damaging influence of excessive axle loads, tire inflation pressure and seasonal variation in climatic condition on the performance of flexible pavements, using the data from Pakistan as a case study. The Mechanistic-Empirical (M-E) design framework has been used for this purpose. The primary objective is to suggest practical proposals for developing M-E pavement design system in Pakistan.

5.2 Design Approach

Damage impact analysis due to increasing axle load, tire inflation pressure and seasonal variation in ambient temperature on pavement structure; typically constructed by the National Highway Authority (NHA) for the traffic conditions corresponding to National Highways with 10 years design life, has been conducted using the M-E design framework in this study.

In the M-E approach, flexible pavements are modeled as layered elastic systems with infinite lateral dimensions, resting on an elastic subgrade layer of infinite depth. Elastic theory implies that each of the pavement layers and the subgrade can be described by their corresponding Elastic Modulus "E" and Poisson's ratio "μ". Each layer is assumed to be homogeneous and isotropic. Tire loads are commonly assumed as circular loads of uniformly distributed vertical stress, equal to tire inflation pressure. The radius of the circular load is given by the following equation (Papagiannakis A. T and Masad E. A., 2008):

$$a = \sqrt{p/_{i\,\pi}} \tag{5.1}$$

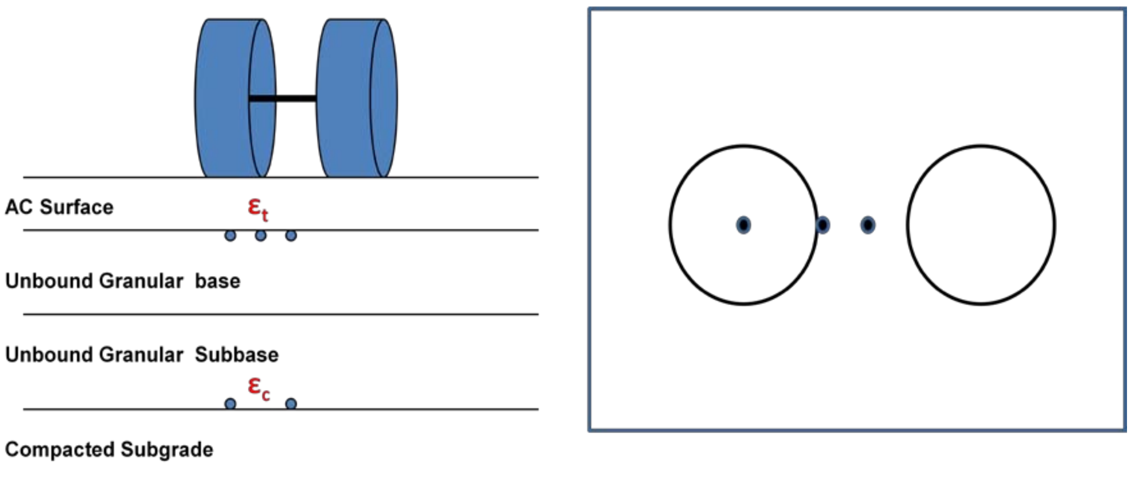
Where a is contact radius in meter (m), p is the vertical load in kN and i is the tire inflation pressure in kN/m^2.

Critical pavement responses e.g., stress, strains and deflections are calculated using theory of elasticity. Analyzing these responses is the mechanistic element of the M-E design system. The empirical component of M-E design constitute relating these critical responses to observable and measurable pavement distresses e.g. pavement surface cracking, permanent deformation or rutting, roughness etc., using distress functions calibrated to local conditions. The analysis parameters employed in this study are discussed in more detail here bellow.

5.2.1 Pavement Response model

A recently developed analysis tool, for layered elastic system to assist in M-E design of pavement structures in Japan has been used for this study. The tool named as General Analysis of Multi-layered Elastic System (GAMES) provide an excellent combination of analysis features and computation speed for linear pavement systems and has been found to produce comparable results with a range of other analysis softwares which are used worldwide (Maina J. W. et. al. 2008, JSCE, 2005). The graphical presentation of results makes it rather more user friendly (Matsui K. et. al., 2007).

Critical responses analyzed in this study were horizontal tensile strain (**ε_t**) at the bottom of the Asphalt concrete layer and vertical compressive strain (**ε_c**) at the top of the subgrade layer as shown in Figure 5.1. These responses were computed for various loading conditions using GAMES. Strains at each point (dots in the Figure 5.1) were computed and the maximum strain responses were later used as inputs in the distress models; also known as transfer functions for damage analysis.



(a) Pavement Section under the Wheel (b) Plan view of the critical locations

Figure 5.1 Locations of critical strains under one wheel (with dual tires) of a Single Axle

5.2.2 Pavement distress models

Number of load repetition to failure (N_f) due to maximum ε_t corresponding to fatigue fracture or bottom up cracking failure and ε_c corresponding to permanent deformation or rutting failure, were computed for a range of loading conditions using the distress functions, proposed by Japan Road Association (JRA). The JRA distress models are basically modified and calibrated version of Asphalt Institute (AI) models (AI, 1999). Many different versions of (AI) models were extensively used in the past for the pavement performance studies. However, capitalizing on the recent advancement in the field of flexible pavement engineering in Japan, the current models suggested by JRA have been used for this study. The JRA distress model for Asphalt concrete (AC) fatigue life also considers the AC mix properties and layer thickness thus producing realistic results. Prior to using the models for extensive study, they were initially tested for the standard loading conditions with the material and climatic conditions of Pakistan and the results were found to be comparable to earlier studies (Huang Y. H., 2004). Moreover, the overall results shown in the later part of this chapter were also found to be in conformity with the actual pavement deterioration trends in Pakistan as highlighted in chapter 3 earlier, which suggest that the models are generally fit to be used for the Pakistani conditions.

In the M-E design system proposed by JRA, a pavement section is considered to be failed when either 20% of lane area is cracked and/or the pavement structure exhibit 15 mm of rut depth (JRA, 2001).

The model to predict the allowable number of load repetition to fatigue cracking (N_{ff}) is a function of tensile strain ($\boldsymbol{\varepsilon_t}$) and asphalt concrete mix stiffness (modulus) "*E*" and is presented here bellow:

$$\boldsymbol{N_{ff}} = \beta a1 \cdot C \cdot 6.617 \cdot 10^{-5} \cdot \left(\frac{1}{\varepsilon_t}\right)^{3.291 \cdot \beta a2} \left(\frac{1}{E}\right)^{0.854 \cdot \beta a3} \tag{5.2}$$

Where;
$$C = 10^{M} \tag{5.3}$$

$$M = 4.84\left[\frac{VFA}{100} - 0.69\right] \tag{5.4}$$

Where; VFA is Volume of voids filled with Asphalt binder (%).

A typical VFA value of 73 %; as suggested by NHA General Specifications (1998) has been used in this analysis.
$\beta a1$, $\beta a2$ and $\beta a3$ are calibration constants.

$$\beta a1 = Ka \cdot \beta a1' \tag{5.5}$$

"*Ka*" is a correction coefficient for asphalt concrete layer thicknesses (h_{ac}) and is expressed as:

$$Ka = \frac{1}{8.27\text{x}10^{-0.11} + 7.83 \cdot e^{-0.11(hac)}} \tag{5.6}$$

The values of $\beta a1'$, $\beta a2$ and $\beta a3$ are given as 5.229x10^4, 1.344 and 3.018, respectively.

The model to predict the allowable number of load repetition to permanent deformation or rutting (N_{fd}) is a function of vertical compressive strain (ε_c) at the top of the subgrade and is expressed as:

$$N_{fd} = \beta s1 \cdot (1.365\text{x}10^{-9} \cdot \epsilon_c^{-4.477*\beta s2}) \tag{5.7}$$

Where; $\beta s1$ and $\beta s2$ are calibration constants, its values are 2.134 x 10^4 and 0.819, respectively.

5.2.3 Pavement Layer Properties

Table 5.1 shows the properties of each layer, assumed for this analysis. The pavement section represents typical design thicknesses and material properties (strong layers) utilized in Pakistan for the National Highway design traffic of about 25 million Equivalent Standard Axle Loads (ESALs), in accordance with AASHTO (1993) pavement design method. The elastic moduli of pavement materials have been worked out from the typical California Bearing Ratio (CBR) test values, which is the common test being performed extensively in Pakistan. The correlations proposed by AASHTO design guide have been used for this purpose.

5.2.4 Axle Load and Tire Inflation Pressure

One of the primary targets of this study was to assess the damaging impact of Legal axle load and the observed excess axle load; as highlighted in chapter 3. However, in order to make this study more useful for pavement designers and road administration agencies; for rational and economical design of flexible pavements, the analysis variables were enhanced to include a wide range of load and tire pressures as shown in Table 5.2.

5.2.5 Seasonal variations in Climate

One of the major advantages of the M-E design approach is the consideration of climatic condition directly into the design process. The interaction of climatic factors with pavement materials significantly affects the performance of flexible pavements. Particularly, the sensitivity of asphalt concrete mix stiffness (modulus) to temperature variations has been well recognized (e.g., AASHTO, 2002). Mean monthly air temperatures data, derived from the 60 years` (1931-1990) historical climate data of

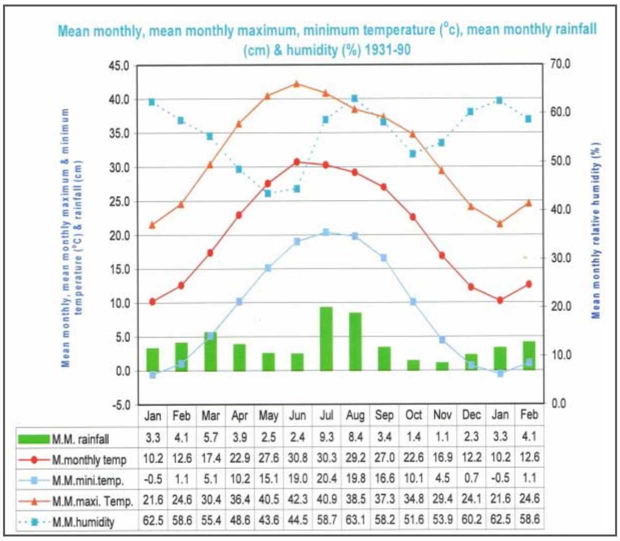
Pakistan; as shown in Figure 5.2 (a) and (b); established in a recent study by Khan. S. U. (2010) has been used in this analysis.

Table 5.1 Pavement Layer Properties

Layer	Thickness (cm)	Modulus (MPa)	Poisson Ratio
AC Surface	5	225 – 6850 (depending on pavement temperature)	0.35
AC Base	10		0.35
Aggregate Base	25	Strong layer = 193 Weak layer = 120	0.40
Granular Subbase	35	Strong layer = 117 Weak layer = 70	0.40
Subgrade	-	Strong layer = 72 Weak layer = 31	0.45

Table 5.2 load and tire pressure conditions

Analysis Conditions	Range
Axle load	58 kN – 176 kN
Tire inflation pressure	551 kPa – 965 kPa



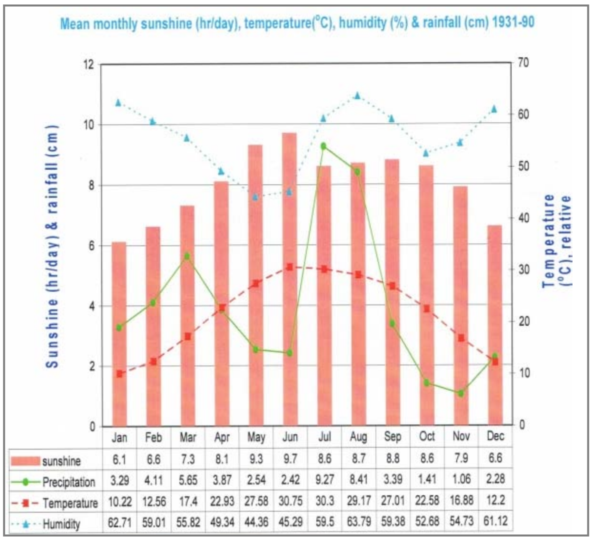


Figure 5.2 (a) Climate of Pakistan (b) Mean monthly Sunshine in Pakistan (after Khan. S. U., 2010)

The mean monthly pavement temperatures were obtained using the following relation which was originally presented by AI and also adopted by JRA:

$$Mp = Ma\left[1 + \frac{2.54}{z+10.16}\right] - \left[\frac{25.4}{9(z+10.16)}\right] + \frac{10}{3} \tag{5.8}$$

Where;

Mp: Mean monthly Pavement Temperature (^{o}C)

Ma: Mean monthly air Temperature (^{o}C)

z: Depth below the surface (cm).

The temperature at the upper third point of the AC Base layer has been used in this analysis (Huang Y. H., 2004).

A number of AC Temperature-Stiffness (moduli) correlations are available in literature (e.g., Scott Wilson, 1998, Bubushait et. al., 1985, Croney D. and Croney P., 1998). For this study the stiffness values were worked out using the correlation established by Scott Wilson pavement engineering, Nottingham, UK. Table 5.3 shows the mean monthly air and mean monthly pavement temperatures and their corresponding AC moduli.

Table 5.3 AC stiffness (moduli) corresponding to mean monthly temperatures in Pakistan

	Jan	**Feb**	**Mar**	**Apr**	**May**	**Jun**	**Jul**	**Aug**	**Sep**	**Oct**	**Nov**	**Dec**
Mean Monthly air Temp(^{o}C)	10	13	17	23	28	31	30	29	27	23	12	10
Mean Monthly Pavement Temp(^{o}C)	15	18	22	29	35	38	37	36	34	29	17	15
Modulus of AC Layer (MPa)	6850	5290	3560	1450	480	225	300	380	660	1450	5790	6850

5.3 Analysis of results

The impacts of variation in axle loading, tire inflation pressure, climate, and properties of pavement materials analyzed in this study have been presented in terms of Relative Damage Factors (RDFs). RDF can be expressed as the damage caused to a pavement structure, by a set of input conditions (i.e. Axle loading, tire inflation pressure, material properties and climate) relative to the damage produced by Standard conditions. In this case the standard load and tire pressure was assumed to be the same as suggested by

AASHTO guide (i.e. 80 kN load on single axle with a set of dual tires, inflated to 551 kPa each).

The RDFs, corresponding to fatigue cracking and permanent deformation or rutting can be expressed as:

$$\mathrm{RDF}_f \quad = \frac{N_{ffs}}{N_{ffi}} \tag{5.9}$$

and;

$$\mathrm{RDF}_d \quad = \frac{N_{fds}}{N_{fdi}} \tag{5.10}$$

Where; RDF_f : Relative damage factor for fatigue cracking, RDF_d : Relative damage factor for rutting. N_{ffs} and N_{fds} are number of repetition for fatigue cracking and rutting under standard axle loading conditions; respectively. N_{ffi} and N_{fdi} are allowable numbers of repetition for fatigue cracking and rutting under any arbitrary loading conditions, respectively. The RDF resulting in larger value among the two is selected as Design RDF.

5.3.1 Effect of axle load and tire inflation pressure

The result show that the performance of flexible pavement is sensitive to not only axle load but also to tire inflation pressure as presented in Figure 5.3 - Figure 5.6. Figure 5.3 shows the effect of increasing axle loading and tire inflation pressure (Tp) on Asphalt Concrete (AC) Fatigue life with Mean Monthly Pavement Temperature (MMT). It was observed that the RDF_f`s for axle loads between 60 – 90 kN, under all Tp range remained under 2.5. However, the RDF_f sharply increases beyond the 90 kN axle loads.

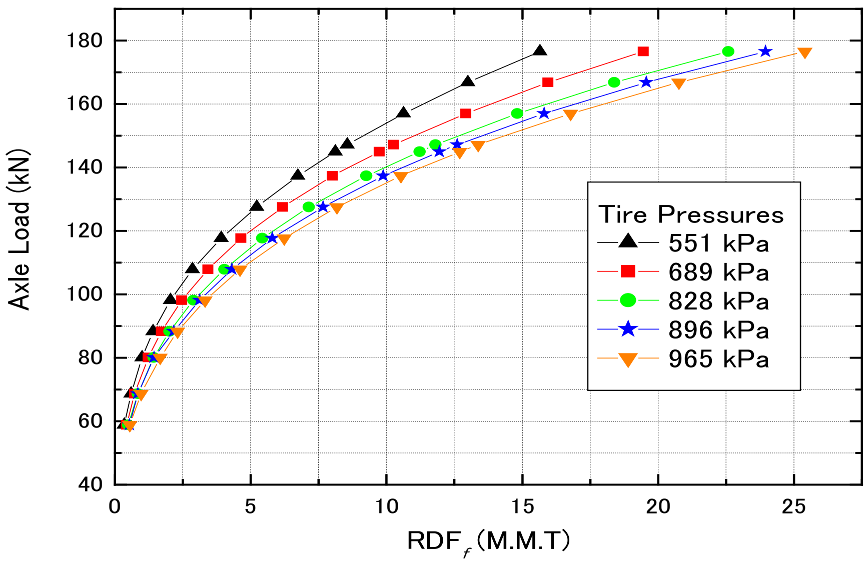


Figure 5.3 Relative Damage Factors for Fatigue Failure (RDF_f)

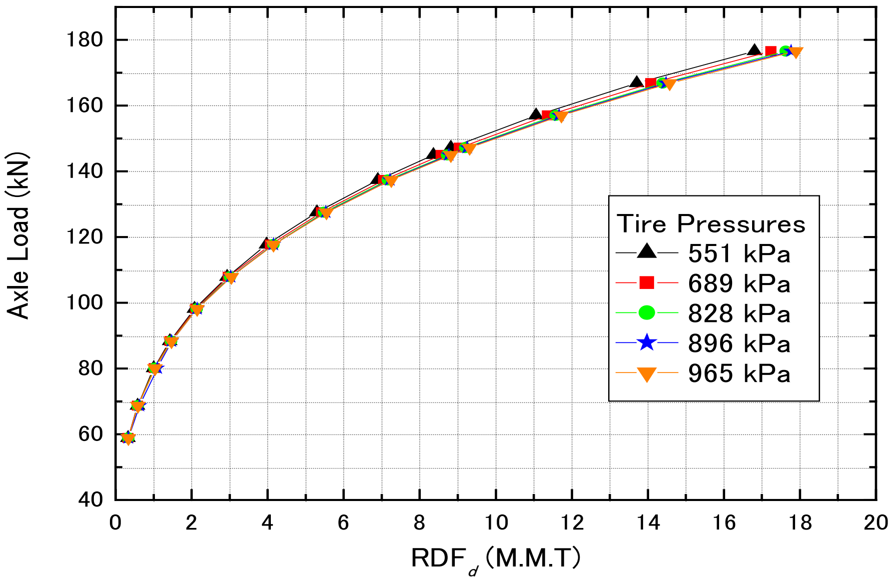


Figure 5.4 Relative Damage Factors for Rutting Failure (RDF$_d$)

This Figure also shows that for any fixed axle load, increasing the Tp resulted in increased RDF$_f$ s. This shows that fatigue performance is highly sensitive to Tp. The phenomenon can be observed even at the standard axle load (80 kN), where the RDF$_f$ increases from 1.0 to 1.66. However, the damaging impact widens for higher axle loads, e.g. at the Legal axle load (118 kN) and the Observed axle load (145 kN), the RDF$_f$ increases by 2.3 and 4.6 points, respectively (for Tp ranging from 551 kPa – 965 kPa).

Figure 5.4 shows that while the Tp increase has minimal effect on rutting performance, the axle load increase has significant effect. This Figure also indicates that for the axle load ranging between 60 – 98 kN, the RDF$_d$ remained around 2.0. However, it keeps on increasing until a maximum value of 17.0 at the axle load of 176 kN. The difference in RDF$_d$`s for each individual load increment remained under 1.0, for all Tp`s, which substantiate the observation that Tp has minimal impact on the rutting performance. A comparison of RDF$_f$`s and RDF$_d$`s at three identical Tps is presented in Figure 5.5.

Figure 5.6 shows the Design RDFs for all loads and Tp ranges. The Design RDFs reveals that pavement performance in Pakistan is more sensitive to Fatigue failure compared to Rutting failure. This result, rather substantiate the actual in-service condition of pavements in Pakistan, as revealed by the pavement condition surveys conducted by NHA observed in Chapter 3. Figure 5.7 summarizes the effect of axle load and Tp on the design life of the pavement structure. The effect on pavement life expectancy due to tire pressure can be observed even at the standard axle load of 80 kN, where the pavement life drops from 10 to 6 years when the Tp was increased from 551 kPa – 965 kPa. At the legal and observed axle loads, the pavement life may be expected to be between 1.5 to 3 years and between 0.8 to 1.2 years, respectively; depending on Tp.

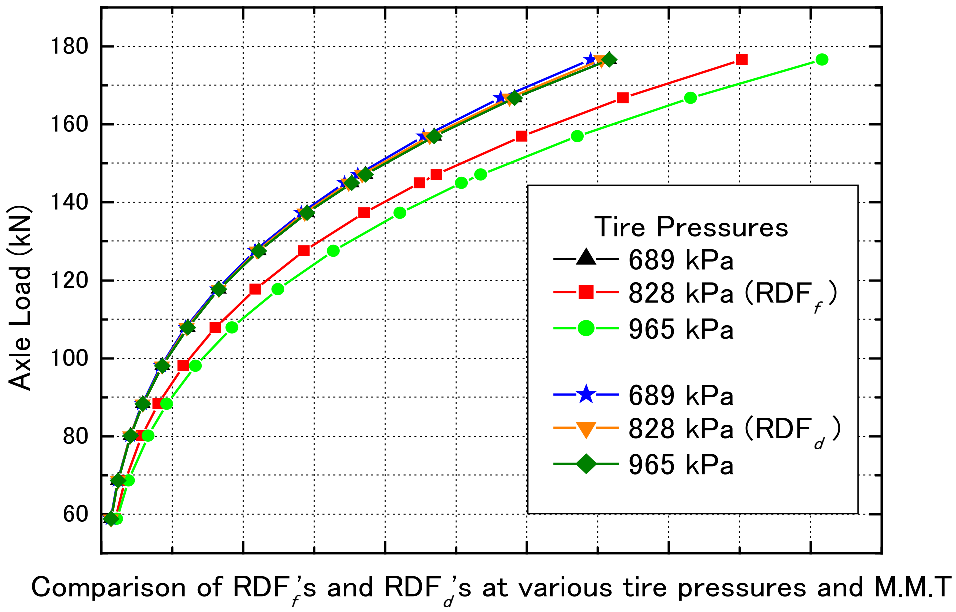


Figure 5.5 Comparison of RDF$_{(f\,\&\,d)}$ at various Tire pressures

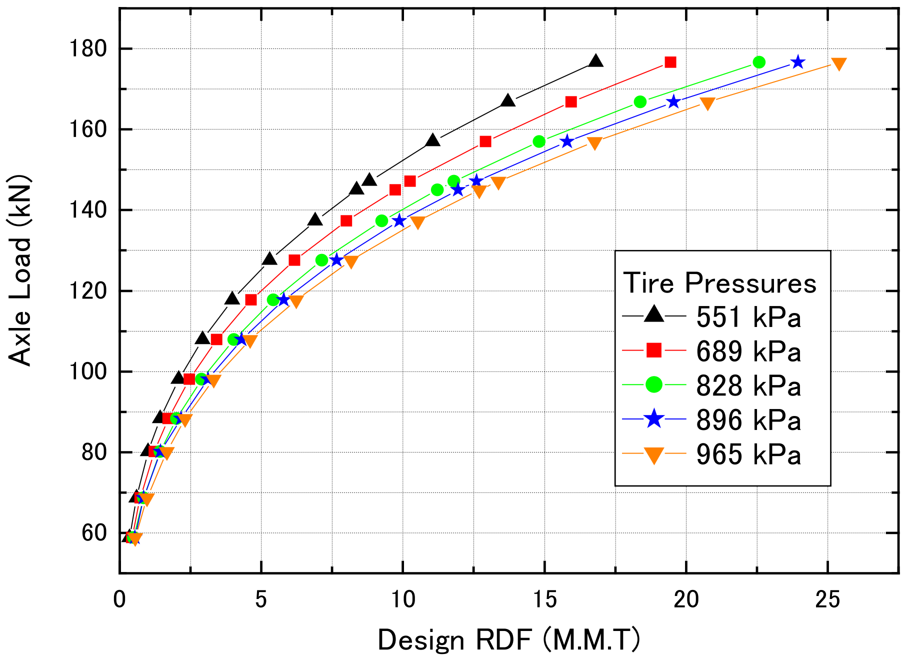


Figure 5.6 Representative Design RDFs at various Tire pressures

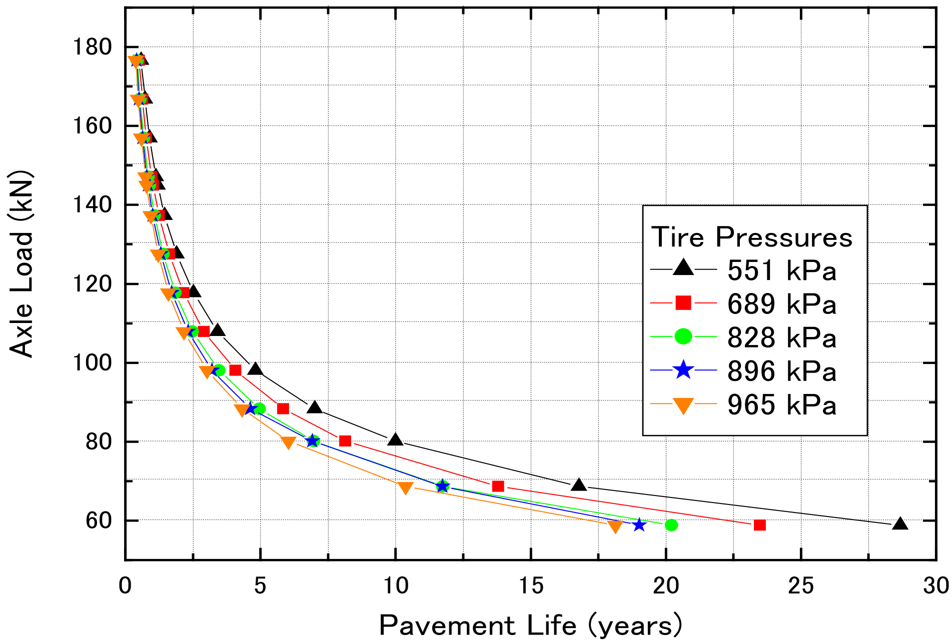


Figure 5.7 Effect of axle load and Tire pressure on pavement design life

5.3.2 Climatic consideration

Figure 5.8 shows the sensitivity of flexible pavement performance to climatic considerations. This Figure shows the effects on Design RDFs due to changes in pavement temperature regime. The results show that the RDFs are much conservative when the Mean Monthly Pavement Temperatures (MMT) and corresponding layer properties are considered in the design. Conversely, the results could be misleading when a single annual temperature (e.g. Mean Annual Temperature-MAT) is selected for the design purpose; as in the case of purely empirical design methods, resulting into overdesign and hence unnecessary waist of public money.

Figure 5.8 also shows the damaging impact of the two extreme seasons i.e. summer and winter, analyzed in this study by considering Mean Summer Monthly Temperature (MST) and Mean Winter Monthly Temperature (MWT), respectively. It shows that the damage impact is much more during the winter season compared to summer. This is however, now understandable in light of the observation noted in section 5.3.1 above, that the pavement performance in Pakistan is governed by the AC fatigue failure. In the winter months, the combined effect of increased AC stiffness (making it brittle) and the repeated application of excessive axle loading coupled with increased Tp, enhance its susceptibility to damage, resulting in to increased surface cracking. Conversely, during the summer months; damage phenomenon shifts to rutting failure and all the rutting occurs during this season. It was also observed that the MST curve runs very close to MMT curve (which was earlier shown to be representing the AC fatigue failure as a general failure mode in Pakistan, refer Figure 5.6). This shows that, while the pavements in Pakistan are generally susceptible to AC fatigue failure; they are also potentially exposed to rutting failure (during summer) due to excessive axle loading.

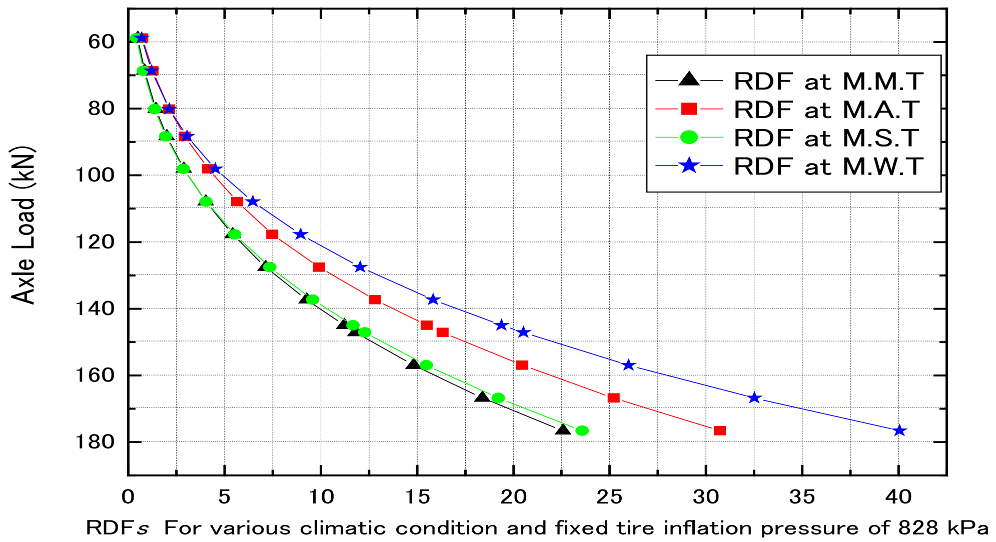


Figure 5.8 Effect of Climate on pavement performance

5.3.3 Effect of stiffness of geomaterials and subgrade

Figure 5.9 and Figure 5.10 show the impact of stiffness of geo-material and subgrade on AC fatigue and pavement rutting performance, respectively. It was observed that AC fatigue performance is significantly affected by the stiffness of unbound base course and to a lesser extent by subbase layer`s stiffness (Figure 5.9). On the other hand, Figure 5.10 reveals that the pavement rutting performance is almost entirely dependent on the stiffness of subgrade. It may be observed that for weak subgrade, the RDF_d raises to a huge value of 88.32 at 176.58 kN axle load and 828 kPa Tp. Figure 5.11 shows the design RDFs at 828 kPa (legal Tp) under various stiffness of geo-materials.

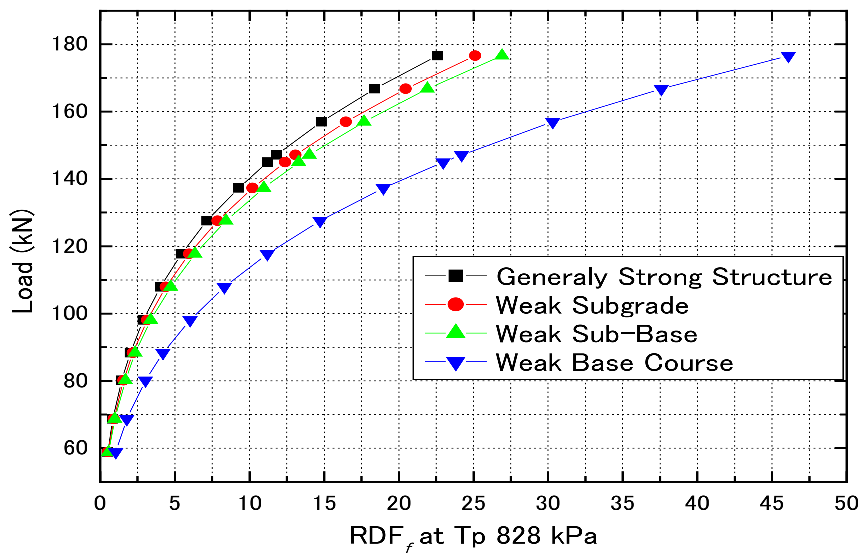


Figure 5.9 Effect of geo-materials stiffness on AC pavement fatigue performance

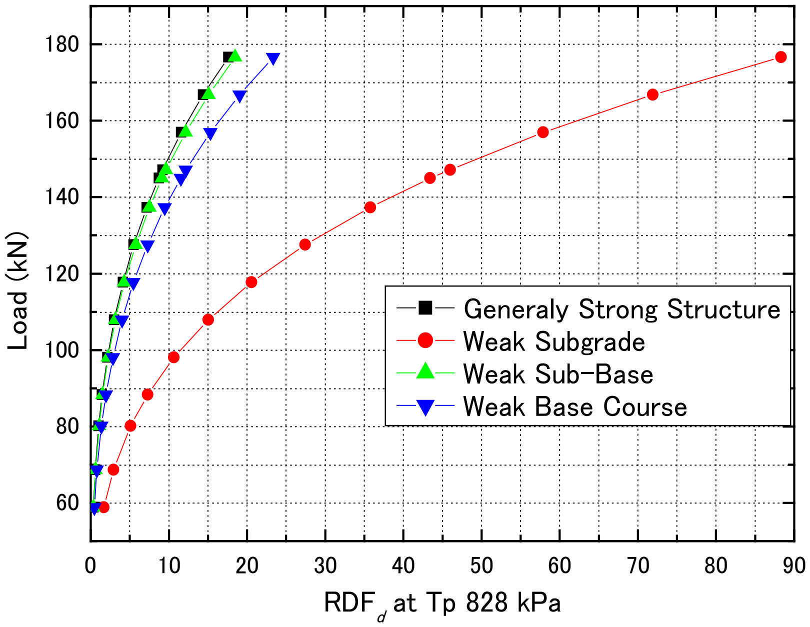


Figure 5.10 Effect geo-materials stiffness on pavement rutting performance (Tp 828 kPa)

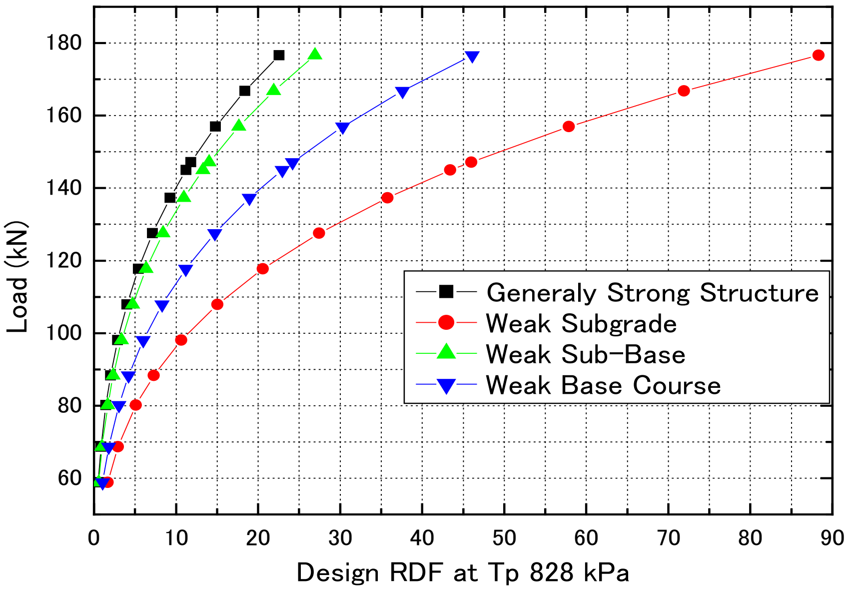


Figure 5.11 Design RDFs under various geo-material conditions (Tp 828 kPa)

5.3.4 Proposed Design RDFs

Figure 5.12 presents the RDF curve being proposed for the representative climatic conditions (MMT) and the legally allowed tire pressure in Pakistan, arrived at as a result of this study.

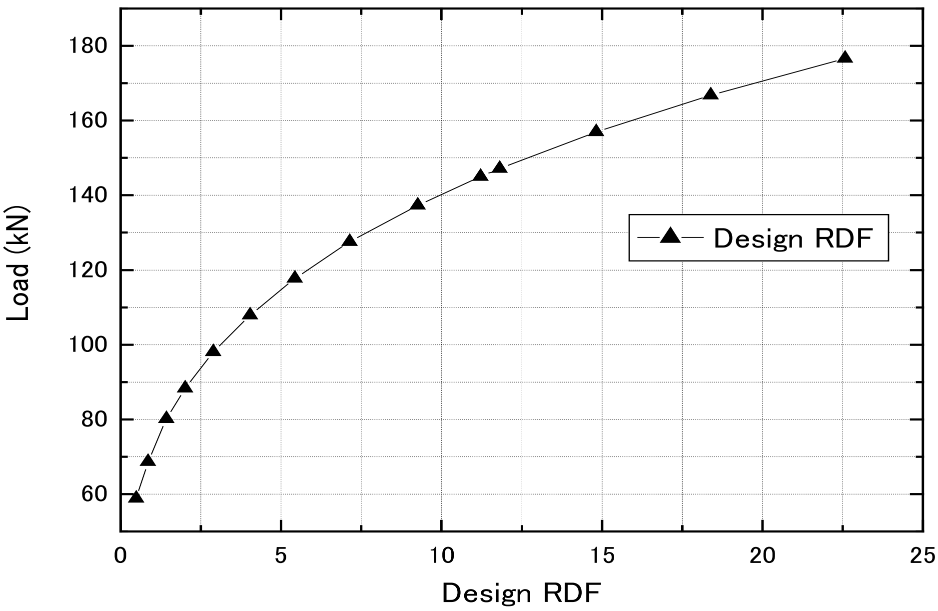


Figure 5.12 Proposed Design RDFs for Pakistan (Tp-828 kPa and MMT)

Figure 5.13 shows the damage ratio (Dr) for each axle load corresponding to the suggested RDF. Dr is defined as the damage caused by a single application of load (JRA 2001). These curves can be readily used with confidence for the network level pavement management purposes, when the detail axle load data is not available and/or cost intensive. However, the design process itself can also be used for rational project level designs with specific project related field data for traffic, climate and, geomaterials and subgrade soils.

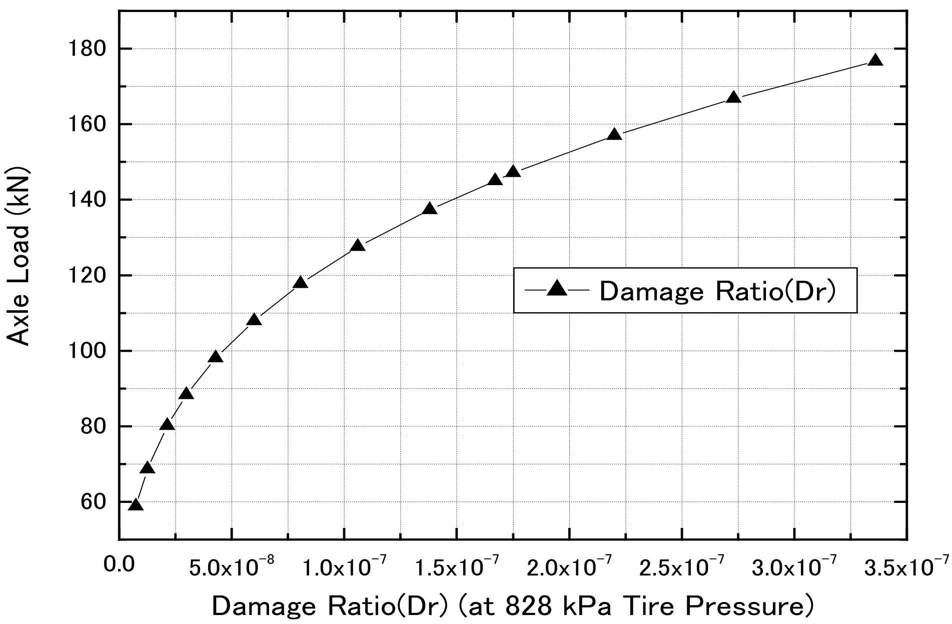


Figure 5.13 Damage Ratios (Tp-828 kPa and MMT)

5.3.5 Practical Implications

As discussed in chapter 3, currently many developing countries including Pakistan are using the AASHTO design guide for design of flexible pavements. The empirical relationships (equations 2.1 – 2.3c, Chapter 2) suggested in the guide for estimating the expected traffic in terms of ESALs is based on the AASHO road test conducted about 50 years ago in a specific geo-environment with limited loading conditions and fixed tire pressure of 551 kPa. The equivalent load factors derived in the said test are still being used without the consideration of the current axle loading, tire pressure, climatic conditions, geomaterials and subgrade soils much different from those encountered in the road test. Since the load factors (identified in this study as RDFs) are based on representative conditions and are derived using the rational M-E design approach, these factors will result into economical and reliable pavement structures.

These factors can be used even with the current empirical design methods, until the economies of the developing countries in general and Pakistan in particular permit to invest in more sophisticated and cost intensive data accumulation and analysis techniques related to traffic (for example using axle load spectra instead of ESALs), climate (for example Enhanced Integrated Climatic Models), improved laboratory characterization of geomaterials etc. A comparison of the damage factors derived in this study and those estimated using AASHTO equations is shown in table 5.4. It can be noted that using the AASHTO factors (which is the general case in Pakistan) for estimating the expected numbers of ESALs will result in an under designed pavement

Table 5.4 Comparison of RDFs with AASHTO load factors

Load (kN)	AASHTO Range (SN:5, Pt:2.5 & Tp 551 kPa)	RDF at Tp 551 kPa	RDF at Tp 689 kPa	RDF at Tp 828 kPa (***Allowed***)	RDF at Tp 896 kPa	RDF at Tp 965 kPa (***Observed***)
58.86	0.26	0.35	0.43	0.49	0.53	0.79
68.67	0.48	0.60	0.72	0.85	0.85	1.37
80.11	1.00	1.00	1.23	1.43	1.44	2.31
88.29	1.51	1.43	1.71	2.01	2.16	3.20
98.10	2.18	2.08	2.46	2.89	3.12	4.55
107.91	3.03	2.93	3.43	4.03	4.31	6.17
117.72	4.09	3.97	4.64	5.43	5.80	8.12
127.53	5.39	5.30	6.17	7.14	7.66	10.58
137.34	6.97	6.90	8.01	9.26	9.88	13.71
147.15	9.98	8.82	10.26	11.81	12.60	17.48
156.96	12.50	11.06	12.92	14.82	15.80	21.92
166.77	15.50	13.71	15.95	18.39	19.56	27.03
176.58	20.41	16.81	19.45	22.58	23.95	32.99

structure susceptible to premature failure when exposed to actual in-service conditions explained earlier. The highlighted rows from top to bottom in the table show comparative factors under standard, legal and observed axle loads in Pakistan, respectively.

5.4 Summary

In this chapter the damaging influences of excessive axle loading, tire inflation pressure, seasonal variation in climatic condition, and properties of pavement geomaterials and subgrade on the performance of flexible pavement were analyzed by using the data from Pakistan as a case study. The Mechanistic-Empirical (M-E) design framework; based on GAMES (General Analysis of Multi-layered Elastic Systems) was used for the analysis. Results were presented in terms of Relative Damage Factors (RDF). Conclusions drawn from the analysis may be summarized as follows:

1) The Mechanistic-Empirical (M-E) Pavement design approach, realistically capture the variations in all factors that influence the pavement performance, resulting in rational pavement design.
2) The performance of flexible pavement is sensitive to not only axle loading but also significantly to tire inflation pressure. The damaging influence of increase in tire pressure keeps on magnifying with each axle load increment.
3) The Design RDF for the Legal axle load (118 kN) and the mean observed axle load (145kN) on single axle with dual tires; with mean observed tire pressure of 896 kPa, was noted to be 5.80 and 11.95, respectively.
4) The flexible pavement performance against distresses such as fatigue cracking and rutting is significantly affected by the stiffness of unbound base course and subgrade, respectively.
5) Consideration of climate regime in the design affects the economy of pavement design. Considering a single value for air/pavement temperature, e.g. mean annual temperature or extreme temperature condition, may result in extravagant pavement design.
6) The damage factors derived in this study can be readily used for network level pavement management when the detail axle load data is time and/or cost intensive.

5.5 Future Research

Based on the results of this study, the following is suggested for future research:

1. Damage analysis due to excessive loading and tire inflation pressure on multi-axle trucks is also required for a complete picture of mixed heavy traffic impact on flexible pavements in Pakistan.
2. The generalized transfer functions suggested in this study need to be carefully calibrated to country specific conditions (traffic, materials and climate) through

long term performance studies for developing M-E design systems for project level designs of flexible pavements in Pakistan.

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Chapter 6 Laboratory Testing for Time Dependent Mechanical Properties of Unbound Granular Material (UGM)

6.1 Introduction

Unbound granular material (UGM) such as uniformly-graded crushed gravel is generally used as base material for railroad tracks; free draining base course material for road pavements and as backfill material of abutments. Deterioration such as differential settlement is usually observed in these structures due to the lateral plastic flow and/or abrasion of the gravels under the cyclic load of the moving traffic. This deterioration is not only a cause of concern for the periodic maintenance and thus associated enormous costs, but also impacts on the riding comfort and safety of the passengers. However, the strength and deformation characteristics of uniformly-graded crushed gravel have not been thoroughly studied under various conditions. Therefore it is essentially required to examine its strength and deformation characteristics under all possible conditions, to be able to suggest adequate measures for the efficient and cost effective maintenance of railroad system. Much of the studies conducted in the past are related to the impact of grain size distribution on the strength and deformation of geomaterial (Kohata et al, 2007); (Kohata & Takeda, 2008). The time-dependency effects on strength and deformation characteristics of these materials have not been studied in-depth.

Time-dependency in the context of geo-materials refer to the effects caused by two factors: *i) Ageing effect* (cementation and weathering) i.e. changes with time in the intrinsic properties like strength, stiffness, deformation etc. due to change in interface and/or internal particle properties caused by physico-chemical processes and *ii) Loading rate effect* (viscous effect) i.e. viscous sliding at inter-particle contact points. Figure 6.1 and Figure 6.2 show the stress-strain relationship due to above outlined two effects respectively.

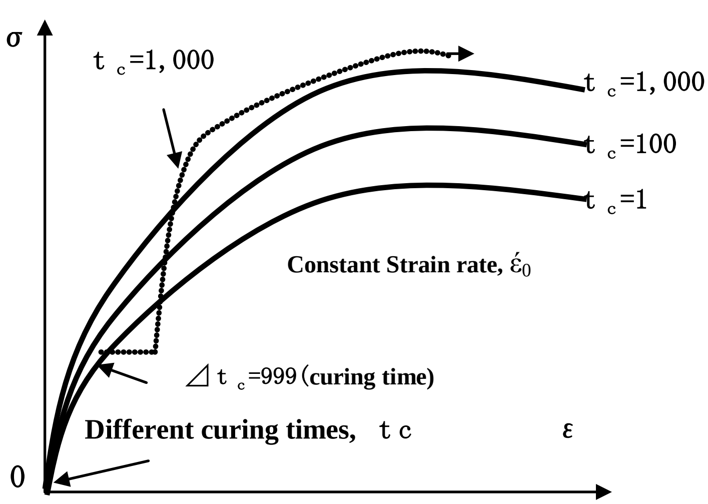


Figure 6.1 Effect of ageing on stress-strain relation

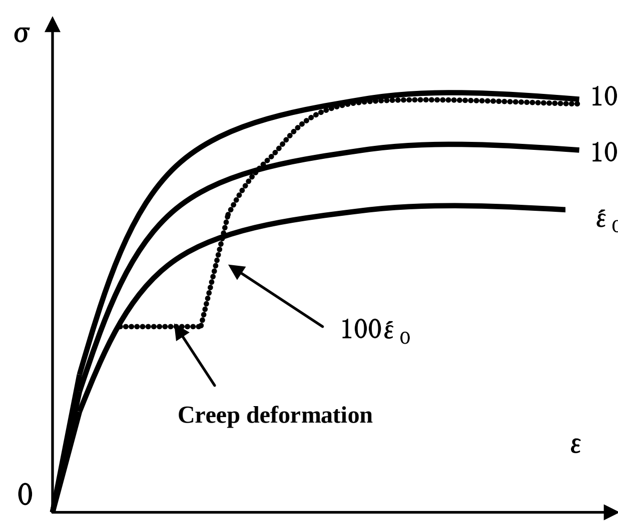


Figure 6.2 Viscous effect on stress-strain relation

The effect on the stress-strain behavior of geomaterials at constant strain rate, changes in stress-strain behavior when the strain rate is suddenly or gradually increased or decreased from a certain value to another and stress-strain behavior when loading is restarted after a stage of creep have been reported as methods for studying the time dependency in strength and deformation characteristics of geomaterials (Tatsuoka et al, 1998); (Matsushita et al, 1998); (Maden et al, 1999); (Di Benedetto et al, 2005).

In this study, triaxial compression tests with monotonic loading (ML) on uniformly-graded crushed gravel were carried out, under various loading conditions including constant strain- rate ($\acute{\varepsilon}_0$), 10 times $\acute{\varepsilon}_0$, 100 times $\acute{\varepsilon}_0$, incremental strain-rate changes during loading with and without creep stages, to consider time dependency in strength and deformation characteristics of this material.

6.2 Sample Preparation And Test Method

Andersite crushed gravel obtained from Kohshu with grain size distribution adjusted to 1/5th of the rail/road base material was used in this study. Figure 6.3 shows the grain size distribution curve of the sample material. The uniformity coefficient was 1.5. Target dry density for all samples was determined 1.65 g/cm^3 to prepare dense specimen. Table 6.1 shows the initial and relative dry density of each sample. It also shows the initial and relative dry density of those samples which were subjected to 10000 numbers of loading cycles (pre-strained). All samples had the relative dry density of more than 90%. Because all the samples were dense, the dry density of the specimens subjected to cyclic loading also remained the same as those of initial dry density.

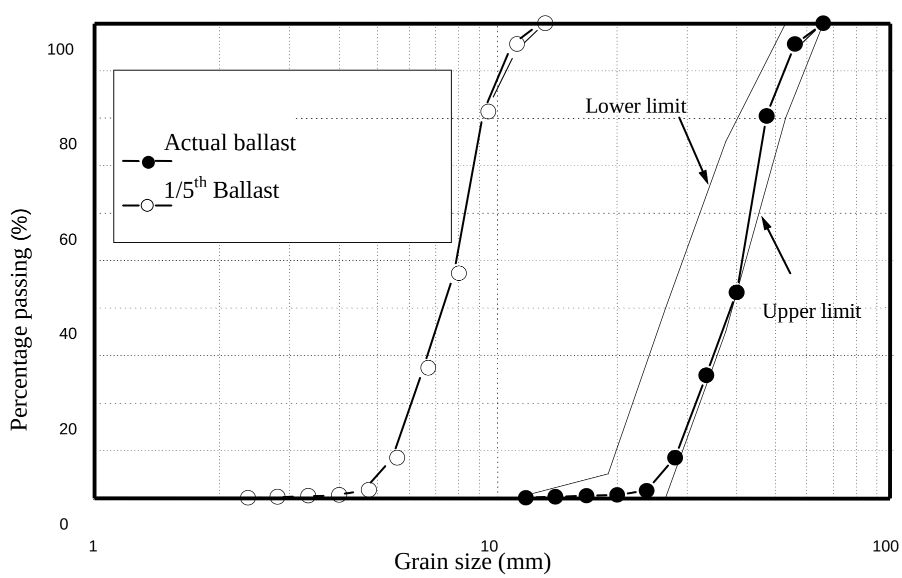


Figure 6.3 Grain size distribution of sample material

Table 6.1 Initial and relative dry density of each sample

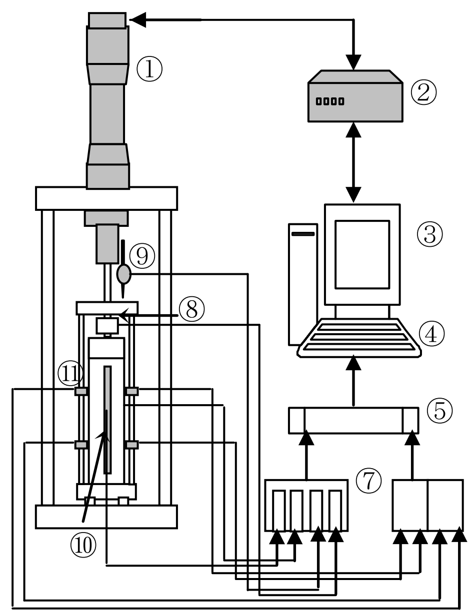
Test case	Specimen condition before shear	Initial dry density (g/cm^3)	D_r (%)	Dry density after cyclic loading (g/cm^3)	Relative density after cyclic loading, D_r' (%)
case1	Virgin	1.666	98.98	—	—
	Prestrained	1.663	98.20	1.662	97.95
case2	Virgin	1.645	93.51	—	—
	Prestrained	1.669	99.74	1.669	99.74
case3	Virgin	1.654	95.87	—	—
	Prestrained	1.669	99.74	1.669	99.74
case4	Virgin	1.649	94.56	—	—
	Prestrained	1.663	98.20	1.663	98.20
case5	Virgin	1.669	99.74	—	—
	Prestrained	1.669	99.74	1.670	100.00

Cylindrical specimens of 15 cm diameter and 36 cm height were prepared in a metal mold in six equal layers, each compacted for three minutes using vibrator (upper loading type vibrator). After preparation, each specimen was subjected to a confining pressure of 29.4 kPa under isotropic stress condition for 14 hours. specimens prepared for the study of cyclic loading history were accordingly subjected to 10000 numbers of loading cycles with the stress amplitude of 100 kPa, within the range of ((q_{cyc}) max= 110 kPa, (q_{cyc}) min= 10 kPa). Triaxial compression tests were carried out with five different loading conditions for both virgin and pre-strained specimens. Table 6.2 summarizes the test conditions employed in this study. Axial strain-rate of 0.015 %/min in Case 1 was chosen as a base rate ($\acute{\varepsilon}_0$). Strain-rate of 0.15 %/min ($10\acute{\varepsilon}_0$) and 1.5 %/min ($100\acute{\varepsilon}_0$) was used in Case 2 and Case 3 respectively. Case 4 was the case of varying strain-rate during loading as shown in the table. In Case 5, the loading condition was the same as of Case 4 but with the introduction of creep stages before each variation in strain-rate. Furthermore, the change in axial loading was conducted within the range of 1 % of the peak level.

Table 6.2 Test Conditions

case1	0.015%/min($\acute{\varepsilon}_0$)
case2	$10\acute{\varepsilon}_0$
case3	$100\acute{\varepsilon}_0$
case4	$\acute{\varepsilon}_0 \rightarrow 10\acute{\varepsilon}_0 \rightarrow \acute{\varepsilon}_0 \rightarrow 100\acute{\varepsilon}_0 \rightarrow \acute{\varepsilon}_0$
case5	$\acute{\varepsilon}_0 \rightarrow_c 10\acute{\varepsilon}_0 \rightarrow_c \acute{\varepsilon}_0 \rightarrow_c 100\acute{\varepsilon}_0 \rightarrow_c \acute{\varepsilon}_0$ c : creep before changing of strain rate

Figure 6.4 shows a schematic diagram of the test apparatus. Direct drive motor (DD motor) type loading equipment was used to avoid occurrence of backlash during load/unload. In order to consider the loosening of top and/or bottom ends of the specimen, which may occur during specimen preparation, or bedding error due to compression of the filter paper, the axial deformation was measured at minimum two locations 180^{o} apart (in plan view) using local deformation transducers (LDT) fixed on approximately the middle 1/2 of the specimen. However, LDTs are only able to measure up to 2 % of axial deformation; therefore, the strain above such range was corrected by subtracting bedding error from relative displacement value of the DD motor. Moreover, dial gauge was also used to measure axial deformation from the displacement of loading rod as well as from common external displacement censor. 2 pairs of non-contact censors were also set up, as shown in the Figure to directly measuring lateral displacement.



① Direct Drive Motor (DD Motor)
② Motor control unit
③ Computer
④ A/D converter
⑤ Noise rejection filter
⑥ Amplifier
⑦ Amplifier for proximeter
⑧ Load cell
⑨ Dial gauge
⑩ Local Deformation Transducer (LDT)
⑪ Proximeter

Figure 6.4 Triaxial Test Apparatus

6.3 Discussion On Test Results

6.3.1 About Bedding Error

Figure 6.5 and Figure 6.6 show the relationship between deviator stress q and axial strain ε_a in the range $\varepsilon_a = 0.0 - 3.5$ % and $\varepsilon_a = 0.0 - 0.1$ % respectively using values obtained from DD motor`s relative displacement and dial gauge. Furthermore, Figure 6.5 also shows local axial strain measured from LDTs up to 1%. A comparison of axial strain values, for all the specimens (both virgin and pre-strained) measured from relative displacement of DD motor and those measured from LDTs reveal that the DD motor values are over-estimated. In addition, in case of pre-strained specimens, the axial strain values by LDT are measured from 10^{-5} order whereas the relative displacement of DD motor did not produce values below 0.01 %. This implies that results obtained from relative displacement of DD motor in

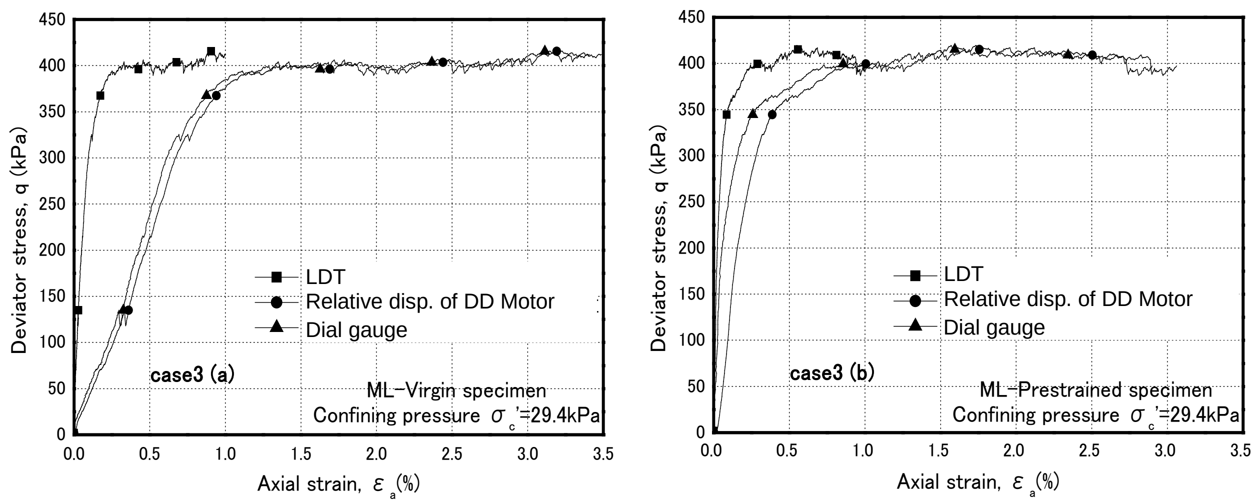


Figure 6.5 Effect of bedding error on $q - \varepsilon_a$ relations ($\varepsilon_a = 0 - 3.5$ %)

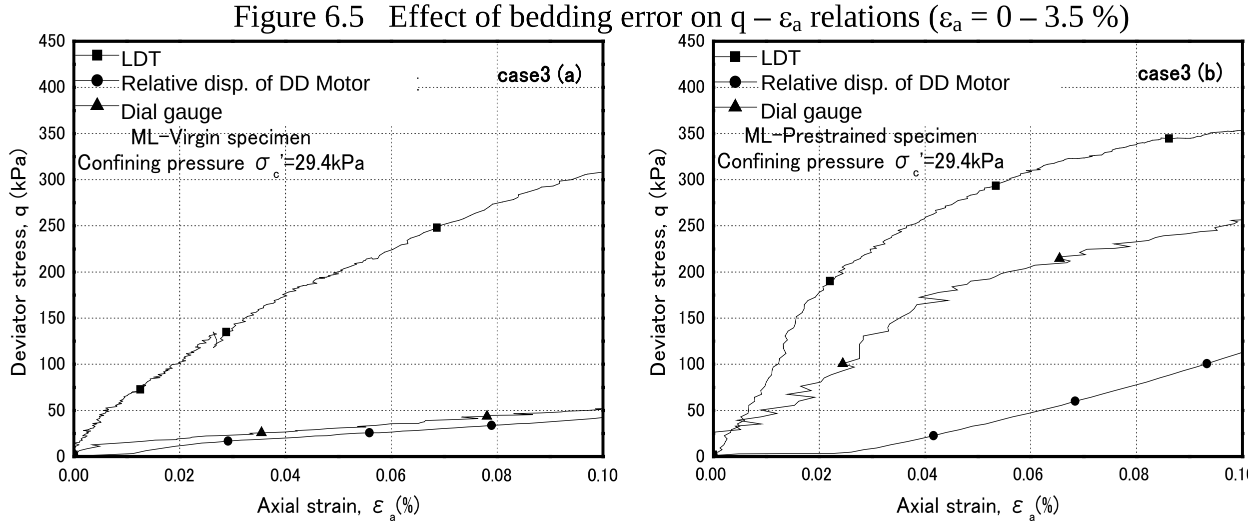


Figure 6.6 Effect of bedding error on $q - \varepsilon_a$ relations ($\varepsilon_a = 0 - 0.1$ %)

the initial loading stage can be misleading. Similarly the values of ε_a obtained from dial gauge are also overestimated, probably due to the effect of bedding error. A comparison of the ε_a values obtained from the relative displacement of DD motor and those obtained from dial gauge indicate that the former are larger values. The difference is particularly remarkable in case of pre-strained specimens. It is therefore considered that the effect of loosening of the topmost layer and the consequent effect, caused by the bedding error, on the results of the dial gauge marginally decreases after 10000 loading cycles. Thus, it is understood that the bedding error may considerably affect the results at the initial loading stage and therefore cannot be ignored.

6.3.2 Strength and Deformation Characteristics

Figure 6.7 and Figure 6.8 show the relationship between deviator stress q and axial strain ε_a in the range: ε_a = 0 – 12 % and ε_a = 0 – 2 % respectively. Figures (a) show results of monotonic loading test on specimens isotropically consolidated to 14 hours (Virgin) and Figures (b) show result of specimens subjected to 10000 numbers of loading cycles after consolidation (pre-strained). Figure 6.7 indicates the maximum deviator stress q_{max} in all cases of virgin and pre-strained to be around 420 kPa and is independent of axial strain rate, strain rate variation and creep stages. The q - ε_a relationship after each of q_{max} also show monotonically decreasing trend in all cases despite of the aforementioned varying conditions. For increasing trend in stress level prior to q_{max}, the variation in strain rate has not caused any noticeable effect in the virgin specimen case (Figure 6.8 (a)).

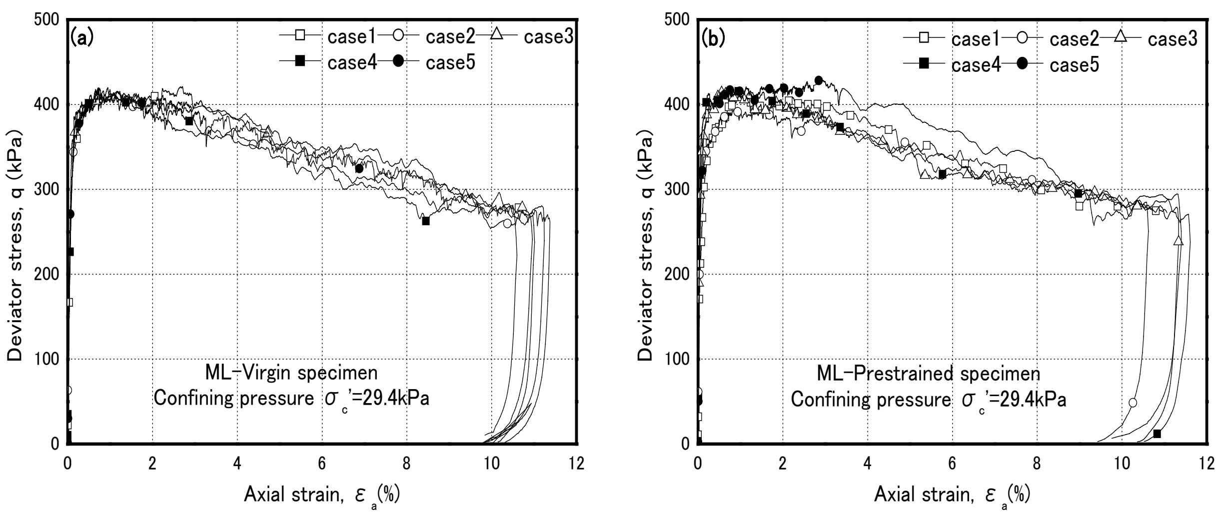


Figure 6.7 q – ε_a relations（0～12%）

However, in case of pre-strained specimens, the stress increment to axial strain (hereinafter called deviator stress increasing ratio) tend to become larger with the increase in strain rate (Figure 6.8 (b)). However, this trend ceases with the increase in strain-rate up to $100\varepsilon_0$ and beyond, which implies that the stress-strain behavior do not entirely depend on the strain-rate. In order to examine this phenomenon in detail, strain-rate variation and creep stages conditions during loading were created in the triaxial compression test, as highlighted in Case 4 and Case 5 respectively.

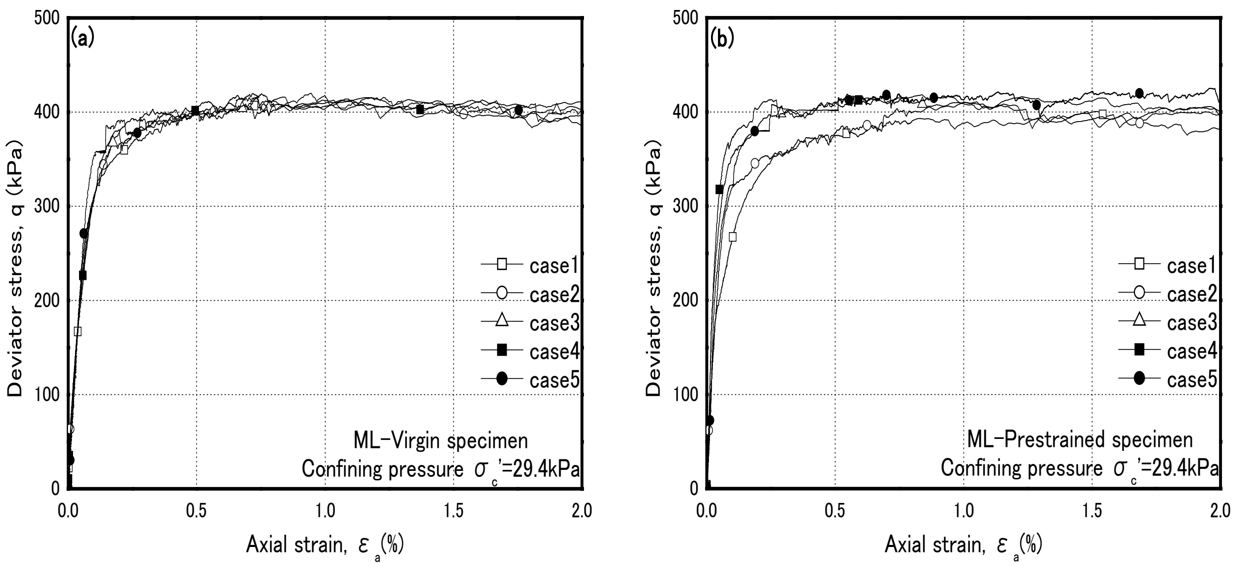


Figure 6.8 q – ε_a relations（0～2%）

Figure 6.9 shows q - ε_a relation in the range of ε_a= 0 – 1 %. In the virgin specimens, deviator stress increasing ratio in Case 5 is quite high compared to that in Case 1 – 4. In the q - ε_a relation after the creep, the deviator stress increases temporarily with increase in strain-rate (Overshooting). Conversely, if the deviator stress was reversed sharply, the axial strain decreased temporarily (Undershooting). Thereafter at constant strain rate the curve asymptotically approaches the same path as in Case 4. In other words, we can say that q - ε_a relation after the creep tends to converge to the constant strain-rate curve.

These results show that the changing trend in q - ε_a behavior after the creep is not due to changes in the intrinsic physical properties caused by ageing effect, rather it can be attributed to have been caused by another primary factor i.e. the viscous effect, which is the delayed deformation due to inter-particle friction, and it can be concluded that the time-dependency has considerable effect on the q - ε_a behavior. Similarly the q - ε_a curve in the pre-strained specimen also show the same trend of converging to the constant strain-rate path after the creep. However, the viscous effect in the pre-strained case is more evident than the virgin case.

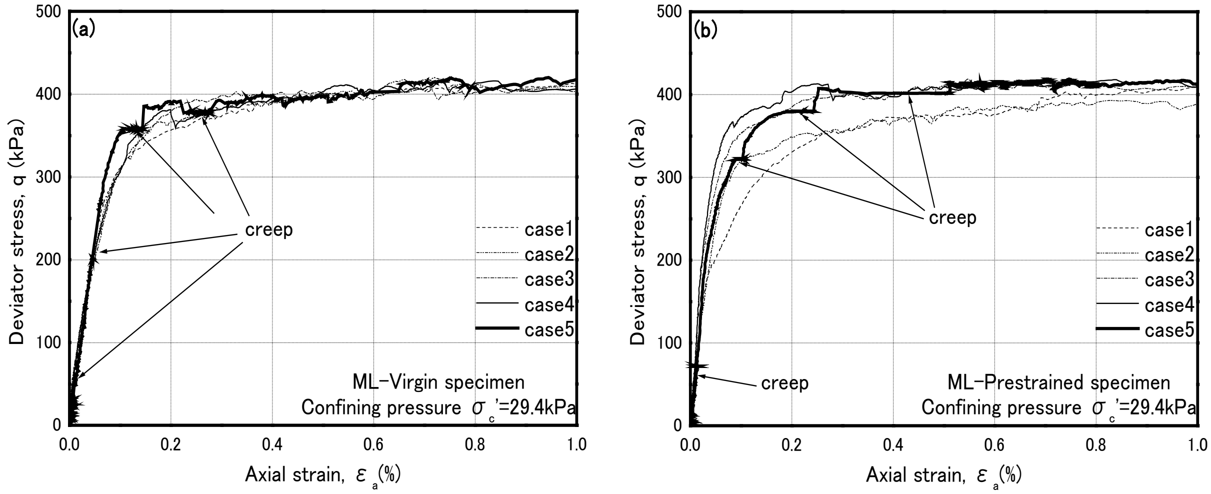


Figure 6.9 q – ε_a relations（0～1%）

Figure 6.10 shows the relationship between deviator stress q and volumetric strain ε_{vol} obtained from triaxial compression tests for conditions in Case 1- 4. Results of virgin and pre-strained specimens are shown in Figure 6.10(a) and Figure 6.10(b) respectively. It can be seen that in both the cases, when the axial strain-rate is constant (Case 1 – 3) the deviator stress increasing ratio tend to be higher for the faster strain rate and the q - ε_{vol} behavior depend upon the strain rate. Even in Case 4, where the strain-rate was changed during loading, the trend of deviator stress increasing ratio is almost the same as in Case 1 – 3 where the deviator stress rises with the increase in axial strain-rate and falls accordingly with the reduction in strain-rate. However this trend is more evident in both virgin and pre-strained cases only up to the level of $100\varepsilon_0$. Thus it can be concluded that the q - ε_{vol} behavior is more affected by the strain-rate rather than strain acceleration.

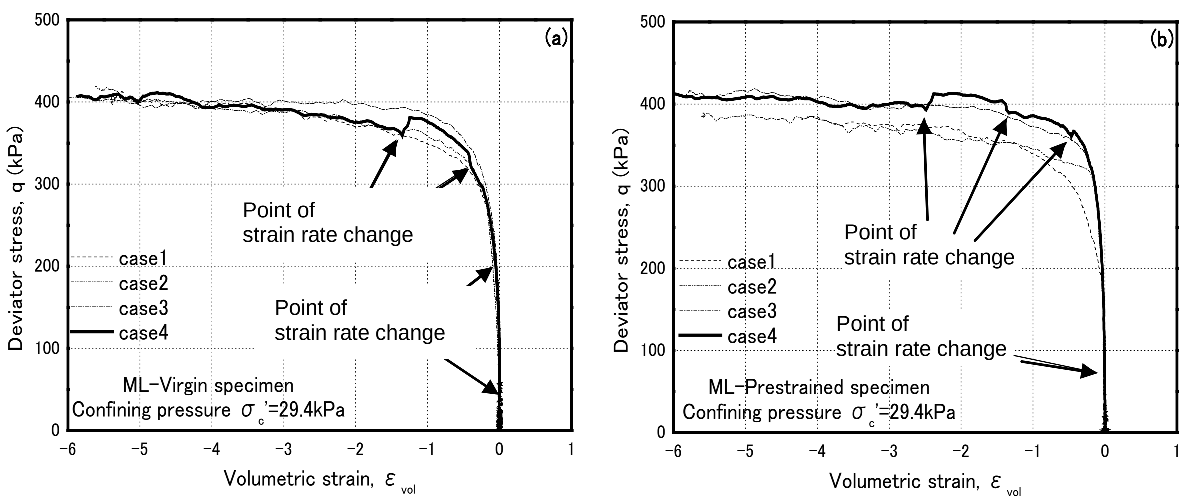


Figure 6.10 Comparison of q - ε_{vol} relation (Case 1-4)

Moreover, these effects were not found in the virgin case and gradually minimize in the pre-strained case with the further advance of volumetric strain. In other words the effect of strain rate over q - ε_{vol} behavior minimizes with the strain progression.

Figure 6.11 shows the relation between deviator stress q and volumetric strain ε_{vol} for Case 1 – 3 and Case 5. Results of virgin and pre-strained specimens are shown in Figure 3.11 (a) and Figure 3.11 (b) respectively. In both virgin and pre-strained cases, the q - ε_{vol} relation after the creep in Case 5 indicates that the deviator stress increases with the strain-rate increment and vice-versa. However, this trend is only remarkable up to the increment of $100\varepsilon_0$ like that in Case 4. In addition, deformation due to creep appear more remarkable in pre-strained condition. These results suggest that the q - ε_{vol} relation after the creep depend much on the viscous effect rather than the ageing effect. Moreover, as in case of q - ε_a relationship; the q - ε_{vol} relation also leads to the same conclusion that the viscous effects in the pre-strained condition are more evident compared to the virgin condition.

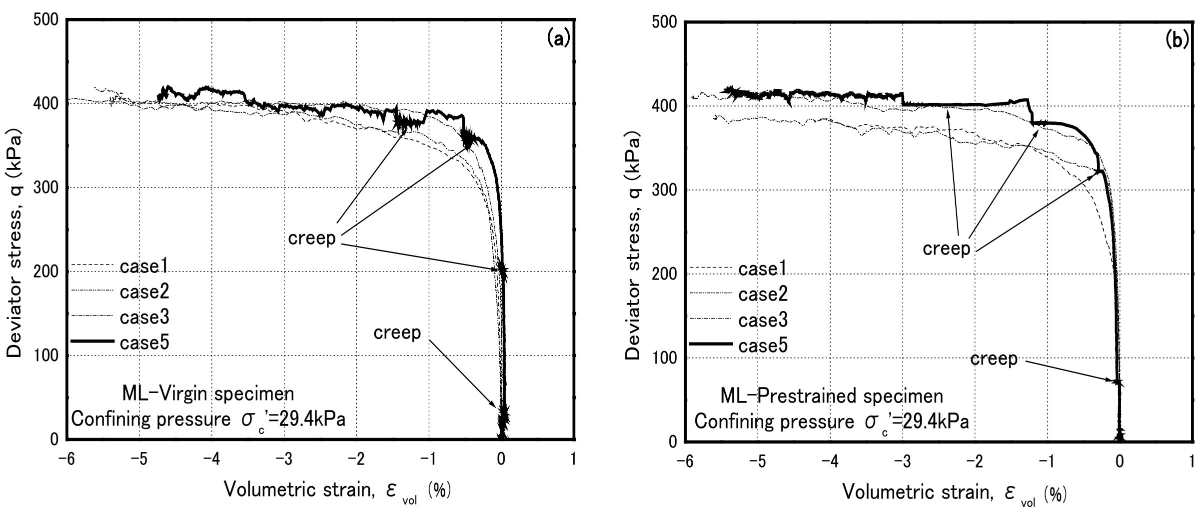


Figure 6.11 Comparison of q - ε_{vol} relation (Case 1-3 and Case 5)

It has been noted from the q - ε_a and q - ε_{vol} relationships that the viscous effect is encountered in uniformly-graded crushed gravel until the time the inter-particle contact state become steady. However, comparing with the experimental results on the influence of viscous effect to the deformation characteristics of sand e.g. (Di Benedetto et al 2002); (Tatsuoka et al 2002), the influence is less evident in uniformly-graded crushed gravel. It is considered to be due to fewer inter-particle contact points in uniformly-graded crushed gravel compared to sand.

6.3.3 Dilatancy Characteristics

Figure 6.12 shows the relationship between volumetric strain ε_{vol} and axial strain ε_a. Results of virgin and pre-strained conditions, within the range of ε_a= 0 - 0.5 %, are shown in Figure 6.12 (a) and Figure 6.12 (b) respectively. The figures indicate positive dilatancy trend in all test Cases 1 – 5, in both virgin and pre-strained conditions in the initial loading stages. In addition, uniform trend in ε_{vol} - ε_a behavior has not been observed in different testing conditions and this non-uniformity is considered to be within the range of dispersion. The influence of loading rate and loading acceleration on ε_{vol} - ε_a behavior was not observed in particular. Thus, the effect of time-dependency on dilatancy was not observed within the extent of this study.

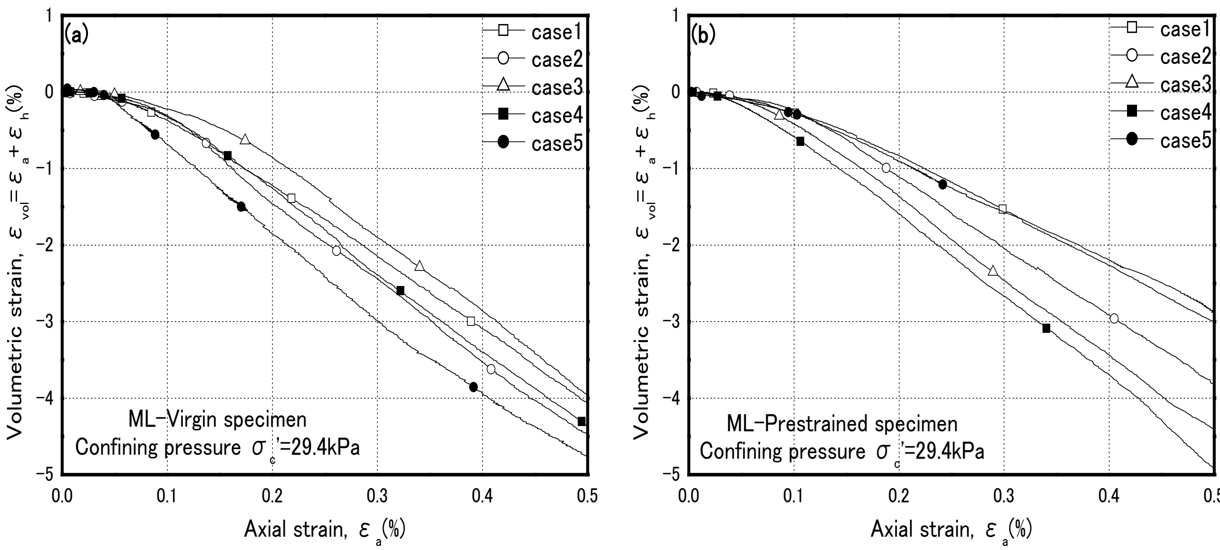


Figure 6.12 ε_{vol} - ε_a relations

6.4 Summary

In order to examine the effect of time-dependency on the strength and deformation characteristics of UGMs (e.g. uniformly-graded crushed gravels), series of triaxial compression tests with 5 different loading conditions including constant strain- rate ($\acute{\varepsilon}_0$), 10 times $\acute{\varepsilon}_0$, 100 times $\acute{\varepsilon}_0$, incremental strain-rate changes during loading; with and without creep stages, were conducted. Conclusions based on the analysis of results may be summarized as follows:

1) The influence of viscous effect to the strength of uniformly-graded crushed gravel was not observed in both cases; with and without cyclic loading history. Therefore the time-dependency effect on the strength properties is considered to be minimal.

2) Noticeable influence of viscous effect was observed on the deformation characteristics of uniformly-graded crushed gravel, which reveal that the time-dependency has considerable effect on the deformation properties of these materials. Moreover, the effect is more evident in materials with cyclic loading history.
3) The influence of loading rate and loading acceleration on ε_{vol} - ε_a behavior was not observed, which transpires that time-dependency has minimal effect on dilatancy characteristics of these materials.

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Chapter 7 An Efficient Field Testing Method for Evaluating Mechanical Properties of Unbound Pavement Layers by Portable FWD

7.1 Introduction

Unbound pavement layers such as aggregate base course, subbase and subgrade are the major structural components of flexible pavement system. The physical properties of geomaterials in these layers greatly affect the performance of pavements. Characterization of these layers in terms of stiffness is an integral part of pavement design and evaluation. Insufficient stiffness of any component may result into premature failure and thus loss to the economy. Therefore the stiffness of these layers is strictly controlled during the construction process.

In Japan, stiffness of unbound pavement layers is evaluated from K value i.e. coefficient of sub grade reaction, obtained from a plate loading test (JRA, 1995). This K value is used in quality control management during execution of these layers. However, a large reaction device is required to perform the plate loading test. Moreover, the location to be tested is restrained for further work for at least one day as the plate loading test require much time. For these reasons, application of $K_{P.FWD}$ values based on portable Falling Weight Deflectometer (P.FWD) tests for construction quality management of unbound pavement layers is attempted in Japan (Tatsumi, Y. et. al., 2004; Kamono, T. et. al., 1999) Portable FWD due to its numerous advantages including, portability; simplicity and ease of measurement; and time efficiency in estimating stiffness moduli has gained popularity in recent years (George, K. P., 2006)

In this study a relationship between K values obtained from P.FWD tests and conventional plate loading tests is discussed. The data for both K values was collected from previous researches and compared with physical tests performed on various geomaterials at different road pavement construction sites using a portable FWD and plate loading tests.

7.2 Field test sites and physical properties of geomaterials

In this study, field tests using P.FWD and plate loading test equipment for road were performed on the following three road construction site:

i) Abuta - Lake Toya IC, HOKKAIDO EXPRESWAY

ii) Route “Lake Toya - Noboribetsu”, Hokkaido Prefecture Highway

iii) Route “Abuta - Toya”, Hokkaido Prefecture Highway

The test sites are shown in Photos 7.1 - 7.3. In the Abuta-Lake Toya IC, HOKKAIDO EXPWY, field tests were performed on the subbase (restraint layer for frost heave) and the base course as shown in Figure 7.1. Test location plan for this site is shown in Figure 7.2. In the Route “Lake Toya - Noboribetsu”, Hokkaido Pref. Highway, tests were performed on the base course as shown in Figure 7.3. Similarly tests on subgrade were performed in the Route “Lake Toya - Noboribetsu”.

Photo 7.1 Construction site at Abuta-Lake Toya IC, HOKKAIDO EXPWY

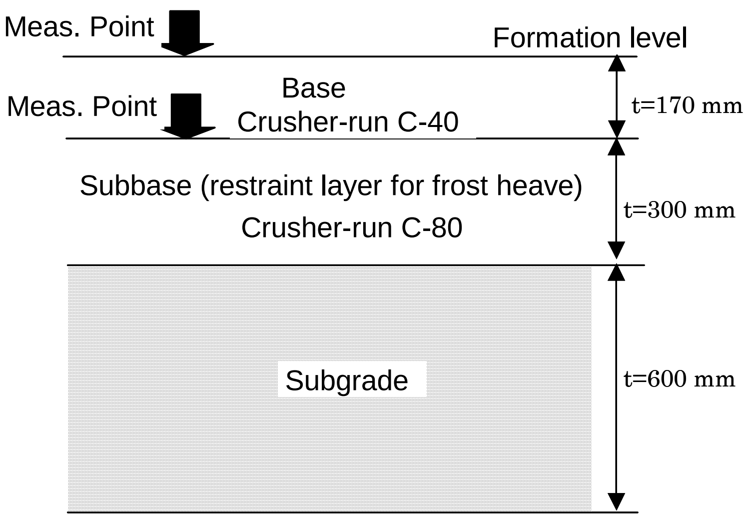

Figure 7.1 Cross-section of Construction site at Abuta-Lake Toya IC, HOKKAIDO EXPWY

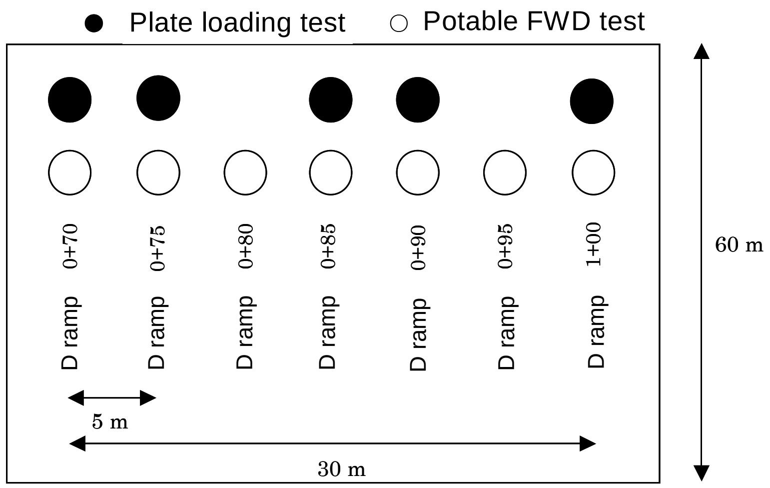

Figure 7.2 Plan view of test point at Abuta-Lake Toya IC, HOKKAIDO EXPWY

Photo 7.2 Construction site at Route “Lake Toya - Noboribetsu”, Hokkaido Pref. Highway

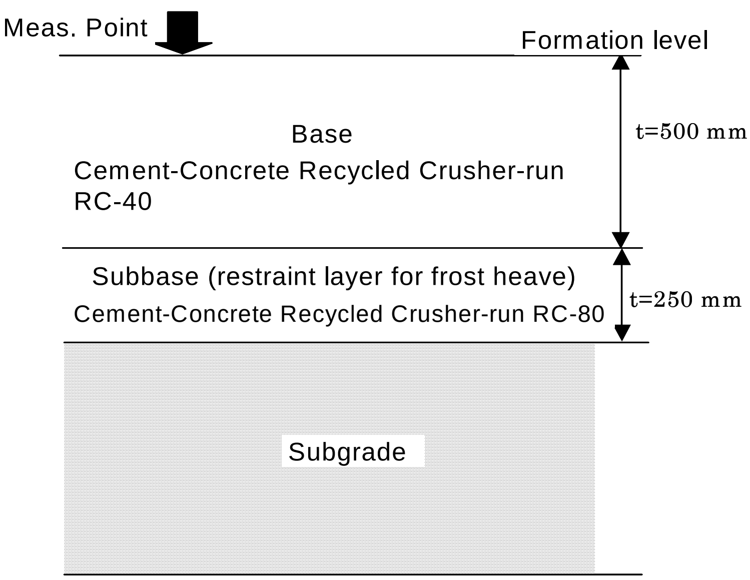

Figure 7.3 Cross-section of Construction site at Route “Lake Toya - Noboribetsu”, Hokkaido Pref. Highway

Photo 7.3 Construction site at Route “Abuta - Toya”, Hokkaido Pref. Highway

Table 7.1 and 7.2 show the physical properties of base, subbase and subgrade geomaterials in the field test sites. Figure 7.4 shows the grain size distribution curves for each geomaterials. In order to investigate the compaction property of base and subbase geomaterials, soil compaction tests (E - b method, JIS A 1210) were performed. The compaction curves of base and subbase geomaterials obtained from soil compaction tests are shown in Figure 7.5. It is indicated that the dry density of crusher-run (C-80), which is fine-grained material, largely dependent on water content. The maximum dry density of C-80 is largest among geomaterials of field test sites investigated in this study.

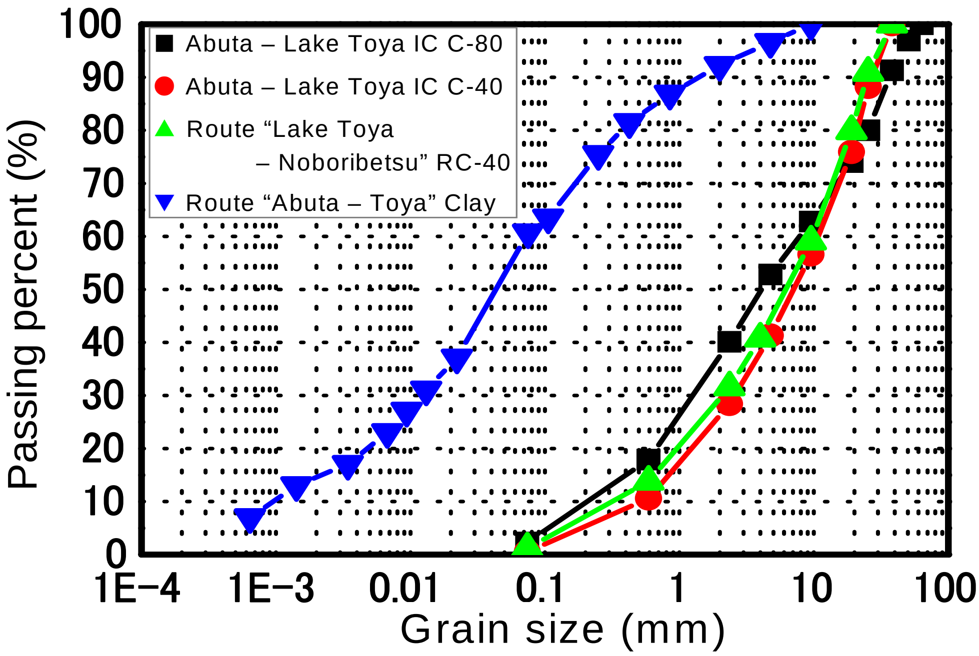

Figure 7.4 Grain size distribution curves of Base, Subbase and Subgrade

7.3 Calculation Method of $K_{P.FWD}$ by portable FWD test

A schematic of a portable FWD test apparatus is shown in Figure 7.6. In this study, the diameter of the loading plate (P.FWD) was 10 cm. whereas, plate diameter generally used for plate loading test is 30 cm. Due to the difference in loading plate diameters used in the two test methods, the displacement from P.FWD load plate is corrected to loading plate of plate load test using the following relation:

$$K_{P.FWD} = (P_{P.FWD.\varphi}/\delta_{P.FWD.\varphi}) \cdot (\varphi_{P.FWD}/\varphi_{PLT}) \tag{7.1}$$

Where;

$P_{P.FWD.\varphi}$: Loading stress at displacement $\delta_{P.FWD.\varphi}$ (kN/m^2)

$\delta_{P.FWD.\varphi}$: Displacement (mm)

$\varphi_{P.FWD}$: Diameter of P.FWD loading plate (cm)

φ_{PLT} : Diameter of loading plate for plate loading test (30cm)

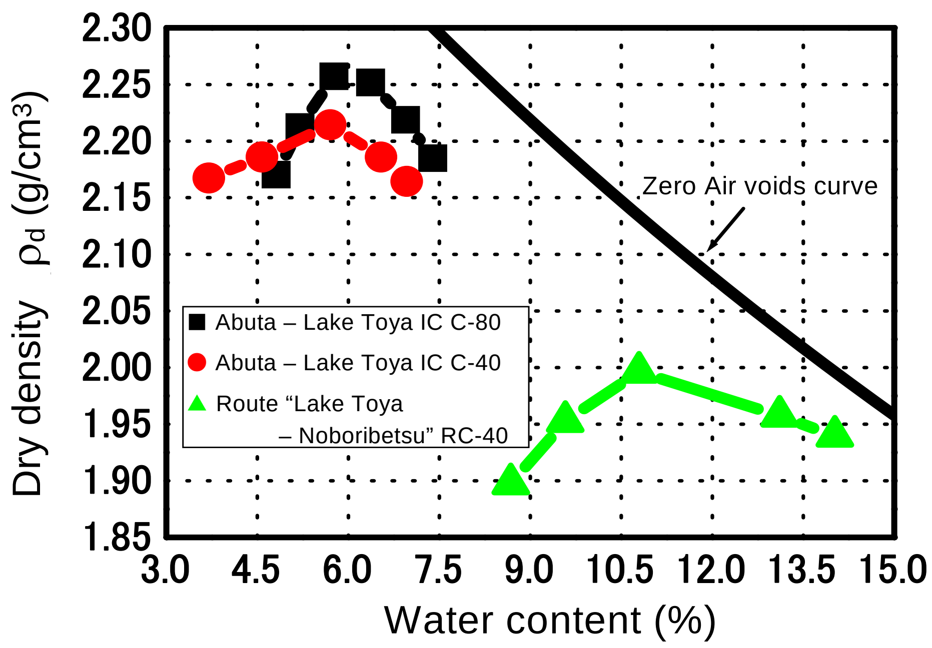

Figure 7.5 Compaction curves of Base and Subbase Geomaterials

Standard displacement for P.FWD loading plates of various diameters is corrected to 30 cm. diameter loading plate (plate load test) as follows:

$$\delta_{P.FWD.\varphi} = 1.25\ \text{mm} \times (\phi_{P.FWD}/30\ \text{cm}) \tag{7.2}$$

Table 7.1 Physical properties of Base and Subbase Geomaterials

Base and Subbase Geomaterial		Abuta-Lake Toya IC C-80	Abuta-Lake Toya IC C-40	Route Lake Toya - Noboribetsu RC-40
Gradation Properties	gravel fraction (%)	59.88	71.57	68.43
	sand fraction (%)	37.87	27.90	30.39
	fine fraction (%)	2.251	0.522	1.179
	60 % grain size, D_{60} (mm)	7.896	10.79	9.803
	30 % grain size, D_{30} (mm)	1.261	2.577	2.084
	10 % grain size, D_{10} (mm)	0.207	0.523	0.315
	coefficient of uniformity, U_c	38.14	20.63	31.12
	coefficient of curvature, U_c'	0.973	1.177	1.406
Density of soil particle, ρ_s (g/cm^3)		2.772	2.637	2.610
Soil classification		GS	GS	GS
Max. dry density on compaction test (E-b method), ρ_{dmax} (g/cm^3)		2.257	2.214	1.996
Optimum moisture content on compaction test (E-b method), w_{opt} (%)		5.762	5.703	10.79

Standard displacement for 9cm, 10cm and 20cm diamtere P.FWD plates is as follows.

$$\varphi_{P.FWD} = 9\text{ cm} \longrightarrow \delta_{P.FWD.\varphi} = 0.375\text{ mm}$$

$$\varphi_{P.FWD} = 10\text{ cm} \longrightarrow \delta_{P.FWD.\varphi} = 0.417\text{ mm}$$

$$\varphi_{P.FWD} = 20\text{ cm} \longrightarrow \delta_{P.FWD.\varphi} = 0.833\text{ mm}$$

Table 7.2 Physical properties of Subgrade Geomaterial

Subgrade Geomaterial		Route Abuta - Toya Clay
Gradation Properties	gravel fraction (%)	7.722
	sand fraction (%)	31.57
	fine fraction (%)	60.71
	60 % grain size, D_{60} (mm)	$7.230*10^{-3}$
	30 % grain size, D_{30} (mm)	$1.200*10^{-2}$
	10 % grain size, D_{10} (mm)	$9.592*10^{-4}$
	coefficient of uniformity, U_c	75.38
	coefficient of curvature, U_c'	2.076
Liquid limit, w_L (%)		53.04
Plastic limit, w_P (%)		21.26
Plasticity index, I_P		31.78
Density of soil particle, ρ_s (g/cm^3)		2.733
Soil classification		CH
Max. dry density on compaction test (A-a method), ρ_{dmax} (g/cm^3)		1.474
Optimum moisture content on compaction test (A-a method), w_{opt} (%)		27.24

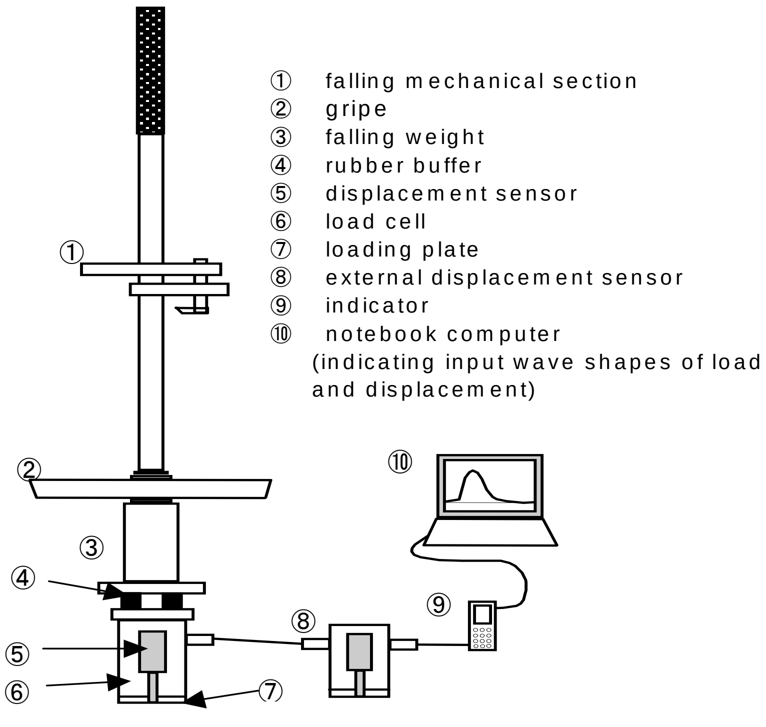

Figure 7.6 Schematic of portable FWD test apparatus

Since 10cm diameter plate has been used for P.FWD, therefore; in the field the falling weight and the drop height is first adjusted carefully for the layer to be tested to obtain a minimum of three displacement values, with the middle value being 0.417 mm. The loading stress corresponding to this displacement is worked out as shown in Figure 7.7. This procedure is called single layer analysis.

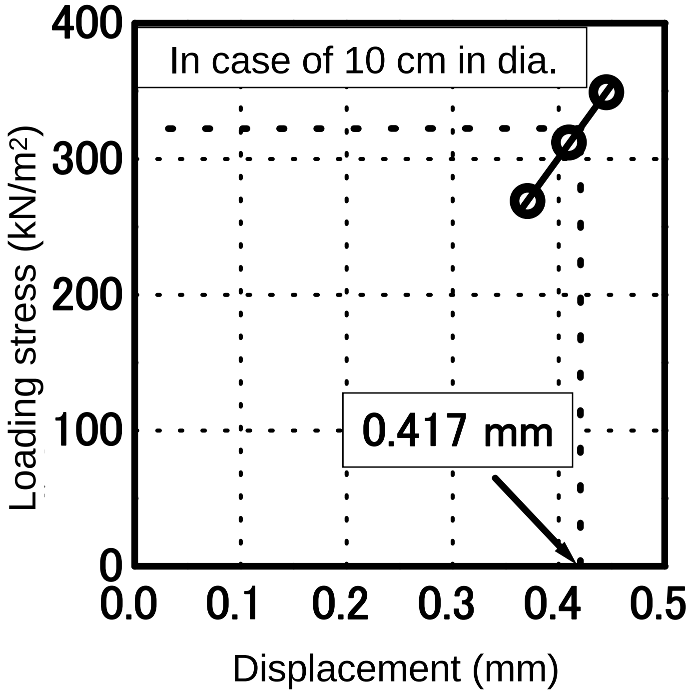

Figure 7.7 Example of relationship between displacement and loading stress
(In case of 10 cm plate diameter)

However, in order to study the influence stiffness of the underlying layer in the pavement structure on $K_{P.FWD}$ value; multi layer analysis was also conducted to estimate the $K_{P.FWD}$ value using software Back Analysis for Layer Moduli (BALM), which is based on multi-layer elastic theory. External displacement sensor was used in portable FWD test to obtain the stiffness of underlying layer. The displacement D_1 on the surface layer is measured by placing the external sensor at a horizontal distance of about the thickness of the surface layer from the center of the loading point. Table 7.3 shows the condition of portable FWD test performed at construction site in this study.

Table 7.3 Condition of portable FWD tests

Field test site	Abuta-Lake Toya IC		Route Lake Toya - Noboribetsu	Route Abuta - Toya
Layer	Base	Subbase	Subbase	Subgrade
Plate diameter	100 mm	100 mm	100 mm	300 mm
Mass of weight	15 kg	10 kg	10 kg	15 kg
Falling height	50 mm spacing from 150 mm upto 250 mm	50 mm spacing from 100 mm upto 200 mm	50 mm spacing from 150 mm upto 250 mm	50 mm spacing from 150 mm upto 250 mm
Number of External displacement sensor	1	1	1	0
Location of External displacement sensor	30 cm from center of loading point	30 cm from center of loading point	30 cm from center of loading point	

The calculation of $K_{P.FWD}$ value by elastic analysis; using two layers model was performed for the values obtained from external displacement sensor at the Abuta-Lake Toya IC and the Route Lake Toya-Noboribetsu. The $K_{P.FWD}$ value was calculated by using the relation:

$$K = 2E/(\pi r(1-\nu^2)) \tag{7.3}$$

Where; r is radius of loading plate of portable FWD apparatus (mm), ν is Poisson's ratio assumed 0.3 (for gravelly soil) and E is elastic modulus obtained from elastic analysis using BALM.

It is to be mentioned that the loading condition for the software BALM used in multi-layer elastic analysis is the loading to flexible surface; on the other hand, the loading condition of portable FWD test is loading to rigid plate. Both conditions are different. However, it is assumed that the loading width of portable FWD test apparatus is relatively small because of the diameter of loading plate of the portable FWD test apparatus (10 cm). Therefore, it is considered that the effect of loading condition on the result is minimal.

7.4 Test results and discussion

7.4.1 Consideration of the scattering of K-value

The relationships between $K_{P.FWD}$ value and K_{30} value are shown in Figures 7.8, 7.9 and 7.10 for subbase C-80, base C-40 and base RC-40, respectively. Figure 7.11 shows the coefficient of variation of each K values for cohesive soil (subgrade) and gravelly soils (subbase C-80, base C-40 and RC-40). Since a displacement of 2.50 mm in subgrade in flexible pavements with asphalt concrete surface is considered to be critical, therefore Figure 7.11 also indicate K values for 2.50 mm displacement of loading plate.

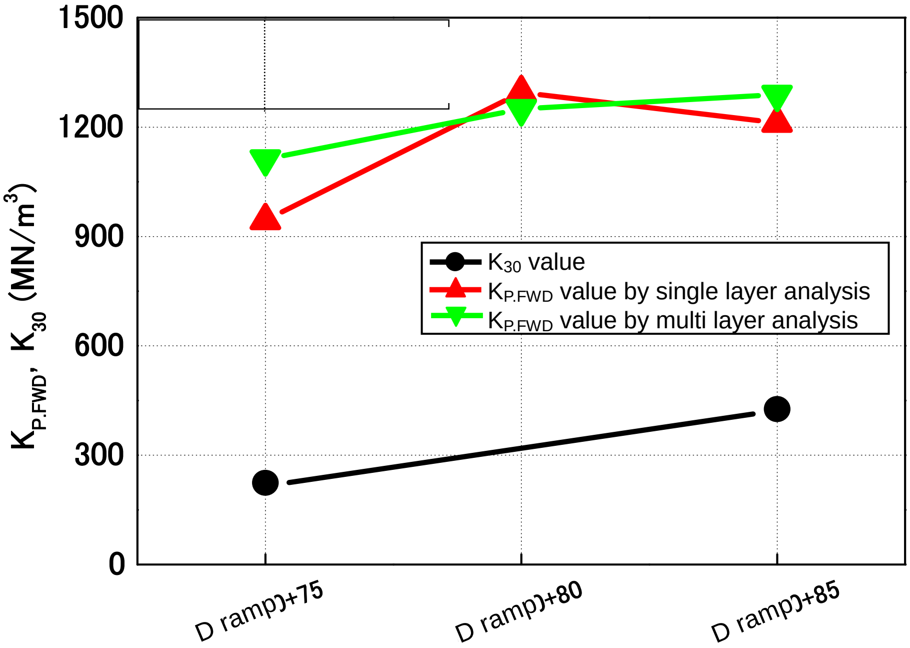

Figure 7.8 $K_{P.FWD}$ and K_{30} value at each measurement point (Subbase C-80)

Figures 7.8 - Figure 7.10 show that the results for $K_{P.FWD}$ obtained from single layer analysis as well as multi-layer analysis are comparable, except in Figure 7.9 where $K_{P.FWD}$ values obtained from multi-layer analysis for base C-40 are on the higher side. It is considered this is due to the effect of increased stiffness of the underlying subbase layer which is much thicker (300 mm) compared to the base course (170 mm). However, while the K_{30} values obtained from plate load tests in all the cases are closely corresponding; the K_{30} values are 3-4 times lower than $K_{P.FWD}$ values. It is considered that this disparity is probably due to the difference in diameters of the loading plates and influence of impact load etc.

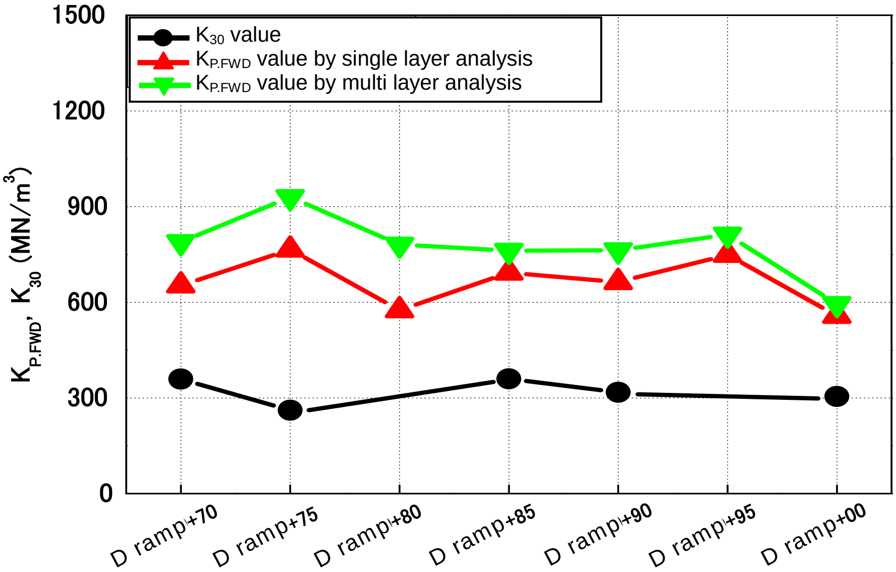

Figure 7.9 $K_{P.FWD}$ and K_{30} value at each measurement point (Base C-40)

The coefficient of variation for K values obtained from plate load test and those from portable FWD (both single and Multi-layer analysis), presented in Figure 7.11 show comparable values for base C-40, whereas; considerable variation was observed in case of subbase C-80. On the other hand, while significant variation exists between K_{30} and $K_{P.FWD}$; the two analysis procedure for portable FWD resulted in almost identical results. It is considered that these variations in results were due to the degree of compaction of each individual test point on unbound granular materials which is affected by air voids and dry density etc.

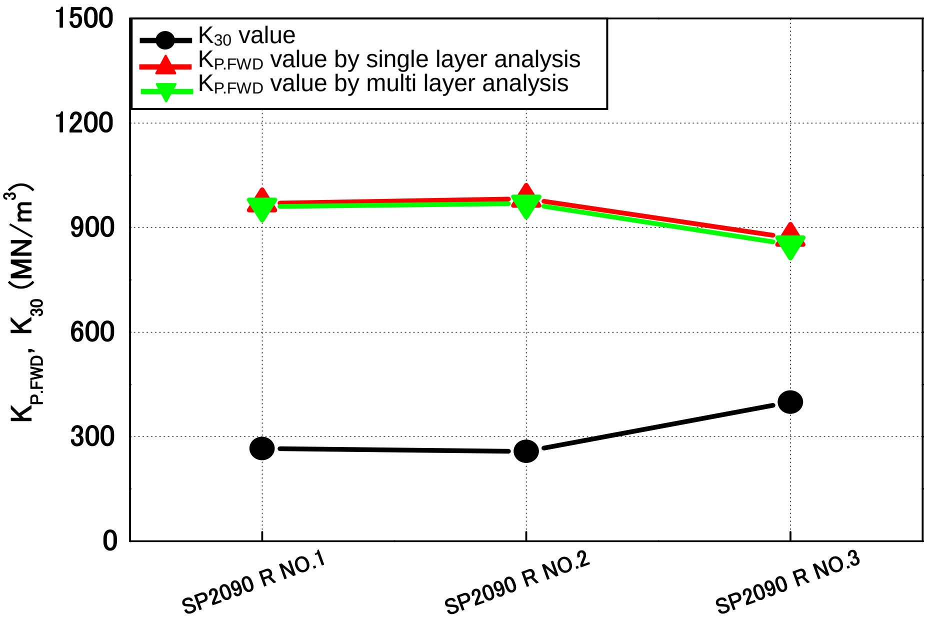

Figure 7.10 $K_{P.FWD}$ and K_{30} value at each measurement point (Base RC-40)

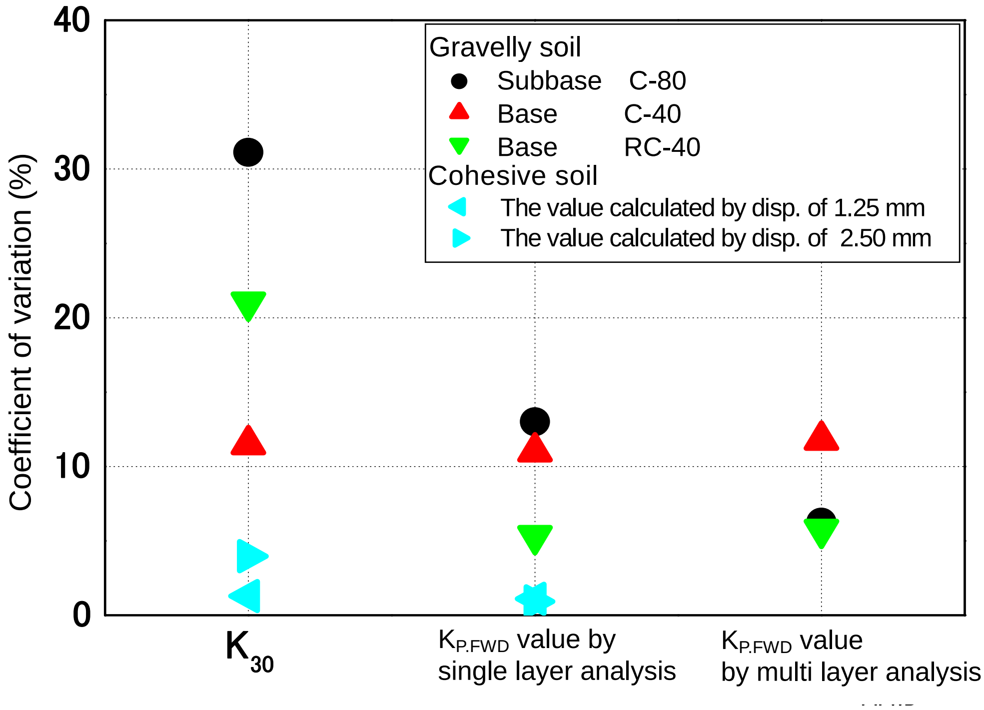

Figure 7.11 Coefficient of variation of each K values

Figure 7.12 shows the comparison of $K_{P.FWD}$ values calculated by single layer analysis and multi-layer analysis. The Figure indicates that multi-layer analysis resulted into higher values for base C-40 and subbase C-80. Moreover, the Figure also shows that the subbase values by both the analysis procedure are larger than the base course C-40, whereas it should have been the other way round. This reconfirms the earlier observation that values are indeed affected by the thickness and higher stiffness of the underlying layer (subbase C-80).

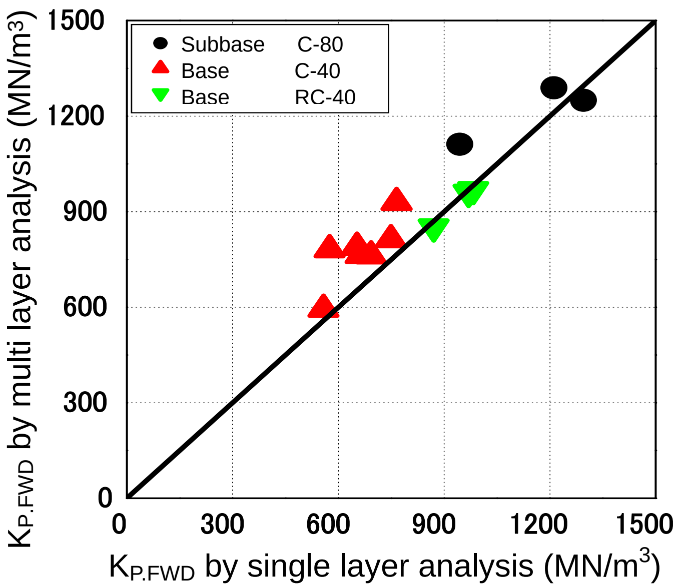

Figure 7.12 Comparison of $K_{P.FWD}$ by single layer analysis and $K_{P.FWD}$ by multi layer analysis

7.4.2 Relationship between $K_{P.FWD}$ and K_{30}

Figure 7.13 - Figure 7.15 show the relationship between $K_{P.FWD}$ and K_{30} for various geomaterials by plotting the corresponding values obtained from the bibliographic survey of the past research. The Figures also show the relationships arrived at as a result of this study. Results of the past researches show that the relationship between $K_{P.FWD}$ and K_{30} for cohesive and granular soil is 1:1 and 1.5:1, as shown in Figures 7.13 and Figure 7.14, respectively. However, the relationship from the statistical analysis of past and current data for granular soil presented in Figure 7.15 revealed that a ratio of 2:1 more suitably express the relationship for granular soil.

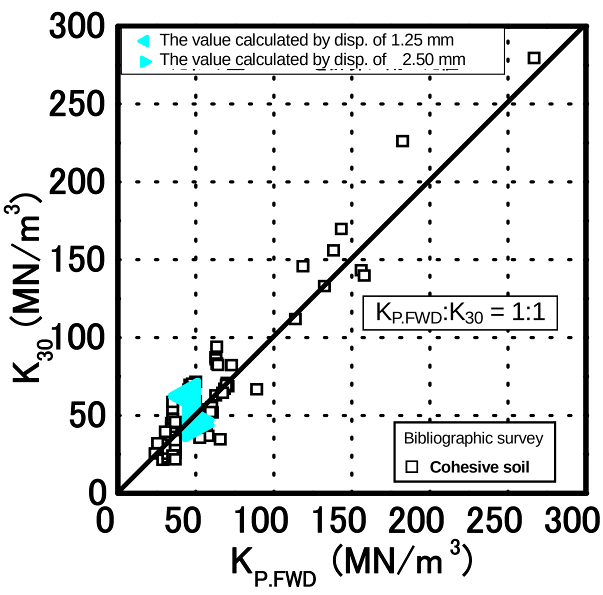

Figure 7.13 $K_{P.FWD}$ and K_{30} relations (Cohesive soil)

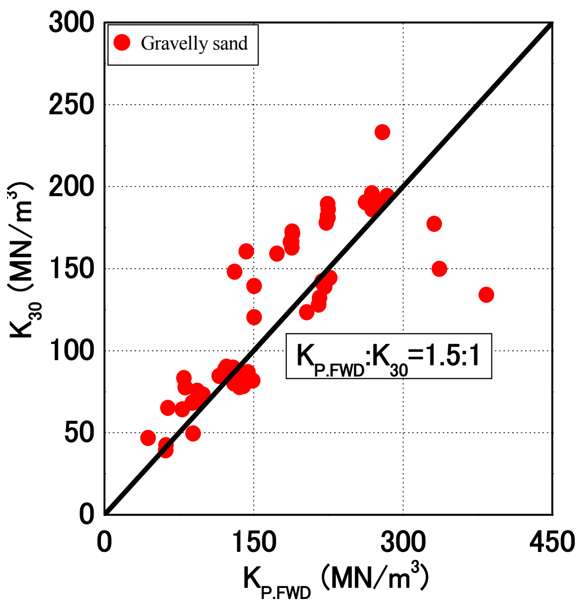

Figure 7.14 $K_{P.FWD}$ and K_{30} relations (Gravelly sand)

Figure 7.16 show the relationship between $K_{P.FWD}$ and K_{30} for all types of geomaterials. Whereas, the data plotted on the log scale show that K_{30} values are scattered in case of subbase C-80 and base RC-40 (due to the probable reasons discussed earlier), the curve fitting the entire data do represent a relationship between $K_{P.FWD}$ and K_{30} which is nearly expressed by the following equation:

$$\log K_{30} = 0.861 + 0.352 \log K_{P.FWD} + 0.077 (\log K_{P.FWD})^2 \qquad (7.5)$$

7.4.3 Conversion to K_{30} value from $K_{P.FWD}$ value

In the construction of pavement structures the execution quality control of subgrade and unbound pavement layers such as subbase and base course is generally assessed from the stiffness of the finished layer expressed in terms of K_{30} value estimated by plate loading tests. Therefore it is necessary to convert the $K_{P.FWD}$ to K_{30} as accurately as possible so as to capitalize on the numerous advantages offered by portable FWD including the testing time efficiency. Thus the test points along the finished surface can be increased and chances of localized discrepancies in layer stiffness can be minimized considerably.

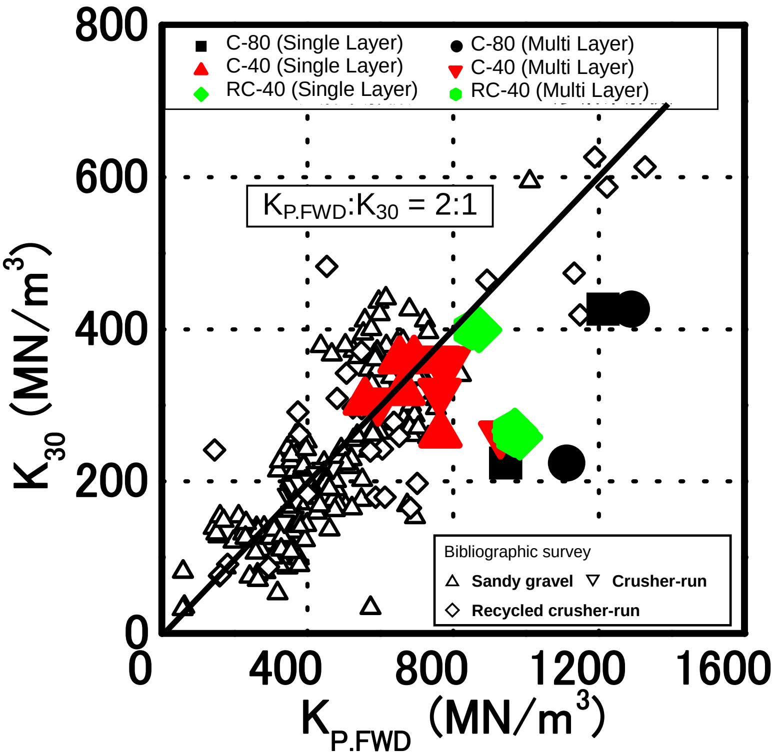

Figure 7.15 $K_{P.FWD}$ and K_{30} relations (Gravelly soil)

The conversion factor "a" expressed in the equation below was examined using the available data of the past research and that obtained in this study.

$$a = \frac{K_{P.FWD}}{K_{30}} \tag{7.6}$$

In general, the stiffness of unbound pavement layers is affected by stress dependency and consequent non linearity in the stress-strain behavior. Therefore, the K value from portable FWD; even for the same type of geomaterials cannot be expected to be a constant value. Moreover, the diameter of the loading plate and characteristics of rubber buffer (which may be affected by the field temperature) being used in the portable FWD may also affect the impact load and thus the resulting displacement values. Also the stiffness of the target layer resting on a comparatively thicker layer (e.g. a thin base course resting on a thick subbase) can be sometimes misleading.

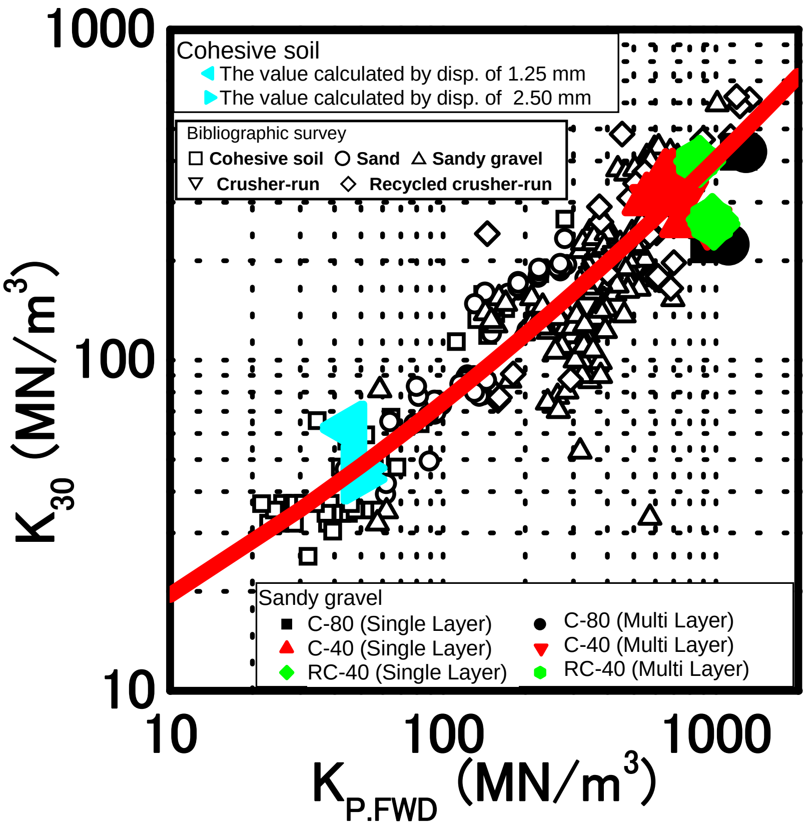

Figure 7.16 $K_{P.FWD}$ and K_{30} relations (All geomaterials)

Keeping in view the above possible anomalies, it was considered to examine the conversion factors for various geomaterials in more detail based on the results of the past research and those observed in this study. Figure 7.17 to Figure 7.19 show the "a" factors corresponding to $K_{P.FWD}$ values. Based on the statistical analysis of entire data, conversion factors are suggested as follows:

Cohesive soil $\quad a = 0.438 \times K_{P.FWD}^{0.211}$ (7.7)

Granular soil $\quad a = 0.666 \times K_{P.FWD}^{0.191}$ (7.8)

All the geomaterials $\quad a = 0.331 \times K_{P.FWD}^{0.300}$ (7.9)

In order to validate these factors K_{30} values converted from $K_{P.FWD}$ and those obtained from actual field testing were analyzed as shown in Figure 7.20, Figure 7.21 and Figure 7.22 for cohesive soil, granular soil and all geomaterials, respectively. It was observed that the conversion factor "a" increases as $K_{P.FWD}$ value increases (or in other words as the stiffness) increases. It is considered that the conversion factor "a" and the proposed equation in this paper is adequate to estimate the K_{30} value from the $K_{P.FWD}$ value.

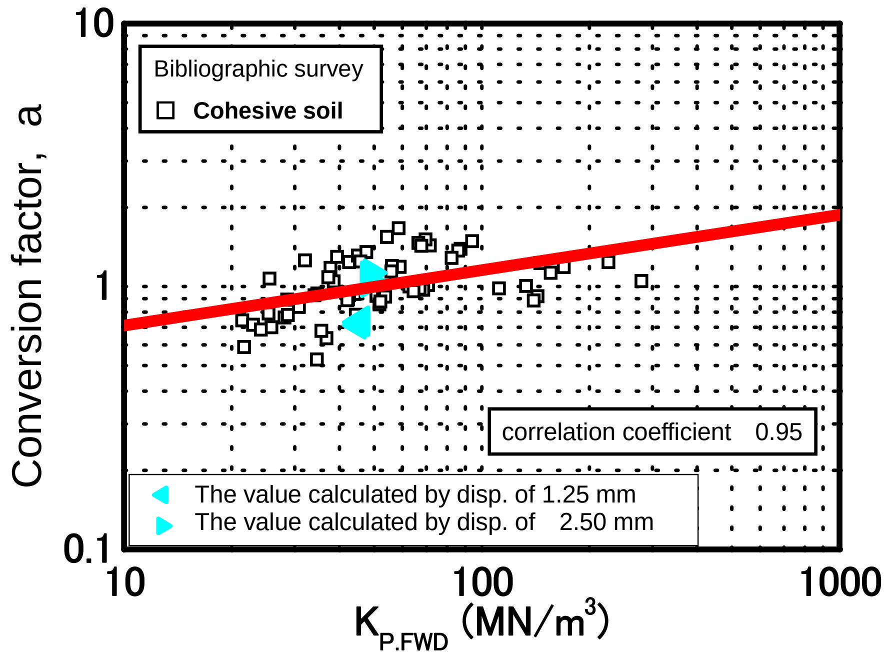

Figure 7.17 $K_{P.FWD}$ and conversion factor "a" relations (Cohesive soil)

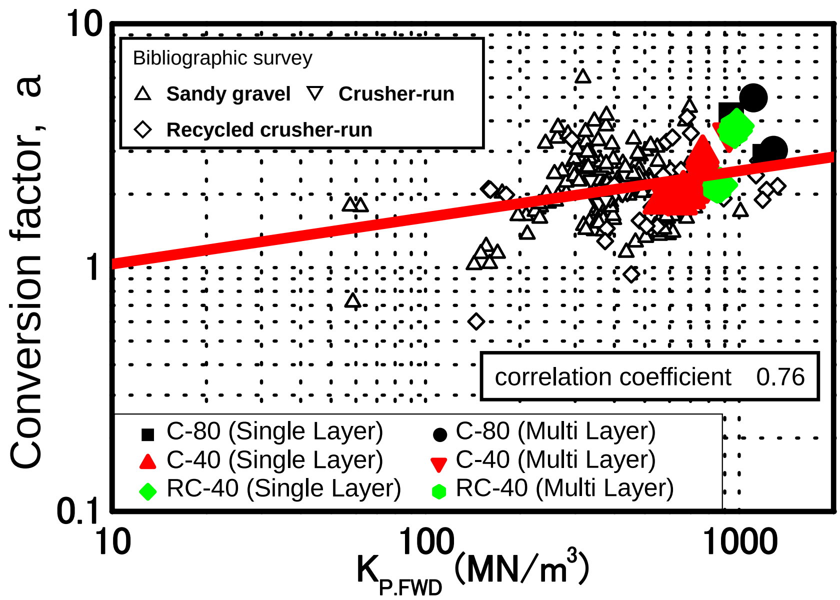

Figure 7.18 $K_{P.FWD}$ and conversion factor "a" relations (Gravelly soil)

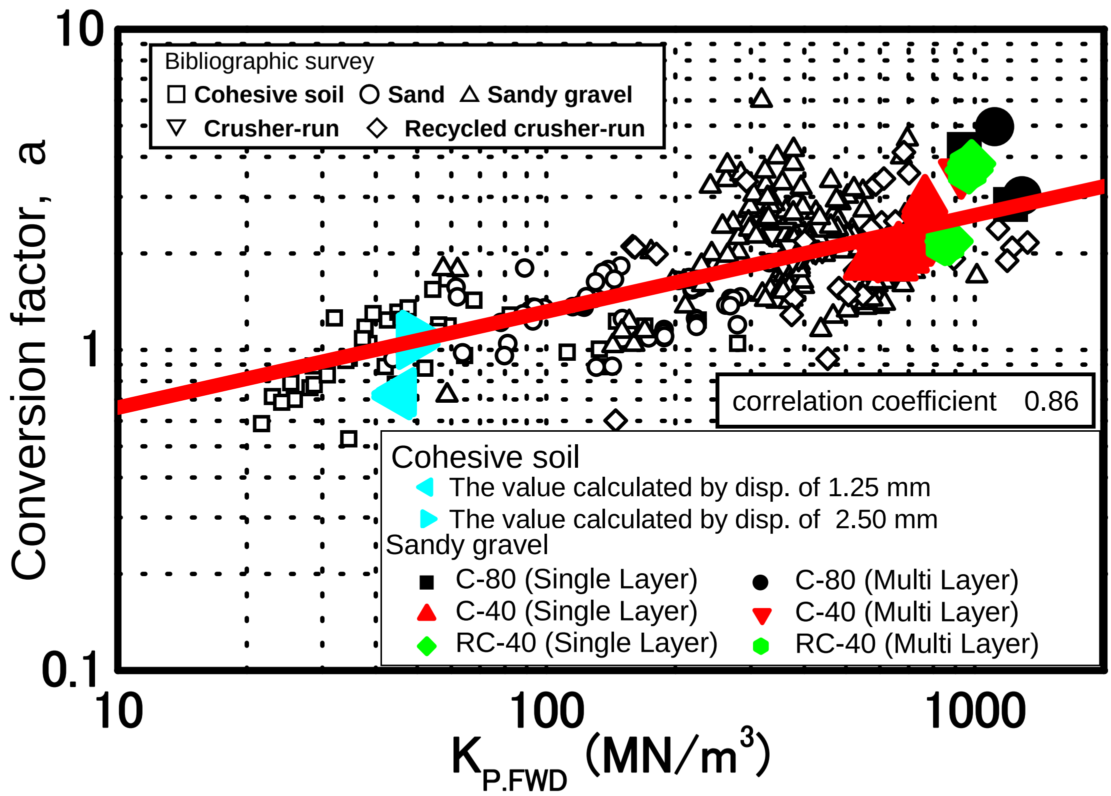

Figure 7.19 $K_{P.FWD}$ and conversion factor "a" relations (All geomaterials)

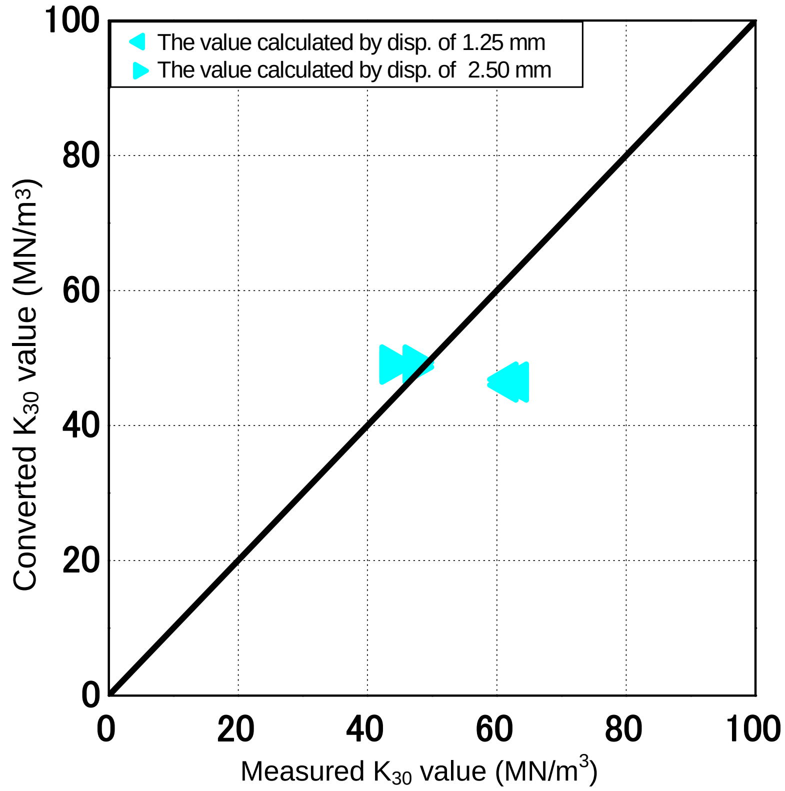

Figure 7.20 Comparison of measured K_{30} value and converted K_{30} Value (conversion by Eq. (7.7))

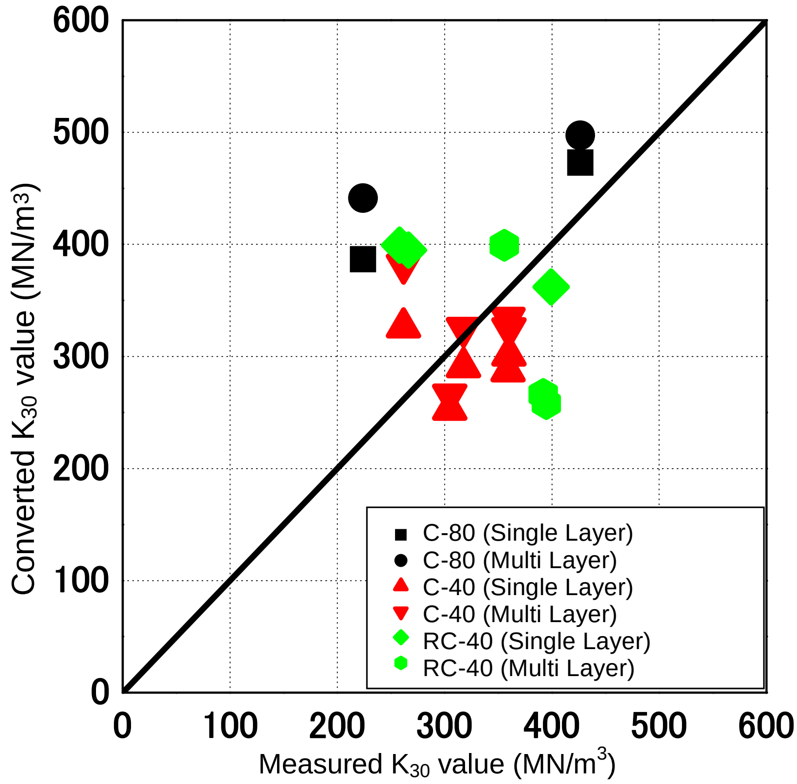

Figure 7.21 Comparison of measured K_{30} value and converted K_{30} Value (conversion by Eq. (7.8))

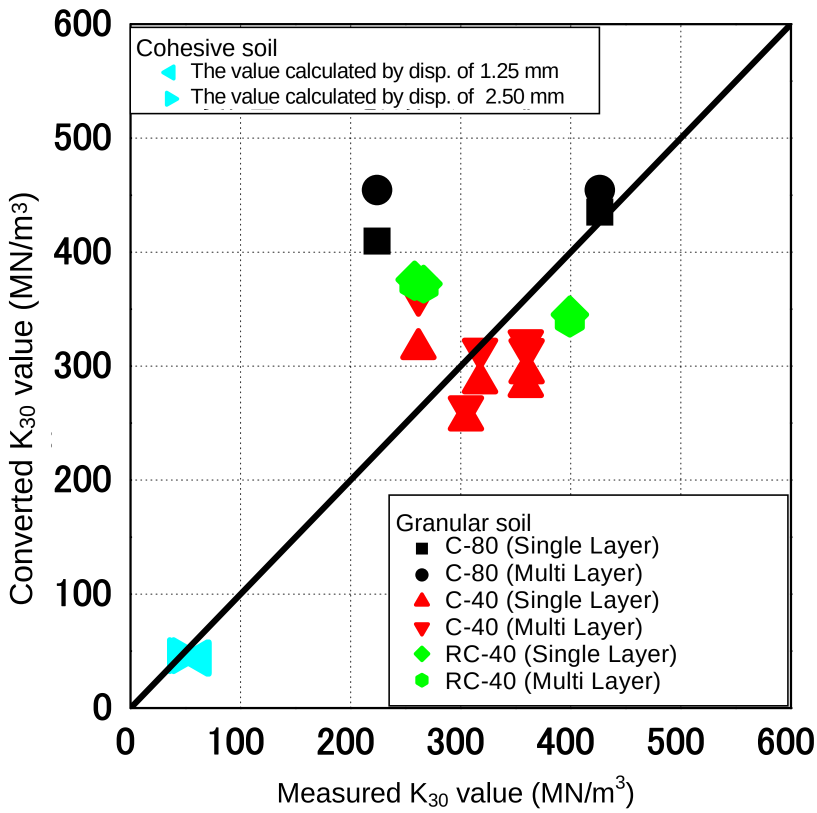

Figure 7.22 Comparison of measured K_{30} value and converted K_{30} Value (conversion by Eq. (7.9))

7.5 Summary

The following conclusions were derived from the bibliographic survey and the field investigation of various pavement geomaterials.

1) The $K_{P.FWD}$ value (by portable FWD test) is larger than the K_{30} value (by plate loading test) for granular materials. It is considered that this factor is due to a difference of diameter of loading plate and difference of extent of impact load and so on.
2) Since the scattering of $K_{P.FWD}$ value obtained from single layer analysis is large in case of subbase material (C-80) and base material (C-40), it is considered that $K_{P.FWD}$ value obtained from single layer analysis is affected by stiffness of underlying layer.
3) The relationship between $K_{P.FWD}$ and K_{30} based on the field tests is about 2:1 for base materials (well-graded gravels) and 1:1 for subbase materials (cohesive soil). However, it was shown that the $K_{P.FWD}$ value is 3 - 4 times of the K_{30} value in some gravelly soil. It is considered that this is affected by a compaction property of gravelly soil.
4) The proposed equation and the conversion factors in this paper are adequate to estimate the K_{30} value from the $K_{P.FWD}$ value.

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Chapter 8 Development of High Stability Asphalt Concrete Mixture

8.1 Introduction

The weather patterns in recent years have considerably changed due to the global warming, resulting in to increased atmospheric temperatures and severe ultra violet rays during summers. In the past one decade alone, the average atmospheric temperature and accordingly the pavement temperature have increased by 3^{o}C and 10^{o}C, respectively (Kazuyuki KUBO et. al., 2006). In order to prevent pavement deterioration on heavily trafficked and/or heavily loaded routes due to distresses such as surface rutting, shoving and raveling, various types of Polymer Modified Asphalt (PMA) pavements or semi-flexible pavements are generally constructed. Some of the PMA types used in Japan are shown in Table 8.1. However, in designated places like excessively loaded road pavement sections, sea/air port cargo terminals, heavy industrial workshops etc. where the loading and handling conditions are extremely harsh, the control of pavement surface rutting or disintegration (due to accidental fuel oil dripping from faulty equipment/vehicle etc.) remain a challenge for pavement engineers. It is therefore imperative to design asphalt mixtures which can sustain the damaging impact of severe loading conditions and are also oil proof (resistant against decomposition of asphalt binder due to fuel oils etc).

8.2 Development of HS Asphalt Concrete

8.2.1 Properties of additive agent and mixture

The high stability (HS) asphalt mixture is a hybrid asphalt mixture, developed by introducing a special thermoplastic resin to polymer modified asphalt type-II (PMA type-II) with a ratio of 20:80, respectively. The special thermoplastic resin is flaky in structure, as shown in Photo 8.1. The resin can either be added to PMA type-II at the PMA production plant or during wet mixing at the asphalt mixture plant.

Photo 8.1: Thermoplastic resin

Properties of the resin are shown in Table 8.2. As shown in this table, the softening point of the resin ranges from 118°C -127°C. By adding this material to PMA type-II, the softening point of the new binder (HS asphalt) was found to be 75.5°C compared to the average softening point of normal PMA type-II which is around 60°C, thus increasing the softening point of the binder by around 15°C; a factor which contributed to the overall increase in the viscosity of the binder and thus resistance to flow under higher temperature conditions. Similarly, marked improvement in other properties e.g. penetration resistance, ductility and toughness etc. was also achieved as shown in Table 8.3.

Table 8.1: The types of $PMA_{(s)}$ in Japan

Item	Applied mixture	Addition of SBS polymer (%)
PMA-type Ⅰ	Dense grade, Gap grade	3-4
PMA-type Ⅱ	Dense grade, Gap grade, SMA	5-6
PMA-type Ⅲ	Dense grade, PA, SMA	7-10
PMA-typeH	Porous Asphalt mixture	11-15

Table 8.2: Properties of special thermoplastic resin

Item	Thomoplastic resin (polyamide)
Softing point (°C)	118-127
Melting viscosity (mPa・s, 180°C)	210
Density (g/cm^3)	1.01

Improvement in physical properties of the binder resulted into asphalt mixture which is stable not only under severe temperature conditions but also highly resistant to disintegration caused by decomposing agents such as oil spills. To demonstrate the oil proofing ability of the HS mix, specimens were subjected to a specially designed (oil bath stability) test by the authors. In this test, the laboratory compacted specimens were soaked in kerosene oil at 20°C for 24 hours followed by marshal stability tests procedure (at 20°C) and the loss in stability was analyzed. Results of this test are shown in Table 8.4 and Photo 8.2.

Table 8.3: Binder properties of the HS and semi-flexible mixtures

Item	**Kind of mixture**	**HS Asphalt mix (20)**	**Semi flexible mixture**	**Test Method**
Item	Use of asphalt mixture	Dense graded	Open graded, Air void=24%	Japan Road Association
	Use of asphalt binder	PMA- type Ⅱ, resin	PMA- type Ⅱ	
Binder properties	Softening point (°C)	75.5	60.0	ASTM D 36
	Ductility (15°C), (cm)	54	100+	ASTM D113-99
	Penetration (25°C), (1/10mm)	35	51	ASTM D 5-97
	Toughness (25°C), (N・m)	25.1	17.4	Benson method (Benson, J.R., 1955)
	Film heating residual penetration percent (%)	82.9	80.4	ASTM D1754-97

Table 8.4: Properties of HS asphalt mixture

Physical properties of HS asphalt mixture	Optimum asphalt content (OAC)		5.8%(PMA Ⅱ =4.6%, resin=1.2%)	ASTM D 1559-89
	Marshall stability test (60°C)	Stability (kN)	27.8	
		Residual stability (%)	87.3%	
	Wheel tracking test	DS (pass/mm)	63,000	EN12697-22, JRA-B003
	Heavy load tracking	Deformation (mm)	3.25	Special test by authors
	Bending test of mixture	Bending Strength (MPa)	11.3	ASTM D 7552-09
		Bend fracture strain	6.2E-03	
	Indirect tensile test	Tensile strength (MPa)	2.5	ASTM D 7369-09
Oil resistance evaluation test at the oil bath	Marshall stability test at the oil bath (20°C)	Stability (kN)	70.8	Special test by authors
		Stability (kN), 24h oilbath	55.1	
		Residual stability (%)[1)]	77.8	
	Wheel tracking test	DS (pass/mm)	63,000	Based on EN12697-22
	Twist test at oil bath	Deformation (mm)	0.79	

1) Residual Stability is the percentage stability of the mix, retained after soaking the specimen in oil bath for 24 hours at 20^{o}C.

Similarly the dynamic stability was also verified by performing heavy wheel tracking test with special loading arrangement of steel tire weighing 175 kg (compared to standard rubber tire weighing 70 kg) at 60^{o}C, as shown in Table 8.5 and photo 8.3. A minor surface deformation of 3.25 mm was observed after the 60 minutes test, thereby confirming the high stability of the mixture against heavy loading as well.

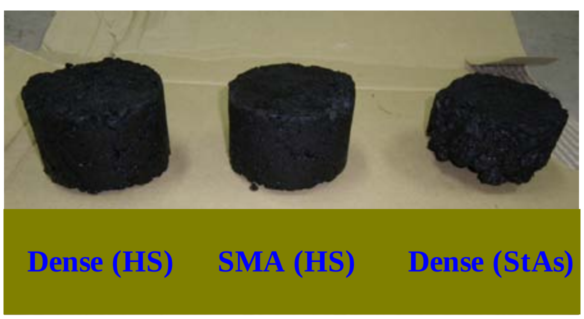

Photo 8.2: Comparison of the Marshall specimen after oil bath test

Photo 8.3: Special loading of Wheel tracking test

Table 8.5: Test conditions of Wheel tracking test

Item	Standard method	Special method
Test temperature	60°C	60°C
Kind of tire	Solid rubber tire	Steel tire
Loading weight	70 kg	175 kg
Contact pressure	630 kPa	343 N/cm
Measurement time	60 min.	60 min.

8.2.2 Characteristic of HS asphalt concrete and applicable location

Characteristic

- Performance of HS asphalt mixture with respect to stability and durability is far better than PMA mixtures.
- HS mix is oil proof and the ability to withstand the damaging effect of oil spill is comparable to semi-flexible pavement mixtures.
- The production and workability of HS mix is comparable to normal asphalt mixture, thus no special arrangement and/or equipment are required.

Applicable location

- Road pavement sections where heavy traffic or excessive wheel loading is anticipated; heavily trafficked intersections and all other road sections susceptible to surface rutting.
- Special heavy duty industrial/commercial yards with severe loading and handling conditions.
- Perpetual (Long life) pavements with extended maintenance cycles.

8.3 Results of test construction and follow-up site investigation

8.3.1 Construction planning

A test pavement section was prepared at the overlay construction site of an airport container terminal. The original work consisted of replacement of existing 50 mm-100 mm normal asphalt pavement surface with semi-flexible pavement (PMA type-II). The existing pavement had developed severe rutting and localized surface disintegration due to heavy loading (both dynamic wheel load and static top lifter load) and oil spill (from heavy vehicles, fork lifter etc), respectively. Construction of the 200 m^2 test section with HS asphalt mixture was completed on 4th January 2009 and follow up investigations were carried out after one year in service, on 11th January 2010.

Figure 8.1 shows the flow diagram of the entire process (from manufacturing to paving) adopted for the HS asphalt mixture. As shown in the figure, the special thermoplastic resin was introduced to the process during wet mixing of aggregate and PMA type-II. The minimum mixing time was set as 60 seconds so that the resin is fully dissolved in the mix. Temperature of the mix was recorded at each stage of the process, as identified by arrows on the left side of the flow diagram.

Structural sections of the test pavement and the comparative pavements are shown in Figure (8.2a) and Figure (8.2b), respectively. The grain size distribution of the aggregate used in HS asphalt mix is shown in Figure 8.3. The grain size distribution is well within the gradation envelope of the airport engineering general specifications (MLIT, CAB, 2010) and is identical to the continuous gradation suggested by Fuller (Fuller and Thompson, 1907), with the intermediate size aggregates adjusted on the finer side in order to achieve a denser mix. A comparison of the target and actual in-situ temperatures and compaction efforts are presented in Table 8.6. These results reveal that the in-situ values (from production to ultimate paving at site) were well within the tolerance limits set during laboratory mix design.

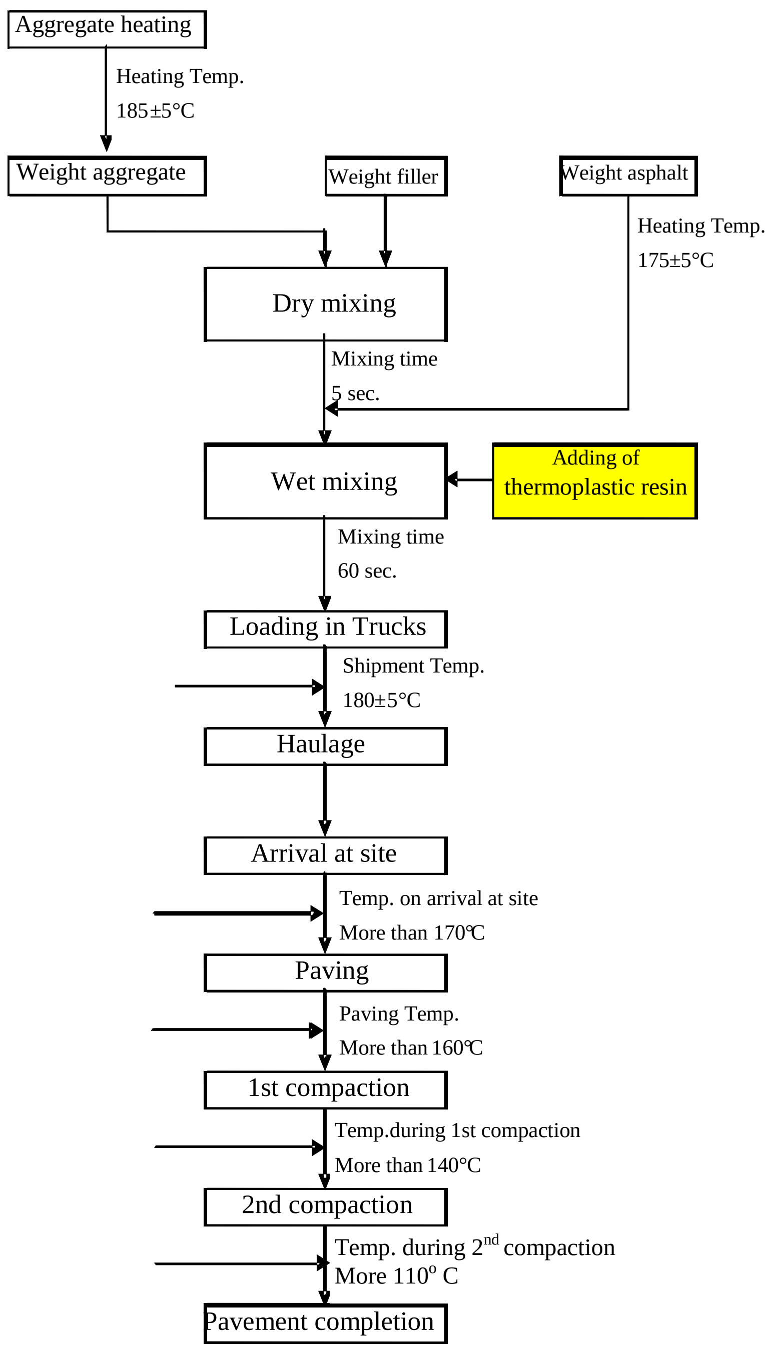

Figure 8.1: Flow diagram, from production to paving of HS asphalt mix.

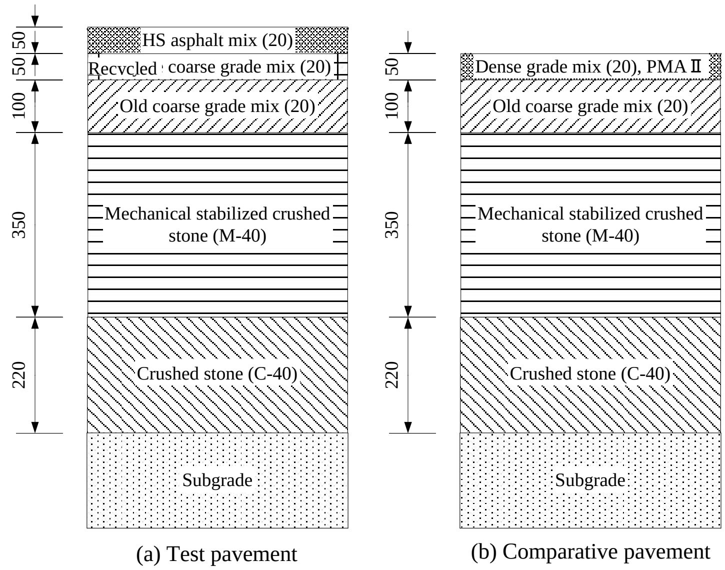

Figure 8.2: Pavement section at test construction

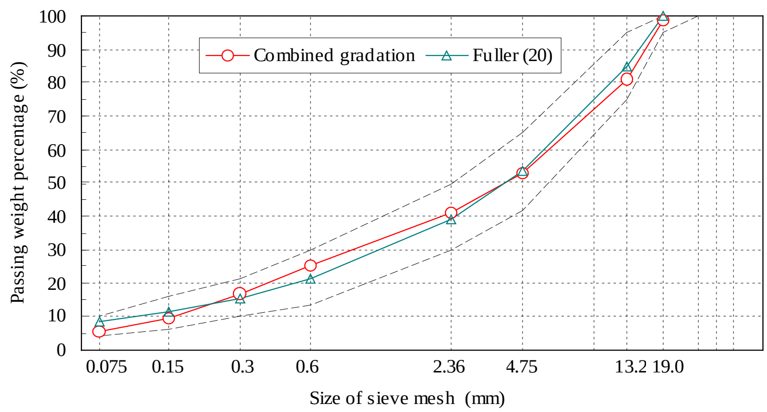

Figure 8.3: Grain size distribution for HS asphalt

Table 8.6: Target value and actual measurement of the mixture temp. and compaction efforts

Item		HS Asphalt mix (20)	In -situ measure
Producing temp. of mix ture		180±5°C	188°C
Arrival temp. of mixture		More than 170°C	185°C
Laying temp. of mixture		More than 160°C	166°C
First compaction	Tandem roller	10 passes	8 passes
	Compaction temp.	More than 140°C	143°C
Second compaction	Tire roller	4 passes	4 passes
	Co mpaction temp.	More than 110°C	114°C

8.3.2 Results of test construction

Table 8.7 shows the thickness, density and degree of compaction of the core specimens retrieved from the test section. Physical properties of the HS asphalt mixture obtained from the cores are presented in Table 8.8. The compaction required was more than 96 %, whereas the mean compaction actually achieved at site was found to be 98.4 %. Furthermore, the physical properties of the mixture obtained from the cores also corresponded well with the standard values fixed as per the laboratory mix design. Comparing the results of Tables 8.6 - 8.8, it can be deduced that the quality standards of the paved mixture can be ensured provided the temperature of the mixture during the construction process is maintained within the tolerance limits.

Table 8.7: Density and Compaction degree of core specimens

Item	Thickness (cm)	Density (g/cm^3)		Compction degree (%)
		Measurement	Standard	
Core specimens	5.65	2.342	2.377	98.5
	4.80	2.317		97.5
	5.53	2.357		99.2
Average	5.33	2.339	-	98.4
Standard	More than 4.8			More than 95

Visual inspection of the test pavement section after one week of usage by the top lifter revealed as follows:

- The pavement surface was fully intact with no signs of rutting, shoving or raveling, even at location of maximum rotational traction under the tire of top lifter (Photo 8.4).
- Slight dragging of the containers over the pavement surface during loading/unloading is a usual phenomenon at the container yards causing the surface wearing with dislodging of aggregate. However, in this case (test pavement) no such damage was observed, though the dragging marks were clearly visible.
- As expected, isolated spots of oil dripping at the pavement surface were observed. However, the surface was intact with no evidence of disintegration.

Table 8.8: Physical properties of asphalt mixture from core specimens

Item	Density (g/cm^3)		Compaction degree (%)	Air void (%)	Saturation degree (%)	Stability (kN)	Flow value (1/100mm)	Residual stability (%)
	Mean	Standard						
Marschall specimen	2.339	2.377	98.4	3.1	81.7	27.8	30	87.3
Standard Value	–	–	95 over	2～5	75～85	8.8 over	20～40	75 over

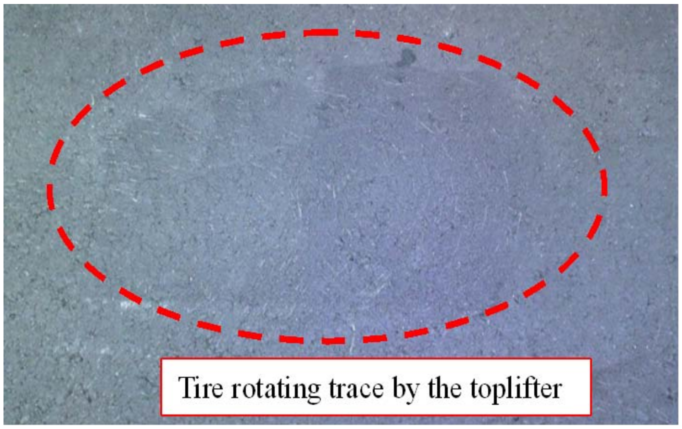

Photo 8.4: Tire rotating trace by top lifter at pavement surface

8.3.3 Results of follow-up site investigation after one year

In order to assess the in-service performance of the test pavement, follow up investigation were carried out after one year of construction. Results are summarized here bellow.

Figure 8.4 shows the comparison of the measured rut depth of both the HS asphalt and the PMA type-II pavements, recorded after one week and one year of service. It was observed that the HS asphalt pavement surface exhibited 40% less rutting compared to PMA type-II pavement, even though the grain size distribution of both the mixtures was identical.

Table 8.9 shows the deflection results of the two pavements, recorded with vehicle-mounted FWD. Corresponding air and pavement temperatures at the time of test are also shown in the table.

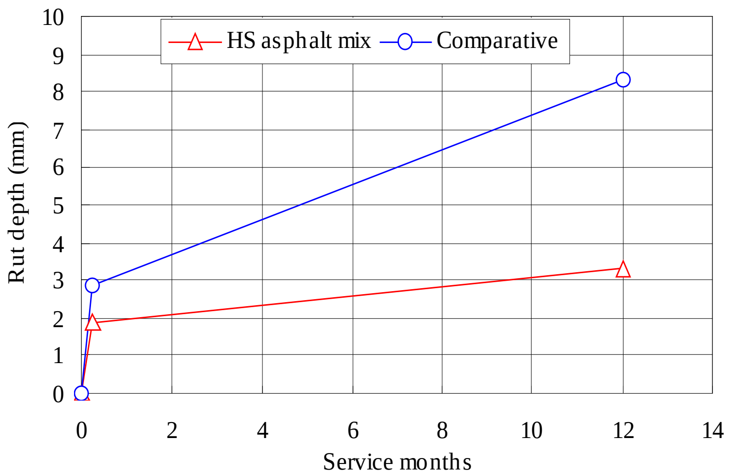

Figure 8.4: Rut depth at HS asphalt mix and comparative section

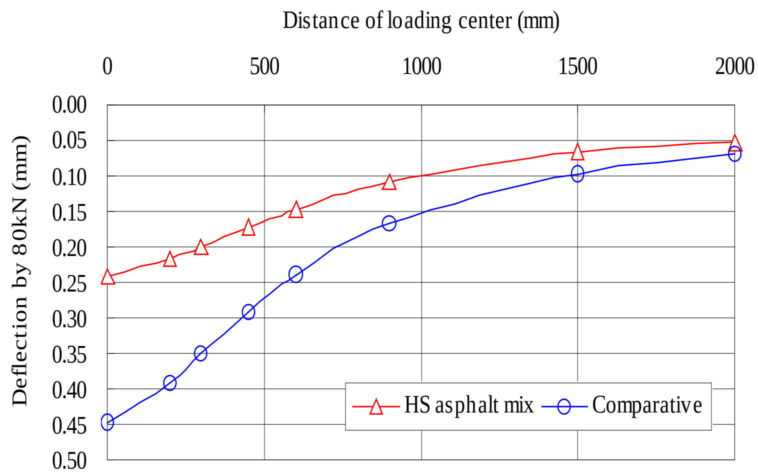

Figure 8.5: Comparison of FWD deflection curve

Comparison of the FWD deflection basins is shown in Figure 8.5. Significant difference can be observed in deflection basins of the two pavements, with HS asphalt pavement exhibiting less deflection compared to the PMA type-II pavement. This is presumably due to the increased combined thickness of the asphaltic layer in case of the test pavement which was 200 mm compared to PMA pavement, where the thickness of asphaltic layer was 150 mm.

Table 8.9: Measured FWD deflection and temperature at test pavement

Section	Load	Deflection (mm)								Temperature (°C)		
	(kN)	D0	D200	D300	D450	D600	D900	D1500	D2000	Air	Surface	ASave
HS asphalt mix	79.50	0.241	0.216	0.199	0.172	0.147	0.108	0.068	0.053	5.3	13.2	7.7
Comparative	78.50	0.447	0.392	0.350	0.291	0.240	0.167	0.097	0.069	5.3	11.2	8.0

Table 8.10: Elastic modulus of each layer using back analysis program BALM

Item	Thickness	Elastic modulus (MPa)		E as corrected at 20°C (MPa)	
	(mm)	HS asphalt mix	Comparative	HS asphalt mix	Comparative
HS asphalt mix	50	20,083	-	11,600	
Binder course	150	5,889	5,889	3,500	3,500
M-40		535	281		
C-40		322	176		
Subgrade		151	107		
Equivalent elastic modulus		8,825		5,200	

Using the deflection data, elastic moduli of each layer were worked out by the "Back Analysis of Layered elastic Moduli" (BALM) program (Matsui et. al., 2000). Comparison of the back calculated moduli for each layer of the two pavement sections is shown in Table 8.10. In case of the test pavement, the back calculated elastic modulus of 200 mm asphaltic layer was 8825 MPa (5200 MPa, corrected at 20^{o}C). Whereas the elastic modulus of the 150 mm asphaltic layer of the comparative pavement was found to be 5889 MPa (3500 MPa, corrected at 20^{o}C). The elastic modulus of the 50 mm HS asphalt layer was estimated using equation-8.1 (AI, 1991), assuming that the modulus of the underlying asphalt layer is equivalent to the modulus of a conventional asphalt concrete.

$$E_{eq} = \left(\frac{h_1 \times E_1^{\;1/3} + h_2 \times E_2^{\;1/3}}{h_1 + h_2} \right)^3 \tag{8.1}$$

Where, E_{eq}: Equivalent elastic modulus of asphalt mixture, MPa, E_1 : Elastic modulus of the first layer of asphalt mixture, MPa, E_2 : Elastic modulus of the second layer of asphalt mixture, MPa, h_1 : Thickness for the first layer of asphalt mixture, mm, h_2 : Thickness for the second layer of asphalt mixture, mm.

The estimated elastic modulus was found to be more than 20,000 MPa (11,600 MPa, corrected at 20^{o}C). Whereas the range of 20^{o}C corrected modulus of semi-flexible pavements is 9000-12000 MPa. Therefore it is considered that the stiffness of HS asphalt is comparable to semi-flexible mixture and that the difference in deflection basins was not only due to the increased thickness but also due to the high stiffness of the HS asphalt concrete layer.

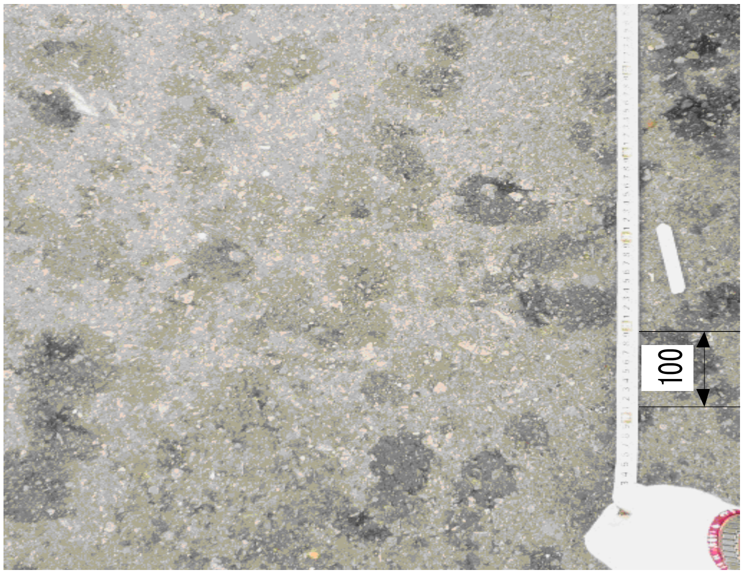

Photo 8.5: Pavement surface with Oil drippings

In order to reconfirm the oil proofing ability of the HS mixture, oil bath stability tests were conducted on core specimens. The stability was found to be 59.2 kN. It is to be mentioned that the oil bath stability of the laboratory specimen was 55.1 kN (Table 8.4), which suggest that the oil proofing ability of HS asphalt mixture did not decline during in-service conditions. This is also evident in Photo 8.5, showing an isolated spot of the pavement surface (about 8 cm in diameter) which remained exposed to oil dripping with no signs of surface distress.

8.4 Summary

Physical properties of the dense graded HS asphalt mixture (with 20 mm maximum aggregate size) and performance evaluation of a test pavement section, constructed in 2009 at an airport cargo container yard (with top lifter FD-420) were examined in this chapter. Conclusions may be summarized as follows:

1) Laboratory test results for standard marshal stability, dynamic stability, bending strength, flexural breaking strain and indirect tensile strength have shown that HS asphalt mixture has better characteristics compared to semi-flexible pavement mixtures.
2) The quality standards of HS asphalt pavement with respect to high stability and durability can be ensured; provided the temperature of the mixture during the production and construction processes is maintained within the tolerance limits.
3) The test pavement section after one year of service was found to be fully intact with no fracture, distortion or disintegration; even under severe loading and maneuvering conditions of the top lifter; thereby confirming the in-service performance of the HS asphalt mixture.
4) The back-calculated modulus of HS asphalt mixture layer was found to be more than 11,000 MPa (corrected at 20^{o}C), thereby exhibiting the high stiffness of the mixture as well.
5) HS asphalt mixture is oil proof and the ability to withstand the damaging effect of oil spill is comparable to semi-flexible pavement mixtures.

Based on the above mentioned results, it is considered that the performance of HS asphalt mixture is most suitable for pavement sections where severe loading (traffic and/or climate) conditions are anticipated (for example heavily trafficked/loaded road section, special industrial/commercial yards, sea/air port container terminals etc.) and for long life pavements as well.

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Chapter 9 Conclusions and Recommendations

Road transport has a significant contribution in the development process of emerging nations, including Pakistan. It plays a pivotal role in cost effective transportation of freight and passengers and provides equal opportunities of access to jobs and trade to all segments of society, thus promoting national cohesion and alleviation of poverty; which are the issues shared by almost all the developing nations around the world. Consequently, there has been a phenomenal shift towards road transportation options during the past few decades. Due to this unprecedented shift coupled with a general tendency of truck overloading in Pakistan, in recent years the newly constructed as well as rehabilitated pavements have shown accelerated deterioration and premature failure causing not only waste of public money but also safety hazard and inconvenience to the road users.

Further the societies in the developed as well as emerging countries now demand not just a road facility but also enhanced serviceability with respect to riding quality and longevity with respect to pavement structural stability. On the other hand there is a worldwide tendency (particularly in developing countries such as Pakistan) of funds constraints for construction of new and maintenance of large portfolios of existing ageing road networks. Therefore pavement researchers are concentrating more towards analytical and coherent design systems to respond to those demands, and for optimal use of resources.

Flexible pavements with hot mix asphalt concrete (AC) surfacing constitutes majority of road pavements around the world predominantly due to its low initial/maintenance cost, easy and quick construction, superior riding quality and skid resistance, etc. The design and performance considerations of flexible pavements includes a multitude of potentially influencing variables including the complex behavior of pavement geomaterials in each layer; resting on a variety of subgrade soils; dynamic traffic loads and an array of climatic conditions.

This research explores the current situation of flexible pavements in Pakistan with a view to suggesting practical proposals towards Mechanistic-Empirical pavement design system in Pakistan.

9.1 Conclusions of the present study

The following conclusions were obtained from the results presented in this thesis.

(1) Applicability of Mechanistic-Empirical design system for flexible pavement structures in Pakistan

The damaging influences due to excessive axle loading tire inflation pressure, seasonal variation in climate and changes in stiffness of unbound pavement material and subgrade soil on flexible pavements were analyzed; using the conditions prevailing in Pakistan as case study. Mechanistic-Empirical pavement design approach was utilized for the study. Results are summarized as follows:

a) The Mechanistic-Empirical (M-E) Pavement design approach, realistically capture the variations in all variables that influence the pavement performance, resulting in rational pavement design.
b) The performance of flexible pavement is sensitive to not only axle loading but also significantly to tire inflation pressure. The damaging influence of increase in tire pressure keeps on magnifying with each axle load increment.
c) The Design RDF for the Legal axle load (118 kN) and the mean observed axle load (145kN) on single axle with dual tires; with mean observed tire pressure of 896 kPa, was noted to be 5.80 and 11.95, respectively.
d) The flexible pavement performance against distresses such as fatigue cracking and rutting is significantly affected by the stiffness of unbound base course and subgrade, respectively.
e) Consideration of climate regime in the design affects the economy of pavement design. Considering a single value for air/pavement temperature, e.g. mean annual temperature or extreme temperature condition, may result in extravagant pavement design.

REMARKS

The damage factors derived in this study can be readily used for network level pavement management; even with the current empirical design system (when the detail axle load data is time and/or cost intensive). However, the analysis and design approach used in this study itself can be adopted as a starting point towards Mechanistic-Empirical pavement design system in Pakistan.

(2) The time dependent mechanical behavior of unbound pavement materials

Time-dependency in the context of geo-materials refer to the effects caused by two factors: *i) Ageing effect* (cementation and weathering) i.e. changes with time in the intrinsic properties like strength, stiffness, deformation etc. due to change in interface and/or internal particle properties caused by physico-chemical processes and *ii) Loading rate effect* (viscous sliding at inter particle contacts). Test method and results of triaxial compression tests with monotonic loading on granular material under various loading conditions has been discussed in chapter 6. The following conclusions were drawn:

a) The influence of viscous effect on the strength of uniformly-graded crushed gravel was not observed in both cases; with and without cyclic loading history.

Therefore the time-dependency effect on the strength properties is considered to be minimal.

b) Noticeable influence of viscous effect was observed on the deformation characteristics of uniformly-graded crushed gravel, which reveal that the time-dependency has considerable effect on the deformation properties of these materials. Moreover, the effect is more evident in materials with cyclic loading history.
c) The influence of loading rate and loading acceleration on ε_{vol} - ε_a behavior was not observed, which transpires that time-dependency has minimal effect on dilatancy characteristics of these materials.

(3) Evaluation of physical properties of unbound pavement layers and subgrade by portable FWD (P.FWD).

The relationship between $K_{P.FWD}$ and K_{30} (based on conventional plate loading test), established as a result of comprehensive field testing and available data from the past research, has been discussed in chapter 7. The following conclusions were derived from the bibliographic survey and the field investigation.

a) The $K_{P.FWD}$ value (by portable FWD test) is larger than the K_{30} value (by plate loading test) for granular materials. It is considered that this factor is due to a difference of diameter of loading plate and difference of extent of impact load and so on.
b) Since the scattering of $K_{P.FWD}$ values obtained from single layer analysis is large in case of subbase material (C-80) and base material (C-40), it is considered that $K_{P.FWD}$ value obtained from single layer analysis is affected by stiffness of underlying layer.
c) The relationship between $K_{P.FWD}$ and K_{30} based on the field tests is about 2:1 for base materials (well-graded gravels) and 1:1 for subbase materials (cohesive soil). However, it was shown that the $K_{P.FWD}$ value is 3 - 4 times of the K_{30} value in some gravelly soil. It is considered that this is affected by a compaction property of gravelly soil.
d) The proposed equation and the conversion factors in this paper are adequate to estimate the K_{30} value from the $K_{P.FWD}$ value.

(4) Development and performance of high stability Asphalt Concrete (AC).

Development of a High Stability (HS) AC mixture and its in-service performance based on tests results of laboratory compacted mixture and core specimens of a test pavement section was discussed in chapter 8. Results are concluded as below:

a) Laboratory test results for standard marshal stability, dynamic stability, bending strength, flexural breaking strain and indirect tensile strength have shown that HS asphalt mixture has better characteristics compared to semi-flexible pavement mixtures.
b) The quality standards of HS asphalt pavement with respect to high stability and durability can be ensured; provided the temperature of the mixture during the production and construction processes is maintained within the tolerance limits.
c) The test pavement section after one year of service was found to be fully intact with no fracture, distortion or disintegration; even under severe loading and maneuvering conditions of the top lifter; thereby confirming the in-service performance of the HS asphalt mixture.
d) The back-calculated modulus of HS asphalt mixture layer was found to be more than 11,000 MPa (corrected at 20^{o}C), thereby exhibiting the high stiffness of the mixture as well.
e) HS asphalt mixture is oil proof and the ability to withstand the damaging effect of oil spill is comparable to semi-flexible pavement mixtures.

9.2 Future research

Based on the findings and conclusions of this study, the following recommendations are made:

1. The generalized transfer functions suggested in this study (chapter 5) need to be carefully calibrated to country specific conditions (traffic, materials and climate) through long term performance studies for a developing M-E design systems for project level designs of flexible pavements Pakistan.
2. Similar research with respect to time dependent strength and deformation properties of unbound pavement materials of various blends, used in flexible pavement structures is needed.
3. Damage analysis due to excessive loading and tire inflation pressure on multi-axle trucks is also required for a complete picture of mixed heavy traffic impact on flexible pavements in Pakistan.
4. Long-term monitoring of the performance of the test pavement section with HS asphalt mixture is needed.