EXPERIMENTAL INVESTIGATION OF SUPERPAVE AND CEMENT TREATED AGGREGATE BASE MIXTURES FOR LONG LIFE ASPHALT PAVEMENTS

Thesis

Submitted in partial fulfillment of the requirements for the degree of

DOCTOR OF PHILOSOPHY

by

PRIYANKA B A

DEPARTMENT OF CIVIL ENGINEERING

NATIONAL INSTITUTE OF TECHNOLOGY KARNATAKA

SURATHKAL, MANGALORE -575 025

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DECLARATION

By the Ph.D Scholar

I hereby declare that the Research Thesis entitled **"Experimental Investigation of Superpave and Cement Treated Aggregate Base Mixtures for Long Life Asphalt Pavements"** which is being submitted to the National Institute of Technology Karnataka, Surathkal in partial fulfillment of the requirements for the award of the degree of **Doctor of Philosophy** in **Civil Engineering**, *is a bona fide report of the research work carried out by me*. The material contained in this Research Thesis has not been submitted to any University or Institution for the award of any degree.

(PRIYANKA B A)
Register No. 145003CV14F06
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Place: NITK Surathkal
Date: 14-02-2019

CERTIFICATE

This is to certify that the Research Thesis entitled **“Experimental Investigation of Superpave and Cement Treated Aggregate Base Mixtures for Long Life Asphalt Pavements”** *submitted by* **Priyanka B A** (Register Number: **145003CV14F06**) as the record of research work carried out by her, is accepted as the Research Thesis submission in partial fulfillment of the requirements for the award of the degree of **Doctor of Philosophy.**

Dr. A.U Ravi Shankar
Research Guide
(Signature with date and seal)

Chairman-DRPC
(Signature with date and seal)

DEDICATED

TO

MY PARENTS,

FAMILY MEMBERS, FRIENDS

AND

TEACHERS

ACKNOWLEDGEMENT

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ABSTRACT

Early deterioration of flexible pavements, due to increased traffic volume, environmental conditions, poor maintenance, and construction quality causes difficulties to road users, all around the world. The structural failures such as fatigue and rutting demand the reconstruction of the pavements which further leads to significant construction cost. One potentially sustainable solution to this problem is to adopt Long Life Asphalt Pavement (LLAP) technology. The fatigue and rutting distresses in the pavements can be minimized to some extent by utilizing Superpave and cement treated aggregate base mixtures with LLAP concept. The LLAPs are designed in such a manner that the response of the pavements to loads (particularly strains) is kept below certain threshold levels.

In the current study two types of Superpave mixtures were prepared, one with Optimum Binder Content (OBC) designed at 4% air voids (Optimum mixtures) and the other with extra binder content of +0.5% over the OBC (Rich mixtures), for asphalt intermediate and base layers of LLAP respectively. The optimum mixtures were prepared with two aggregate gradations having two Nominal Maximum Aggregate Sizes (NMAS), 25mm and 19mm named as SP1 and SP2 respectively, for intermediate layers to enhance the rutting resistance. Rich mixtures were prepared with the same aggregate gradations for asphalt base layer to improve the fatigue resistance. Viscosity Graded (VG) 30 asphalt, Crumb Rubber Modified Binder (CRMB) of grade 60 and Polymer Modified Binder (PMB) of grade 40 were used as binders. The specimens were prepared as per Superpave mix design and were compacted in Superpave Gyratory Compactor (SGC). The performance of these mixtures was assessed in the laboratory through volumetric properties, Indirect Tensile (IDT) strength, rutting resistance, fatigue behavior, resilient modulus, and moisture susceptibility characteristics. In general, mixes with PMB 40, showed better properties. In case of IDT strength, rutting resistance, resilient modulus and ITS tests, optimum mixtures performed better compared to rich binder mixtures. However, in case of fatigue behaviour and moisture susceptibility tests, rich binder mixtures performed better compared to optimum mixtures. For all mixture types, SP1 gradation showed better results than SP2, except for moisture susceptibility, in which both gradations performed almost the same.

Cement Treated Aggregate (CTA) mixtures were also prepared with two aggregate gradations having two NMAS, 37.5mm and 45mm named as CTA1 and CTA2 respectively, for base course of LLAP to enhance the structural capacity with increased stiffness. Cement contents of 3, 5 and 7 % were used in the mixtures, and the modified compaction test was carried out to prepare specimens at their respective Optimum Moisture Content (OMC) and Maximum Dry Density (MDD). The performance of these mixtures was evaluated in laboratory through compressive strength, flexural strength, split tensile strength, modulus of elasticity and flexural fatigue behavior. The experimental investigations indicate that all the mixtures satisfied the 7-day compressive strength and 28-day flexural strength requirements as specified by Indian Roads Congress (IRC) for flexible pavement design. For all mixture types, CTA1 gradation showed better results than CTA2.

The fatigue and rutting criteria of pavement sections proposed in the study were evaluated using KENPAVE software. In the analysis mainly eight pavement sections (denoted as S1, S2, S3, S4, S5, S6, S7, and S8) with different combinations of layers and materials were considered. The thickness of the layers in these sections was decided to obtain critical strains within permissible limits (tensile strain < 70 micro strain and compressive strain < 200 micro strain) and were chosen using trial and error method. The sections were divided on the basis of the mixtures used in asphalt intermediate and base layer and base course. From the results it was observed that, in case of SASW load, the critical strains were found to be within limits for pavement sections S2, S3, S7 and S8. The experimental results and analysis on pavement sections with proposed mixtures for intermediate and base asphalt layers and base course show that they can be considered as a better alternative for conventional pavements.

Keywords: Long life asphalt pavement, Superpave, modified binder, cement treated aggregate, KENPAVE.

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NOMENCLATURE

AASHTO	Association of State Highway and Transportation Officials
AC	Asphalt Content
AI	Asphalt Institute
APA	Asphalt Pavement Alliance
ARZ	Above Restricted Zone
ASTM	American Society for Testing and Materials
BRZ	Below Restricted Zone
CBR	California Bearing Ratio
CRMB	Crumb Rubber Modified Binder
CTA	Cement Treated Aggregate
CTB	Cement Treated Base
DR	Damage Ratio
FL	Fatigue Life
G_b	Specific gravity of asphalt
G_{mb}	Bulk density of compacted specimen
G_{mm}	Maximum theoretical density of the mixture
G_{sb}	Bulk specific gravity of aggregates
G_{se}	Effective specific gravity of aggregates
GB	Granular Base
GSB	Granular Sub Base
H	Depth
HMA	Hot Mix Asphalt
IDT strength	Indirect Tensile strength
IRC	Indian Roads Congress

ITS	Indirect Tensile Strength
LLAPs	Long Life Asphalt Pavements
LVDT	Linear Variable Deflection Transducer
MAS	Maximum Aggregate Size
MDD	Maximum Dry Density
MoRT&H	Ministry of Road Transport and Highways
M_R	Resilient Modulus
msa	million standard axles
N_f	Fatigue Life
N_{ini}	Initial number of gyrations
N_{des}	Design number of gyrations
N_{max}	Maximum number of gyrations
NMAS	Nominal Maximum Aggregate Size
N_R	Rutting Life
OAC	Optimum Asphalt Contents
OBC	Optimum Binder Content
OMC	Optimum Moisture Content
OPC	Ordinary Portland Cement
P_b	Asphalt content percentage by total weight of mixture
PG	Performance Grade
PMB	Polymer Modified Binder
P_{mm}	Percentage by weight of total loose mixture
P_s	Aggregate content, per cent by total weight of mixture
RAP	Recycled Asphalt Pavement
RBL	Rich Binder Layer

RBM	Rich Binder Mixture
RZ	Restricted Zone
SADW	Single Axle Dual Wheel
SASW	Single Axle Single Wheel
SGC	Superpave Gyratory Compactor
SMA	Stone Matrix Asphalt
SP	Superpave
TRZ	Through Restricted Zone
TSR	Tensile Strength Ratio
V_a	Air Voids in Total Mix
VFA	Voids Filled with Asphalt
VG	Viscosity Graded
VMA	Voids in Mineral Aggregates
W_a	Weight of specimen in air
WBM	Water Bound Macadam
WMM	Wet Mix Macadam
WRT	Wheel Rut Tester
W_{ssd}	Saturated Surface Dry (SSD) weight of specimen
W_w	Weight of specimen in water

CHAPTER 1

INTRODUCTION

1.1 GENERAL

Transportation plays a very important role in the socio-economic development of a country. Road transportation is generally the most effective and preferred mode of transport, for both freight and passenger movement, due to easy accessibility and adaptability to individual needs. India has one of the largest road networks of over 54.83 lakh km in the form of National Highways, Expressways, State Highways, Major District Roads, Other District Roads and Village Roads, which carry about 85% of the passenger traffic and 60% of the freight traffic (Ministry of Road Transport and Highways (MoRT&H), Annual Report 2017-2018).

Based on the materials and layers used, pavements are mainly classified as flexible, rigid and composite types. Most of the Indian roads are flexible types with sub base, base and surface course over the compacted subgrade layer. The conventional Hot Mix Asphalt (HMA) mixtures with dense aggregate gradation are generally used in the pavement surface course. The Water Bound Macadam (WBM) or Wet Mix Macadam (WMM) are used as base and sub-base course, both of which are good load distributing layers with sufficient material properties to transfer the loads coming from the top layers. The roads in India are performing poorly with pavement life much less than the expected life (Basu et al. 2013). The life of the pavement is associated with factors such as design, construction quality, material types, traffic volume, axle load characteristics, environmental conditions and the maintenance. The increasing heavy axle loads and high volume of vehicular traffic on Indian highways have damaged existing arterial road network and heavy investments are needed for restoring it to a desired serviceability level. The repeated application of traffic loads and climatic factors such as temperature variation and moisture are causing premature structural distress to asphalt pavements in the form of fatigue cracking, rutting along wheel tracks, raveling and potholes (Kumar et al.2006). This early pavement deterioration needs maintenance, rehabilitation and sometimes reconstruction due to cumulative damage occurred in the asphalt layers, causing inconvenience to the road users along with higher maintenance and rehabilitation cost. Further, the demand for increased road length and the reconstruction and maintenance of pavements have also resulted in the fast depletion of naturally available road materials. Gradually decreasing resources and increasing cost of new construction and rehabilitation are motivating the highway agencies and concessionaires to step beyond the conventional methods and look for value engineered options for pavement type selection involving alternative design, and pavement materials, and construction of pavements with extended life.

The longer service life of the pavement holds a direct relationship with the fatigue and rutting resistance (Newcomb et al. 2001, Al-Khateeb and Basheer 2009, Sidess and Uzan 2009, Cao et al. 2016). The tensile strain and the vertical compressive strain experienced in the pavement critically affect the fatigue and rutting resistance. The long life can be achieved in the pavement structure by keeping these two strain values within the allowable limits. Considering the above factors, the Long Life Asphalt Pavement (LLAP) structure serves as an effective solution for extending the life of the Indian highways. In many countries including United States, LLAP is known as "Perpetual Pavement" and is a proven technology with performance period above 50 years (Asphalt Pavement Alliance (APA) 2002).

1.2 LONG LIFE ASPHALT PAVEMENT

The concept of LLAP was first launched in 2000, by the APA in a joint promotional effort with Asphalt Institute, National Asphalt Pavement Association, and the State Asphalt Pavement Associations of USA (APA 2002). The concept is considered as the long-term solution for increased traffic volumes and related pavement rehabilitation and user delay costs, and it can be applied to any pavement structure. However, these pavements are more important on high volume roads and on urban areas where new roads are being built. A long life pavement is a well-designed and well-constructed pavement where, the structural elements last indefinitely provided that the designed maximum individual load and environmental conditions are not exceeded and that appropriate and timely surface maintenance is carried out (European National Highway Research Laboratories 2009). The primary purpose of long life pavements is to resist bottom-up fatigue cracking in the HMA layers and to resist rutting of the subgrade. If properly rehabilitated through periodic repair for surface distress, these pavements present no major structural failures. They are very appealing alternative to concrete pavements, especially for large metropolitan areas (Merrill et al. 2006, Scholz et al. 2006, Sinha 2008, Tarefder and Bateman 2012).

1.2.1 Structure of Long Life Asphalt Pavement

LLAPs are comprised of HMA layers over a pavement foundation. The basic concept of a long life pavement is shown in Figure 1.1.

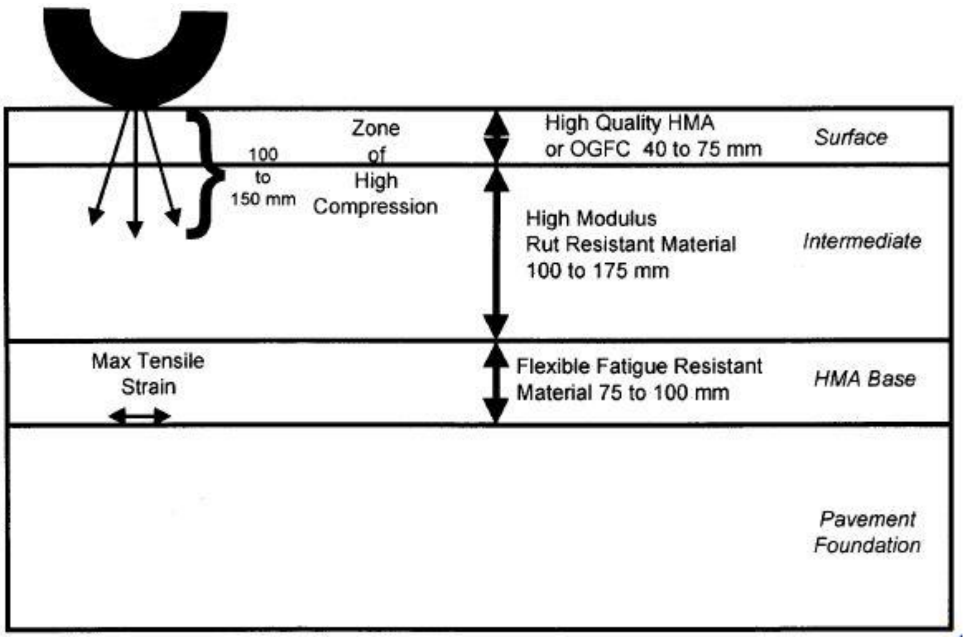


Fig. 1.1 Long Life Asphalt Pavement Design Concept

(Source: APA 2002, Newcomb et al. 2010)

The three structural layers of HMA present in the LLAP are engineered to withstand the distresses associated with its location. The asphalt layer of the LLAP consists of an impermeable, wear resistant and renewable surface course, a rut resistant and durable intermediate course and a fatigue resistant and durable base course which is also known as Rich Binder Layer (RBL**)** (APA 2002). In LLAP, the structural distresses are confined to the top layer and if the distresses exceed the acceptable limits, only periodic renewal is sufficient to regain the structure. The HMA surface layer is supposed to provide comfort, durability, stability, skid resistance, noise reduction, surface water drainage etc. The intermediate layer of HMA is designed specifically to carry most of the traffic load, and therefore, it must be rut-resistant and durable. The base layer of HMA is required to resist fatigue cracking which is generally achieved in two approaches. First, the total pavement thickness can be made great enough such that the tensile strain at the bottom of the base layer is insignificant. Alternatively, the HMA base layer can be made extra flexible by increasing the binder content (Tarefder and Bateman 2012**)**. These three HMA layers are generally constructed on a solid foundation which may consist of base course, sub base course and subgrade. The layer thicknesses are generally variable depending on the traffic loading, and materials/mix-designs. The LLAPs are designed in such a manner that the response of the pavements to loads (particularly strains) is kept below certain threshold levels. To achieve LLAP structures with improved performance, the selection of proper materials for each of the surface course, intermediate course, and base course is crucial. Most of the researchers used Stone Matrix Asphalt (SMA) mixtures in the surface layer and Superpave mixtures in the intermediate and asphalt base layers of LLAP, because of their better performance. In addition, cement treated materials can also be used in base and sub base layer of LLAPs which provide stable support that is necessary for the high traffic volumes (and even overloading) typically seen India (Cao et al. 2016).

1.3 SUPERIOR PERFORMING ASPHALT PAVEMENT

Marshall method was commonly adopted by many countries and researchers, to design asphalt mixtures, and is still the mostly used method in India. In this method, a fixed number of Marshall hammer blows are provided as the compactive effort on either sides of the specimen and this is generally 75 and 50 blows for dense graded and gap graded mixtures respectively. Any mix design procedure functions based on the assumption that, the density achieved in the laboratory due to the applied compactive effort is equivalent to the density in the field. In field generally compaction is attained during the construction phase (primary compaction) and also due to the compaction by moving traffic loads (secondary compaction), and this leads to a significantly higher field density, compared to that obtained in laboratory. The increase in Marshall compactive effort to achieve an increased density is not a suitable solution since it increases the breakdown of aggregates. The concept of developing a laboratory asphalt mix design procedure, to yield approximately the same density as expected in field, with minimum aggregate breakdown was the primary driving force for the development of gyratory compactors. Due to drawbacks of Marshall mix design Strategic Highway Research Program (SHRP) established a concept for the design of asphalt mixes referred to as Superior Performing Asphalt Pavements (Superpave). The Superpave mixes have been widely used by developed countries over the last few years. They are currently being implemented by European Union, Japan and South Korea but the developing countries are still working with the conventional mixes, that are Marshall mixes (Swami et al. 2004). This technology has a tremendous potential to be implemented in India, which will play for itself with higher performance and longer lasting roads (Swami et al. 2004). The Superpave is a comprehensive asphalt mixture design system intended to ensure good field performance of long lasting asphalt pavements under various traffic loading and climatic conditions (Sargand and Kim 2003). One of the key features in Superpave mix design is the use of Superpave Gyratory Compactor (SGC) to compact the specimen in laboratory. The SGC has new operational characteristics and can provide information about the compactability of the particular mixture by capturing data during compaction.

1.4 CEMENT TREATED AGGREGATE BASE

The pavement performance depends primarily on the properties of each constituent material, and these properties can be improved by suitable stabilization techniques (Nusit et al. 2015). In order to enhance the structural capacity of pavements, cement treated materials are used nowadays in different layers (George 1990). Generally, Cement Treated Aggregates (CTAs), cement treated soil, cement treated soil aggregate mixture etc. can be used in the base or sub-base layers. Among all these materials, CTAs are expected to perform well because of the higher cement and aggregate content in them (Lim and Zollinger 2003). Portland Cement Association (1979) defines “Cement Treated Aggregate (CTA) as a mixture of aggregate material and measured amount of Portland cement and water that hardens after compaction and curing to form a durable paving material”. The reduced stress on the subgrade, improved load carrying capacity and additional strength and support without any increase in the total pavement thickness, are significant merits of CTA mixtures (Ismail et al. 2014). Furthermore, cement treated materials in base and sub-base layers of pavements increase the stiffness of layers, which enhances the fatigue behaviour of asphalt layers, and reduces the rutting in subgrade (Saxena et al. 2010). The Chinese pavement experts designed LLAPs with cement treated base layers and found that the use of cement stabilized materials as a base layer increases service life, structural capacity and reduces the need of a thick asphalt layer and contributes to the preservation of the environment and the natural resources (Sultan and Guo 2016).

1.5 NEED AND SIGNIFICANCE OF PRESENT INVESTIGATION

As India is attaining greater modernization, the number of vehicles on the road is increasing significantly. This is imposing severe distress on the roads in the form of increased fatigue cracking and rutting, which directly increases the maintenance cost and resource consumption. LLAPs can improve this situation as they are capable of maintaining the pavement performance for longer periods without requiring major structural rehabilitation. Even though the concept is established in some of the developed countries (like USA etc.) in many parts of the world the idea has not yet been tried, especially the detailed laboratory investigation of the mixtures involved. Scientific investigation is necessary for the better understanding of implementing LLAP design.

1.6 OBJECTIVES AND SCOPE OF THE PRESENT STUDY

The current study is aimed to prepare optimum and rich Superpave mixtures of two aggregate gradations using different types of binders applicable to the asphalt intermediate and base layers for LLAP. Similarly, cement treated aggregate mixtures of two aggregate gradations using different percentages of cement contents applicable to base course for LLAP. Various laboratory tests have been conducted on the prepared Superpave and cement treated aggregate base mixtures to evaluate the performance characteristics.

The main objective of the present research work is to design pavement sections satisfying the necessary criteria for long life pavements with different Superpave and cement treated aggregate base mixtures in asphalt and base course layers.

The scope of this study includes the review of previous research findings related to design criteria, composition and performance characteristics of LLAP, and Superpave and CTA base mixtures that can be used in different layers of LLAPs. Optimum and rich Superpave mixtures were prepared by adopting two aggregate gradations and using different asphalt binders for asphalt intermediate and base layers respectively. Optimum mixtures were prepared with Optimum Binder Content (OBC) designed at 4% air voids, whereas rich mixtures were prepared by adding extra binder content of +0.5% over the OBC. Aggregate gradations with Nominal Maximum Aggregate Size (NMAS) 25mm and 19mm were adopted for Superpave mixtures in the current study. A conventional asphalt binder, Viscosity Graded (VG) 30 asphalt, which is commonly used in India in asphalt mixtures, and two modified binders, Polymer Modified Binder (PMB) 40 grade and Crumb Rubber Modified Binder (CRMB) 60 grade, were used. SGC was used to prepare cylindrical specimens for most of the tests. Volumetric properties, Indirect Tensile strength, rutting resistance, fatigue behaviour, resilient modulus and moisture susceptibility characteristics of Superpave mixtures were evaluated in the laboratory to determine performance characteristics.

CTA mixtures for base course of LLAP were prepared by considering two different aggregate gradations (NMAS 45mm and 37.5mm) and cement contents of 3, 5 and 7

% by weight of total mixture. The modified proctor tests were performed to determine the Optimum Moisture Contents (OMC), and the Maximum Dry Density (MDD) of the mixes. The properties such as compressive strength, flexural strength, split tensile strength and modulus of elasticity of different CTA mixes were evaluated as per standard test procedures. Flexural fatigue performance of CTA mixes was also determined by carrying out repeated load tests on beam specimens using repeated load testing equipment. The fatigue life data obtained are represented and analyzed using S-N curves to establish fatigue equations. Furthermore, the application of Superpave mixtures and CTA mixtures in the prescribed layers of pavement sections proposed in the study were analyzed using KENPAVE software and the responses were checked.

1.7 ORGANIZATION OF THE THESIS

The present work has been divided into seven chapters and compiled in this thesis for the purpose of better understanding and clarity of the proposed problem.

Chapter 1 includes importance of pavements with extended life in India. It also covers brief note on LLAP, Superpave and CTA base mixes, objectives and scope of the present work.

Chapter 2 provides the detailed review of the literature about the LLAP, which includes composition, performance characteristics, pavement analysis and design. Information gathered about the research works carried out so far on Superpave and CTA mixtures are also presented.

Chapter 3 presents the details of various materials used during laboratory investigation, aggregate gradation and the methodology adopted to prepare and test Superpave and CTA mixtures.

Chapter 4 discusses the results and discussion on volumetric properties, Indirect Tensile strength, rutting resistance, fatigue behaviour, resilient modulus and moisture susceptibility of optimum and rich Superpave mixtures prepared with conventional and modified binders.

Chapter 5 provides the results and discussion on compaction characteristics, compressive strength, flexural strength, split tensile strength, modulus of elasticity and flexural fatigue behaviour of CTA base mixtures prepared with varying cement contents.

Chapter 6 details about the KENPAVE analysis carried out on pavement sections with proposed Superpave and CTA mixtures in prescribed layers to analyze the critical strains of LLAP.

Chapter 7 Presents the conclusions drawn based on experimental investigation and analytical study.

CHAPTER 2

REVIEW OF LITERATURE

According to APA (2002) LLAP is defined as "an asphalt pavement designed and built to last longer than 50 years without requiring major structural rehabilitation or reconstruction, and needing only periodic surface renewal in response to distresses confined to the top of the pavement". When most of the LLAP sections were constructed for 50 years of design life, some researchers have designed pavement sections with lesser life (30 and 40 years) also (Park et al. 2005, Sidess and Uzan 2009, Chai et al. 2012).

2.1 DESIGN CRITERIA FOR LONG LIFE ASPHALT PAVEMENT

The tensile strain acting in the horizontal direction at the bottom of asphalt layer and the compressive strain acting in the vertical direction on the top of subgrade are considered as the critical strains in any pavement structure. So the general principle of LLAP design is to keep these strain responses in some particular limits, and many researchers have adopted a maximum limit of 70 micro strains and 200 micro strains for horizontal tensile strain and vertical compressive strain respectively (Park et al. 2005, Walubita and Scullion 2010, El-Hakim 2013). Indian Roads Congress (IRC) 37 (2012) also suggests same criteria for developing LLAPs for a design life of 50 years. It is assumed that limiting the tensile strain makes the fatigue life of the asphalt base layer so high (virtually infinite) and the vertical compressive strain criterion limits the rutting on subgrade. However, some researchers were of the opinion that only the limiting strain at the bottom of asphalt layer, commonly known as Fatigue Endurance Limit (FEL), is only significant in LLAP design (Sidess and Uzan 2009, Tarefder and Bateman 2009). According to them the subgrade strain criterion is liberal considering the traffic loads in LLAPs and the design method can protect subgrade from shear deformation. Also it was claimed that the contribution of subgrade and sub base layers to rutting distress is very low and rutting is generally confined to the top 50mm of the pavement structure. An FEL of 70 micro strains was adopted by many researchers and the layer thickness was adjusted to obtain the same (Lee et al. 2007, Sidess and Uzan 2009, Maher and Uzarowski 2010) without considering the compressive strain at subgrade.

2.2 LONG LIFE ASPHALT PAVEMENT COMPOSITION

Since each of the HMA layers in LLAP are tailored to resist specific distresses, the material selection, mix design and performance testing need to be specialized for each material layer (Newcomb et al. 2001). Pavement structures with conventional asphalt and non-asphalt mixtures may not be sufficient in LLAPs mainly due to their reduced resilient modulus values, which make them difficult to achieve the tensile and compressive strain criteria (Palit et al. 2004, Jitsangiam et al. 2013). A typical LLAP structure consists of, but is not limited to, impermeable, durable, and wear resistant top layers, a stiff and thick rut-resistant intermediate layer for structural strength; and a flexible fatigue-resistant bottom layer resting on a permanent, stable foundation. The layer thicknesses are generally variable depending on the traffic loading, environmental location, and materials/mix-designs. However, the rut-resistant intermediate layers are often the thickest element, providing sufficient load carrying capability (APA 2002, Walubita and Scullion 2010).

Lee et al. (2007) suggested structure of perpetual pavement with rut resistant, impermeable and wear resistant surface course (100-150 mm thick), a rut resistant and durable high modulus base layer (200-250 mm thick), and a sub-base layer above the natural subgrade. A high modulus asphalt mixture prepared using a hard grade asphalt binder (penetration grade 20-30) and a typical dense gradation having 25mm NMAS was used in the base layer. Dense graded HMA with unmodified binder for base layer, other than dense graded mixtures with polymer modified asphalt for intermediate layer and SMA or Open Graded Friction Course (OGFC) with a smaller sized aggregate such as 9.5 or 12.5 mm mixtures for surface layer were recommended by Newcomb and Hansen (2006) and their uses were discussed. Walubita et al. (2010), suggested section has five asphalt mixture layers and a stiff base or a stabilized subgrade over the natural subgrade. The asphalt layers included a sacrificial layer composed of porous friction course (1-1.5 inch thick), impermeable load carrying layer with heavy duty SMA mix (12.5mm NMAS with thickness 2-3 inch), transitional layer composed of Stone Filled Hot Mix Asphalt (SFHMA) mix (19mm NMAS, 2-3 inch thick), stiff load carrying layer with same SFHMA mix (25mm NMAS, 8 inch to variable thickness) and a stress relieving impermeable layer such as Superpave rich bottom layer (12.5mm NMAS, 2-4 inch thick), and all mixtures are suggested using Performance Graded (PG) binder. After studying 14 perpetual pavements across different states in the United States (US), Tarefder and Bateman (2009) suggested sections with thickness varying from 13 to 30 inches, which consisted of a surface layer, a rut resistant intermediate layer and a fatigue resistant base layer on a solid foundation with granular sub-base or treated subgrade. Texas Department of Transportation generally uses a perpetual pavement structure with 2 to 3.5 inch renewable HMA surface, structural load – bearing, stiff, rut resistant HMA base or multiple HMA layers (thickness variable based on pavement design, minimum 8 inch), rich bottom layer (minimum 2 inch thick), a moisture resistant pavement foundation (minimum 6 inch thick, minimum 240MPa design modulus) over a natural subgrade (Walubita and Scullion 2010). Sidess and Uzan (2009) proposed a structure of perpetual pavement for Israeli conditions, with 300mm thick HMA layers over a granular sub base layer whose thickness ranges between 150 and 650 mm depends on the subgrade strength. The HMA layer included 50mm of rich base layer (dense graded with a maximum aggregate size of 19mm and PG 70-10 binder), 40mm of upper layer (SMA mixture with a maximum aggregate size of 19mm and PG 76-10 binder), 120mm intermediate layer (coarse aggregates with maximum aggregate size of 25mm and PG 76-10 binder) and a complementary HMA layer lying on the top of the asphalt base layer. Uzarowski et al. (2008) designed a perpetual pavement section by altering the conventional deep strength pavement by incorporating a rich bottom layer. It consists of 40mm wearing course (with SMA 12.5), 120mm asphalt binder course (Superpave 19 mix for upper binder and Superpave 25 mix for lower binder layer), 80mm rich bottom layer (Superpave 19 with PG 70-28 asphalt cement modified mix), 150mm granular base and 370mm granular sub-base.

Most of the researchers used SMA in the surface layer and Superpave mixtures in the intermediate and asphalt base layers of LLAP, because of their better performance. Some researchers suggested using OGFC also in the surface, to improve the visibility by reducing splash and spray and to reduce pavement noise (Newcomb and Hansen 2006, Tarefder and Bateman 2009, Maher and Uzarowski 2010, Chai et al. 2012). In the case of RBL, in which binder content is generally higher than the optimum (OBC) in order to improve the flexibility of the layer, modified Superpave mixtures are generally used. Binder content of Rich Binder Mixture (RBM) can be increased by adding extra binder (0.3 to 0.5 %) to the OBC of the mixture prepared with 4% air voids. In another method, the mix design can be done to achieve a lesser air void content of 3%, and the corresponding binder content can be used (Sidess and Uzan 2009, Maher and Uzarowski 2010, Abou-Jaoude and Ghauch 2011). The increased binder content and reduced air voids in RBM help to achieve higher fatigue resistance.

El-Hakim et al. (2009) suggested 50mm surface layer with Superpave or SMA 12.5mm (with PG 76-28 binder), 90mm intermediate layer of Superpave or SMA 19mm (with PG 76-22 binder) and 120mm rich bottom mix layer of Superpave 25mm (with PG 64-22 binder) followed by an open graded drainage layer above the subgrade. Uzarowski et al. (2008) used SMA 12.5 for surface layer (40mm thick), Superpave 19 mix and 25 mixtures for upper and lower binder layers (120mm thick). Superpave 19 modified mixture with PG 70-28 asphalt cement having significantly higher polymer content was used in RBM to improve its fatigue endurance. Walubita and Scullion (2010) also suggested a similar structure with Superpave mixture with PG 70-22 or higher PG asphalt-binder as the intermediate layer and SMA as the surface layer. Two Perpetual pavement sections with and without RBL have been recommended by Tarefder and Bateman (2012) for New Mexico. Superpave mixtures with fine gradation, (19mm and 12.5mm NMAS) were used for surface and intermediate layers, and a coarser mix with 25mm NMAS for RBL. The RBL was designed for 3% air voids with PG 64-22 binder for flexibility, whereas modified PG 76-22 and PG 70-22 binders were used in other layers.

2.3 PERFORMANCE CHARACTERISTICS

Performance of both LLAP structures and mixtures in field and laboratory was noted to be very impressive. Uzarowski et al. (2008) determined the mechanistic properties of different asphalt mixtures in perpetual pavement (SMA, Superpave 19 and 25, RBM) using dynamic modulus, rut resistance and fatigue endurance tests. The rutting behaviour was checked using asphalt pavement analyzer as per AASHTO TP-63-03 and all mixtures showed rut depth lesser than 5mm after 8000 cycles. Field trails for four thick, flexible pavement structures in Kansas, US were conducted by Romanoschi et al. (2008) to investigate the suitability of the perpetual pavement concept and the longitudinal and transverse strains were observed to be lower than 70 microstrain. Dynamic and triaxial resilient modulus tests were also conducted to measure the stiffness of asphalt mixtures and subgrade soils respectively. Walubita et al. (2010) also conducted dynamic modulus and repeated load permanent deformation tests for SFHMA mixes in laboratory and verified with field tests, and observed that they are generally very stiff mixes with high moduli values and less temperature sensitive to provide adequate rutting resistance. SFHMA was recommended by Texas as the main structural load-carrying layer in a perpetual pavement structure. El-Hakim (2013) conducted resilient modulus test, dynamic modulus test and the thermal stress restrained specimen test in laboratory for Superpave 25, 19 and 12.5 mixtures and also for an RBM with Superpave 25.

Yang et al. (2009) describes about the first perpetual asphalt pavement test road in China having three sections, S1, S2 and S3, with different pavement structure combinations and thicknesses. S1 was designed to have a tensile strain less than 70 micro strains at the bottom of the asphalt layer, while S2 and S3 were designed to have 125 micro strains. The only difference between S2 and S3 was the use of a modified asphalt binder in S3 (for fatigue layer of 7.5 cm thick) compared to the unmodified binder in S2. Comparison was also done with control sections S4 and S5 having 15cm thick large stone permeable asphalt mixture layer (between the asphalt layers and pozzolanic-treated materials) and a semi-rigid base pavement structure (15cm thick asphalt layer on a thick pozzolanic-treated base) respectively. In order to measure the pavement responses, asphalt strain gauges, earth pressure cell, pavement temperature sensors, axle position measurement gauge, weigh-in-motion etc. were installed. The maximum strain was observed for S5, whereas S1 showed the minimum. The layer thickness, load, pavement temperature and truck speed influenced the measured stress. Further study on these sections conducted by Timm et al. (2011) indicated that the sections S1, S2, and S3 satisfy the criteria of perpetual pavements, though S1 was found to be overdesigned for the prevailing conditions. Falling Weight Deflectometer study was carried out by Chai et al. (2012) on long-term pavement performance site constructed with semi-perpetual materials. The Falling Weight Deflectometer deflection testing was undertaken using 40, 60, 80, and 120 kN loading by placing the geophones at 0, 200, 450, 600, 900 and 1500 mm from the loading position. The tensile strain at the bottom of asphalt layer for the pavement structure was observed to be varying from 102 to 187 micro-strains and the vertical compressive strain on top of the subgrade from 65 to 98 micro-strains. The authors reported that the structural overlay (40mm thick open graded asphalt layer) at 30th year of service life would improve the strains to satisfy the fatigue endurance limit and limiting subgrade strains of a perpetual pavement structure.

Many researchers concentrated on the performance of RBM including the effect of additional binder content, type of binder etc. Maher and Uzarowski (2010) described the application of RBM technology to the Red Hill Valley Parkway in Hamilton, Ontario, Canada. Asphalt pavement analyzer was used to check accelerated performance testing of RBM and the accumulated permanent deformation was less than the 5 mm set in the specification. A pavement response system was installed in field section of perpetual pavement to measure pressure, moisture, strains, and temperature at different layers. The field monitoring confirmed that the induced maximum tensile strains under standard axle loading were within limits. Four dense graded HMA mixes with unmodified and polymer-modified asphalt binders were prepared by Hajj et al. (2011) and they were evaluated at both optimum and rich binder contents. The performance of mixtures was assessed through fatigue, rutting and resilient modulus tests in laboratory. The rutting resistance of the rich mix and its corresponding optimum mix were observed to be similar, whereas it was much better for polymer modified optimum and rich mixes compared to the unmodified mixes. The mechanistic-empirical analysis showed that even though RBMs improved the fatigue performance of HMA pavements, polymer-modified mixes provided significant advantages in both fatigue and rutting performance. Lee et al. (2007) prepared conventional and high modulus asphalt mixtures by Marshall method and dynamic modulus, moisture susceptibility, wheel tracking and fatigue tests were conducted. The resistances of the high modulus mixture against moisture, rutting, and fatigue damage were better than those of the conventional mix. A full scale performance test showed that with lesser thickness, the high modulus sections provided the tensile strain values at the bottom of the asphalt layer lower than those of the conventional mix sections. Similar observations were made by Liu and Wang (2011) for an asphalt mixture developed for asphalt treated base layer of perpetual pavement. The resistances of the mixture with high binder content against moisture and fatigue damage were found to be better than those with low binder content.

For a field evaluation the Ministry of Transportation Ontario constructed a control section with conventionally designed flexible pavement and two perpetual pavement sections, one with RBM as the lower binder layer and the other with Superpave 25mm mix (Lane et al. 2006). El-Hakim (2013) conducted structural and economic evaluation of perpetual asphalt pavement design and compared with conventional asphalt pavement design. Three test sections were constructed and two of these sections were perpetual pavement sections with and without rich mix layer, designed and constructed to determine the use of rich mix layer at the bottom of the perpetual pavement. The tensile and compressive strains in the pavement layers were recorded with sensors and were correlated with laboratory test results by using several linear regression models. It was observed from the models that perpetual pavement section with RBM had the lowest tensile strain at the bottom of the asphalt layer. Similar comparison was made by Zhu and Ni (2015) in China, for perpetual pavement sections with and without RBM and a semi rigid base (cement treated base) asphalt pavement section. Two binder contents were considered for RBM, 5.5% (OBC from Marshall Test) and 6% (binder content which provided highest fatigue life). According to the authors, higher asphalt content in RBL would strengthen the cracking resistance ability, but at the same time, weaken the rutting resistance ability. Even though perpetual pavement without RBL was provided satisfactory rutting and cracking resistance, perpetual pavement with RBL obtained much smaller maximum tensile stress and was observed to be more cost effective in the long run.

2.4 PAVEMENT ANALYSIS AND DESIGN

Pavement design methods are generally categorized into two, empirical methods and mechanistic-empirical methods. Empirical methods are established on experience gained in practice and from observation of the performance of existing or specially constructed pavements under different traffic conditions. Hveem and associates developed the first empirical methods using California Bearing Ratio (CBR) method during 1930's. In 1972, the American Association of State Highway Officials (AASHO) developed an empirical pavement design guide based on an equation (prediction model) with coefficients that were statistically obtained from the AASHO test road. The main drawback of empirical methods were, they are restricted to a particular extent of pavement and traffic loads only, and they are insufficient to account a new material or different traffic loads outside the range considered (Lav et al. 2006). This leads to the development of mechanistic empirical methods for pavement design. In this method, the pavement structure and load configuration are assumed. Generally the pavement structure is simplified to three distinct layers (Dormon and Edwards 1968). A better approach to the design of LLAPs is the mechanistic-empirical method. This approach uses the elements of a rational engineering analysis of the reaction of the pavement in terms of stresses, strains and displacements in the context of the pavement's expected life (Newcomb et al. 2010).

Pavement analysis is generally conducted to determine the responses, including stresses strains and displacements, in a pavement structure during the application of a wheel load. The horizontal tensile strains developed at the bottom of the surface layer, which control the fatigue cracking, and the vertical compressive strains, developed at the top of the subgrade, which control the permanent deformation, are considered as the critical strains in any pavement structure. If the design life is less than the governing failure criterion in terms of number of standard axles, then the related pavement configuration is considered as satisfactory and acceptable as a valid design. Otherwise, layer thickness and/or material properties are adjusted to reach an acceptable configuration (Lav et al. 2006). In the design of pavement sections the thickness of each layer is achieved depending on the individual material characteristics and traffic conditions, to satisfy certain requirements. In LLAP, the thickness design procedure is based on limiting the critical responses in pavement layers. The critical pavement responses considered are tensile strain at the bottom of asphalt concrete layer for fatigue cracking and compressive strain on top of the subgrade for rutting.

Park et al. (2005) proposed a simplified pavement response model called ILLIPAVE finite element program for determining the layer thickness and modulus to develop long life pavements. Pavement responses were predicted by varying layer thickness and modulus in intermediate and base layer. Many researchers reported the use of a probabilistic mechanistic–empirical pavement analysis program named PerRoad, which incorporates Monte Carlo simulation to obtain pavement reaction to loading and an evaluation of potential damage. Generally four basic data sets are required for PerRoad such as thickness, environmental data, material properties and traffic. Timm and Newcomb (2007) used PerRoad 2.4 for design of perpetual pavement in US and modeled each pavement layer as linear elastic using Waterways Experiment Station Layered Elastic Analysis characterized with the elastic stiffness and Poisson’s ratio of materials. The structural design performed using the AASHTO 1993 methodology was verified using PerRoad by Uzarowski et al. (2008) and Maher and Uzarowski (2010). Timm et al. (2011) conducted analysis of five test sections in China using PerRoad by providing surveyed thicknesses, material properties derived from back calculation, and load spectra from an on-site weigh-in-motion system as inputs. From the analysis, horizontal tensile strain at the bottom of the HMA, vertical stress at the top of the first layer beneath the HMA, and horizontal stress at the bottom of the first layer beneath the HMA were evaluated.

El-Hakim et al. (2009) designed both conventional and perpetual pavement sections using AASHTO DARwin software and their structural, technical and economic evaluations were performed. The stresses, strains and pavement deflections were calculated at the layer interfaces and the pavement surface using ELSYM 5 and WESLEA for Windows 3.0 programs. ELSYM5 generally provides a multi-layer elastic solution for a pavement subjected to static loads. Hajj et al. (2011) used the program to analyze the perpetual pavement sections in Nevada, U S. Axle load, tyre pressure, resilient modulus and Poisson’s ratio of materials are used as input parameters in ELSYM5. Tarefder and Bateman (2012) used AASHTO’s Mechanistic-Empirical Pavement Design Guide (MEPDG) to design and analyze perpetual pavement sections for New Mexico State Highways. There are three levels of input in the MEPDG analyses. Material properties obtained from laboratory testing are used in Level 1. In Level 2, these properties are determined from existing correlation equations, whereas in Level 3, they are calculated from various index properties including soil classification, plasticity, aggregate gradation, binder content by using correlations or equations. To simulate the stress distribution in the different pavement sections, Zhu and Ni (2015) used a two-dimensional finite element model developed with ABAQUS software.

For predicting perpetual pavement performance, Crovetti et al. (2008) used KENLAYER computer program, which is a part of KENPAVE software package developed by Young (1975) for pavement analysis and design. KENLAYER allows designing the pavement as a stress-dependent multilayer system and it provides details regarding the stress, strain and deflection under single or dual wheel systems with different axle configurations. A computer program called FPAVE was developed by IIT Kharagpur in 1997 for the computation of stresses in a pavement structure, which was later modified as IITPAVE (Das and Pandey 1999). Any combination of traffic and pavement layer combination can be tried using IITPAVE by providing inputs similar to KENPAVE and it gives strains at critical locations as outputs. IRC also suggests this program for the design of flexible pavements, including perpetual pavement sections (IRC 37 2012). A satisfactory pavement design can be achieved through iterative process by varying layer thicknesses or, if necessary, by changing the pavement layer materials. Basu et al. (2013) used IITPAVE program to obtain the maximum tensile strain, vertical strain in subgrade, tensile strain in cementitious layer and also to design various perpetual pavement sections. Hernando and Del Val (2016) used multilayer linear elastic analysis to study the response (stresses and strains) of different semi rigid long life pavement sections proposed in Spain to determine which sections meet the fatigue criteria.

2.5 SUPERPAVE MIXTURES

Superpave (acronym for Superior Performing Asphalt Pavements) is a comprehensive asphalt mix design and analysis system established based on the Strategic Highway Research Program in 1987 (Federal Highway Administration 2009). It represents an improved, performance-based system to specify asphalt binders and mineral aggregates, to perform asphalt mixture design and also to analyze pavement performance. The Superpave mix design procedure is standardized by Asphalt Institute (AI 2001) in the Superpave series No. 2 (SP-02) manual, which includes mix design practices, material selection (asphalt binder and mineral aggregate), asphalt mixture volumetrics etc.

Superpave generally uses performance system for testing, specifying and selecting asphalt binders. The commonly used Performance Graded (PG) binders are PG 64-22, 70-22, 76-22, 64-28, 52-34 etc. and they are selected based on the climate and traffic conditions. For these PG binders the physical property requirements are constant among all grades and are differentiated based on the temperature at which these requirements must be met. In order to select suitable aggregates, AI suggest considering ‘consensus’ and ‘source’ properties of aggregates. The consensus aggregate properties decided based on SHRP are coarse and fine aggregates angularity, flat and elongated particles and clay content. The source properties include toughness, soundness and proportion of deleterious materials. The aggregate gradation is generally represented using the NMAS (the sieve opening through which all of the aggregate may, but need not, pass so that maximum 10% of the aggregate may be retained on that sieve) and the Maximum Aggregate Size (MAS – one sieve size larger than NMAS. It is the smallest sieve opening through which the entire amount of aggregate is must pass). The Superpave gradation should be within a set of control points specified at the NMAS, at an intermediate size (2.36mm) and at the smallest size (0.075mm), and it has a Restricted Zone (RZ) through which the gradation line should never pass. The RZ resides along the maximum density gradation between the intermediate size (either 4.75mm or 2.36mm) and the 0.3mm size. Aggregate gradation requirements for 19mm MAS and 12.5mm NMAS as per AI, SP-02 is presented in Figure 2.1. SP-02 manual specifies gradations with different NMAS such as 37.5, 25.0, 19.0, 12.5, 9.5 mm, along with RZ values.

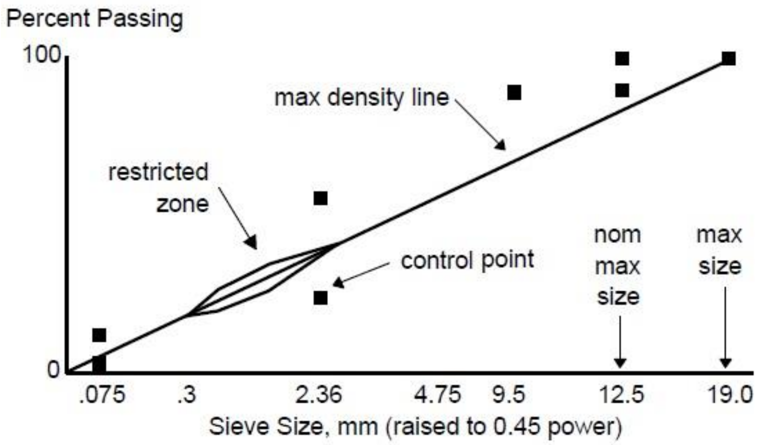


Fig. 2.1 Superpave Gradation Chart for 12.5mm NMAS Mixture

Some researchers suggested that Superpave gradation bellow the RZ has significantly better performance (Roberts et al. 1996). Even though Superpave recommended gradations passing below the RZ (BRZ), it is not a compulsory requirement. Several highway agencies have reported successful use of gradation passing above the RZ (ARZ) and even satisfactory performance was experienced for grading passing through the RZ (TRZ). Kandhal and Mallick (2001) evaluated the effect of mix gradations, both complying with and violating the Superpave RZ, on rutting potential using different types of aggregates. Even though statistical analysis indicated a significant difference between rut depths obtained in mixes using different aggregate types and gradations, the one passing TRZ did not provide higher rut depths compared to the other two gradations. Kandhal and Cooley (2002) compared the rutting resistance of both coarse-graded (BRZ) and fine-graded (ARZ) Superpave mixtures with 9.5 and 19.0 mm NMAS using asphalt pavement analyzer, superpave shear tester, and repeated load confined creep test. Granite and crushed gravel coarse aggregates and four fine aggregates (sandstone, limestone, granite, and diabase) were used with PG 64-22 binder. No significant difference in rutting resistance was observed between coarse and fine-graded Superpave mixtures. Sargand and Kim (2003) prepared Superpave mixes with three aggregate gradations having 12.5mm NMAS and three different polymer modified PG 70-22 binders (unmodified, Styrene Butadiene Styrene (SBS) and Styrene Butadiene Rubber (SBR) modified). The mixtures were evaluated for rutting and fatigue resistance by conducting laboratory tests such as triaxial repeated load test, uniaxial static creep test, diametral resilient modulus test, indirect tensile strength test, asphalt pavement analyzer test, and the flexural beam fatigue test. Test results indicated that the effects of gradation variation on rut and fatigue resistance were small, but the polymer modified mixes were more rut and fatigue resistant compared to the unmodified mixtures. Superpave gradations, both above and below the RZ showed similar rut resistance in the National Center for Asphalt Technology (NCAT) test track in Florida. Based on the test results and observations from other parts of US, Florida Department of Transportation allowed fine graded mixtures for different traffic levels. These changes were adopted to improve the quality of the mixture and to reduce the production and constructability issues associated with coarse-graded mixtures (Gokhale et al. 2005). Al-Khateeb et al. (2017) evaluated the effect of RZ on volumetric and compaction properties of Superpave mixtures. Fifteen different aggregate gradations having 19mm, 12.5mm and 9.5mm NMAS and passing ARZ, BRZ, crossover through RZ (CRZ), hump through RZ (HRZ) and TRZ were used in the investigation. Authors conclude that TRZ mixtures are favorable over BRZ and ARZ mixtures. The study does not support the complete removal of the RZ from the Superpave aggregate gradation criteria.

Nukunya et al. (2002) reported that mixtures that are graded BRZ have poorer rutting performance than those that are graded ARZ or TRZ. Superpave mixes with gradation above RZ, two through RZ (one closer to above RZ and the other closer to below RZ) and two below RZ (with different fine aggregate angularity values) were designed for low volume local roads by Kim et al. (2006) and tested for rut resistance. Good rutting resistance was observed for finer graded (above RZ and the through RZ that is close to above RZ gradation) mixtures compared to the coarser ones. Chun et al. (2012) observed that the existing Superpave mix design criteria including voids in mineral aggregates, gradation control points, and effective asphalt content are not effectively related to rutting and cracking performance and hence the Dominant Aggregate Size Range (DASR)– Interstitial Component (IC) model was considered.. In an another study, Al-Khateeb et al. (2018) investigated the effect of aggregate gradation on rutting performance of Superpave mixtures. Two aggregate gradations passing ARZ and BRZ were used to prepare specimens and compared. It was noticed from the results that asphalt mixtures with ARZ gradation exhibited higher resistance to failure than mixtures with BRZ gradation.

Based on the laboratory and field performance and previous experiences, researchers have compared Superpave mixtures with conventional mixtures produced by Marshall design methods and confirmed the superiority of the former ones. Along with this comparison, Asi (2007) aimed to evaluate the suitability of locally available aggregates in Superpave mixes. Two mixtures prepared with local materials using Superpave and Marshall mix design procedures were tested for Marshall stability, loss of Marshall stability, indirect tensile strength, loss of indirect tensile strength, resilient modulus, fatigue life, rutting, and creep. The Superpave mixtures showed better performance in all tests due to their improved aggregate structure, lower asphalt content and lower dust proportion compared to Marshall mixes. Similar observations were made by Jitsangiam et al. (2013) also, in an evaluation of Superpave mix design procedure conducted for Thailand pavement conditions. A comparison of Superpave and Marshall mix design methods for Indian condition was carried out by Swami et al. (2007) along with evaluating the effect of angle of gyration, number of gyrations on mix properties like density, stability, indirect tensile strength. It was observed that Superpave mixes fulfilled all the criteria for easy and good construction at lesser binder content than the Marshall mixes. Khan and Kamal (2008) conducted tests for indirect tensile strength, creep performance and moisture sensitivity on Superpave and Marshall mixtures. The Superpave mixes showed improved performance in terms of low accumulated strains, high modulus of resilience, less moisture sensitivity and better rut resistance than the Marshall mixes. In a study, Ahmad et al. (2014) investigated the performance characteristics of Superpave and Marshal method of mix design for HMA mixtures prepared using two granite aggregate sources. Wheel tracking test, dynamic creep test, indirect tensile resilient modulus test were conducted in the laboratory. Test results showed that Superpave-designed mixtures utilize less OBC, more superior and least susceptible to permanent deformation and had higher resilient modulus values compared to Marshall mixtures. Palit et al. (2004) compared laboratory performance of asphalt mixtures prepared using three aggregate gradations (Superpave; gradation specified by the Ministry of Surface Transport, India for asphalt concrete; one gap gradation). Fatigue and permanent deformation characteristics, temperature and moisture susceptibility, and oxidative aging of mixes were evaluated and observed the better overall performance of Superpave mixes compared to the mixes having other aggregate gradations. SU and Hachiya (2008) carried out a study to access the possibility of implementing the Superpave mix design procedure for use in airfield. Superpave mixtures were compared with Marshall mixtures to quantitatively access the rutting resistance by wheel tracking tests. The result indicates the superiority of Superpave mixtures and the authors suggest that it can be used in heavy duty airport pavements. Khedr and Breakah (2012) compared Superpave and Marshall mix designs by using different coarse and fine Egyptian gradations. Superpave mixes showed lower optimum asphalt content compared to Marshall mixes. The reduction was higher in the case of coarse graded mixes. Samples prepared using the SGC had higher values in stability and flow than those prepared using the Marshall method. In another study, Gupta and Veeraragavan (2009) observed improved performance of Superpave method by investigating the performance of SBS polymer modified and conventional asphalt mixes compacted by SGC and Marshall compactor. The comparison between both the compaction methods was established in terms of fatigue life, resilient modulus, retained Marshall stability and indirect tensile strength ratio. Higher density values were observed for SGC specimens and this may be due to the higher compaction effort. Marshall and Superpave mix designs using local aggregates to study the suitability of the Superpave mix design as compared with the Marshal mix design for low volume roads and shoulders was evaluated by Habib et al. (1998). The results of volumetric analysis show that the calculated estimated asphalt content for Superpave mixtures was less than Marshall mixtures. Wang et al. (2000) used Superpave and Marshall mix design procedures for the two Superpave and a typical Taiwan mixtures respectively, to compare the volumetric and mechanical performance properties. SGC was used to compact the specimens. Volumetric analysis showed that Taiwan mixture hardly met the Superpave volumetric requirements and it contained less than 1% air void at design number of gyrations, which suggests that the mixture is highly prone to rutting. Mechanical test result indicated that the Superpave mixtures were more resistant to permanent deformation than the Taiwan mixtures.

2.6 ASPHALT BINDERS

Superpave generally uses PG binders and they are selected based on the climate and traffic conditions. In many countries, the usage of PG binder system is limited and they widely adopt the penetration graded asphalt binders for paving mixtures. Even the softer binder 80/100 was tried by some researchers in Superpave mixtures also. Ahmad et al. (2014) used penetration grade (PEN) 80/100 and PEN 60/70 binders in Superpave mixtures based on climatic conditions. These binders are equivalent to PG 64 and PG 70 respectively. SU and Hachiya (2008) used straight asphalt with a penetration of 60-80 along with gradations having different MAS to prepare Superpave mixtures for heavy duty airport pavements. Asphalts with penetration grade of 60/70, were reported in various studies on Superpave mixtures by many researchers (Al-Khateeb et al. 2013, Al-Khateeb et al. 2017, Al-Khateeb et al. 2018). Superpave mixtures were prepared by Gogula et al. (2003) using PG 52-28, PG 64-22, PG 58-28, and PG 70-28 binders. Habib et al. (1998) prepared Superpave mixtures with PG 58-22 (AC-10) binder and gradation having 19mm NMAS. Khedr and Breakah (2011) used PG 64-16 binder to study the effect of using a fine or coarse gradation, as related to the RZ, on the rutting behaviour of asphalt concrete in flexible pavements.

2.7 MODIFIED ASPHALT

Properties of asphalt and asphalt mixes can be improved by incorporating certain additives or a blend of additives. Asphalt treated with these additives or modifiers is known as “Modified Asphalt” and is expected to provide higher life mixtures depending upon the degree of modifications and type of additives used. Tia et al.

(1994) reported that Haas et al. (1983) defines these modifiers as: "An asphalt cement additive is a material which would normally be added to and/or mixed with the asphalt before mix production, or during mix production, to improve the properties and/or performance of the resulting binder and/or the mix, or where an aged binder is involved, as in recycling, to improve or restore the original properties of the aged binder." The possible advantages of binders and pavements with commonly used modifiers like rubber and polymer include increase in softening point, viscosity, ductility, fracture toughness, elastic modulus, flexural strength, creep resistance, reduction in embrittlement by aging, rut susceptibility and low temperature cracking, enhanced Marshall stability, resilient modulus, tensile strength and traction, and overall improvement in performance both in the laboratory and field (Alexander 1968, Shim-Ton et al. 1980, Denning and Carswell 1981, Kortschot and Woodhams 1984, Jew et al. 1986, Carpenter and VanDam 1987, Lee and Demirel 1987, Shuler at al. 1987, Nahas et al. 1990, Choquet and Ista 1992, Dhalaan et al. 1992, Tia et al. 1994, Zaman et al. 1995, Hossain et al. 1999, Palit et al. 2004, Hamzah et al. 2006).

Asphalt binders subjected to suitable modification will improve the cracking and rutting resistance in Superpave mixtures and this prompted researchers to use different types of polymer modified binders in Superpave mixtures. Sirin et al. (2008) evaluated the potential performance of unmodified and SBS polymer modified (3% by weight of total binder) Superpave mixtures and the results from laboratory tests showed higher rutting resistance and indirect tensile strength of SBS polymer modified mixtures. In an investigation, Sargand and Kim (2003) studied effect of polymer modification on rutting and fatigue resistance of Superpave mixtures prepared using three different PG 70-22 binders (unmodified, SBS and SBR modified). The SBS and SBR modified binder resulted in mixes having increase in rut and fatigue resistance, in comparision with unmodified mixtures. Romanoschi et al. (2006) determined dynamic modulus, bending stiffness and fatigue properties of four Superpave mixtures used in the construction of base layers of Kansaa flexible pavements. Three mixtures were prepared with PG 64-22 binder and one mixture was prepared with PG 70-28 SBS polymer modified binder. The mix containing modified binder had much higher fatigue life, while having similar dynamic moduli with those of mixes with unmodified binders. Two different types of dense graded Superpave HMA mix were developed by Shaffie et al. (2015) to evaluate performance by conducting resilient modulus test. The base asphalt of 80/100 penetration grade and nanopolyacrylate (NPA) polymer modified asphalt (prepared by adding 6% NPA polymer to base asphalt) were used. The results indicated that all the mixes passed Superpave volumetric properties criteria and higher resilient modulus was observed for NPA polymer modified asphalt mix which in turn demonstrates the better resistance to rutting than those prepared using base asphalt. Mrawira and Elizondo (2008) investigated Superpave mixtures prepared using PMB (PG 76-10). The conventional asphalt, PG 64-19 was modified by adding 1.5% Etilen glicidil acrilato polymer. The effect of asphalt type on the rutting and resilient modulus characteristics of asphalt mix was evaluated by Radhakrishnan et al. (2017). The mix was prepared using unmodified asphalts (VG 10, VG 30 and VG 40), PMBs (PMB grade 40 and PMB grade 70 both with EVA and SBS as modifier) and CRMB grade 60. Gupta and Veeraragavan (2009) studied the benefit of SBS polymer modified asphalt mixes on fatigue performance. Conventional asphalt of grade 60/70 and asphalt modified with SBS polymer (PMB 70) were considered and repeated load indirect tensile test, Marshall stability, indirect tensile strength ratio tests were conducted. Results obtained from the investigation show the superiority of SBS modified mixes over the conventional mixes in all the tests. Lee et al. (2008) evaluated the volumetric properties of asphalt mixtures prepared using control binder (PG 64-22), SBS modified binder (PG 76-22) and CRMB (prepared by adding 10% and 15% ambient crumb rubber modifier by weight of the control binder, PG 64-22). The results from the study indicated that crumb rubber modified mixtures showed the higher voids in mineral aggregates values than the control and SBS modified mixtures.

Researchers have also tried to use rubber modified asphalt in Superpave with an aim to improve the mix properties. Palit et al. (2004) investigated performance of unmodified and crumb rubber modified Superpave asphalt mixes. The crumb rubber generated by scraping old tires of trucks and buses was used to modify 80/100 asphalt. Compared to normal mixes, crumb rubber modified mixes provided improved fatigue and permanent deformation characteristics, lower temperature susceptibility and greater resistance to moisture damage. Al-Mansob et al. (2017) compared the performance of asphalt mix produced with 80/100 penetration grade asphalt and with epoxidized natural rubber modified asphalt in terms of dynamic creep, rutting resistance and moisture susceptibility. Author concludes that asphalt mixture performance can be enhanced by using epoxidized natural rubber as modifier. Lee et al. (2008) determined the stiffness and permanent deformation properties of recycled crumb rubber modified mixtures and stated that addition of crumb rubber of more than 25% satisfied the Superpave mixture performance criteria. Lee et al. (2008) evaluated volumetric properties of crumb rubber modified asphalt mixtures (prepared by adding 10% and 15% rubber to control binder of PG 64-22) as a function of four different compaction temperatures (116, 135, 154 and 173^{o} C) using two compaction methods (Superpave and Marshall) in the laboratory. The results indicated that the compaction temperatures significantly affected the volumetric properties of crumb rubber modified mixtures regardless of the compaction methods.

2.8 CEMENT TREATED AGGREGATE BASE

Generally, CTA as a road base material is produced by using coarse natural or crushed aggregates and designed as a heavy traffic base (Ebrahim Abu El-Maaty Behiry 2013). The amount of cement and aggregates is an important factor in the performance of CTA, and generally an increase in cement content results in increase in strength. But very high level of cement is not economical moreover it does not necessarily guarantee acceptable long-term pavement performance. Guthrie et al. (2002) conducted strength and long-term durability tests in laboratory to determine the optimum cement content for stabilizing limestone and recycled concrete aggregates to use as base materials. The aggregates treated with 1.5, 3.0, and 4.5 % cement were cured for 7 and 28 days and tested for compressive strength, shrinkage, durability and moisture susceptibility. Durability of the CTA was evaluated in the South African wheel tracker erosion test and moisture susceptibility in the tube suction test. The authors suggest a cement content of 3 and 1.5 % for lime stone and recycled aggregates respectively, which satisfied the requirements for compressive strength, durability, and moisture susceptibility. Burns and Tillman (2006) investigated the effects of aggregate mineralogical composition, fines content, cement content, and freeze-thaw cycling on the performance of cement-treated aggregate. Mica, limestone, diabase and granite aggregates were used with 3, 4, 5, and 6 % cement contents, and the mineralogy was found to make a significant difference in the strength. Increase in the cement content increased the measured compressive strength of cylinders. The fines content had a protective effect to the durability of the cylinders subjected to freezing and thawing cycles. A mixture of crushed rock and 2% Portland cement, named Hydrated Cemented Treated Crushed Rock Base (HCTCRB), is commonly used as a base course material in Australia (Jitsangiam and Nikraz 2009, Siripun et al. 2009). After hydration period, HCTCRB is retreated by breaking the cementitious bonds generated during the hydration time, in order to maintain the properties of the unbound material. Researchers conducted different tests including static and dynamic triaxial tests, to assess the performance of the mixture, and models were developed for resilient modulus and permanent deformation characteristics. Siripun et al. (2009) observed that the effect of hydration periods and added water on the performance of HCTCRB is significant. IRC 37 (2012) has also suggested flexible pavement sections with cementitious materials, which includes cement treated soils, aggregates or both, for base and subbase layers, along with a crack relief aggregate interlayer. Jiang and Fan (2013) prepared cement stabilized crushed rock (limestone) material with cement contents 2 – 5 % (with increment of 0.5%), and by subjecting for curing periods 0, 3, 7, 14, 28, 60, 90, 120 and 180 days. Different compaction methods were adopted to simulate the field compaction and the results showed that compressive strength, tensile strength and elastic modulus increase with increase in the cement content and curing time increase, while the ratio of compressive strength to tensile strength decreases with the increase in curing time. Mechanistic evaluation of Cement Treated Base (CTB) mixture using crushed granite coarse aggregates and 0 – 6 % cement was carried out by Ismail et al. (2014). The samples prepared for compressive strength, indirect tension test, elastic modulus and flexure strength tests were subjected to 7, 28 and 60 days curing period. The cement content, curing time, moisture content and dry density had significant effect on the performance of CTB, and based on the study authors recommend cement content of 4% for pavement base layer. Barišić et al. (2016) evaluated elastic and mechanical properties of a new type cement stabilized material using steel slag and gravel for pavement bases. The authors concluded that measuring the ultrasound pulse velocity is a suitable way to predict the mechanic and elastic properties of cement-stabilized materials. Nusit and Jitsangiam (2016) performed fatigue and damage studies of CTB. Damage developments of CTB specimens tested under different loading conditions were characterized. Also the cyclic flexural beam tests were performed to determine the fatigue damage evolutions of CTB specimens. The test results showed that damage evolutions of CTB specimens subjected to cyclic bending forces were influenced by the levels of applied strain. Jitsangiam et al. (2016) conducted beam-fatigue test on CTB in laboratory under the strain-controlled (constant strain) and stress-controlled (constant stress) testing conditions with varying cement contents of 3 – 10 %. General purpose Portland cement and granite/diorite was used to prepare CTB mixtures and the authors observed that cement content affects the fatigue characteristics of the CTB test specimens. Test results showed that, under the same applied strain level, specimens with higher cement content and initial cyclic flexural stiffness have more fatigue failure resistance compared to that of specimens with lower cement content and initial stiffness.

The cement treatment has been utilized to a greater extent for recycled aggregates to improve their performance and to make use in pavement layers. Many states in the US and transportation agencies currently prescribe the use of recycled asphalt pavement materials combined with cement for a stabilized base course for both flexible and rigid pavements (Rupnow et al. 2011). Base materials were prepared by Taha et al. (2002) using different recycled-virgin aggregates blends with cement contents 0, 3, 5, and 7 %. Samples were cured for 3, 7, and 28 days by keeping them in plastic bags at room temperature, and compaction and compressive strength tests were conducted. It was observed that recycled aggregates can act as a pavement structural component with the addition of cement, and also when subjected to longer curing periods. Lim and Zollinger (2003) used conventional crushed limestone, recycled concrete materials and cement to prepare cement treated aggregate base mixtures with 19mm MAS and satisfying the Texas Department of Transportation Portland CTB gradation. The mixtures were varied by changing the quantity of material retaining on 4.75mm sieve and passing 75μ sieve and cement content (4 – 8 %), and the samples were tested for 1, 3, 7 and 28 days of curing. Compressive strength and modulus of elasticity were determined for mixtures and they were observed to be mostly governed by the applied cement content whereas the effects of coarse aggregate and fines contents were less significant. Equations were proposed for the development of compressive strength and modulus of elasticity with curing period. Puppala et al. (2011) used cement dosages of 2 and 4 % only with recycled aggregates based on the suggestion of Texas Department of Transportation, since higher cement treatment results in stiffer bases which results in high temperature cracks. Elastic modulus values of treated and untreated aggregates determined using repeated triaxial test showed the enhancements with cement treatment. Test results were analyzed to determine the structural coefficients for pavement design purpose, which showed the greater structural support of cement-treated recycled asphalt pavement layers when compared with untreated aggregates. Along with recycled asphalt pavement content the quantity of cement was also observed to be an important parameter in the performance of base materials with different recycled asphalt pavement and cement contents (Yuan et al. 2011). Rupnow et al. (2011) added 4 to 8 % cement with different types of natural aggregates (gravel and limestone) and recycled limestone aggregates to conduct tests for compressive strength (on cylindrical specimen), flexural strength, length change and elastic modulus. The results show that all the treated materials can be used for base course construction and the optimum cement content for each case was determined. The average 28-day compressive and flexural strengths for all material types showed an increasing trend with cement content, whereas the elastic modulus showed a decreasing trend. Behiry (2013) used varying cement contents (4, 5, 6, and 7 %) in mixtures with recycled concrete aggregate and traditional limestone aggregates, and were tested after different curing periods (1, 3, 7, 28 days). The fine contents in the mixtures were also varied and tested for compaction, California Bearing Ratio, compressive strength, elastic modulus, flexural strength and indirect tensile strength. The cement treatment provided a significant improvement in the modulus and compressive strength, where the latter showed a linear increase with the curing time for both treated recycled and natural aggregates. Taherkhani and Farokhi (2014) used different cement contents (3, 5, 7, 9 and 11 %) to stabilize recycled asphalt and concrete materials for base or sub-base course in highway pavement. Both OMC and MDD increased with increase in cement contents and, hence the compressive test was used to determine the optimum cement content. Steel fibers were also added to mixture with optimum cement content (9%) in order to enhance the strength further and the same was evaluated through bending and compression strength tests.

CTAs can be suitably utilized in pavements to meet LLAP criteria with reduced thickness for layers, and some researchers have made efforts to accomplish the same. Tongji University, China has designed and constructed three perpetual pavement sections with 20 – 32 cm thick cement treated sub base layer having cement contents 2, 4 and 6 % (Cui et al. 2007). Two grades of cement treated course were used: one with modulus 900 to 1500 MPa and the other with modulus 500 to 900 MPa. As the modulus of cement treated sub-base increased, the tensile strain of the HMA and the compressive strain at the top of the subgrade were observed to be smaller. In a long-term pavement performance site constructed in Queensland, Chai et al. (2012) reported the usage of 200mm thick cementitious stabilized crushed rock as base layer. Basu et al. (2013) modified cement treated pavement sections suggested by IRC to meet perpetual pavement criteria and considered them as perpetual semi rigid pavement. Zhu and Ni (2015) also used cement treated base layer of thickness 18cm in a perpetual pavement section considered for a comparative study conducted in Jiangsu Province, China. Semi-rigid long life pavement sections proposed in Spain for the heaviest traffic conditions include CTB which consists of soil cement, gravel cement, high resistance gravel cement and compacted lean concrete (Hernando and del Val 2016). In Europe, the long life semi-rigid pavement sections for heavily traffic conditions has relatively thick asphalt layer (17-30 cm) on 20 – 30 cm of CTB (European National Highway Research Laboratories 2009).

2.9 SUMMARY OF LITERATURE REVIEW

From the above literature review it is observed that, the three structural layers of HMA present in the LLAP are engineered to withstand the distresses associated with its location. The bottom HMA layer (thickness 75 – 100 mm) made of rich asphalt mixture with low air voids increases the fatigue resistance of the LLAP. A rut resistant mixture is utilized in both intermediate and top layers having thickness 100 – 175 mm and 40 – 75 mm respectively. Literature suggests SMA mixtures in Surface layer and Superpave mixtures in intermediate and asphalt base layers because of their better performance. The layer thicknesses are generally variable depending on the traffic loading, environmental location, and materials/mix-designs. In the case of bottom HMA layer, in which binder content is generally higher than the optimum in order to improve the flexibility of the layer, modified Superpave mixtures are generally used. Binder content of rich asphalt mixture can be increased by adding extra binder (0.3 to 0.5 %) to the OBC determined at 4% air voids. The tensile strain and the vertical compressive strain experienced in the pavement structure critically affect the fatigue and rutting resistance. Generally, the LLAP is designed to keep the tensile strain at the bottom of the asphalt layer and compressive strain at the top of the subgrade within the maximum limit of 70 micro strain and 200 micro strain respectively. It is clear from the literature review that, Superpave mixtures perform better than conventional mixtures prepared by Marshall mix design. Superpave mixtures with gradation ARZ provides better resistance in terms of rutting. The efforts are made to use different types of binders with different gradations, and to evaluate their performance both in field and laboratory. The usage of suitable modified asphalt binders improves the mechanical properties of asphalt mixtures. In Superpave mix design generally, SGC is used to compact the test specimens. CTAs can be suitably utilized in base and sub-base layers of pavements to meet LLAP criteria with reduced thickness for layers and researchers have tried the same. Cement contents from 3 – 10 % can be used based on minimum requirement and economic feasibility.

Analysis and design of pavement sections by determining the responses on different layers were generally carried out using various software programs. A satisfactory pavement design can be achieved through iterative process by varying layer thicknesses or, if necessary, by changing the pavement layer materials.

CHAPTER 3

MATERIALS AND METHODS

3.1 MATERIALS USED

The main materials used in this study are aggregates, asphalt binder, mineral filler and cement. The following sections provide brief information pertaining to the materials selected to prepare Superpave and CTA base mixture.

3.1.1 Aggregates

Aggregate is a collective term for the mineral materials such as sand, gravel, and crushed stone that are used with a binding medium (such as water, asphalt, Portland cement, lime, etc.) to form compound materials (such as asphalt concrete and Portland cement concrete). The quality of aggregates is very important and it should be hard, durable and clean. In this study crushed granite aggregates collected from a local quarry were used. Physical properties of aggregates were tested as per IS 2386 (1963) methods and the results are presented in Table 3.1.

Table 3.1 Properties of Aggregates

Property	Test	Method	Results
Strength	Aggregate Impact Value	IS 2386 (P-4)	21%
	Los Angeles Abrasion Value	IS 2386 (P-4)	22%
Water Absorption	Water Absorption	IS 2386 (P-3)	0.18%
Particle shape	Combined Flakiness and Elongation Index	IS 2386 (P-1)	27.3%
Specific Gravity	Specific Gravity	IS 2386 (P-3)	2.69

3.1.2 Asphalt Binder

In this study, one conventional asphalt and two types of modified asphalt were used as the binder material in Superpave mixtures. Viscosity Graded (VG) 30 asphalt, a commonly used asphalt type in India, was the normal asphalt used in this study. Modified asphalt types including Polymer Modified Binder (PMB) grade 40 and Crumb Rubber Modified Binder (CRMB) grade 60 were also used to prepare Superpave mixtures. The asphalt types used in the study were supplied by Mangalore Refineries and Petroleum Limited and Hincol, Mangalore, Karnataka, India. Each asphalt was tested for different properties as per IS codes and found to be satisfying IS 73 (2013) and IRC SP 53 (2010) specifications for normal asphalt and modified asphalt types respectively. The properties of asphalt binders are listed in Tables 3.2 and 3.3.

Table 3.2 Properties of Normal Asphalt (VG 30)

Property Tested	**Test Method**	**Results Obtained**	**IS 73 Requirements**
Penetration at 25°C, 0.1 mm, 100g, 5s	IS 1203	63	45 Minimum
Softening point, (R&B), °C	IS 1205	54	47 Minimum
Specific Gravity	IS 1202	1.00	-
Flash point, COC, °C	IS 1448	249	220 Minimum
Ductility at 25°C (5 cm /minute pull), cm	IS 1208	> 100	-
Absolute Viscosity at 60°C, Poises	IS 1206 Part 2	2950	2400 – 3600
Kinematic Viscosity at 135°C, cSt	IS 1206 Part 3	380	350 Minimum
Test on residue from rolling thin film oven test:			
Viscosity ratio at 60°C	IS 1206 Part 2	3.1	4.0 Maximum
Ductility after thin film oven test at 25°C, cm	IS 1208	55	40 Minimum

Table 3.3 Properties of Modified Asphalt

Property Tested	Test Method	Results	
		CRMB 60	PMB 40
Penetration at 25°C, 0.1 mm, 100g, 5s	IS 1203	45 (30-50)	38 (30-50)
Softening point, (R&B), °C	IS 1205	69 (Min. 60)	67 (Min. 60
Flash point, COC, °C	IS 1209	283 (Min. 220)	251 (Min. 220)
Elastic recovery of half thread in ductilometer at 15°C, per cent	Annex 2 of IRC SP 53	61 (Min. 60)	87 (Min. 60)
Thin film oven tests and test on residue:			
Loss in mass, per cent	IS 9382	0.084 (Max. 1)	0.049 (Max. 1)
Increase in softening point, °C	IS 1205	3 (Max. 5)	3.2 (Max.5)
Reduction in penetration of residue, at 25°C per cent	IS 1203	41 (Max. 35)	24 (Max. 35)
Elastic recovery of half thread in ductilometer at 25°C, per cent	Annex 2 of IRC SP 53	33 (Min. 50)	64 (Min. 50)

3.1.3 Mineral Filler

Finely divided mineral matter is generally used as mineral filler in asphalt mixtures. In this study granite stone dust and hydrated lime were used for this purpose, limiting the quantity of lime to 2% by weight of aggregates. Hydrated lime provides better resistance to degradation of mixture in the presence of moisture by increasing the stiffness, strength, and toughness of the mastic, and produces better resistance to stripping by improving the asphalt-aggregate interfacial bonding (Kim et al. 2008). This also improves the permanent deformation characteristics and fatigue endurance of asphalt mixtures, particularly at higher temperatures (Mohammad et al. 2000). The filler material was graded as per Table 3.4 suggested by Ministry of Road Transport and Highways (MoRT&H 2013).

Table 3.4 Gradation Requirement for Mineral Filler

IS Sieve (mm)	Cumulative % by weight of total aggregate passing
0.6	100
0.3	95-100
0.075	85-100

3.1.4 Cement

Cement is a binder, a substance that sets and hardens on drying and also reacts with carbon dioxide in the air dependently, and can bind other materials together. The cement used in the present investigation to prepare CTA mixtures was Ordinary Portland Cement (OPC) 43 grade conforming to IS 8112 (2013) and the basic properties are tabulated in Table 3.5.

Table 3.5 Physical Properties of Cement

Sl. No	Test Conducted	Results Obtained	Requirements as per IS 8112 (2013)
1	Specific gravity	3.14	–
2	Standard consistency, %	32	–
3	Setting time - Initial (minutes) - Final (minutes)	65 375	> 30 <600
4	Fineness of cement (m^2/kg) (Blaine's air permeability)	330	>225
5	Soundness (mm) –Le Chatelier test	2.50 (Expansion)	<10

3.1.5 Water

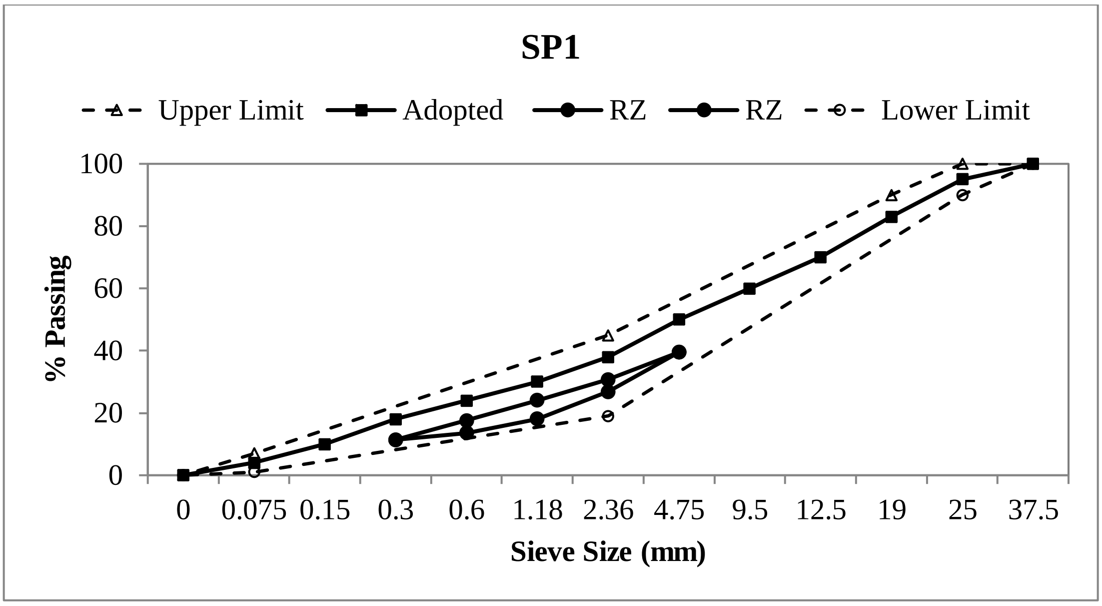
Potable tap water available in the institute laboratory was used for casting and curing of all CTA specimens in the present investigation.

3.2 AGGREGATE GRADATION

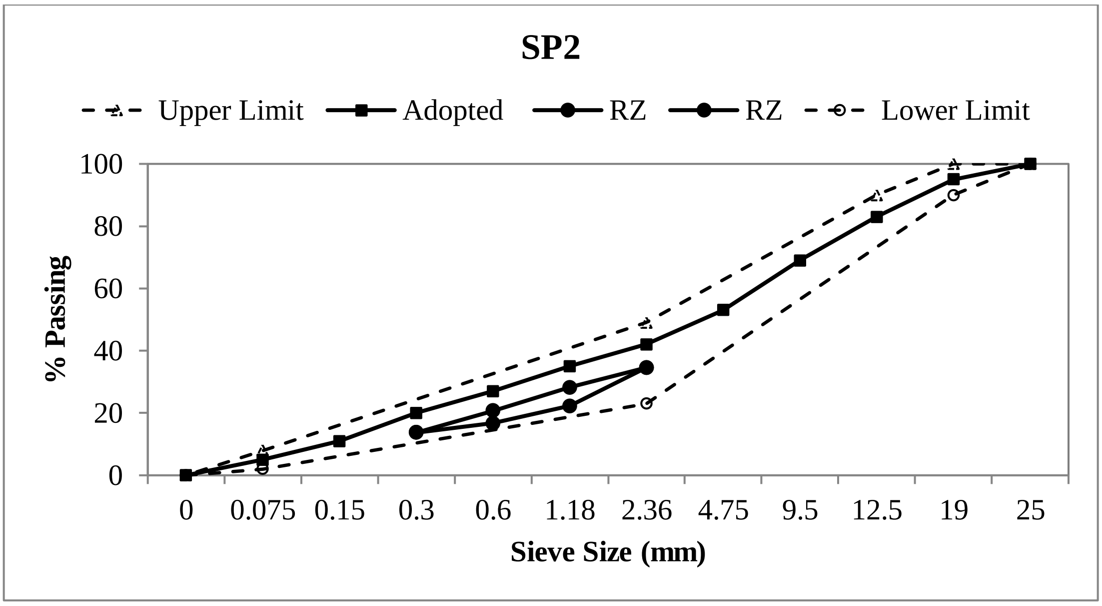
In this study, two different aggregate gradations with Nominal Maximum Aggregate Sizes (NMAS) 25mm and 19mm were used in the preparation of Superpave mixture and are abbreviated as SP1 and SP2, respectively. The gradations considered for the study are above the Superpave Restricted Zone (RZ) and adopted from Asphalt Institute (AI) in the Superpave Series No. 2 (SP-02) manual. The gradation ranges and adopted values for SP1 and SP2 are presented in Table 3.6 and Figures 3.1 (a-b).

Table 3.6 Aggregate Gradation of Superpave Mixtures

Mixture	**SP1**			**SP2**		
Sieve Size (mm)	**Nominal Maximum Aggregate Size- Control Point (Percentage Passing)**					
	25.0mm			**19.0mm**		
	Range	**RZ**	**Adopted**	**Range**	**RZ**	**Adopted**
37.5	100		100			
25	90-100		95	100		100
19	$\leq$ 90		83	90-100		95
12.5			70	$\leq$ 90		83
9.5			60			69
4.75		39.5-39.5	50			53
2.36	19-45	26.8-30.8	38	23-49	34.6-34.6	42
1.18		18.1-24.1	30		22.3-28.3	35
0.6		13.6-17.6	24		16.7-20.7	27
0.3		11.4-11.4	18		13.7-13.7	20
0.075	1-7		4	2-8		5



(a) SP1



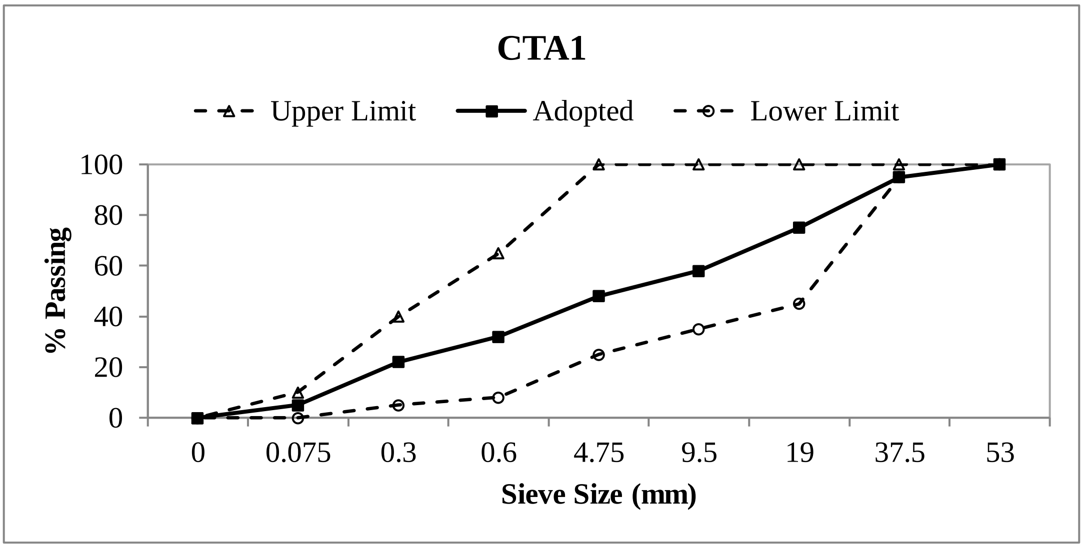
(b) SP2

Fig. 3.1 Aggregate Gradations for Superpave Mixtures

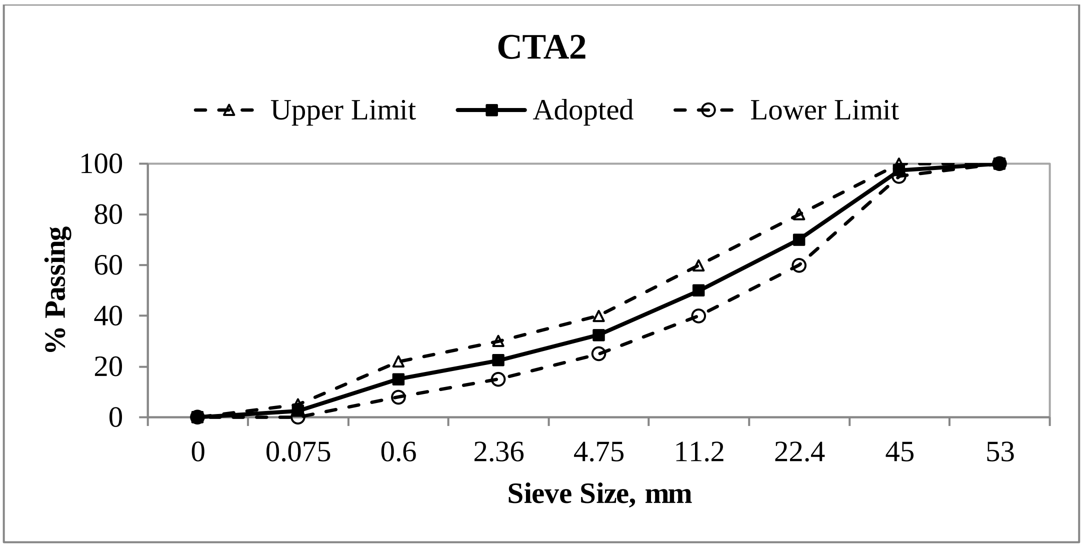
In order to prepare CTA mixtures, two aggregate gradations with two NMAS, 37.5mm and 45mm were considered and are named as CTA1 and CTA2 respectively. Both the gradations were adopted from MoRT&H (2013). The gradation ranges and adopted values for CTA1 and CTA2 are presented in Table 3.7 and Figures 3.2 (a-b). The collected aggregates were sieved as per the sieve sizes in the adopted gradation and material retaining on each sieve were separated. During mixture preparation, these separated aggregates were mixed based on the gradation requirement, and this way of aggregate mixing helped to maintain uniformity in all mixtures.

Table 3.7 Aggregate Gradation of CTA Mixtures

IS Sieve Size (mm)	CTA1	Adopted	CTA 2	Adopted
	Cumulative % by weight of total aggregate passing			
53	100	100	100	100
45	-	-	95-100	97.5
37.5	95-100	95	-	-
22.4	-	-	60-80	70
19	45-100	75	-	-
11.2	-	-	40-60	50
9.5	35-100	58	-	-
4.75	25-100	48	25-40	32.5
2.36	-	-	15-30	22.5
0.6	8-65	32	8-22	15
0.3	5-40	22	-	-
0.075	0-10	5	0-5	2.5



(b) CTA1



(b) CTA2

Fig. 3.2 Aggregate Gradations for CTA Mixtures

3.3 MIXTURE NOTATIONS

In this current investigation, both optimum and rich Superpave mixtures were prepared with one conventional asphalt, two modified asphalt for both SP1 and SP2 aggregate gradations. Similarly, the CTA mixtures were prepared with varying cement contents of 3, 5 and 7 % by weight of mixture for CTA1 and CTA2 aggregate gradations. For convenience to describe, these mixes are named as listed in Tables 3.8 and 3.9.

Table 3.8 Superpave Mixture Constituents and Notations

Aggregate Gradation	**SP1**	**SP2**
Mixture Constituents	**Notations**	
VG 30 Optimum	1-VG	2-VG
PMB 40 Optimum	1-PB	2-PB
CRMB 60 Optimum	1-CB	2-CB
VG 30 Rich	1-VR	2-VR
PMB 40 Rich	1-PR	2-PR
CRMB 60 Rich	1-CR	2-CR

Table 3.9 CTA Mixture Constituents and Notations

Aggregate Gradation	CTA1	CTA2
Cement Content (%)	**Notations**	
3	1-C3	2-C3
5	1-C5	2-C5
7	1-C7	2-C7

3.4 METHODOLOGY

3.4.1 Superpave Mixtures

Superpave mix design method as per the specification laid down by the Asphalt Institute (AI) in the Superpave Series No. 2 (SP-02) manual was adopted for the present study. The Superpave mixture requirement specified by AI, SP-02 is presented in Table 3.10. Loose Superpave mixtures were used to determine the maximum theoretical density (G_{mm}). Cylindrical specimens were prepared to evaluate the volumetric properties, Indirect Tensile (IDT) strength, fatigue behaviour, resilient modulus and moisture susceptibility characteristics of Superpave mixtures. In order to study the rutting behaviour, slab specimens were prepared.

Table 3.10 Superpave Volumetric Mixture Design Requirements

Mix design parameters		Requirement
Air void content, %		4.0
Voids in Mineral Aggregate (VMA), %	25mm NMAS	12.0 min.
	19mm NMAS	13.0 min.
Voids Filled with Asphalt (VFA), %		65 – 75
Dust-to-Binder Ratio		0.6 – 1.2
Tensile Strength Ratio (TSR), % AASHTO T 283		80 min.

Following the NMAS requirements as recommended in AI, SP-02, the diameters of the cylindrical specimen were selected as 150mm and 100mm for SP1 and SP2 respectively. However, fatigue and resilient modulus of all the mixtures were determined using 100mm in diameter test specimens. Hence, the specimens prepared with 150mm diameter were first prepared at their respective design asphalt contents (at OBC and OBC+ 0.5% extra binder) and then 100±0.5mm diameter test specimens were cored from the center (Figure 3.3) and were subsequently cut to 68±0.5mm in height (Figure 3.4).

Fig. 3.3 Coring of Superpave Specimen

Fig. 3.4 Cutting of the Specimen

Initially, the test specimens were prepared by adding trial asphalt content of 4 per cent (for 25mm NMAS mixtures) and 4.5 per cent (for 19mm NMAS mixtures) by total weight of mixture in Troxler 4140 Superpave Gyratory Compactor (SGC), shown in

Figure 3.5. This trial asphalt content was preferred based on the NMAS (AI, SP-02). Once the air voids at trial asphalt contents were determined, then the specimens were compacted at varying asphalt binder contents such as, estimated asphalt content, ± 0.5% of estimated asphalt content and + 1.0% of the estimated asphalt content by weight of total mix. Generally, asphalt mixtures are designed at specific level of compactive effort. In Superpave this is a function of the design number of gyrations, N_{des}. N_{des} is used to vary the compactive effort of the design mixture and it is a function of traffic level. Traffic is represented by the design Equivalent Single Axle Loads (ESALs). The range of values for N_{des} is shown in Table 3.11. Two other gyration levels are also of interest, the initial number of gyrations (N_{ini}) and the maximum number of gyrations (N_{max}). Generally, the test specimens are compacted using N_{des} gyrations. In this study, N_{des} of 125 gyrations corresponding to high traffic (design ESALs ≥ 30 millions) was considered according to SP-02 manual. Design for higher volume roads requires a higher gyration number (Zhao 2011).

Table 3.11 Superpave Gyratory Compactive Effort (AI, SP-02)

Design ESALs (millions)	Compaction Parameters		
	N_{ini}	N_{des}	N_{max}
< 0.3	6	50	75
0.3 to < 3	7	75	115
3 to < 30	8	100	160
≥ 30	9	125	205

Fig. 3.5 Superpave Gyratory Compactor

Following procedure was adopted for mix preparation and compaction.

Loose Mixture Preparation:

- The aggregates were proportioned and mixed as per the adopted gradation and heated to a temperature of 150 – 170 °C.
- The asphalt heated to 150 – 165 °C was added to the hot aggregates in required quantity (trial asphalt content, estimated asphalt content, ± 0.5% of estimated asphalt content and + 1.0% of the estimated asphalt content by weight of total mix) and was thoroughly mixed by maintaining a temperature of 150 – 165 °C.

- For modified asphalts, the aggregate and binder temperature should be raised to 165 – 185 °C and the mixture temperature to 150 – 170 °C.
- After mixing the mix was placed in conditioning oven for 2 hours ± 5 minutes corresponding to their compaction temperature to simulate binder aging and absorption during asphalt pavement construction.

Compaction in SGC:

- The mix was placed in respective pre-heated SGC moulds (Figure 3.6 (a-b)) of diameter 150mm and 100mm. The mould with a puck inserted received the asphalt mixture for making specimens.

(a) **(b)**

Fig. 3.6 SGC Mould: (a) 150mm Diameter (b) 100mm Diameter

- After levelling the top surface, the mould was kept inside the SGC and the glass door was closed.
- In the menu status, the pressure was set to 600kPa, angle of gyration to 1.25°, gyration rate to 30 rpm, number of gyrations to 125 and number of dwell gyrations to 10.
- When the START button was pressed, the ram moved down to apply the fixed pressure of 600kPa to the mix. The mould then tilted to 1.25° while the upper

and lower pucks remain parallel to each other and perpendicular to the original axis of the cylinder. While maintaining the pressure and preventing the mould from rotating, the mould was gyrated at 1.25° about the original central axis at 30rpm.

- As the specimen was being compacted, its height was measured after each gyration and displayed to the nearest 0.1 mm. The dot matrix printer printed the data.
- After completion of 125 gyrations and 10 dwell gyrations, the ram automatically moved up.
- Then the mould was taken out and the specimen was removed through the top of the mould with the extruder.
- The diameter, weight in air and weight in water of the specimens were noted.

3.4.1.1 Volumetric Properties

Maximum Theoretical Density

Maximum Theoretical Density of the mixture (G_{mm}) is measured for the mixture of aggregates and asphalt in loose uncompacted form, since it can provide the value after the absorption of asphalt by aggregates. Loose Superpave mixtures were prepared to determine G_{mm} and the test was conducted as per ASTM D 2041 (2011), using Asphalt Mixture Density Tester shown in Figure 3.7 and the procedure is described below.

1. The Superpave mixture was prepared using oven-dry aggregates, and the particles were separated by hand, taking care to avoid fracturing the aggregates, so that the particles of the fine aggregate portion were not larger than about 6mm. The mixture was cooled to room temperature.
2. The sample was placed directly into a cylindrical container of the Asphalt Mixture Density Tester. The container was weighed with the mixture and the net mass (mass of mixture only) was designated as A.
3. Sufficient water was added at a temperature of approximately 25°C to cover the mixture completely and then the container was closed with lid.

4. The container was placed in the machine with the mixture and water, and agitation was started immediately to remove air trapped in the mixture by gradually increasing the vacuum pressure (using a vacuum pump connected to it) until the residual pressure manometer reads 3.7 ± 0.3 kPa. The vacuum was achieved within 2 minutes. Once the vacuum was achieved, vacuum and agitation were continued for 15 ± 2 minutes.
5. The vacuum pressure was gradually released using the bleeder valve and the weighing in water was done. For determining the weight in water, the container and contents were suspended in water for 10 ± 1 minutes, and then the mass was determined. The mass of the container and mixture under water was designated as C.

The maximum specific gravity of the mixture was calculated using Equation 3.1.

$$G_{mm} = \frac{A}{[A - (C - B)]} \tag{3.1}$$

where,

G_{mm}	=	Maximum theoretical density of the mixture,
A	=	Mass of dry sample in air, g,
B	=	Mass of bowl under water, g, and
C	=	Mass of bowl and sample under water, g.

The theoretical maximum density for Superpave mixtures with 4.5% and 5% asphalt content by weight of mixture were determined by the specified method. The effective specific gravity of the aggregates was determined using Equation 3.2 for each case and the average of the two values were considered.

$$G_{se} = \frac{P_{mm} - P_b}{\frac{P_{mm}}{G_{mm}} - \frac{P_b}{G_b}} \tag{3.2}$$

where,

G_{se}	=	Effective specific gravity of aggregates
G_{mm}	=	The average theoretical maximum specific gravity determined as per ASTM D 2041
P_{mm}	=	Percentage by weight of total loose mixture
P_b	=	Asphalt content percentage by total weight of mixture
G_b	=	Specific gravity of asphalt

The G_{mm} of mixtures with different asphalt contents was then calculated as follows (Equation 3.3):

$$G_{mm} = \frac{P_{mm}}{\frac{P_s}{G_{se}} + \frac{P_b}{G_b}} \tag{3.3}$$

P_s = Aggregate content, per cent by total weight of mixture

Fig. 3.7 Asphalt Mixture Density Tester

Bulk Specific Gravity of Aggregates

The bulk specific gravity of aggregates (G_{sb}) for each specimen was calculated by knowing the specific gravities of the different materials used. It was calculated from Equation 3.4.

$$G_{sb} = \frac{100}{\frac{W_1}{G_1} + \frac{W_2}{G_2} + \frac{W_3}{G_3} + \frac{W_4}{G_4}} \tag{3.4}$$

where,

W_1	=	% by weight of coarse aggregates in total aggregate
W_2	=	% by weight of fine aggregates in total aggregate
W_3	=	% by weight of filler in total aggregate
W_4	=	% by weight of lime in total aggregate
G_1	=	Specific gravity of coarse aggregates
G_2	=	Specific gravity of fine aggregates
G_3	=	Specific gravity of filler
G_4	=	Specific gravity of lime

Bulk Density of Compacted Sample

Bulk density of each compacted specimen (G_{mb}) was calculated from Equation 3.5.

$$G_{mb} = \frac{W_a}{W_{ssd} - W_w} \tag{3.5}$$

where,

W_a	=	Weight of specimen in air
W_w	=	Weight of specimen in water
W_{ssd}	=	Saturated Surface Dry (SSD) weight of specimen

Air Voids in Total Mix (V_a)

Voids in total mix are the volume of small pockets of air between the coated aggregate particles throughout a compacted mix, expressed as a percentage of bulk volume of compacted mix. Equation 3.6 was used to determine V_a.

$$V_a = \frac{G_{mm} - G_{mb}}{G_{mm}} \times 100\ \% \tag{3.6}$$

where,

G_{mm} = Maximum theoretical density of the mixture

G_{mb} = Bulk density of the compacted specimen

Voids in Mineral Aggregates (VMA)

VMA is the volume of inter granular void space between the aggregate particles of the compacted paving mixture that includes the air voids and the volume of the asphalt not absorbed into the aggregates. Equation for VMA is given in below (3.7):

$$\mathrm{VMA} = 100 - \frac{\mathrm{G_{mb}} . \mathrm{P_S}}{\mathrm{G_{sb}}} \tag{3.7}$$

where,

G_{sb} = Bulk specific gravity of total aggregate

Voids Filled with Asphalt (VFA)

VFA is the percentage of the volume of the air voids that is filled with asphalt and was calculated using Equation 3.8.

$$\mathrm{VFA} = \frac{\mathrm{VMA} - \mathrm{V_a}}{\mathrm{VMA}} \times 100 \tag{3.8}$$

Dust to Binder Ratio or Dust Proportion (DP)

Dust to binder ratio was calculated as the percent by mass of the material passing the 0.075mm sieve (by wet sieve analysis) divided by the effective asphalt binder content (expressed as percent by mass of mix). The effective asphalt binder content was calculated from Equation 3.9.

$$\mathrm{P_{be}} = -(\mathrm{P_s} \times \mathrm{G_b}) \times (\frac{\mathrm{G_{se}} - \mathrm{G_{sb}}}{\mathrm{G_{se}} - \mathrm{G_{sb}}}) + \mathrm{P_b} \tag{3.9}$$

where,

P_{be} = Effective asphalt content, percent by total mass of mixture

P_s = Aggregate content, percent by total mass of mixture

G_b = Specific gravity of asphalt

G_{se} = Effective specific gravity of aggregate

G_{sb} = Bulk specific gravity of aggregate

P_b = Asphalt content, percent by total mass of mixture

Dust to binder ratio of each specimen was calculated using Equation 3.10.

$$DP = \frac{P_{0.075}}{P_{be}} \tag{3.10}$$

where,

$P_{0.075}$ = Aggregate content passing the 0.075mm sieve, percent by mass of aggregate

P_{be} = Effective asphalt content, percent by total mass of mixture

A sample calculation for the volumetric properties of Superpave mixture is presented in Appendix I.

3.4.1.2 Optimum and Rich Binder Content

Any asphalt mixture should have necessary binder to coat the aggregates completely and to fill a desired portion of VMA, but its quantity should not be high to result into problems like instability, bleeding etc. Initially the specimens were prepared and compacted with trial asphalt content of 4% (for 25mm NMAS) and 4.5% (for 19mm NMAS) by providing N_{des} = 125 gyrations. The Optimum Binder Content or Optimum Asphalt Content (OAC) for Superpave mixtures is usually selected to produce 4.0% air voids at N_{des} gyrations. If the air void content of mixtures with trial asphalt content varies from four percent, an estimated design asphalt content to achieve 4% air voids at N_{des} was determined and the estimated design properties at this estimated design asphalt content were calculated. The estimated asphalt content was calculated using the Equation 3.11.

$$P_{b\ estimated} = P_{bi} - (0.4 \times (4 - V_a)) \tag{3.11}$$

where,

$P_{b,\ estimated}$ = estimated asphalt content, percent by mass of mixture

P_{bi} = initial (trial) asphalt content, percent by mass of mixture

V_a = percent air voids at N_{des} (trial)

Once the estimated asphalt content was determined, the specimens were compacted at varying asphalt binder contents such as, estimated asphalt content, ± 0.5% of estimated asphalt content and + 1.0% of the estimated asphalt content by weight of total mix. These four asphalt contents are the minimum required for Superpave mix design. The mixtures properties were then evaluated to determine a design asphalt binder content. The binder content (estimated asphalt content, ± 0.5% of estimated asphalt content and + 1.0% of the estimated asphalt content) was plotted against air voids and the binder content corresponding to the specified air voids (4%) was found from the plots. In the present study, the binder content at 4% air voids was taken as OBC for Superpave mixtures. All the properties obtained at OBC were compared with the specification values to ensure that they are in the required limits.

Once the OBC has been determined, an extra binder content of 0.5% was added to this OBC to prepare rich binder mixtures. The air voids of these mixtures may vary from 3 – 4 %.

3.4.1.3 Indirect Tensile Strength

Indirect Tensile (IDT) strength is a measure of tensile strength of asphalt mixtures, measured along the diametrical plane of cylindrical specimen. This value provides an assessment of relative quality of asphalt mixtures and estimate of their rutting or cracking characteristics.

IDT strength of both optimum and rich Superpave mixtures was determined as per ASTM D 6931 (2012). The specimens prepared were kept in water bath at 25°C for about one hour (more than 30 minutes, but lesser than 2 hours is recommended). The specimen was placed over the bottom loading strip and then the upper portion of mould was lowered for the top loading strip to touch the specimen. The specimen was adjusted to align the loading strips along its diametrical plane, and then the testing mould was placed in the Marshall stability testing equipment as shown in Figure 3.8. A vertical compressive load was applied, by maintaining a deformation rate of 50mm/minutes and the maximum load required for specimen failure was noted. The IDT strength was calculated using Equation 3.12.

$$S_t = \frac{2000\ P}{\pi D t} \times 100 \tag{3.12}$$

where,

S_t	=	Tensile strength (kPa)
P	=	Failure load (N)
D	=	Diameter of specimen (mm)
t	=	Thickness of specimen in (mm)

Fig. 3.8 IDT Strength Test Setup

3.4.1.4 Rutting Resistance

Due to the increased traffic loads, surface and other layers of asphalt pavements deform leading to longitudinal depression along the wheel path, commonly known as rutting or permanent deformation. Nevelt and Thanfold (1988) defined rutting as the accumulation of unrecoverable strain in lesser magnitudes due to the heavy loads coming over the pavements. In this study, the rutting resistance of Superpave mixtures was assessed using Wheel Rut Tester (WRT), in dry condition at a testing temperature of 60°C according to EN 12697-22 (BSI 2003), shown in Figure 3.9. Slab specimen of dimensions 300mm×300mm×50mm fabricated with air voids of 7.0±0.1% at their respective design asphalt contents was used for the test. As presented in Figure 3.9, WRT is a small size wheel tracking test device consists of a loaded rubber wheel with total wheel load of 750N and contact pressure of 700kPa. It works with a principle of simple harmonic motion at a speed of 42 ± 1 passes per minute. Three slab specimens of each Superpave mixture were subjected to 8000 cycles (tracked back and forth**)** and the depth of deformation was measured at mid-point by means of Linear Variable Differential Transducers (LVDTs) (Shiva Kumar and Suresha 2017).

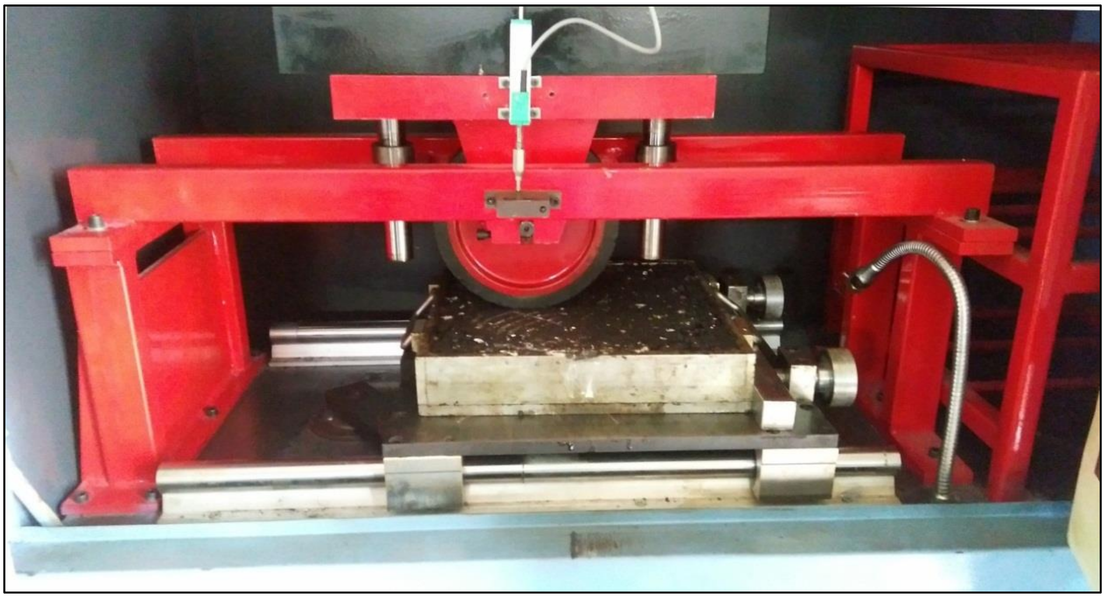


Fig. 3.9 Wheel Tracking Device

The slabs for each Superpave mixture were prepared at the corresponding binder contents and bulk density values. The aggregates required for the rutting sample was taken by measuring the required quantities according to the adopted gradation using these bulk density values and volume of mould (4500cm^3).

The procedure carried out is briefed here:

1. The aggregates were heated and mixed uniformly. Then the asphalt heated was mixed with the hot aggregate.

2. This loose Superpave mixture was compressed in a sturdy steel mould (300 × 300 × 50 mm) using a Wheel Rut Shaper to the required density and thickness (Figure 3.10).
3. Prior to slab compaction, all the asphalt mixtures were subjected to Short Term Oven Ageing (STOA) for two hours corresponding to their compaction temperature to simulate binder aging and absorption during asphalt pavement construction (Bonaquist 2011, Martin 2014).
4. The compacted specimen is shown in Figure 3.11. After 24 hours of casting, the slab was placed and all the sides were encased with confining plates in an environmental chamber for 6 hours at 60°C before testing (Kandhal and Alen, 2003).
5. The wheel was brought into contact with slab surface. The slab was subjected to 8000 wheel cycles (16000 passes) or until the rut depth reached 12.5mm, at which the test was halted (NCHRP 508 2003, Uzarowski et al. 2006, Yildirim et al. 2007, Shaheen et al. 2016) and the depression on the slab surface was recorded. A set of specimens after test are presented in Figure 3.12.

Fig. 3.10 Wheel Rut Shaper

Fig. 3.11 Compacted Rutting Slab Specimen

Fig. 3.12 Rutting Specimens after Test

3.4.1.5 Fatigue Behaviour

Fatigue failure is one of the main distress mechanisms causing degradation of pavements. Fatigue cracking due to repeated loading, results in crack initiation, crack propagation and eventually catastrophic failure of the material due to unstable crack growth (Gupta and Veeraragavan 2009). Fatigue behaviour of Superpave mixtures was assessed using Repeated Load Testing machine shown in Figure 3.13 (Ravi Shankar et al. 2013). The device is a modified version of similar equipment reported by Palit et al. (2001). This is a dynamic diametrical tensile test and the load is applied to the cylindrical specimen in a positive sinusoidal pattern. The dynamic loading is applied using the hydraulic loading system present in the machine, and is transferred to the specimen through a movable shaft. A cooling system is attached to control temperature of the machine and the pressure can be adjusted to maintain balance between input and output loads. The specimen arrangement is sown in Figure 3.14. The specimen is fixed in between two steel strips present at the top and bottom of the testing setup. The position of the specimen is adjusted in such a way that, it is exactly below the loading shaft and to apply the load along its diametrical plane. Two vertical and two horizontal LVDTs are connected with the specimen to measure the deflections. The machine is capable of applying load with frequency from 1 to 5 Hz and rest period 0 to 0.9 seconds. The machine is attached with a PC and can be controlled using a software 'fatigue 4.0', which is also used to provide various input values.

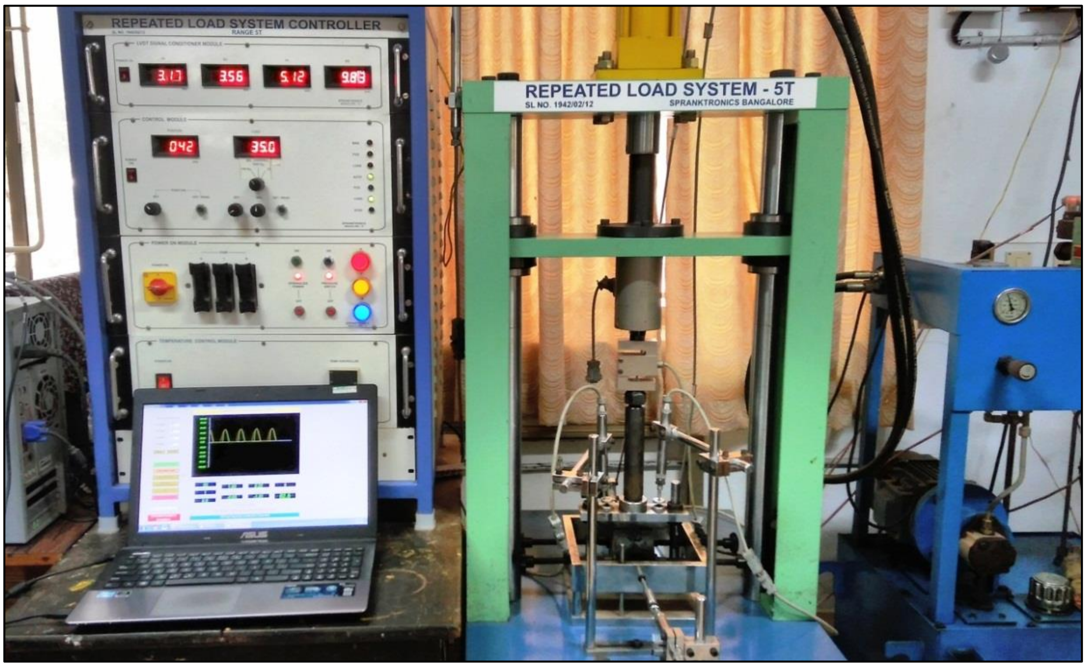


Fig. 3.13 Repeated Load Testing Machine

Fig. 3.14 Specimen Arrangement in Repeated Load Testing Machine

In this study, Superpave specimens were tested with loadings of 100kg, 300kg and 500kg. The specimens were subjected to 1Hz frequency and 0.9 s rest period, and number of cycles required for failure was considered as Fatigue Life (FL).

Other than the mixture characteristics, applied load is also a significant factor affecting the FL of the mix, along with the dimensions of the tested specimen, and hence FL value alone cannot be used to represent the fatigue behaviour of a mixture. In order to obtain a more accurate picture about the fatigue behaviour of optimum and rich Superpave mixtures, the FL values were related with the corresponding tensile stress, which includes load applied to the specimen and its dimensions. The tensile stress was calculated using Equation 3.13.

$$\text{Tensile Stress (kPa)} = \frac{2000\ \text{P}}{\pi \text{dt}} \tag{3.13}$$

where,

P = Load applied to the specimen in fatigue test, N

d = Diameter of the specimen, mm

t = Thickness of the specimen, mm

3.4.1.6 Resilient Modulus Test

Resilient modulus is an important input parameter for the design, evaluation, and analysis of the pavement structures. It is the measure of pavement response in terms of dynamic stresses and corresponding strains (Asi 2007). This repeated load indirect tension test was carried out using Repeated Load Testing machine (which is a new version of Repeated Load Testing machine mentioned in section 3.4.1.5) shown in Figure 3.15 (Panda and Mazumdar 2002, Palit et al. 2004, Panda et al. 2013). The test method was selected because of its simplicity and the ease with which samples can be prepared (Palit et al. 2004, Gupta and Veeraragavan 2009). In this study, all the Superpave specimens were tested with 150kg load (for comparison) at three different temperatures of 25°C, 30°C and 35°C (Imaninasab 2016). A constant test temperature was maintained using an environmental air chamber. Each specimen of Superpave mixtures was placed inside the chamber at the set temperature for 3 hours before testing. The specimen arrangement is similar to fatigue test mentioned in section 3.4.1.5 (Figure 3.16). The test was conducted in controlled stress mode and the loading was repeated for every 0.1s followed by a rest period of 0.9s. The compressive loads were applied in a haversine pattern with frequency of 1Hz on the vertical diametrical plane of cylindrical specimens. A pair of LVDTs were employed both in vertical and horizontal direction to measure the deformations in the specimens all throughout the test. A software and data acquisition card was used to record the deformations during the first 100 to 200 load repetitions in the specimen. The following expressions (Equations 3.14 and 3.15) (Kennedy 1978, Panda and Mazumdar 2002, Gupta and Veeraragavan 2009) were used for determining Poisson's ratio (μ), and resilient modulus (M_R) from experimental data.

Poisson's ratio, $$\mu_R = 3.59\ (H_R/V_R) - 0.27 \tag{3.14}$$

Resilient modulus, $$M_R(\mathrm{MPa}) = P\ (0.27 + \mu_R)/(H_R\,.\,h) \tag{3.15}$$

where,

P = Applied peak constant load (N)

h = Height of specimen (mm)

H_R, V_R = Horizontal and vertical deformations (mm)

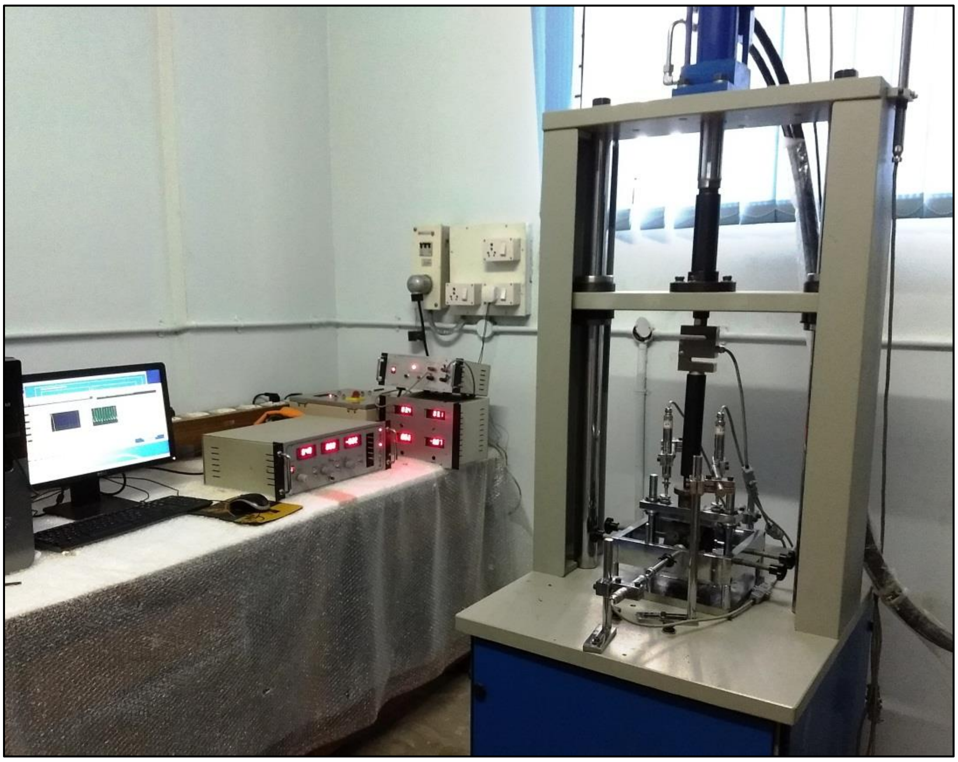


Fig. 3.15 Repeated Load Testing Machine

Fig. 3.16 Specimen Arrangement in Repeated Load Testing Machine

3.4.1.7 Moisture Susceptibility

Moisture susceptibility of asphalt mixtures is one of the main reasons for distresses in flexible pavements, which leads to loss of strength, stripping, ravelling, fatigue damage and permanent deformation. The detrimental effects of water in asphalt mixtures and the pavement distresses due to it were recognized from the 1930s itself. The moisture damage can be defined as the degradation of the mechanical properties of the material due to the presence of moisture in its microstructure (Caro et al. 2008, Hamzah et al. 2015). The moisture susceptibility of Superpave mixtures was assessed using a parameter, Tensile Strength Ratio (TSR).

Tensile Strength Ratio

The Tensile Strength Ratio (TSR) of asphalt mixtures is an indicator of their resistance to moisture susceptibility. The test was carried out according to AASHTO T 283 (2014) specifications, by loading a Superpave specimen with compressive load acting parallel to and along the vertical diametric-loading plane. This method covers preparation of compacted asphalt mixtures and the measurement of the change of diametric tensile strength resulting from the effects of water saturation and laboratory accelerated stripping phenomenon with freeze-thaw cycle. The result may be used to predict long-term stripping susceptibility of asphalt mixtures and evaluate liquid anti-stripping additives that are added to asphalt or pulverized mineral materials such as hydrated lime, which are added to the mineral aggregate.

The test is similar to IDT test mentioned in section 3.4.1.3, but in this test, the specimens are prepared with 7±0.5 % air void content to maximise the effect of moisture action. SGC specimens were prepared, by providing lesser number of gyrations to produce the required air void content. The number of gyrations for each mixture was estimated based on the method suggested by AI, SP-02 manual. It suggests a relation between the actual density of specimen at design air voids (G_{mb}) and the density estimated based on the diameter and height of the specimen after the design number of gyrations (125 in this case) (Est. G_{mb}), using a Correction Factor, C as presented in Equation 3.16.

$$\text{Correction Factor} = \frac{\text{Est. } G_{mb}}{G_{mb}} \tag{3.16}$$

The required actual density of specimen at 7% air voids (G_{mb} at 7%) is 93% of the G_{mm}. The same Correction Factor (from Equation 3.16) can be applied to estimate the density of specimen at 7% air voids based on diameter and height (Est. G_{mb} at 7%), as shown in Equation 3.17.

$$\text{Est. } G_{mb} \text{ at } 7\% = \text{Correction Factor} \times G_{mb} \text{ at } 7\% \tag{3.17}$$

From this Est. G_{mb} at 7%, corresponding estimated height of specimen for 7% air voids (Est. h at 7%) can be calculated. The gyrations v/s height data of specimen at design air voids can be used to identify the number of gyrations required producing a height of Est. h at 7% and the same can be adopted to prepare specimens at 7% air void content.

Test procedure for determination of Indirect Tensile Strength (ITS) is as follows:

1. Specimens for each Superpave mixture were prepared at corresponding binder contents by applying the number of gyrations to produce 7% air voids.
2. Two sets of specimens were prepared for testing, i.e. one to be tested dry and the other to be tested after partial saturation and moisture conditioning with a freeze-thaw cycle.
3. One set of specimens were brought to temperature of 25±1°C, by keeping them in water bath maintained at test temperature for 2 hours. These specimens are called as unconditioned specimens.
4. Another set of specimens were placed in the vacuum container filled with water at room temperature for 30 minutes. The vacuum was removed and specimens were submerged in water for 5 to 10 minutes.
5. Then specimens were placed in plastic bags containing 10 ±0.5 ml of water and sealed and kept in freezer at temperature of -18 ±3°C for minimum period of 16 hours (Figure 3.17).

Fig. 3.17 Specimens in Freezer

6. The specimens were then kept in water bath for 24 ±1 hours maintaining 60 ±1°C temperatures. This complete process in steps (3), (4), (5) and (6) is called a freeze and thaw cycle.
7. The specimens were then kept in another water bath for 2 hours maintaining temperature of 25 ±1°C. These specimens are called conditioned specimens for ITS test.
8. The conditioned and unconditioned specimens were tested for ITS using the same mould and method adopted for IDT strength mentioned in section 3.4.1.3, and ITS was calculated using Equation 3.18.

$$\mathrm{ITS} = \frac{2000\,\mathrm{P}}{\pi \mathrm{Dt}} \tag{3.18}$$

where,

ITS	=	Indirect Tensile Strength (kPa)
P	=	Failure load (N)
D	=	Diameter of specimen (mm)
t	=	Thickness of specimen in (mm)

9. The ratio of the ITS value of the conditioned subset to that of the unconditioned subset is termed as Tensile Strength Ratio (TSR) and is calculated using Equation 3.19.

$$\text{TSR} = \frac{S_2}{S_1} \text{x } 100 \tag{3.19}$$

where,

S_1 = Average tensile strength of the dry (unconditioned) subset, kPa

S_2 = Average tensile strength of the conditioned subset, kPa

3.4.2 Cement Treated Aggregate Base Mixtures

In this investigation, Cement Treated Aggregate (CTA) mixtures were prepared with varying cement contents of 3, 5 and 7 % by weight of total mixture, which are normal range of cement application of CTA base (Nusit et al. 2015, Jitsangiam et al. 2016). In base course stabilization, typically low cement contents (between 3 and 6%) are used to control shrinkage in the field (Sounthararajah et al. 2018). Austroads (2012) recommended that the minimum cement content required for stabilized material is 3% (percentage by dry mass of granular material) to achieve the structural performance characteristics in service. However, a cement content of 10% is considered as the maximum value for economic use in road construction (NAASRA 1970, Nusit et al. 2017, Sounthararajah et al. 2018). Accordingly, the cement content of CTA test specimens in this study was limited to the minimum value of 3% and the maximum value of 7%. To maintain the uniformity and accuracy in mixtures, sieve analysis for the entire aggregate sample was done and they were separated according to sieve sizes. For mix preparation, these separated aggregates were mixed as per the selected gradation. Modified compaction test was carried out as per IS 2720 (Part-8 1983) to prepare specimens at their respective Optimum Moisture Content (OMC) and Maximum Dry Density (MDD). The Portland cement and fines were initially mixed in the ribbon mixer of 125 kg capacity until a homogeneous color was get, and then the rest of aggregates were added and mixed thoroughly for one min. Finally, the estimated quantity of water was added and mixed for another 3 min (Guotang et al. 2017). After proper mixing, the fresh CTA mixture was poured into steel moulds in three layers for proper compaction. The wet mix was thoroughly compacted and then allowed to stand in cool place at ambient room temperature for about 24 hours. The specimens of different dimensions were prepared in order to test the hardened properties. After 24 hours, the specimens were demoulded and cured in water tank at room temperature for the required number of days (3, 7, 28 and 90 days). In order to evaluate the strength and other characteristics, the various specimens were cast. The details of the specimens used for various tests are given in Table 3.12.

Table 3.12 Specimen Details for Various Tests

No.	Types of Test	Specimen Type	Specimen Dimension (mm)	Relevant Standards
1	Compressive strength	cube	150	IS 516-1959
2	Flexural strength	Beam	100*100*500	IS 516-1959
3	Split tensile strength	Cylinder	150(dia)*300(ht)	IS 5816-1999
4	Modulus of Elasticity	Cylinder	150(dia)*300(ht)	IS 516-1959
5	Flexural fatigue testing	Beam	100*100*500	-

The mechanical properties such as compressive strength, static flexural strength, modulus of elasticity and split tensile strength were determined as per relevant Indian standards. For each test minimum three specimens were casted and tested using calibrated machines and the average values of the results obtained were considered.

3.4.2.1 Flexural Fatigue Testing

The flexural fatigue tests on CTA samples were carried out on beam specimens of dimensions 100 × 100 × 500 mm using Repeated Load Testing machine shown in Figure 3.18 (Palankar et al. 2017). The machine consists of a double acting hydraulic cylinder with suitable mounting flanges. It is associated with a power pack unit consisting of pump coupled with motor (1 HP, 3-phase, 1440 rpm), valves and filters, heat exchanger (cooling system), servo valve, pressure gauge etc. Load sensing devices were used to sense the applied load to the specimen during testing. The loading is generally with half sinusoidal waveform (zero -maximum load-zero). The application frequency can be between 1-5 Hz with or without rest period. In the test Control Unit was used to monitor the load and the repetitions. It is connected to the PC with an ADD ON Card to acquire or log the data. Five specimens were tested for each mix at each stress level. The static flexural strength of the mixes was recorded at 90 days of curing, before the fatigue test was conducted. The beam specimen was loaded at the same span (i.e. 400mm) as it was loaded in case of static flexural tests. The specimens were subjected to loading using constant amplitude half sinusoidal wave form at a frequency of 4Hz without any rest period. The test setup was calibrated applying initial loading and the frequency of loading was maintained constant throughout the test for all specimens. The minimum load is maintained as zero while the maximum load was adjusted based on the required stress ratio (ratio of applied stress to the modulus of rupture of CTA). Fatigue testing is a very time consuming and expensive process and a large number of samples have to be tested. An upper limit of 1,00,000 cycles was selected in this investigation. The test was terminated when the failure of the specimen occurred or the upper limit was reached, whichever was earlier. In the experimental investigation, it was found that the specimen reached the upper limit at stress ratio 0.60. Hence, in this study, the stress ratios were limited to 0.70, 0.75, 0.80 and 0.85. As the uncertainty involved in this test is very high, the maximum stress level was restricted to 0.85. The fatigue tests were conducted at different stress ratios i.e. 0.70, 0.75, 0.80 and 0.85 to obtain a relationship between different stress ratios (SR) and the number of cycles to failure (N). The test was conducted at the end of 90 days of curing of CTA specimen in order to eliminate the errors occurring, due to the strength development of CTA mixes after 28 days of curing. The Fatigue life (N) i.e. the number of cycles up to failure for each sample was recorded. The specimen failed after test is shown in Figure 3.19.

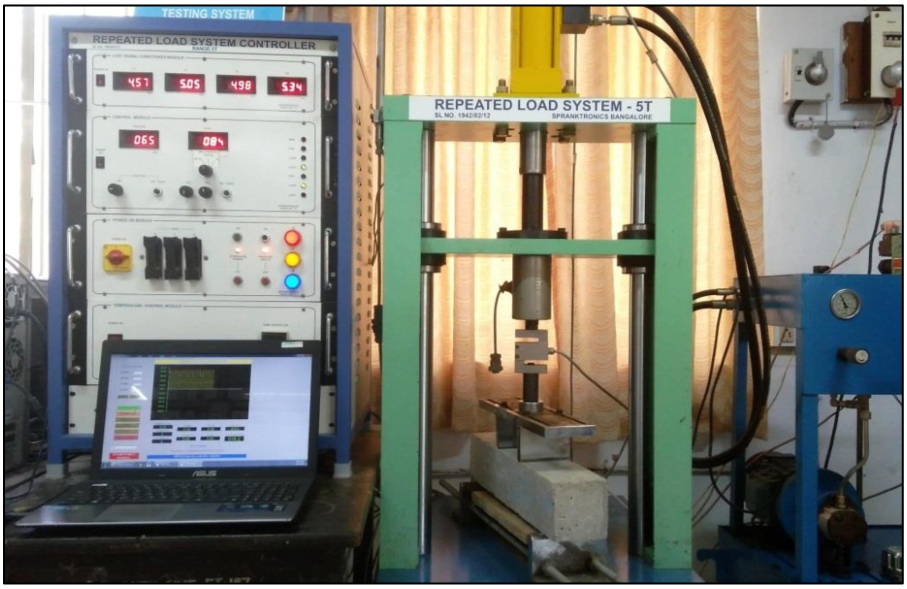


Fig. 3.18 Repeated Load Testing Machine

Fig. 3.19 Failed CTA Specimen after Test

3.4.3 KENPAVE Analysis

In this present research work, the critical stress strain analysis has been carried out to predict the performance of pavement structures proposed in the study. To examine the performance of pavement sections, multilayer KENLAYER analysis was carried out to compute stresses and strains. The process of computing the stresses and strains in a multilayer flexible pavement system is highly complex and time consuming even after assuming that all the layers are homogenous, isotropic and continuous. It is with this background that many researchers have developed multilayer analysis algorithms like DAMA, ILLI-PAVE, MICH-PAVE, VESYS, PDMAP, ELSYM-5, BISAR, CHEVE and KENLAYER etc., which are very effective in solving majority of multilayer problems. However, KENLAYER algorithm, developed by Yang (2004) is considered to be the best performed algorithm in many of the reported case studies and accurate enough to give satisfactory stress, strain values. It also offers much flexibility like number of classes of axle loads and number of seasons to be incorporated while inputting the data. Hence, in the present work, the KENLAYER program is used and analyzed. This KENLAYER analysis includes damage analysis and distress models to predict the life of the new pavement. The damage analysis is based on the horizontal tensile strain at the bottom of specified layers, usually the surface layer, and the vertical compressive strain at the top of the sub-grade layer. Instead of reading in the

Z coordinates, simply specifying the total Number of Layers for Top Compression (NLTC), the Layer Number for Bottom Tension (LNBT), and the Layer Number for Top Compression (LNTC), the program will determine the Z coordinates of all necessary points and compute the required strains. If several radial coordinate points are specified under single wheel or several x and y coordinate points under multiple wheels, the program will compare the strains at these points and select the most critical ones for damage analysis. Any combination of traffic and pavement layers can be tried using KENPAVE by providing inputs like number of layers, layer thickness, Poisson's ratio, resilient modulus, tyre pressure and wheel load, and critical strains are obtained as outputs.

CHAPTER 4

OPTIMUM AND RICH SUPERPAVE MIXTURES

4.1. GENERAL

In this chapter, the observations made on properties of optimum and rich Superpave mixtures prepared with conventional binder and modified asphalt binders are discussed. Mixtures prepared with asphalt modified with suitable additives in appropriate proportions perform better than mixes with conventional asphalt. Optimum mixtures were prepared with OBC designed at 4% air voids, whereas rich mixtures were prepared by adding extra binder content of +0.5% over the OBC. The volumetric properties are initially discussed and this is followed by a detailed discussion on the strength related properties like IDT strength, rutting resistance, fatigue behaviour, resilient modulus and moisture susceptibility. The results obtained from the experiments are analyzed and discussed.

4.2 EXPERIMENTAL INVESTIGATION

Superpave mixtures were prepared with aggregate gradations SP1 and SP2, using one type of conventional binder, VG 30 and two types of modified asphalt binders, PMB 40 and CRMB 60. Cylindrical specimens were prepared in SGC at trial asphalt content of 4 and 4.5% (for SP1 and SP2 mixtures respectively) to determine the estimated asphalt content and then they were prepared at estimated asphalt content, ±0.5% of estimated asphalt content and + 1.0% of the estimated asphalt content by weight of mix to check volumetric properties. IDT strength, fatigue, resilient modulus and TSR tests were conducted on cylindrical specimens prepared at respective OBC and at +0.5% extra binder over OBC for each mixture, whereas rutting test was conducted on slab specimens.

4.3 RESULT AND DISCUSSION

4.3.1 Volumetric Properties

The VMA, VFA, percent air voids, and dust proportion are very important to asphalt mixtures, because these volumetric properties significantly affect the durability and stability of mixtures (AI 1993, Wang et al. 2000). Volumetric properties of both SP1 and SP2 mixtures with VG 30, PMB and CRMB are presented in Tables 4.1 – 4.7 and Figures 4.1 – 4.3. In SP1 mixtures, estimated asphalt content to achieve 4% air voids at 125 gyrations was determined as 5.05, 4.67 and 4.52 % for VG, CB and PB respectively, whereas it was respectively 5.21, 4.75 and 4.60 % for SP2 mixes. G_{mm} was observed to be decreasing with asphalt content for all the six mixtures, whereas G_{mb} increased with asphalt content first, attained a maximum value and then decreased, which is shown in Figure 4.1. Air voids were decreasing with asphalt content, following the general trend in asphalt mixtures, and the values were in the range 5.44 – 3.23 % and 5.27 – 3.25 % for SP1 and SP2 gradations respectively. Figure 4.2 shows that all the mixtures are satisfying the minimum VMA requirement of 12% and 13% for SP1 and SP2 mixtures respectively, as specified by Asphalt Institute (AI) in the Superpave Series No. 2 (SP-02) manual.

Table 4.1 Properties of Superpave Mixtures with Conventional and Modified Binders at Trial Asphalt Content

Mixture	**1-VG**	**1-CB**	**1-PB**	**2-VG**	**2-CB**	**2-PB**
Trial AC (%)	**4.0**	**4.0**	**4.0**	**4.50**	**4.50**	**4.50**
G_{mm}	2.493	2.503	2.508	2.471	2.478	2.482
G_{mb} (g/cc)	2.327	2.361	2.375	2.329	2.363	2.376
V_a (%)	6.64	5.69	5.31	5.76	4.64	4.26
VMA (%)	15.77	14.57	14.04	16.16	14.93	14.45
VFA (%)	57.91	60.96	62.18	64.33	68.95	70.51
D/B ratio	1.04	1.08	1.10	1.12	1.14	1.15

Table 4.2 Properties of SP1 Mixture with VG 30 Binder (1-VG)

Property	Asphalt content by weight of mix			
	4.55	**5.05**	**5.55**	**6.05**
G_{mm}	2.472	2.453	2.434	2.416
G_{mb} (g/cc)	2.344	2.352	2.346	2.338
Va (%)	5.15	4.12	3.62	3.23
VMA (%)	15.64	15.82	16.46	17.20
VFA (%)	67.08	73.96	78.03	81.22
D/B ratio	0.91	0.82	0.74	0.68
OBC (%)	5.15			

Table 4.3 Properties of SP1 Mixture with CRMB (1-CB)

Property	Asphalt content by weight of mix			
	4.17	**4.67**	**5.17**	**5.67**
G_{mm}	2.496	2.477	2.458	2.440
G_{mb} (g/cc)	2.363	2.373	2.368	2.354
V_a (%)	5.35	4.22	3.66	3.50
VMA (%)	14.64	14.73	15.33	16.27
VFA (%)	63.45	71.38	76.15	78.52
D/B ratio	1.03	0.91	0.82	0.74
OBC (%)	4.82			

Table 4.4 Properties of SP1 Mixture with PMB (1-PB)

Property	Asphalt content by weight of mix			
	4.02	**4.52**	**5.02**	**5.52**
G_{mm}	2.507	2.488	2.469	2.451
G_{mb} (g/cc)	2.371	2.386	2.377	2.371
V_a (%)	5.44	4.12	3.76	3.25
VMA (%)	14.20	14.12	14.90	15.54
VFA (%)	61.71	70.84	74.77	79.12
D/B ratio	1.09	0.96	0.86	0.77
OBC (%)	4.69			

Table 4.5 Properties of SP2 Mixture with VG 30 (2-VG)

Property	Asphalt content by weight of mix			
	4.71	**5.21**	**5.71**	**6.21**
G_{mm}	2.463	2.444	2.426	2.408
G_{mb} (g/cc)	2.339	2.342	2.338	2.326
Va (%)	5.04	4.17	3.61	3.39
VMA (%)	15.97	16.29	16.88	17.75
VFA (%)	68.45	74.38	78.62	80.90
D/B ratio	1.07	0.97	0.88	0.81
OBC (%)	5.28			

Table 4.6 Properties of SP2 Mixture with CRMB (2-CB)

Property	Asphalt content by weight of mix			
	4.25	**4.75**	**5.25**	**5.75**
G_{mm}	2.487	2.468	2.450	2.431
G_{mb} (g/cc)	2.356	2.364	2.358	2.352
Va (%)	5.27	4.23	3.75	3.25
VMA (%)	14.94	15.11	15.78	16.43
VFA (%)	64.76	72.04	76.23	80.19
D/B ratio	1.21	1.08	0.98	0.89
OBC (%)	4.98			

Table 4.7 Properties of SP2 Mixture with PMB (2-PB)

Property	Asphalt content by weight of mix			
	4.10	**4.60**	**5.10**	**5.60**
G_{mm}	2.497	2.478	2.459	2.441
G_{mb} (g/cc)	2.366	2.376	2.369	2.361
V_a (%)	5.24	4.14	3.69	3.28
VMA (%)	14.45	14.56	15.26	15.98
VFA (%)	63.73	71.59	75.81	79.49
D/B ratio	1.27	1.13	1.01	0.92
OBC (%)	4.75			

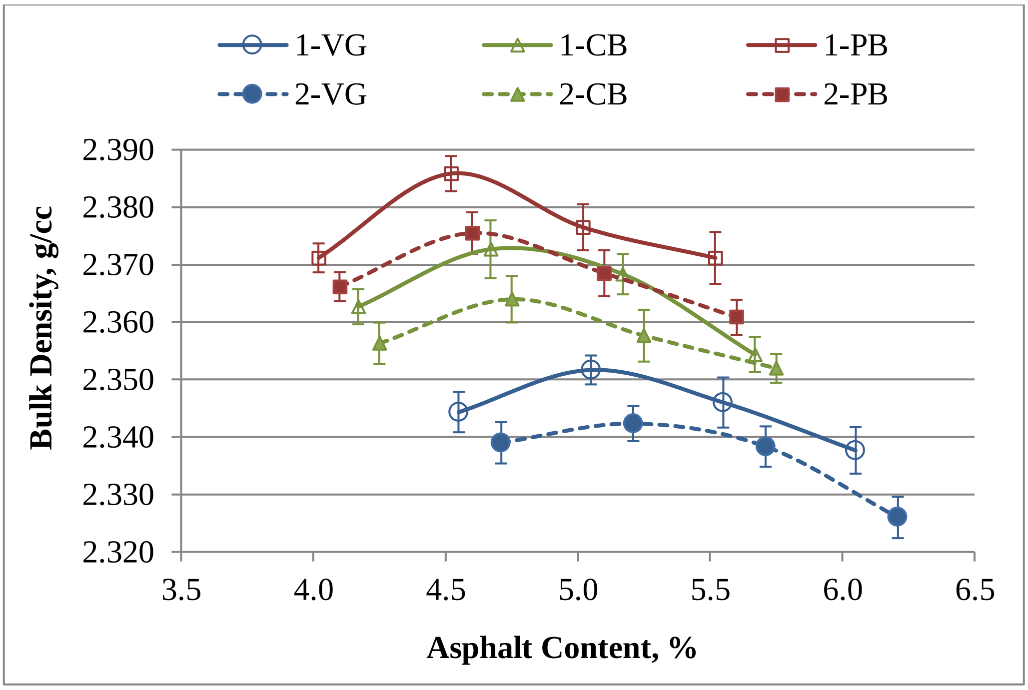


Fig. 4.1 Bulk Density of Superpave Mixtures with Conventional and Modified Binders

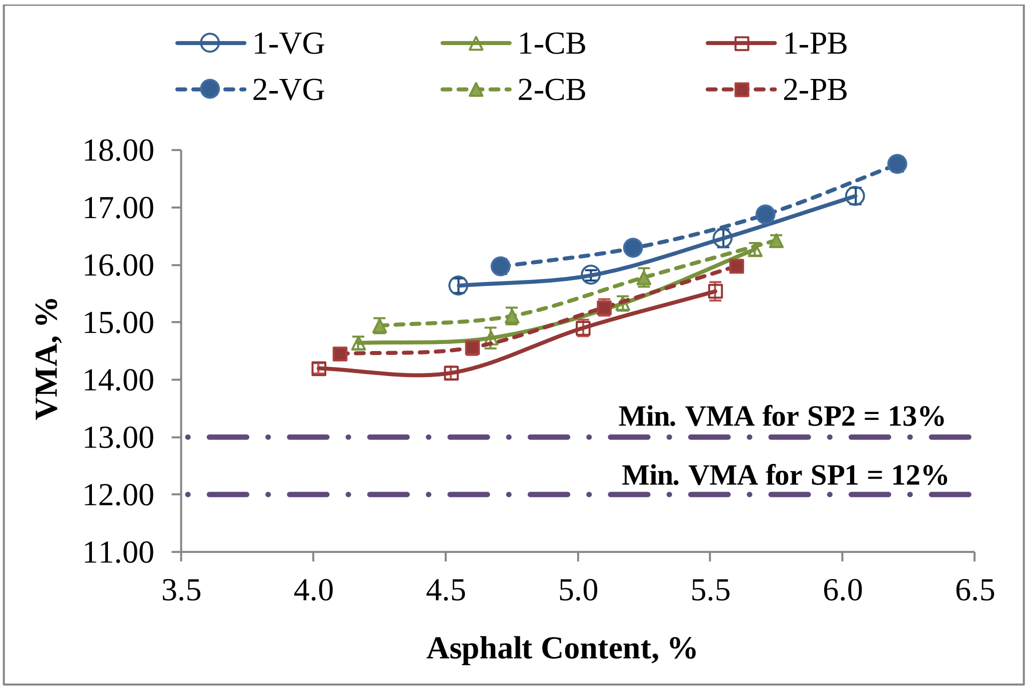


Fig. 4.2 VMA of Superpave Mixtures with Conventional and Modified Binders

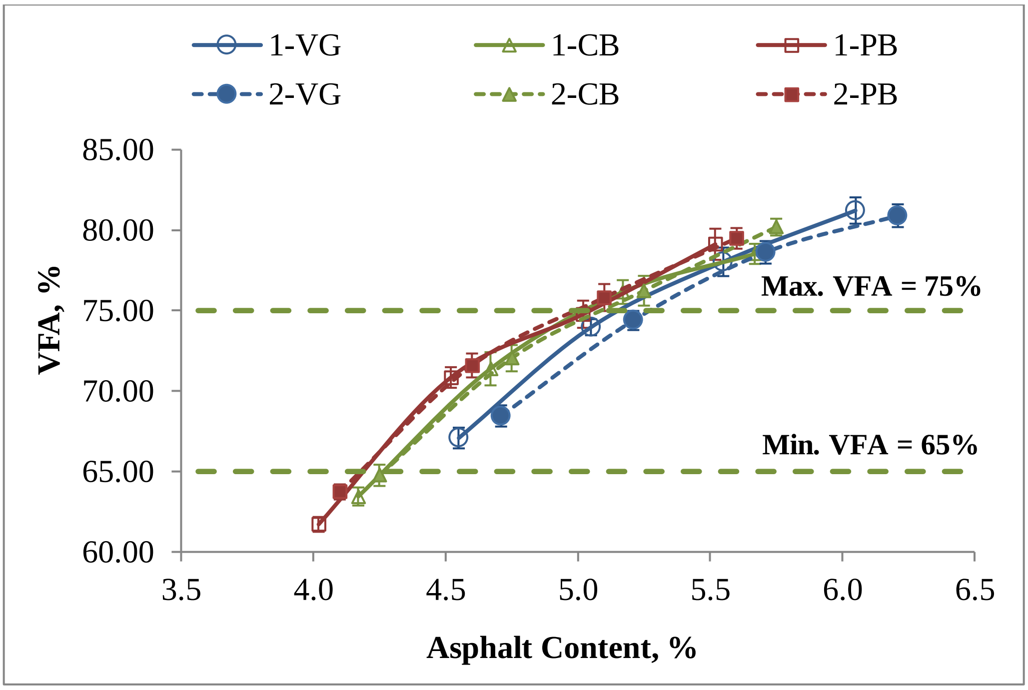


Fig. 4.3 VFA of Superpave Mixtures with Conventional and Modified Binders

In SP1 mixtures, OBC was determined as 5.15, 4.82 and 4.69 % for VG, CB and PB respectively, whereas it was respectively 5.28, 4.98 and 4.75 % for SP2 mixes. Decreasing trend of OBC for mixtures with PMB and CRMB may be attributed to the lesser absorption of binder by the aggregates (Sargand and Kim 2003). The OBC of the SP2 mixtures are slightly higher compared to the SP1 mixtures. This can be explained by the higher surface area in SP2 mixtures, where more binder is needed to coat the finer aggregates (Khedr and Breakah 2012, Ahmad et al. 2014). Properties of all the mixtures at OBC are presented in Table 4.8. Highest density values were obtained for PB mixtures which are due to the improved properties of PMB. The dust to binder ratio were showing a decreasing trend with asphalt content and for all the mixtures prepared at OBC the values were within specified design requirements of 0.6 – 1.2. For 1-VG and 2-VG mixes, the dust proportion found to be on the lower side of the criteria range, which resulted in the high OBC and high VFA values in the mix (Ahmad et al. 2014). VFA values were also within the specified design requirement of 65 – 75 % for samples prepared with OBC and it showed a gradual increasing trend with asphalt content (Figure 4.3). The binder content for 1-VR, 1-CR, 1-PR mixtures were calculated as 5.65, 5.32 and 5.19 % respectively, whereas it was respectively 5.78, 5.48 and 5.25 % for 2-VR, 2-CR, 2-PR. The properties of mixtures prepared at OBC+0.5% extra binder (rich binder mixtures) are presented in Table 4.9. The air void of all the mixtures were within the range of 3.53 – 3.68 %. The extra binder fills the voids in the mineral aggregate and thus creates a low air-void mixture (Tarefder and Bateman 2012).

Table 4.8 Properties of Superpave Mixtures with Conventional and Modified Binders at OBC

Mixture	**1-VG**	**1-CB**	**1-PB**	**2-VG**	**2-CB**	**2-PB**
OBC (%)	5.15	4.82	4.69	5.28	4.98	4.75
G_{mm}	2.449	2.471	2.482	2.442	2.460	2.473
G_{mb} (g/cc)	2.351	2.373	2.384	2.342	2.362	2.374
V_a (%)	3.99	4.00	3.95	4.08	3.97	3.97
VMA (%)	15.92	14.86	14.33	16.36	15.39	14.73
VFA (%)	74.97	73.11	72.45	75.09	74.20	73.05
D/B ratio	0.80	0.88	0.92	0.95	1.03	1.09

Table 4.9 Properties of Rich Binder Mixtures with Conventional and Modified Binders

Mixture	**1-VR**	**1-CR**	**1-PR**	**2-VR**	**2-CR**	**2-PR**
AC (%)	5.65	5.32	5.19	5.78	5.48	5.25
G_{mm}	2.430	2.453	2.463	2.423	2.441	2.454
G_{mb} (g/cc)	2.342	2.362	2.375	2.335	2.355	2.364
V_a (%)	3.62	3.68	3.58	3.63	3.53	3.65
VMA (%)	16.68	15.68	15.10	17.04	16.09	15.54
VFA (%)	78.32	76.53	76.32	78.71	78.04	76.51
D/B ratio	0.73	0.80	0.83	0.87	0.93	0.98

4.3.2 Indirect Tensile Strength

IDT strength was determined for optimum and rich Superpave mixtures and the results are presented in Figures 4.4 and 4.5 respectively. Tensile strength was found to be higher for SP1 mixtures for all types of asphalt binders and this can be attributed to the presence of more coarse aggregate sizes in the mixture compared to SP2. In case of optimum mixtures PB produced the highest strength among all mixture types and VG had the least. This is due to the characteristics of asphalt binder. Rich binder mixes with the same aggregate gradations and binder types exhibited lesser strength compared to optimum mixtures and this may be due to the increased flexibility of the rich binder mixtures. Similar findings were noticed in the studies conducted by Zhao (2011) on HMA performance properties.

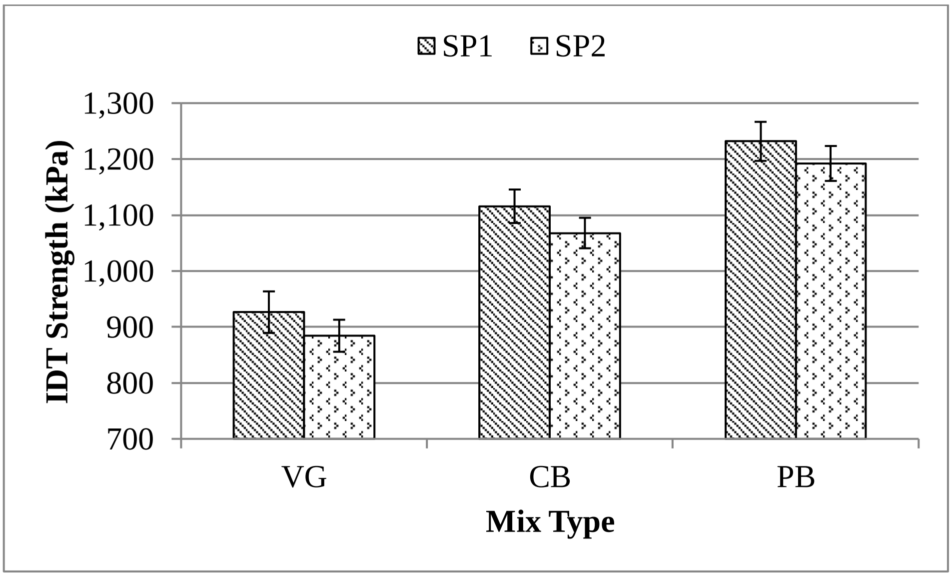


Fig. 4.4 IDT Strength of Optimum Mixtures with Conventional and Modified binders

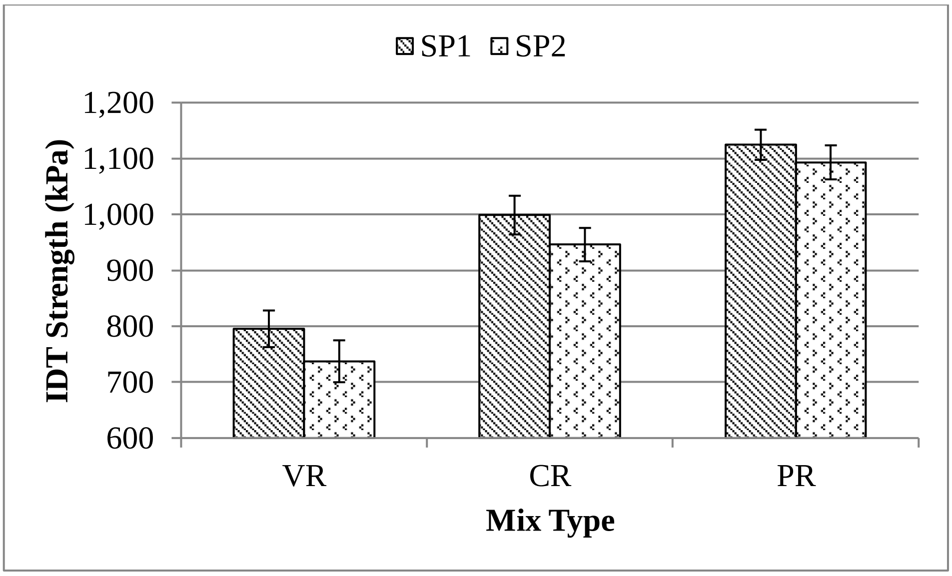


Fig. 4.5 IDT strength of Rich Mixtures with Conventional and Modified binders

4.3.3 Rutting Resistance

Wheel Tracking Device was used to determine the rut deformation for each Superpave mixture and the results are presented in Figures 4.6 and 4.7. The deformation recorded at all wheel cycles was lesser for PB and CB mixtures, which is an indication of better rut resistance. The improved rut resistance for these mixtures is due to the higher viscosity of the modified binders (Sirin et al 2008). The test was conducted for 8,000 wheel cycles, and the final deformation was obtained as 5.46, 4.86, 4.19, 6.18, 5.43, and 4.65 mm for 1-VG, 1-CB, 1-PB, 2-VG, 2-CB and 2-PB respectively.

As expected, the rich mixtures with both gradations were found to be more susceptible to rutting compared to optimum mixtures. The increase of binder content led to a higher rutting depth due to the reduction of friction between aggregate particles. However, all mixes provided acceptable rutting resistance, with a maximum rut depth of 7.72mm, which is significantly below 12.5mm, the recommended value (by NCHRP 508 2003, Uzarowski et al. 2006, Yildirim et al. 2007, Shaheen et al. 2016), even if 0.5% extra binder was added to the OBC. After 8000 cycles, the deformation was obtained as 6.91, 6.30, 5.51, 7.72, 6.93 and 6.00 mm for 1-VR, 1-CR, 1-PR, 2-VR, 2-CR and 2-PR respectively. The difference of rutting values is thought to be attributed to different binder properties of mixtures at a test temperature of 60°C (Lee et al. 2007).

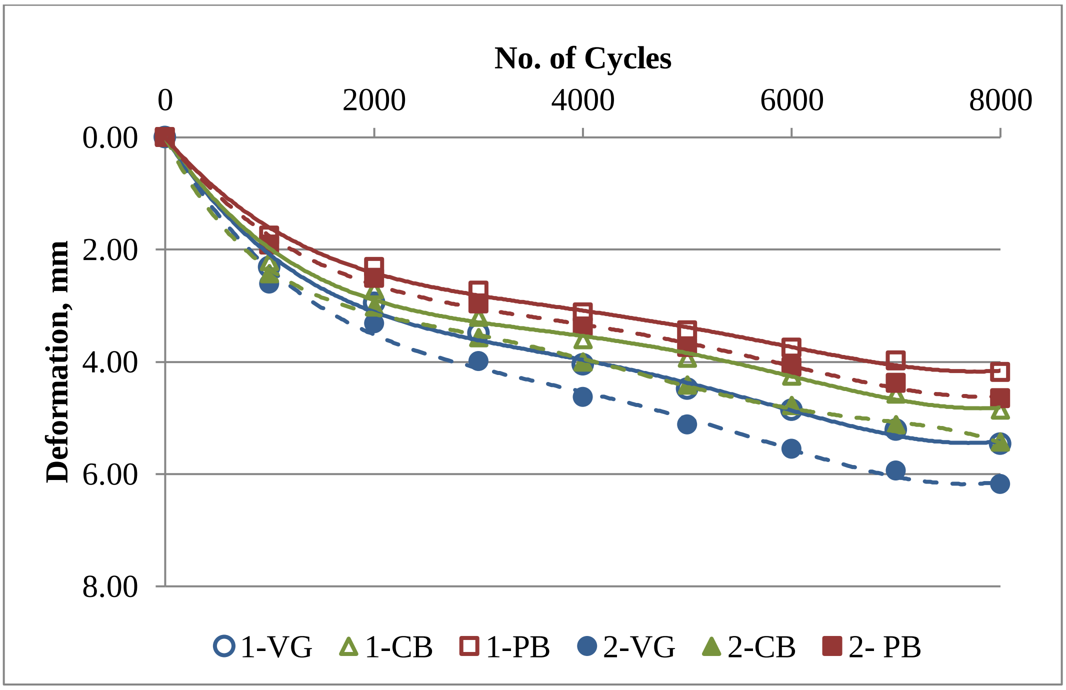


Fig. 4.6 Rutting Deformation of Optimum Superpave Mixtures

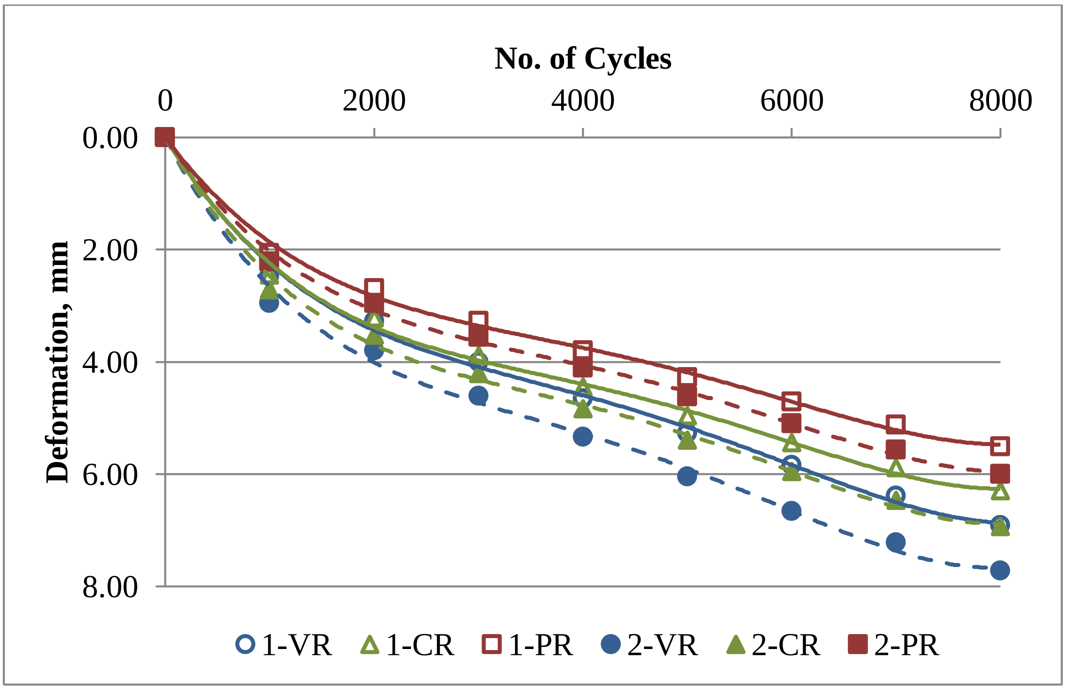


Fig.4.7 Rutting Deformation of Rich Superpave Mixtures

4.3.4 Fatigue Behaviour

Both optimum and rich Superpave mixtures were tested for 100, 300 and 500 kg dynamic loads and the results are tabulated in Tables 4.10 and 4.11 respectively. From the results, it can be noticed that, for a particular mixture, with the increase in applied load, the specimen fails faster with lesser number of cycles, providing lesser FL. The improved fatigue behaviour of SP1 mixtures, due to their aggregate structure compared to SP2, is evident from the increased FL values obtained for these mixes. All the mixes with modified binders showed higher resistance to fatigue failure (with an upper hand for mixes with PMB) as compared to mixes with conventional binder, irrespective of the applied load. The better fatigue performance of modified mixes is rendered by the added elasticity from polymer and rubber addition (Sargand and Kim 2003). Furthermore, rich Superpave mixtures had higher fatigue life values compared to optimum mixtures. Similar findings were noticed by Shaheen et al. (2016) in an investigation of Superpave mixtures prepared with different binder types and binder contents. This improved fatigue life of rich mixtures may be due to the extra 0.5% binder content added to the optimum binder, which increases the thickness of the binder coating, and lowers the internal friction between the aggregate particles and creates softer mixtures (Shaheen et al. 2016).

The dimensions of the test specimen and the load applied to it, also affect the FL value and hence FL was represented along with the tensile stress value as shown in Figures 4.8 and 4.9. This gives a clear indication of the fatigue behaviour of mixes and shows that polymer modified mixes are having better fatigue behaviour followed by crumb rubber modified and conventional mixes both in case of optimum and rich mixtures.

Table 4.10 FL of Optimum Superpave Mixtures

Mixture	Applied Load (kg)	Applied Load (N)	Fatigue Life
1-VG	100	981	9141
	300	2943	4784
	500	4905	1908
1-CB	100	981	10570
	300	2943	5789
	500	4905	2514
1-PB	100	981	11445
	300	2943	6357
	500	4905	2820
2-VG	100	981	8693
	300	2943	4592
	500	4905	1725
2-CB	100	981	10036
	300	2943	5460
	500	4905	2338
2-PB	100	981	10652
	300	2943	6145
	500	4905	2642

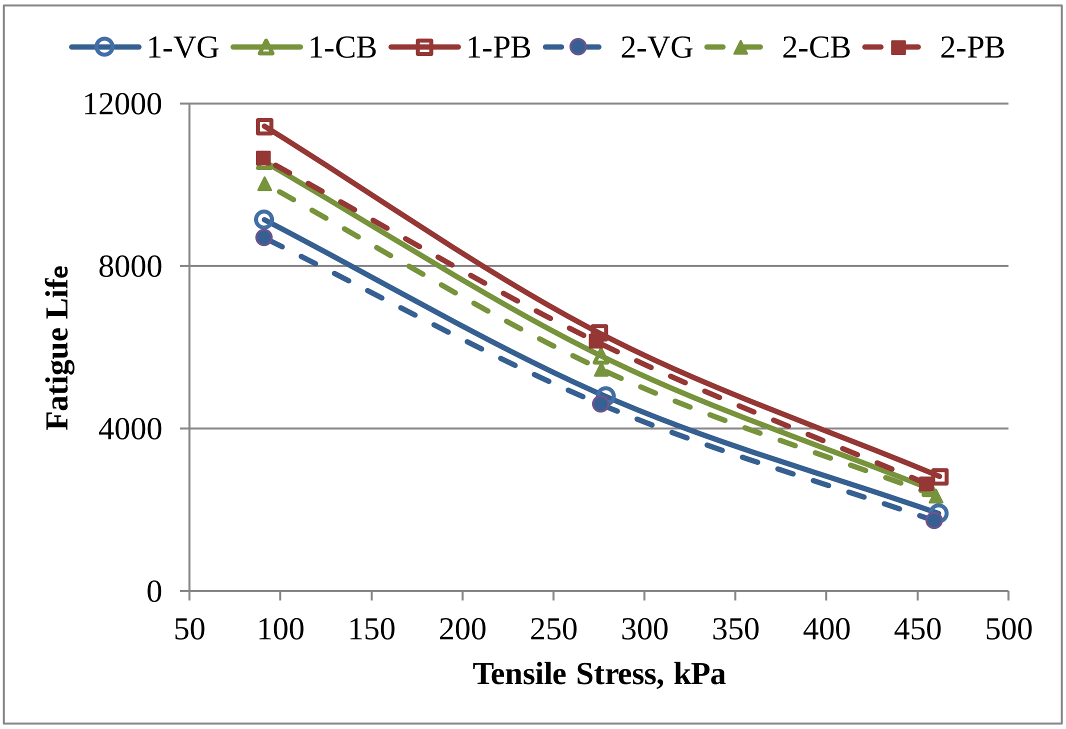


Fig. 4.8 Variation of Tensile Stress with FL for Optimum Superpave Mixtures

Table 4.11 FL of Rich Superpave Mixtures

Mixture	Applied Load (kg)	Applied Load (N)	Fatigue Life
1-VR	100	981	12632
	300	2943	6798
	500	4905	2696
1-CR	100	981	14692
	300	2943	8121
	500	4905	3420
1-PR	100	981	15348
	300	2943	8915
	500	4905	3917
2-VR	100	981	12052
	300	2943	6118
	500	4905	2213
2-CR	100	981	13941
	300	2943	7557
	500	4905	3189
2-PR	100	981	14912
	300	2943	8366
	500	4905	3452

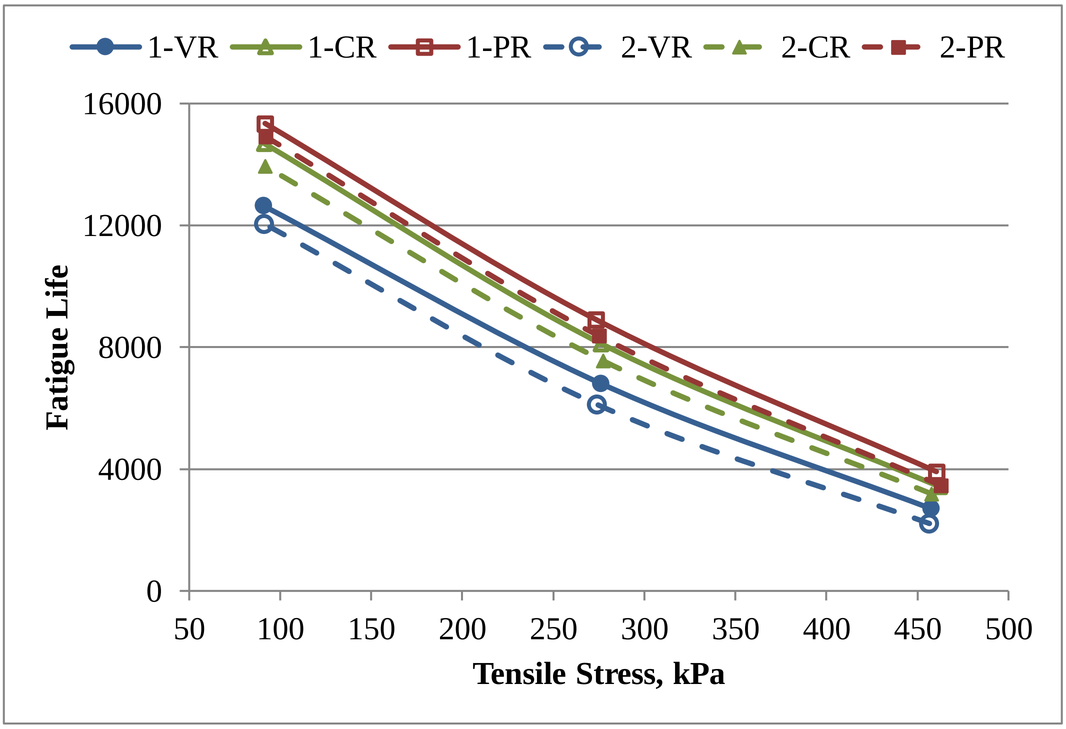


Fig. 4.9 Variation of Tensile Stress with FL for Rich Superpave Mixtures

4.3.5 Resilient Modulus Test

The resilient modulus of asphalt mixtures is calculated by using the measured horizontal and vertical deformations (Kennedy 1977) and is defined as the ratio between applied stress and recoverable strain (Ping and Xiao 2008). The Poisson's ratio (μ) and resilient modulus of elasticity (M_R) of all Superpave mixtures were tested at three most prevailing temperatures of 25°C, 30°C and 35°C by applying 150kg load. The test results of both optimum and rich mixtures are presented in Tables 4.12 and 4.13 respectively. It can be seen from the tables that, as the temperature increases, the resilient modulus values significantly reduce regardless of the binder types. Also Poisson's ratio values tends to increase with an increase in temperature. At higher temperatures, the mixes tend to soften and lose strength with an increase in elastic deformation (Al-Abdul-Wahhab and Al-Amri 1991, Gupta and Veeraragavan 2009). Among two aggregate gradations used, higher resilient modulus was observed for SP1 compared to SP2 and the performance of optimum mixtures was observed to be better than rich mixtures. The results also reveal that increasing the binder content by 0.5% had relatively less impact on asphalt mixture stiffness. The modulus of all the Superpave mixtures obtained at different temperatures is considerably higher than those of conventional mixtures as specified by IRC for flexible pavement design (IRC 37 2012).

Table 4.12 Resilient Modulus of Optimum Superpave Mixtures

Mix Type	Resilient Modulus (M_R) (MPa)			Poisson's Ratio (μ)		
	25°C	30°C	35°C	25°C	30°C	35°C
1-VG	2654	2225	1543	0.35	0.36	0.36
1-CB	3226	2619	1858	0.28	0.31	0.33
1-PB	3562	2823	2079	0.27	0.30	0.31
2-VG	2308	1970	1364	0.37	0.38	0.39
2-CB	2962	2536	1710	0.30	0.34	0.36
2-PB	3295	2684	1873	0.27	0.33	0.34

Table 4.13 Resilient Modulus of Rich Superpave Mixtures

Mix Type	Resilient Modulus (M_R) (MPa)			Poisson's Ratio (μ)		
	25°C	30°C	35°C	25°C	30°C	35°C
1-VR	2164	1746	1217	0.38	0.40	0.41
1-CR	2832	2202	1662	0.33	0.35	0.37
1-PR	3063	2478	1865	0.32	0.34	0.34
2-VR	1950	1565	1025	0.43	0.42	0.42
2-CR	2520	2086	1433	0.35	0.36	0.38
2-PR	2662	2293	1554	0.32	0.36	0.35

4.3.6 Moisture Susceptibility

4.3.6.1 Tensile Strength Ratio

The ITS test was conducted on all Superpave specimens prepared with 7% air voids for both unconditioned and conditioned cases to determine the moisture resistance of mixtures in terms of TSR and the results presented in Figures 4.10 – 4.11 and Tables 4.14 – 4.15 are obtained. Higher ITS and TSR values were observed for mixtures with modified binders in both gradations compared to those with unmodified binder. From the Figures 4.10 and 4.11, it can be seen that for both conditioned and unconditioned cases, ITS value was observed to be higher in the case of optimum mixtures. The TSR values of all mixtures are greater than the Superpave minimum criterion of 80% and can be concluded that all the mixtures have sufficient resistance to moisture damage. Comparing the TSR for the twelve mixture types from the Table 4.14 and Table 4.15, it is clear that rich binder mixtures are less susceptible to moisture than optimum binder mixtures and this may be attributed to the thicker film of the rich binder coatings on the aggregate particles.

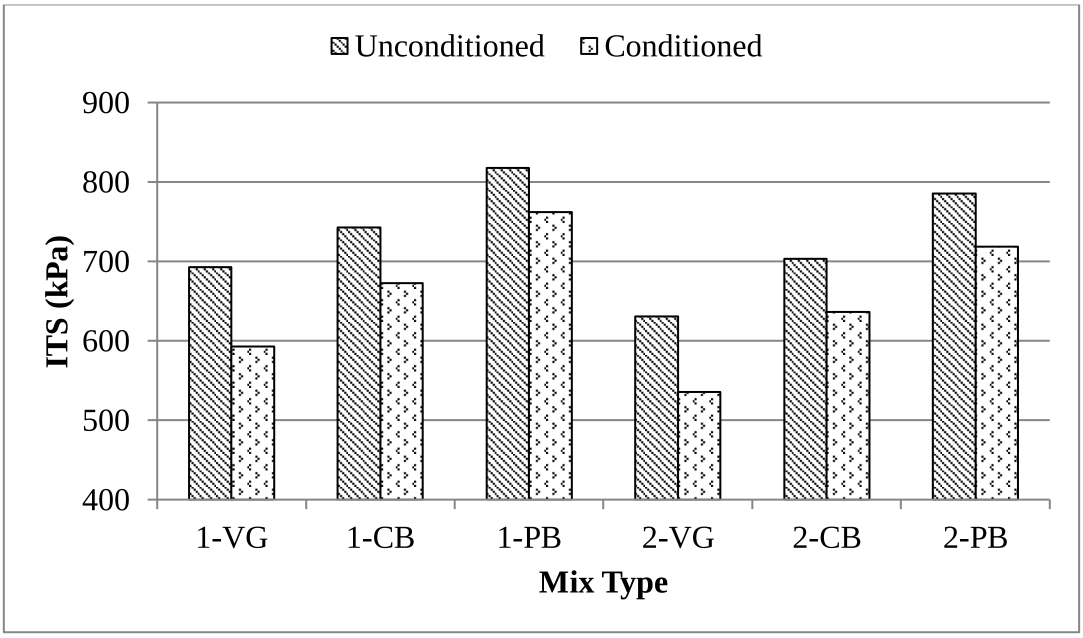


Fig. 4.10 ITS of Conditioned and Unconditioned Optimum Superpave Mixtures

Table 4.14 TSR of Optimum Superpave Mixtures with Modified and Unmodified Binders

Mix Type	1-VG	1-CB	1-PB	2-VG	2-CB	2-PB
TSR (%)	85.53	90.54	93.19	84.92	90.47	91.49

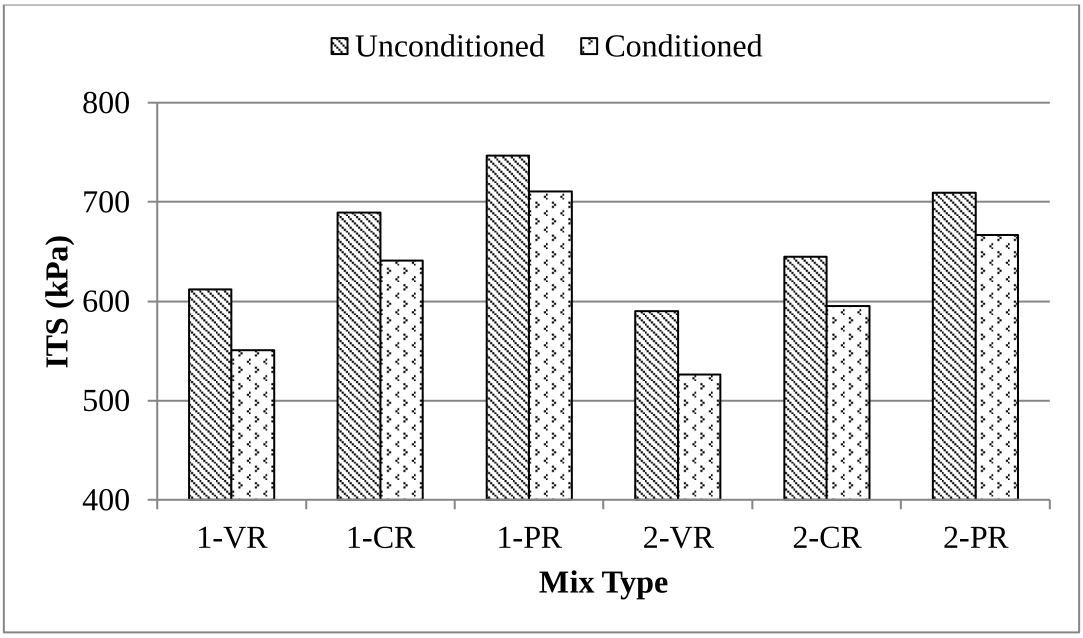


Fig. 4.11 ITS of Conditioned and Unconditioned Rich Superpave Mixtures with Modified and Unmodified Binders

Table 4.15 TSR of Rich Superpave Mixtures with Modified and Unmodified Binders

Mix Type	**1-VR**	**1-CR**	**1-PR**	**2-VR**	**2-CR**	**2-PR**
TSR (%)	89.96	93.04	95.17	89.14	92.29	94.02

4.4 SUMMARY

In this chapter the preparation and laboratory performance of optimum and rich Superpave mixtures with one conventional asphalt binder, VG 30 and two modified asphalt binders, PMB 40 and CRMB 60 were discussed. Volumetric properties showed that, for both SP1 and SP2, PMB produced better mixes compared to other binders. This is due to the improved characteristics of PMB as listed in Table 3.3 (Chapter 3). Among all optimum Superpave mixtures prepared in the study, PB mixes had the minimum OBC (4.69% for SP1 and 4.75% for SP2) whereas it was the highest for mixes with VG 30 binder (5.15% for SP1 and 5.28% for SP2). In case of IDT strength, rutting, resilient modulus and ITS tests, optimum mixtures performed better compared to rich binder mixtures. However, in case of fatigue behaviour and moisture susceptibility tests, rich binder mixtures performed better compared to optimum mixtures. The improved performance of rich mixtures is due to the extra 0.5% binder content added to the optimum binder, which increases the thickness of the binder coating, and lowers the internal friction between the aggregate particles and creates softer mixtures. The increase in fatigue behaviour of rich mixtures was found to be 28 – 40 % compared to optimum mixtures. This confirms the research idea of using rich binder mixture for the asphalt base layer of LLAP structure. For polymer modified mixes, the reduction in resilient modulus due to increase in temperature was about 14 – 33 % and 17 – 31 % for optimum mixtures and rich mixtures respectively. The modulus of all the Superpave mixtures obtained at different temperatures are 1.3 to 2.4 times greater than that of conventional mixtures as specified by IRC for flexible pavement design (IRC 37 2012). The improved stiffness of the optimum mixtures compared to conventional mixtures (currently recommended in India), makes it a promising substitute mixture for the intermediate layer of LLAP. Even though the resilient modulus values of rich mixtures are less than optimum mixture, it can resist the fatigue cracking experiencing in the bottom layer of LLAP to some extent. Moisture resistance of all the Superpave mixtures with all the binders was evident from the TSR, where all specimens showed a value higher than 84.9%. The presence of polymer and rubber particles in the asphalt binder provided higher moisture resistance to the mixture with TSR above 90.4% than conventional binder. Comparing the final rut deformations of optimum mixtures (5.46, 4.86, 4.19, 6.18, 5.43, and 4.65 mm for 1-VG, 1-CB, 1-PB, 2-VG, 2-CB and 2-PB respectively) with that of rich binder mixtures (6.91, 6.30, 5.51, 7.72, 6.93 and 6.00 mm for 1-VR, 1-CR, 1-PR, 2-VR, 2-CR and 2-PR respectively), it can be concluded that optimum binder mixtures are more suitable for intermediate layer of LLAP.

All laboratory test conducted in the study indicated that SP1 gradation produced better mixtures than SP2, except in the case of moisture resistance, which was observed to be similar for both gradations. The better performance of SP1 mixtures is due to the higher NMAS and the presence of more coarse aggregate sizes, compared to SP2 (Shiva Kumar and Suresha 2017).

CHAPTER 5

CEMENT TREATED AGGREGATE BASE MIXTURES

5.1 GENERAL

Cement stabilized mixtures are normally used as base or sub-base course within pavement structures so as to improve the structural capacity of pavement structure in terms of strength and stiffness (Farhan et al. 2016). The performance of pavements with stabilized base has historically been very good. Good quality and uniform base material are two factors for LLAPs (Chen et al. 2011). Cement stabilized aggregate primarily consists of graded aggregate, fines, a specified percentage of cement and a proper amount of water. The differences between CTA and normal concrete are that CTA contains a small amount of cement and is field prepared by rolling compaction. Although there are similarities between CTA and roller compacted concrete in preparing method and component materials, roller compacted concrete usually has a similar cement content with which utilized in normal concrete (Farhan et al. 2016, Guotang et al. 2017). In CTA, cement hydration starts from the time water being added. Cement will hydrate with water in the gaps of compacted mixture. The hydrated products with adhesive capacity like fibrous C–S–H gel interweave together to connect fine aggregates and fill the inner voids of framework. The thickness of newly formed hydration products at cement particles surface increases, resulting in the reduction of the distance between particles as the hydration proceeds. Gradually a semi rigid structure is constructed. Hence, the effect of cement can be called as "Physical (filling)-chemical (hydration binding) action" (Guotang et al. 2017).

In this chapter mechanical properties and flexural fatigue performance of CTA mixtures are discussed. The cement content of CTA test specimens in this study was limited to the minimum value of 3% and the maximum value of 7%.

5.2 EXPERIMENTAL INVESTIGATION

CTA mixtures were prepared with aggregate gradations CTA1 and CTA2, using cement contents of 3, 5 and 7 % by weight of mixture. The modified proctor tests were performed to determine the OMC, and the MDD. Cube specimens were prepared to check compressive strength at different curing periods. Cylindrical specimens were used to determine modulus of elasticity and split tensile strength, while static flexural strength and flexural fatigue tests were performed on prisms samples.

5.3 RESULT AND DISCUSSION

5.3.1 Compaction Characteristics

Compaction tests were conducted as per IS 2720 (1983), on aggregate base treated with cement contents of 3, 5 and 7 % immediately after mixing. The results of MDD and OMC obtained are tabulated in Table 5.1.

Table 5.1 Compaction Test Results of CTA Mixes

Mixture Type	MDD (g/cc)	OMC (%)
1-C3	2.227	7.47
1-C5	2.274	8.16
1-C7	2.310	9.0
2-C3	2.20	7.04
2-C5	2.245	7.66
2-C7	2.271	8.20

It can be observed that, increasing the cement content in the range of 3 – 7 % increased the density slightly and the density of CTA1 mixtures was found to be higher for all cement contents. This can be attributed to the presence of more percentage of fine aggregate particles in the mixture compared to CTA2. OMC of CTA1 mixtures was also slightly greater than that of CTA2 and this may be due to larger specific surface area of finer materials. Additionally, as shown in Table 5.1, incorporation of cement result in an increase in density since the specific gravity of cement (3.14 g/cc) is higher than that of fines (2.7). The increase rate in density was about 0.004 g/cc for every 2% increase in cement content.

5.3.2 Compressive Strength

Although cement treated base courses within pavement structure are normally designed based on tensile stress at the bottom of this layer, compressive strength is one of the widely adopted methods to characterize cement stabilized mixtures due to its simplicity and their established correlations with other properties (Farhan et al. 2016). Compressive strength tests of CTA specimens were conducted as per IS: 516-1959 after 3, 7, 28, and 90 days of curing and the results are depicted in Figure 5.1. As can be seen from Figure, the general trend is an increase in compressive strength value as cement content increase, which is expected since cement will increase the strength of material's matrix and the bond between particles due to increase in the hydration products. In addition, there was an improvement in this parameter with curing time, since hydration is a time dependent process (Barišić et al. 2015, Farhan et al. 2016). It can be noticed that as the cement content increased from 3 to 7 %, the 28 days compressive strength of CTA1 and CTA2 mixtures increased from 10.3 to 14.8 MPa and 9.1 to 12.9 MPa, respectively. Among two aggregate gradations used, higher compressive strength was observed for CTA1 mixtures. All CTA mixtures tested in this study satisfied the 7-days compressive strength requirements of 4.5 to 7 MPa as specified by IRC for flexible pavement design (IRC 37 2012).

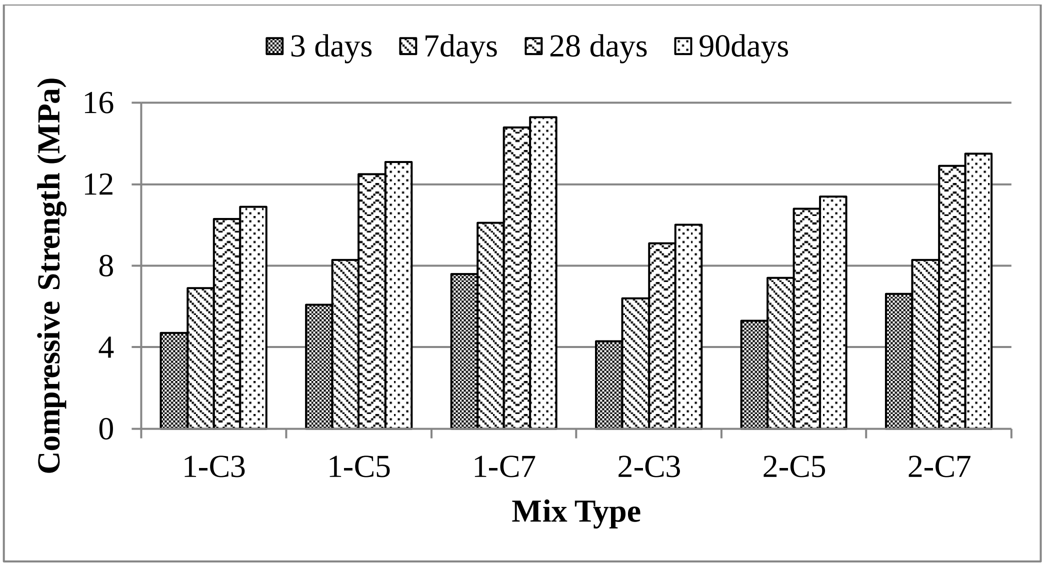


Fig 5.1 Compressive Strength of CTA Mixtures at Different Curing Periods

5.3.3 Tensile Properties and Modulus of Elasticity

The tensile strength of CTA is always considered to be a significant material parameter for designing pavement structures, since the bottom of the treated aggregate layer suffers the tensile stress. In general, flexural beam tests and split tensile tests have been employed to evaluate the tensile strength of treated aggregate mixture. Values deduced from those tests differ from each other due to the difference in stress distribution under various testing conditions (Xuan et al. 2012, Ebrahim Abu El-Maaty Behiry 2013). The flexural test is a preferable test used since it reflects really stress condition in field (Guotang et al. 2017).

The flexural strength of CTA mixtures was determined as per IS: 516 (1959), while the split tensile strength was evaluated as per IS: 5816 (1999). Figures 5.2 and 5.3 shows the flexural strength of CTA after 7, 28 and 90 days of curing, and split tensile strength after 28 days of curing respectively. From the figures, it can be observed that flexural and split tensile strength values increased proportionally with higher amounts of cement and/or long curing period. Similar to compressive strength all the CTA mixtures achieved the minimum 28-days flexural strength requirements of 1.4MPa as suggested by the IRC for flexible pavement design (IRC: 37-2012).

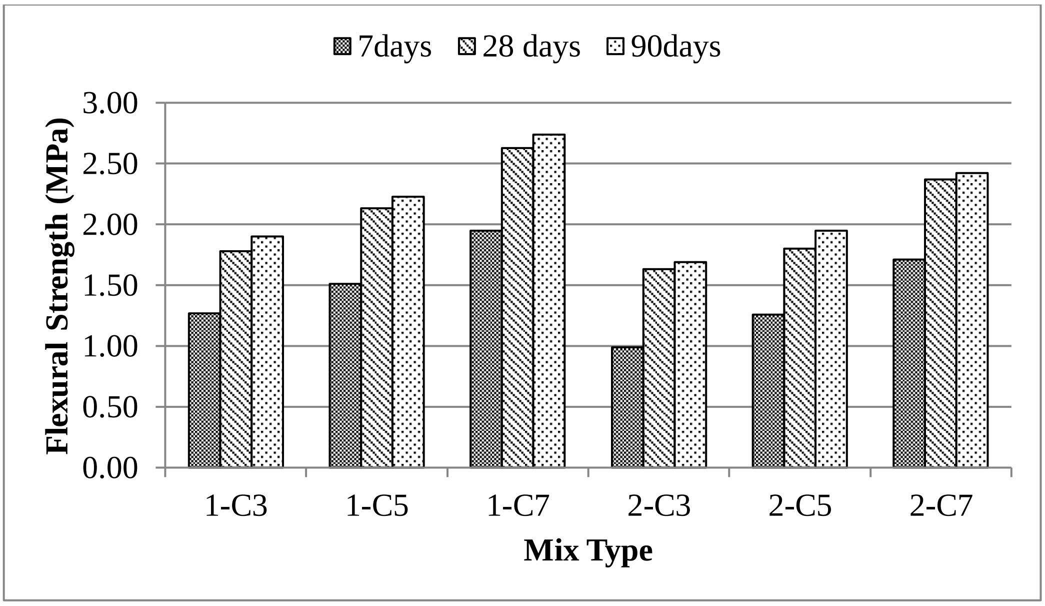


Fig. 5.2 Flexural Strength of CTA Mixtures at Different Curing Periods

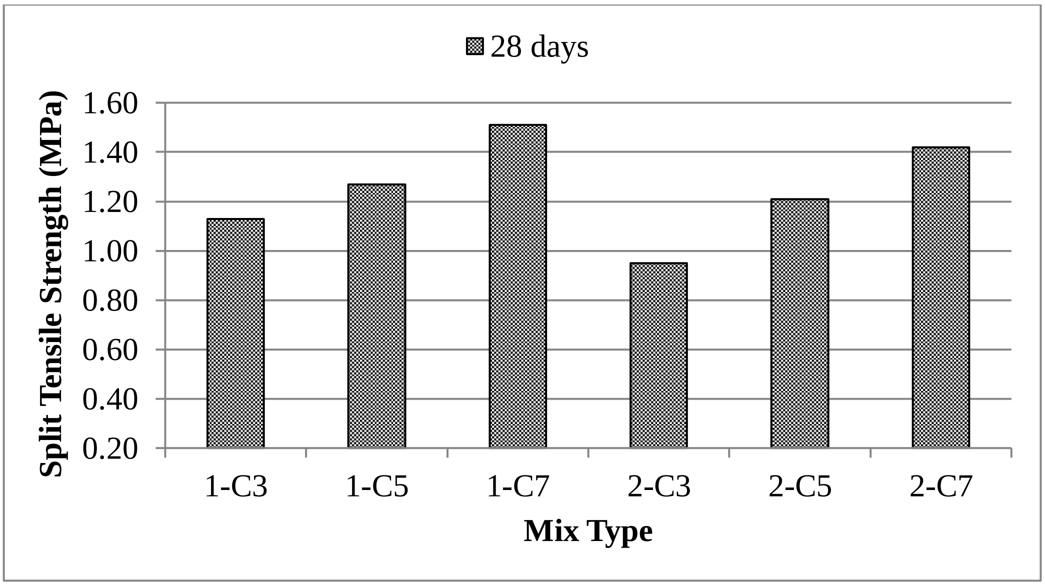


Fig. 5.3 Split Tensile Strength of CTA Mixtures at 28-Days of Curing

The test for modulus of elasticity of CTA mixtures was conducted as per IS 516:1959 after 28 days of curing. It can be said, based on the results presented in the Figure 5.4 that the greater the cement content, the greater the stiffness of the mixture. It can be inferred that, because the stiffness of aggregate is the same for all mixtures having different cement contents, the stiffness of mixture is controlled, to a large extent by the stiffness of the matrix (fines plus cement) and its bond with aggregate particles

(Farhan et al. 2016). The range of results obtained is considerably higher than those currently recommended for the design of cemented-treated base pavements in India (5000 MPa) (IRC 37-2012). Increase in cement contents strengthens the interface between the aggregates and cement paste, which in turn increases the modulus of elasticity (Barišić et al. 2016).

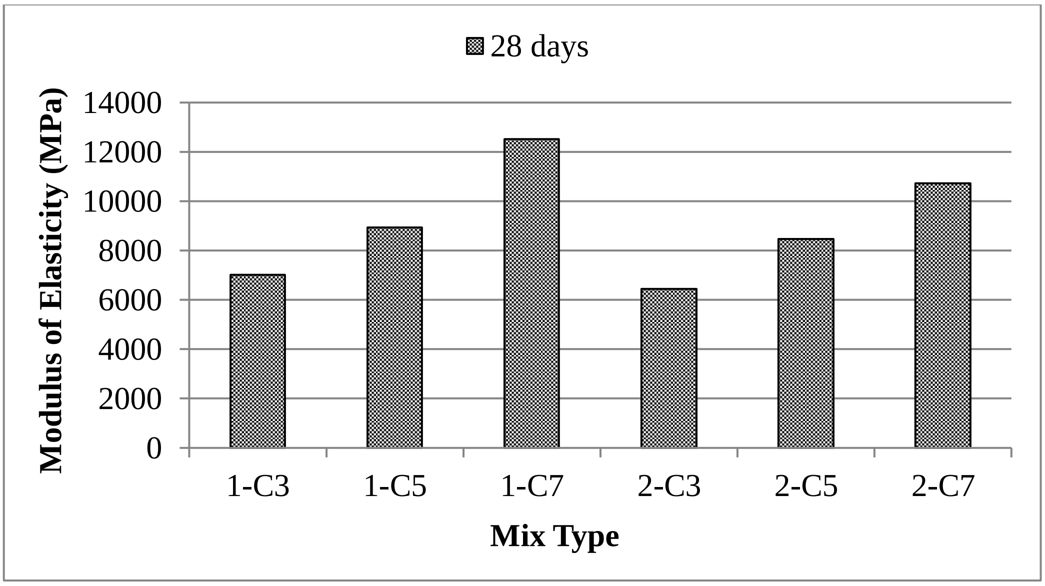


Fig. 5.4 Modulus of Elasticity of CTA Mixtures at 28-Days of Curing

5.3.4 Flexural Fatigue Performance

The primary deterioration mode of cement treated bases is fatigue cracking caused by repeated application of traffic induced stresses. A greater understanding of the performance of cement treated granular materials under repetitive heavy traffic loading is essential for pavement engineers to prevent the premature failure of cement treated bases in service. In the last few decades, researchers have given increased attention to both laboratory and field characterization of the flexural fatigue behaviour of cement treated granular materials. Many laboratory based studies (Sobhan and Das 2007, Yeo 2012, Austroads 2014, Gnanendran and Paul 2016, Jitsangiam et al. 2016) and large scale experimental studies (Hugo and Ebbs 2004, Du Plessis et al. 2008, Cai and Wang 2013) have been undertaken on the fatigue performance of cement treated granular materials and several fatigue models have been developed to predict their in service fatigue live ranging from 15 to 40 years (Southararajah et al. 2018)

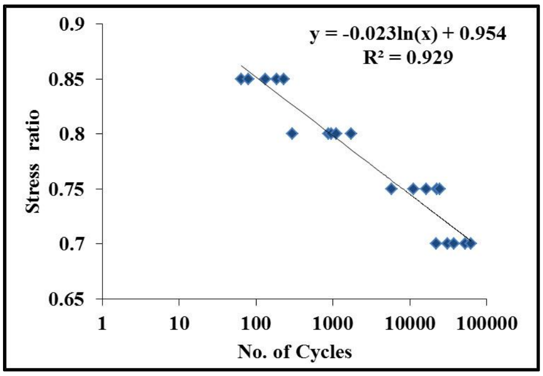
Fatigue testing is a very time consuming and expensive process and large number of samples have to be tested. In the present study, the flexural fatigue performance of CTA specimens prepared with cement contents of 3, 5 and 7 % have been investigated. The specimens were subjected to different stress ratios (0.70, 0.75, 0.80 and 0.85) and the number of cycles for failure of the specimen was determined.

S-N Curve

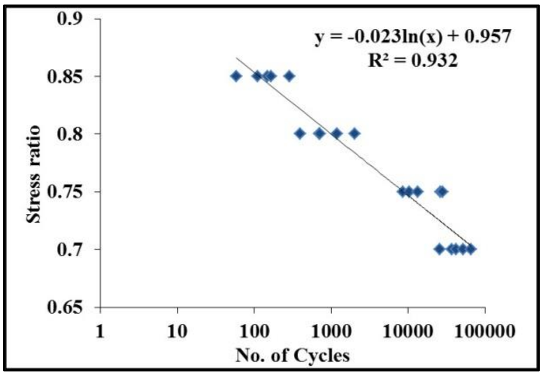
Most of the researchers adopted the relationship between stress level (ratio of maximum applied stress to the modulus of rupture) and the number of repetitions ‘N’ causing failure, to predict the fatigue behaviour of the material (Majumder et al. 1999). The relationship established is known as Wohler equation and is shown by S-N Curve or Wohler curve (Oh 1986). The use of S-N curve or Wohler curve is the most basic method of representing the fatigue behaviour of concrete and cement treated base specimen. S-N curve is an important parameter in the analysis of fatigue data in which ‘S’ denotes the stress amplitude and ‘N’ denotes the number of cycles to complete failure. This S-N curve enables one to predict the mean fatigue life of cement treated base mixtures under given stress level or amplitude of cyclic stress (Roylance 2001, Sounthararajah et al. 2018).The fatigue life (N) i.e. the number of cycles up to failure for all CTA mixtures are tabulated in Table 5.2. The S-N curves obtained by plotting stress ratio (SR) v/s number of cycles (N) up to failure are presented in Figure 5.5 (a-f).

Table 5.2 Fatigue Life of CTA Mixtures

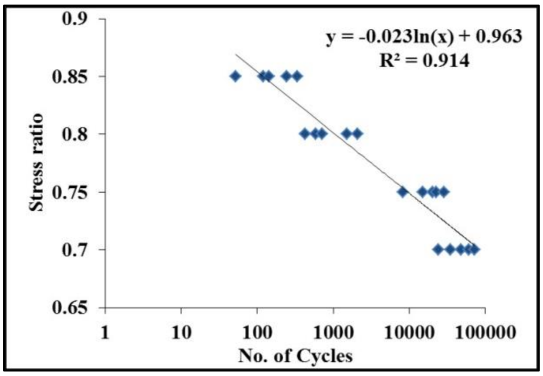
Mix Type	Specimen no	Stress Ratio			
		0.85	0.8	0.75	0.7
	No of cycles to failure (Fatigue Life), N				
1-C3	1	64	298	5741	21873
	2	80	869	11278	31064
	3	134	947	16334	37583
	4	185	1105	22574	52625
	5	231	1735	24369	61773
1-C5	1	58	391	8637	25809
	2	110	698	10336	37129
	3	150	721	13317	42204
	4	166	1187	26263	51751
	5	287	2021	27924	66248
1-C7	1	52	424	8354	24369
	2	121	596	15055	35256
	3	142	717	20613	47965
	4	244	1520	22634	61634
	5	340	2118	29132	72232
2-C3	1	61	247	6626	19827
	2	78	624	9373	31411
	3	111	724	12985	39671
	4	140	1010	17858	43410
	5	228	1107	22052	56787
2-C5	1	55	433	4599	24912
	2	69	524	7688	30060
	3	133	815	16800	35175
	4	174	952	23305	48244
	5	256	1458	26916	61617
2-C7	1	69	413	8591	20686
	2	92	617	15285	27713
	3	125	970	18794	44178
	4	162	1042	21386	53155
	5	283	1424	29539	65291



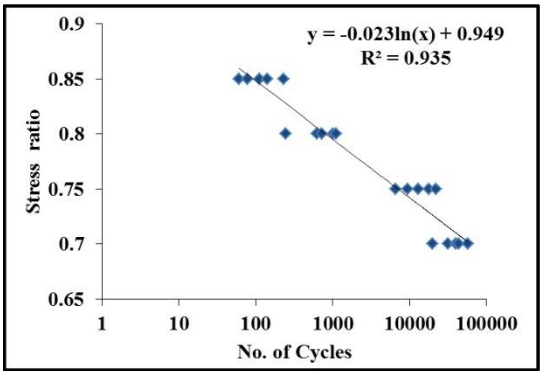
(a) 1-C3



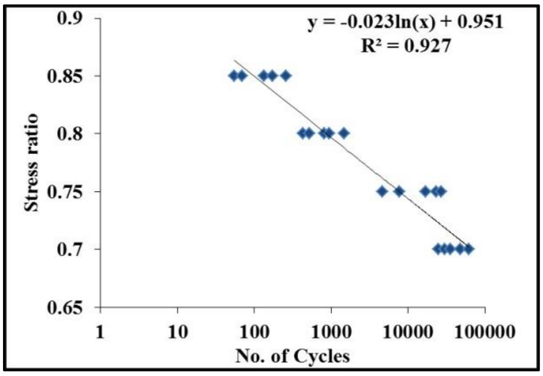
(b) 1-C5



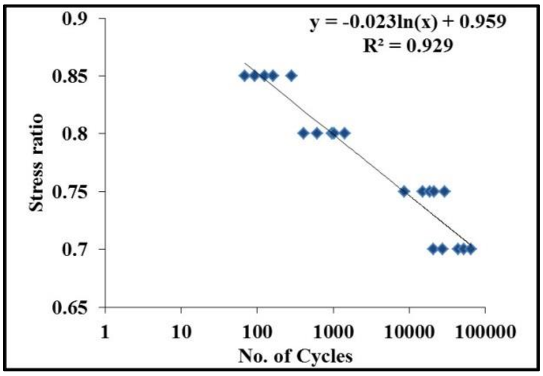
(c) 1-C7



(d) 2-C3



(e) 2-C5



(f) 2-C7

Fig. 5.5 (a-f) S–N Curves for Various CTA Mixtures

Table 5.3 Relationship between Fatigue Cycle (N) and Stress Level (SR)

Mix Type	Equations	R^2
1-C3	$\ln(N) = (0.954 - SR)/0.023$	0.929
1-C5	$\ln(N) = (0.957 - SR)/0.023$	0.932
1-C7	$\ln(N) = (0.963 - SR)/0.023$	0.914
2-C3	$\ln(N) = (0.949 - SR)/0.023$	0.935
2-C5	$\ln(N) = (0.951 - SR)/0.023$	0.927
2-C7	$\ln(N) = (0.959 - SR)/0.023$	0.929

From the Table 5.2, it can be noticed that CTA1 mixtures display higher resistance to fatigue failure as compared to CTA2, irrespective of the applied stress level. The fatigue life of mixes decreased with decrease in cement contents. This may be due to the presence of weak aggregate-paste interface, which may lead to higher and faster propagation of the crack leading to earlier failure. It was observed that specimens exhibit lower fatigue life when subjected to higher stress ratios, while at lower stress ratios, specimens exhibited higher fatigue lives. The failures of the specimens were visually examined and were found to have failed within the middle one third spans. The equations generated from S-N curves for different CTA mixes are presented in Table 5.3 and they can be utilized for estimation of fatigue cycle at any stress level. The statistical correlation coefficient values from the Table 5.3 for different CTA mixes and different stress levels were found to be in the range 0.91 to 0.94 indicating statistical significance.

5.4 SUMMARY

The present chapter summaries the results of the mechanical properties and fatigue experiments carried out on CTA mixtures for base course of LLAP. Based on the previous studies and the suggestion by IRC, cement content of the mixtures was limited to the minimum value of 3% and the maximum value of 7%. The two gradations CTA1 and CTA2 significantly contribute for improving the strength characteristics of the mixtures. Cement contents and curing periods also had substantial effect on the strength development of CTA mixtures. The incorporation of cement contents in CTA mixes with two different gradations slightly affected the OMC and MDD. As the cement content of mixtures, CTA1 and CTA2 increased from 3 to 7 %, the 28 days compressive and flexural strength increased from 9.1 to 14.8 MPa and 1.63 to 2.64 MPa respectively. The compressive strength values increased gradually up to 28 days of curing and beyond this there is a marginal increase. For a curing age of 7 days, the compressive strength ranged from 6.4 to 10.1 MPa; for 28 days, it ranged from 9.1 to 14.8 MPa; and for 90 days, it ranged from 10 to 15.3 MPa for both CTA1 and CTA2 mixtures. The benefits of the CTA mixtures proposed to use in the base course of LLAP are experimentally proved in terms of its increased modulus of elasticity values and fatigue behaviour. All the CTA mixtures satisfied the 7-days compressive strength requirements of 4.5 to 7 MPa and 28-days flexural strength requirements of 1.40 MPa as specified by IRC for flexible pavement design. Mixtures with 3% cement content slightly lowered the mechanical properties in both CTA1 and CTA2, which is mainly due to the weak aggregate-paste interface. However, even with least cement content of 3%, the mixtures exhibited satisfactory performance to be used in pavements. The flexural fatigue performance of all CTA mixes has been investigated. The specimens were subjected to different stress ratios (0.70, 0.75, 0.80 and 0.85) and the number of cycles for failure of the specimen was determined. The fatigue data were represented using S-N curves. It can be noticed that CTA1 mixtures display higher resistance to fatigue failure as compared to CTA2, irrespective of the applied stress level. The fatigue life of mixes decreased with decrease in cement contents. The presence of weak aggregate-paste interface may cause higher and faster propagation of the crack leading to earlier failure.

The entire laboratory test conducted in the study indicated that CTA1 gradation produced better mixtures than CTA2. All the mixes attained sufficient mechanical properties required for application in base course of pavements. Considering the physical and economical aspects, mixtures 1-C3 and 2-C3 can be used for pavement construction since it meets the compressive strength and flexural strength requirements.

CHAPTER 6

PAVEMENT ANALYSIS AND DESIGN

6.1 GENERAL

Pavement design is one of the earliest branches in civil engineering. Dated to early 1920's, the thickness of pavement was purely based on experience and same thickness was used for a section of highway, though widely different soils were encountered. When years passed, various methods were developed by different agencies for determining the thickness of pavement required. Researchers observed that thickness was not alone the basis for design of pavement, but other parameters including stresses, strains, deflections, shear under the application of present and expected future loadings (external stresses) were also considered in the design analysis and possible developments were made in different stages to determine those parameters effectively. Widely known is that, the pavement design is being performed using the following methods:-

- Empirical method
- Analytical method
- Numerical method
- FEM analysis

6.2 FLEXIBLE PAVEMENT

The pavement structure in a flexible pavement generally deflects or bends to accommodate the traffic loads coming over it. The structure consists of different layers including the top wearing layer and the bottom subgrade which acts as the foundation. The load distribution in flexible pavements is accomplished through these layers. Each layer transfers the load to the layer below by spreading into a wider area thereby the stress is reduced from top to bottom. Thus the surface layer, which generally consists of asphalt mixtures, carries the maximum load, whereas the subgrade takes the minimum.

6.3 STRESSES IN FLEXIBLE PAVEMENTS

In order to characterize the behaviour of a flexible pavement under the action of wheel loads, Yang (2004) considered it as a homogeneous half-space. A half-space has an infinitely large area and an infinite depth with a top plane on which the loads are applied. The original Boussinesq theory was based on a concentrated load applied on an elastic half space. The responses in a pavement structure (stresses, strains and deflections) due to a concentrated load can be integrated to obtain them due to loading over circular area. Several theories are available for the analysis of these responses in a flexible pavement based on the behaviour of pavement materials used. But because of the large number and the complexity of factors involved, no single theory is likely to account for all aspects in the design and analysis of flexible pavements. The stresses in flexible pavements are mainly calculated using three layer concepts, with an assumption that a uniformly distributed load is applied over a circular contact area, though it is not completely true in the case of wheel loads applied through pneumatic tyres. Considering layer system generally the analytical solution to the state of stress or strain has several assumptions as listed below:

- The material properties of each layer are homogeneous
- Each layer has a finite thickness except for the lower layer, and all are infinite in the lateral directions
- Each layer is isotropic, i.e. the property at a specific point is the same in every direction
- Full friction is developed between layers at each interface
- Surface shear forces are not present at the surface
- The stress solutions are characterized by the material properties for each layer, Poisson’s ratio and elastic modulus.

The critical stresses that can be calculated using three layer systems include,
σ_{z1}: Vertical stress at interface1; σ_{z2}: Vertical stress at interface 2

σ_{r1}: Horizontal stress at the bottom layer 1; σ_{r2}: Horizontal stress at the bottom layer 2

Figure 6.1 shows a three layered pavement system, having surface, base and sub-grade as the three layers. h_1, E_1, μ_1 are the depth, modulus of elasticity and Poisson's ratio of surface course. h_2, E_2, μ_2 and h_3, E_3, μ_3 are the corresponding values of base course and sub-grade respectively. The sub-grade is considered to be of infinite thickness. P is the load applied, while p is the tyre pressure. Figure 6.2 illustrates the critical failure points in the flexible pavements.

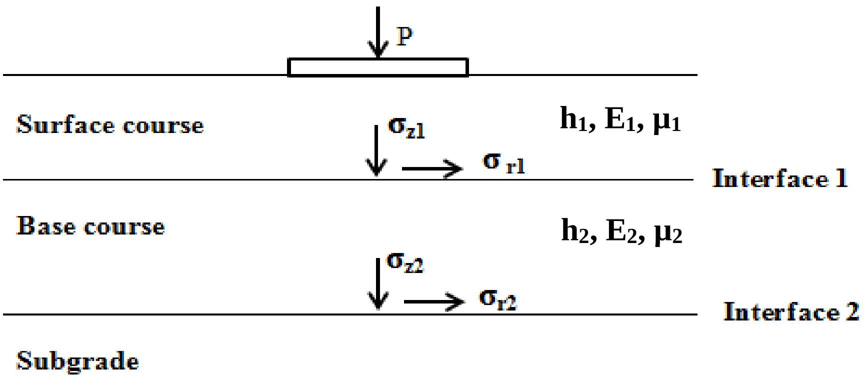


Fig. 6.1 Three Layer System

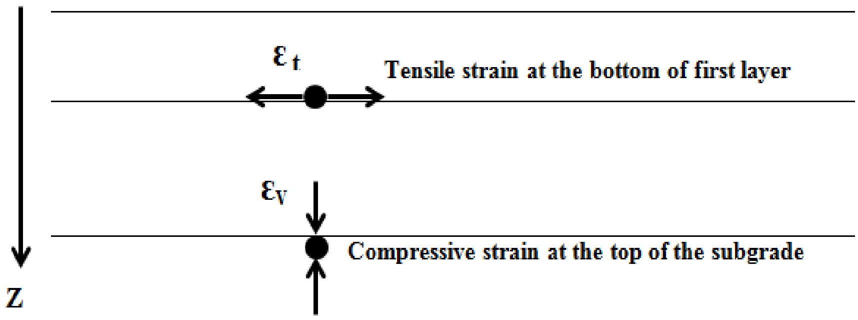


Fig. 6.2 Failure Modes and Critical Strains for Flexible Pavement

The vertical stress on the top of subgrade is an important factor in pavement design. The function of a pavement is to reduce the vertical stress on the subgrade so that detrimental pavement deformations will not occur. The allowable vertical stress on a given subgrade depends on the strength or modulus of the subgrade. To combine the effect of stress and strength, the vertical compressive strain has been used most frequently as a design criterion. The stresses in a two layer system depend on the modulus ratio E_1/E_2 and the thickness radius ratio h_1/a. Vertical surface deflection and vertical interface deflection are other two criteria used in the pavement design.

In this investigation, KENPAVE Software was used to analyze proposed pavement sections with Superpave and CTA mixtures in asphalt and base course layers respectively. Analysis of pavement structures is carried out to assess the pavement responses and is also useful for designing pavement sections. In the analysis generally traffic volume, number of layers, thickness and properties of individual layers are provided as input parameters which gives stresses, strains, deflections, damage ratio etc. as output. In the design of pavement sections the thickness of each layer is achieved depending on the individual material characteristics and traffic conditions, to satisfy certain requirements. In LLAP, the thickness design procedure is based on limiting the critical responses in pavement layers. The critical pavement responses considered are tensile strain at the bottom of asphalt concrete layer for fatigue cracking and compressive strain on top of the subgrade for rutting.

6.4 KENPAVE SOFTWARE

A mechanistic empirical software package called KENPAVE developed by Yang (2004) at the University of Kentucky, was used to analyze the pavement responses and it uses KENLAYER program exclusively for flexible pavements with no joints. The structural analysis of flexible pavement for KENLAYER is based on the Burmister layer theory (Gedafa 2006). KENLAYER allows designing the pavement as a stress-dependent multilayer system and it provides details regarding the stress, strain and deflection under single or dual wheel systems with different axle configurations. An analysis of damage due to fatigue and rutting can be made by dividing each year into a maximum of twelve periods (months), each with a unique set of material properties depending on the seasonal climatic factors. Rutting and fatigue lives are calculated based on the tensile and compressive strains and damage ratio, the ratio of actual load repetitions to the allowable load repetitions, can also be determined. For analysis all layers are assumed to be linearly elastic with a constant elastic modulus for each layer. There are several input parameters for analysis of pavement in KENPAVE and some of them adopted for the current study are listed below.

General Inputs

- The number of periods in a year is 1.
- The number of load group is 0, 1, 2 or 3 depending on the wheel configuration.
- The number of layers varies among 2, 3, 4 and 5.
- The number of Z coordinates is calculated depending upon the number of interfaces and the intermediate points for analysis.
- All layer interfaces are assumed to be bonded.
- SI units are used for calculations.

Loading Inputs

- Types of loading are Single Axle Single Wheel (SASW), Single Axle Dual Wheel (SADW), Tandem and Tridem Axles with dual wheel at the end of each axle.
- SASW/SADW, Tnadem and tridem axles load limits in India are 100kN, 186kN, 235kN respectively (IRC 37 2012).
- The contact radius of circular loaded area is provided as 15.08cm, 10.66cm, 10.28cm and 9.44cm for SASW, SADW, Tandem and tridem axles respectively.
- The contact pressure on circular loaded area is 700kPa.
- Centre to center distance between 2 dual wheels along Y-axis is 32.5cm.
- Centre to center distance between 2 axles along X-axis is 142cm.

The thickness of each layer is measured in cm. For all the pavement sections the required thicknesses are considered by trial and error method.

Material Property Inputs

The resilient modulus (M_R) values of sub grade and granular layers for pavement structures were calculated using Equations (6.1) to (6.3) recommended by the IRC 37 (2012). The Poisson's ratios of subgrade and granular layers were selected as 0.4 and 0.35 respectively.

$$M_R\ (\mathrm{MPa}) = 10 \times \mathrm{CBR} \text{ for CBR} \leq 5 \tag{6.1}$$

$$M_R(\mathrm{MPa}) = 17.6 \times (\mathrm{CBR})^{0.64} \text{ for CBR} > 5 \tag{6.2}$$

$$M_{R\,\mathrm{granular}}(\mathrm{MPa}) = 0.2 \times h^{0.45} \times M_{R\,\mathrm{subgrade}} \tag{6.3}$$

where,

CBR	=	California Bearing Ratio of sub grade (%)
$M_{R\ granular}$	=	Resilient modulus of granular base and sub-base
$M_{R\ subgrade}$	=	Resilient modulus of subgrade soil (MPa)
H	=	Thickness of granular base and sub-base (mm)

The fatigue and rutting performance of pavement structures in terms of tensile strain at the bottom of the asphalt layer and compressive strain on the surface of subgrade was evaluated using fatigue and rutting models (Equations 6.4 and 6.5 respectively) suggested by IRC 37 (2012).

$$N_f = 2.21 \times 10^{-04} \times [1/\varepsilon_t]^{3.89} \times [1/M_R]^{0.854} \tag{6.4}$$

where,

N_f	=	Number of cumulative standard axles to produce 20% cracked surface area
ε_t	=	Tensile strain at the bottom of asphalt surfacing (micro strain)
M_R	=	Resilient modulus of asphalt surfacing (MPa)

$$N_R = 4.1656 \times 10^{-08}[1/\varepsilon_Z]^{4.5337} \tag{6.5}$$

where,

N_R	=	Number of cumulative standard axles to produce rutting of 20mm
ε_z	=	Vertical subgrade strain (micro strain)

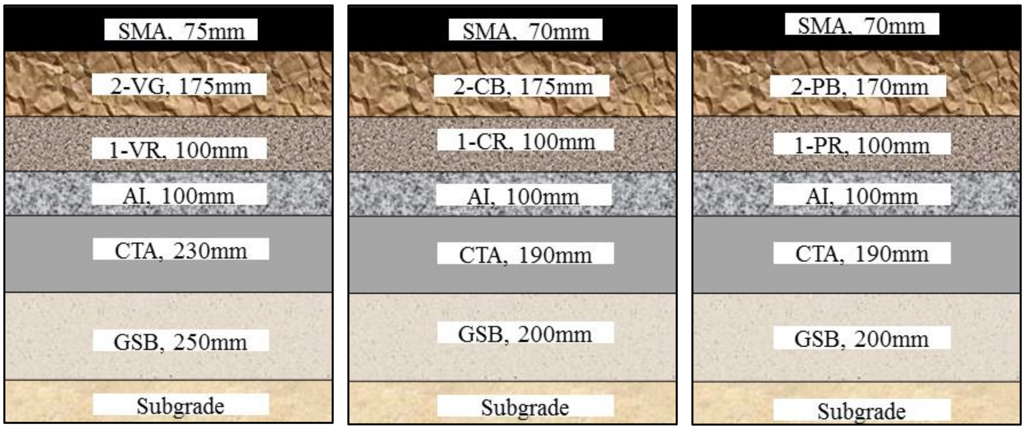
Crack relief layer: A Stress Absorbing Membrane Interlayer (SAMI) using modified asphalt provided over the cementitious layer delays the cracks propagating into the asphalt layer. A crack relief layer of wet mix macadam of thickness 100mm sandwiched between the asphalt layer and treated layer is much more effective in arresting the propagation of cracks from the cementitious base to the asphalt layer. The aggregate layer becomes stiffer under heavier loads because of high confining pressure. In this study, resilient modulus and Poisson's ratio of Aggregate Interlayer (AI) was adopted as 450MPa and 0.35 respectively (IRC 37 2012).

***Traffic parameter and Subgrade*:** A vehicle may have different number of axles, and the load is distributed to these axles and transferred to the pavement surface through the wheels. A standard truck has two axles, front axle with two wheels and rear axle with four wheels. But to carry high loads multiple axles are provided. Since the design of flexible pavements is by layered theory, only the wheels on one side needed to be considered. A LLAP is intended to perform well even in the highest traffic and the weakest subgrade conditions. Hence in the current study, a dual two lane carriageway road with 75 percent of the number of commercial vehicles in each direction has been considered based on IRC 37 (2012). The design traffic of 20 years projected to 50 years at 5% growth rate was calculated as 903 msa and the same was adopted for the analysis.

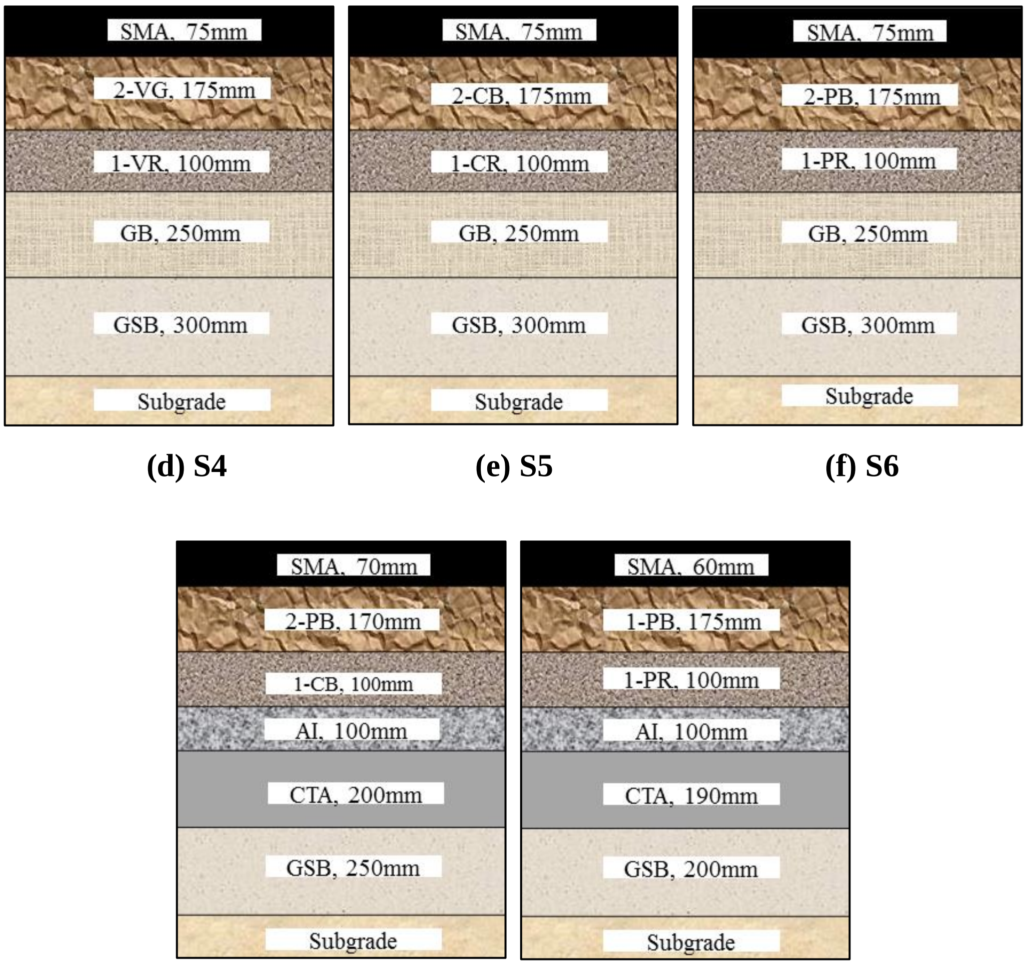
Subgrade is defined as a compacted layer, generally of naturally occurring local soil, assumed to be 300 to 500 mm in thickness depending on traffic volume, just beneath the pavement crust, providing a suitable foundation for the pavement. For high volume roads IRC limits the design California Bearing Ratio (CBR) to 8% and above. Taking into consideration the traffic volume in this study, the subgrade CBR of 8% was selected.

6.5 ANALYSIS OF PROPOSED PAVEMENT SECTIONS

The prime objective of this study was to propose Superpave mixtures with improved characteristics for the asphalt interlayer and base layer of LLAP. Also to propose CTA mixtures with enhanced stiffness for base course of LLAP. The vertical displacement, vertical stress, tensile strain, compressive strain, fatigue life (N_f), rutting life (N_r) and damage ratio are some parameters which projects the application of Superpave and CTA mixtures in the prescribed layers. In the analysis carried out using KENPAVE software, mainly eight pavement sections (denoted as S1, S2, S3, S4, S5, S6, S7 and S8) with different combinations of layers and materials were considered and are shown in Figures 6.3 (a-h). The sections were divided on the basis of the mixtures used in asphalt intermediate layer, asphalt base layer and base course. The sections with Granular Base (GB) and Sub Base (GSB) layers were considered as they are most commonly used in India. Since the usage of cement treated layers is expected to provide better LLAP sections, analysis was carried out for the sections with cemented base and granular sub base also. The thickness of the layers in these sections was decided to obtain critical strains within permissible limits (tensile strain < 70 micro strain and compressive strain < 200 micro strain) and were chosen using trial and error method. Using the in-built KENLAYER program, the KENPAVE software analyses the pavement responses on the sections mentioned above.



(a) S1 **(b) S2** **(c) S3**



(d) S4 **(e) S5** **(f) S6**

(g) S7 **(h) S8**

Fig. 6.3 (a-h) Pavement Sections with Different Layer Compositions

Based on the literature (Newcomb and Hansen 2006, Uzarowski et al. 2008, El-Hakim et al. 2009, Tarefder and Bateman 2009, Maher and Uzarowski 2010, Walubita and Scullion 2010, Chai et al. 2012) SMA mixture was used in the asphalt surface layer of all the pavement sections (Figure 6.3 (a-h)). The SMA mixtures were prepared only for the purpose of analysis, using PMB 40 binder **(**Details of SMA gradation used and mix design properties of mixture are presented in Appendix II). The resilient modulus and Poisson's ratio was evaluated at the standard temperature for pavement design (35^{o}C) and the result was obtained as 1372MPa and 0.34 respectively. For intermediate and base layers of pavement sections, the resilient modulus and Poisson's ratio values of the Superpave mixtures (1-VG, 1-CB, 1-PB, 2-VG, 2-CB, 2-PB, 1-VR, 1-CR, 1-PR, 2-VR, 2-CR and 2-PR) obtained from the modulus test conducted at 35°C (as listed in Table 4.12 and 4.13, Chapter 4) was adopted, wherever the mixtures are used in the pavement sections. From the experimental investigation, performance of CTA1 mixture was found to be better than the CTA2. Even with least cement content, mixture 1-C3 exhibits satisfactory performance to be used in pavements. Considering the physical and economical aspects, for CTA base layer of pavement sections the elastic modulus of the mixture 1-C3 obtained from the modulus test (as presented in Figure 5.4 of chapter 5 i.e., 7012MPa) was adopted, wherever the mixture is used in the pavement sections. In view of the long-term effect due to shrinkage, only 50% of modulus value was taken for CTA layer in the analysis. The Poisson's ratio of CTA layer was adopted as 0.25 based on IRC 37 (2012).

6.6 RESULT AND DISCUSSION

In KENPAVE software, thickness of different layers, material properties and loading conditions are provided as general input parameters and coefficients of rutting, fatigue etc. can also be provided for detailed analysis. The vertical stresses and vertical displacement values generated at all the layer interfaces under different axle load conditions are presented in Tables 6.1 and 6.2 respectively. The results showed that the stress values were getting reduced for all the axle load cases, whereas the displacement was higher for tandem and tridem axles in all the pavement sections. The maximum stress over the subgrade and maximum displacement at all the interfaces was observed for sections with GB and GSB layer. The number of layers, thickness, type of mixture used, and load influenced the stress and displacement values.

Table 6.1 Displacement Values for Different Pavement Sections for All Axle Loads

Displacement (mm)					
Pavement Sections	**H (mm)**	**SASW**	**SADW**	**Tandem**	**Tridem**
S1	0	0.58	0.54	0.72	0.82
	350	0.59	0.52	0.67	0.74
	450	0.55	0.49	0.64	0.72
	680	0.50	0.46	0.61	0.68
	930	**0.42**	**0.40**	**0.55**	**0.62**
S2	0	0.54	0.50	0.68	0.77
	345	0.54	0.48	0.63	0.70
	445	0.51	0.46	0.61	0.68
	635	0.46	0.43	0.58	0.65
	835	**0.39**	**0.37**	**0.52**	**0.59**
S3	0	0.51	0.47	0.65	0.75
	340	0.50	0.45	0.60	0.67
	440	0.46	0.43	0.58	0.65
	630	0.43	0.40	0.55	0.62
	830	**0.38**	**0.36**	**0.51**	**0.58**
S4	0	0.72	0.65	0.84	0.93
	350	0.72	0.63	0.78	0.86
	600	0.66	0.58	0.73	0.82
	900	**0.62**	**0.54**	**0.70**	**0.77**
S5	0	0.71	0.64	0.79	0.88
	350	0.70	0.61	0.76	0.85
	600	0.65	0.57	0.72	0.81
	900	**0.61**	**0.54**	**0.69**	**0.76**
S6	0	0.70	0.63	0.78	0.87
	350	0.70	0.60	0.75	0.84
	600	0.64	0.56	0.71	0.80
	900	**0.59**	**0.53**	**0.69**	**0.76**
S7	0	0.47	0.44	0.62	0.71
	340	0.46	0.42	0.57	0.64
	440	0.43	0.40	0.55	0.62
	640	0.40	0.38	0.53	0.60
	890	**0.34**	**0.33**	**0.48**	**0.55**
S8	0	0.51	0.47	0.65	0.75
	335	0.50	0.45	0.60	0.67
	435	0.47	0.43	0.58	0.65
	625	0.43	0.40	0.56	0.63
	825	**0.37**	**0.35**	**0.51**	**0.58**

Table 6.2 Vertical Stress Values for Different Pavement Sections for All Axle Loads

Vertical Stress (kPa)					
Pavement Sections	**H (mm)**	**SASW**	**SADW**	**Tandem**	**Tridem**
S1	0	700.00	700.000	700	700
	350	129.63	88.937	83.946	71.67
	450	86.20	60.630	57.805	49.332
	680	13.95	12.117	13.073	11.369
	930	**7.84**	**7.290**	**8.645**	**7.655**
S2	0	700.00	700.00	700.00	700.00
	345	116.47	79.95	75.68	64.64
	445	73.98	51.94	49.86	42.59
	635	14.28	12.35	13.35	11.59
	835	**9.01**	**8.29**	**9.61**	**8.45**
S3	0	700.00	700	700	700.00
	340	115.33	79.237	75.011	64.08
	440	73.26	51.438	49.39	42.20
	630	14.13	12.217	13.226	11.49
	830	**8.91**	**8.2**	**9.527**	**8.38**
S4	0	700.00	700.00	700.00	700.00
	350	72.44	53.38	50.76	43.23
	600	26.76	22.69	22.68	19.30
	900	**11.73**	**10.91**	**12.11**	**10.43**
S5	0	700	700.00	700.00	700.00
	350	64.531	48.09	45.92	39.10
	600	24.443	20.90	21.07	17.95
	900	**11.011**	**10.28**	**11.55**	**9.96**
S6	0	700.00	700.00	700.00	700.00
	350	61.51	46.04	44.05	37.51
	600	23.55	20.19	20.45	17.43
	900	**10.73**	**10.03**	**11.32**	**9.78**
S7	0	700.00	700.00	700.00	700.000
	340	117.64	80.98	76.58	65.408
	440	76.10	53.57	51.31	43.821
	640	14.68	12.65	13.57	11.785
	89	**8.11**	**7.52**	**8.86**	**7.832**
S8	0	700.00	700.00	700.00	700.00
	335	115.00	79.03	74.81	63.92
	435	73.05	51.27	49.23	42.07
	625	14.06	12.15	13.17	11.44
	825	**8.86**	**8.16**	**9.48**	**8.35**

Figures 6.4, 6.5 and Tables 6.3, 6.4 show the damage analysis results for all the pavement sections. The thickness of HMA layers of all the sections was adjusted within the recommended range (by Newcomb et al. 2001, APA 2002, Newcomb et al. 2010) in a trial and error basis to make the tensile and compressive strains less than the critical values, considering complete bonding between the pavement layers. The modification of the material composition along with this thickness adjustment enhances the fatigue and rutting life of the proposed pavement sections. From Figure 6.4 and 6.5, it can be seen that in case of SASW load, the pavement sections S2, S3, S7 and S8 meets the LLAP criteria as the tensile and compressive strains are below the critical limits. The maximum tensile and compressive strain values were observed for pavement sections with GB and GSB layers (S4 S5and S6); whereas it was minimum for sections with CTA base and GSB layer (S1, S2, S3, S7 and S8). This indicates that, the usage of CTA base layer is a better option to achieve LLAP criteria, than using GB layer. The strain experienced on the pavement structure in the vertical and horizontal direction gives two different allowable load repetitions- rutting life (N_r) and fatigue life (N_f) respectively (Al-Khateeb et al. 2007). If the actual load repetition crosses any of these two allowable repetitions, the strain in that particular direction exceeds the limit and cause damage to the pavement. Damage ratio, which is the ratio of actual load repetitions to the allowed repetitions, is a crucial parameter in pavement design. In no case, the actual load repetitions shall be more than the allowed repetitions, which indicates the pavement failure. The desired value of this ratio is less than 1 and it permits further traffic movement on the road (Deepthi et al. 2013). From Table 6.4 it was observed that, for SASW load, the damage ratio was less than 1 for the sections with CTA base and GSB layer. The results showed that the fatigue life and rutting life values were getting increased for all the axle loads, whereas the damage ratio was lesser for SADW, tandem and tridem axle loads in all the pavement sections. Figure 6.6 (a-d) presents the view of LGRAPH in KENPAVE.

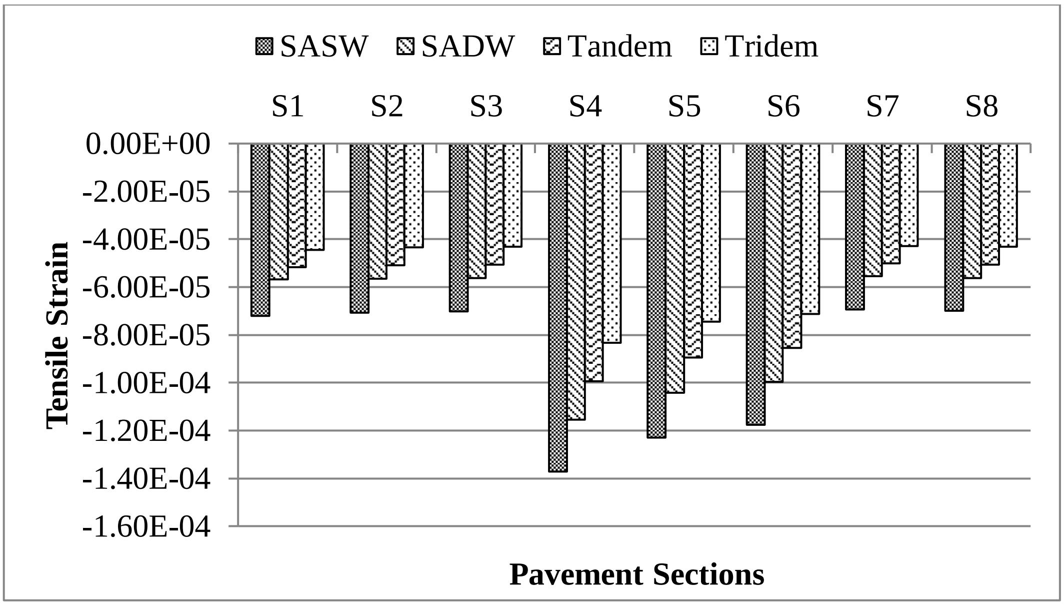


Fig. 6.4 Tensile Strain Values for Different Pavement Sections and Axle Loads

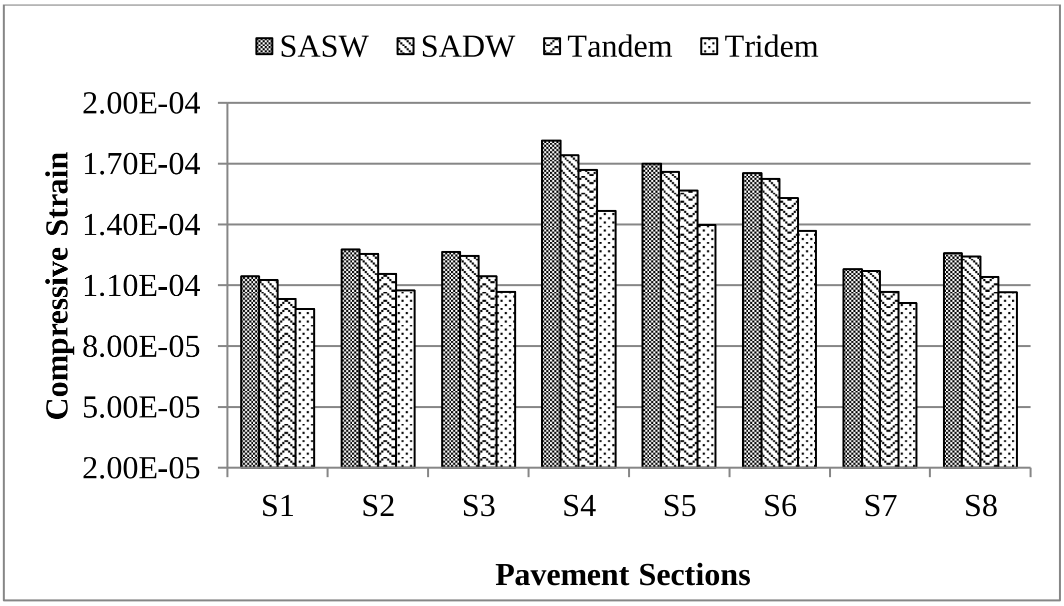


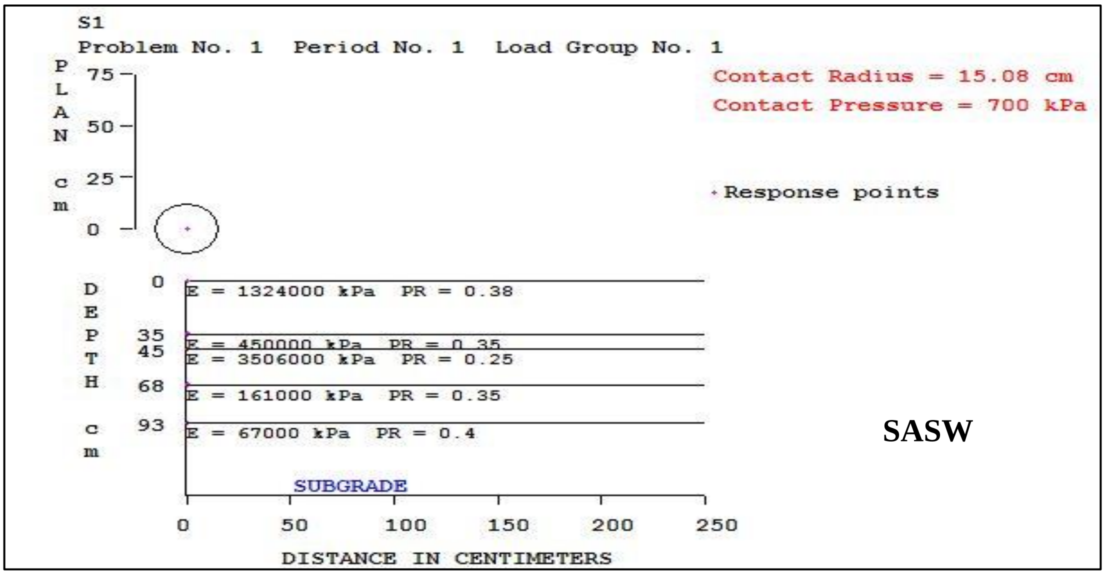
Fig. 6.5 Compressive Strain Values for Different Pavement Sections and Axle Loads

Table 6.3 Fatigue Life and Rutting Life Values for Different Pavement Sections and Axle Loads

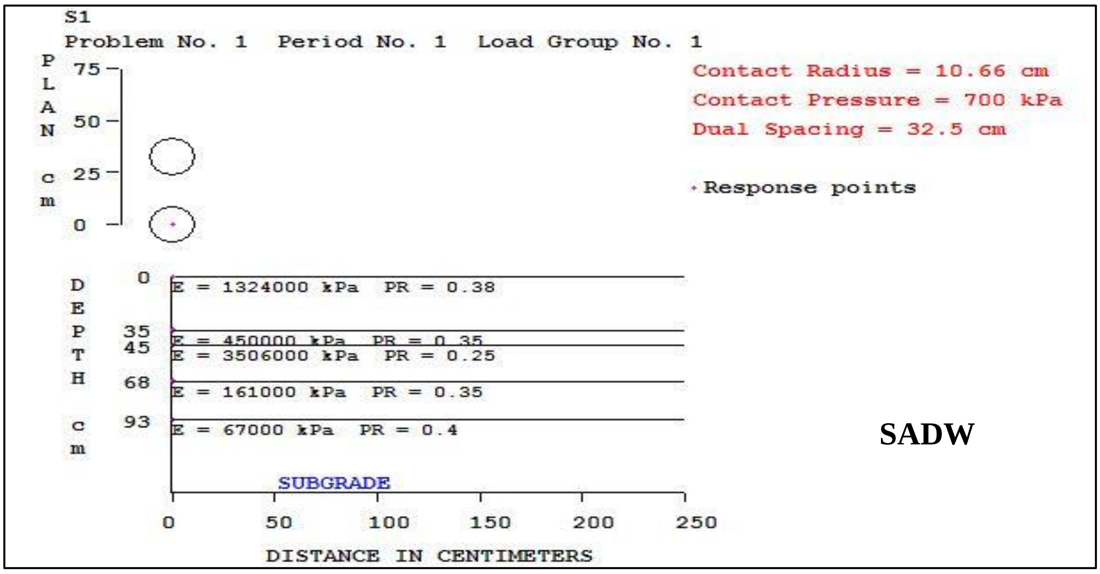
Pavement Sections	SASW		SADW	
	N_f	N_R	N_f	N_R
S1	6.22E+09	3.33E+10	1.58E+10	4.91E+10
S2	5.62E+09	1.88E+10	1.34E+10	2.94E+10
S3	5.37E+09	1.96E+10	1.27E+10	3.07E+10
S4	5.05E+08	3.82E+09	9.92E+08	5.59E+09
S5	6.47E+08	5.14E+09	1.24E+09	7.37E+09
S6	7.21E+08	5.81E+09	1.37E+09	8.28E+09
S7	5.60E+09	2.80E+10	1.34E+10	4.20E+10
S8	5.11E+09	2.00E+10	1.20E+10	3.13E+10
Pavement Sections	**Tandem**		**Tridem**	
	N_f	N_R	N_f	N_R
S1	2.26E+10	3.09E+10	4.01E+10	6.14E+10
S2	2.01E+10	2.02E+10	3.71E+10	4.09E+10
S3	1.92E+10	2.09E+10	3.55E+10	4.22E+10
S4	1.76E+09	4.59E+09	3.52E+09	1.01E+10
S5	2.25E+09	5.74E+09	4.55E+09	1.25E+10
S6	2.50E+09	6.30E+09	5.08E+09	1.37E+10
S7	2.01E+10	2.70E+10	3.69E+10	5.38E+10
S8	1.82E+10	2.13E+10	3.38E+10	4.28E+10

Table 6.4 Damage Ratio Values for Different Pavement Sections and Axle Loads

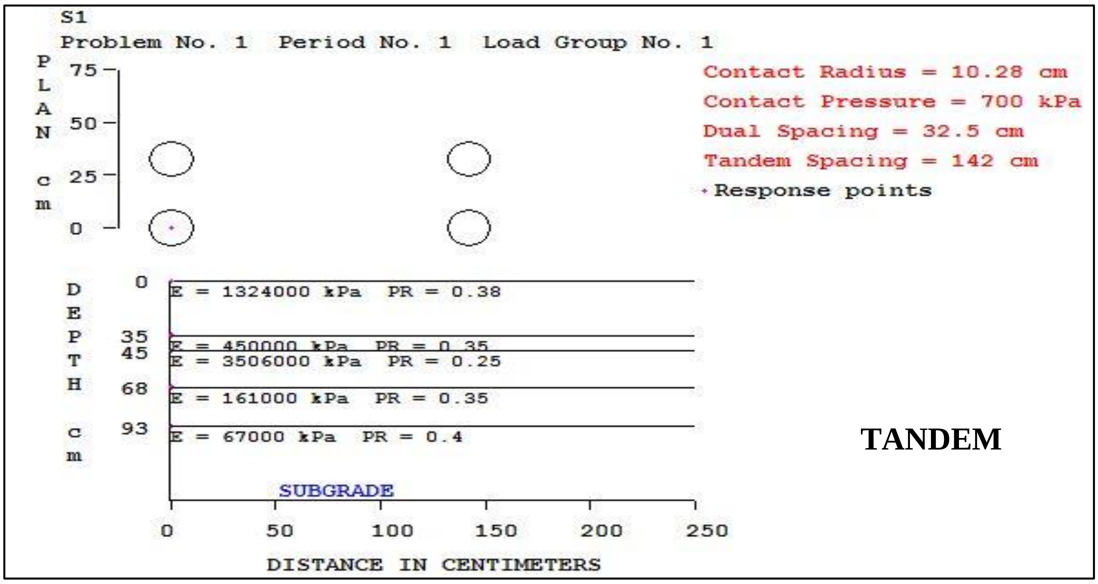
Pavement Sections	Damage Ratio (%)			
	SASW	SADW	Tandem	Tridem
S1	0.15	0.06	0.07	0.06
S2	0.16	0.07	0.07	0.05
S3	0.17	0.07	0.07	0.05
S4	1.79	0.91	0.58	0.32
S5	1.40	0.73	0.44	0.23
S6	1.25	0.66	0.39	0.20
S7	0.17	0.16	0.07	0.05
S8	0.18	0.08	0.07	0.05


S1
Problem No. 1 Period No. 1 Load Group No. 1
Contact Radius = 15.08 cm
Contact Pressure = 700 kPa
Response points
PLAN cm
DEPTH cm
E = 1324000 kPa PR = 0.38
E = 450000 kPa PR = 0.35
E = 3506000 kPa PR = 0.25
E = 161000 kPa PR = 0.35
E = 67000 kPa PR = 0.4
SUBGRADE
DISTANCE IN CENTIMETERS
SASW

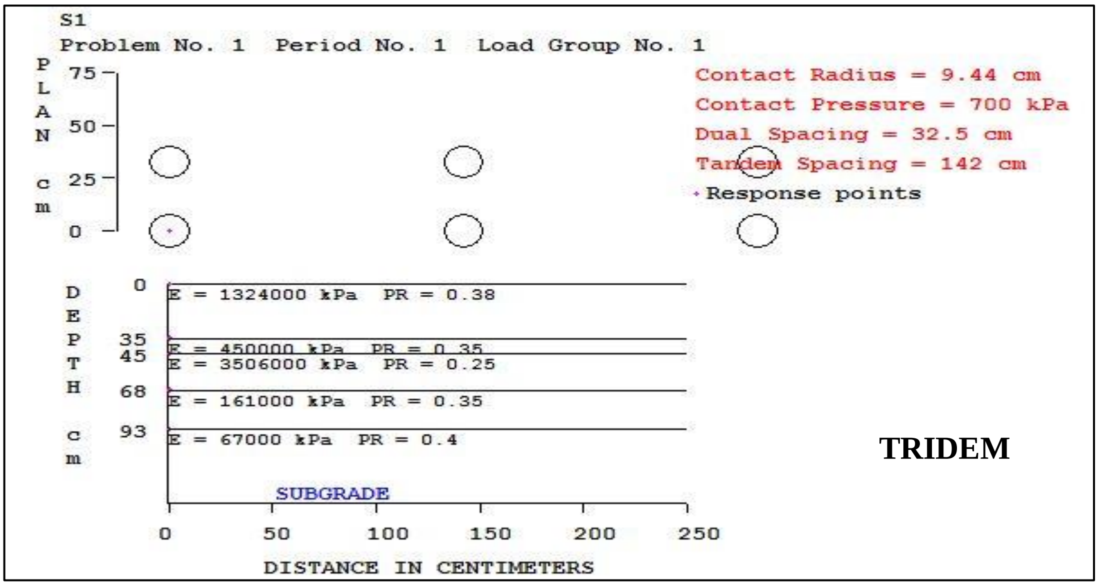
(a)


S1
Problem No. 1 Period No. 1 Load Group No. 1
Contact Radius = 10.66 cm
Contact Pressure = 700 kPa
Dual Spacing = 32.5 cm
Response points
PLAN cm
DEPTH cm
E = 1324000 kPa PR = 0.38
E = 450000 kPa PR = 0.35
E = 3506000 kPa PR = 0.25
E = 161000 kPa PR = 0.35
E = 67000 kPa PR = 0.4
SUBGRADE
DISTANCE IN CENTIMETERS
SADW

(b)



(c)



(d)

Fig. 6.6 (a-d) LGRAPH Snapshots for Different Axles

6.7 SUMMARY

In this chapter, an attempt has been made to assess the responses of proposed pavement structures subjected to SASW, SADW, tandem and tridem axle loading using KENPAVE software. In the analysis traffic volume, number of layers, thickness and properties of individual layers were provided as input parameters which give stresses, strains, deflections, damage ratio etc. as output. It was observed from the analysis that, when the number of wheels and axles increases the stresses and strains in the pavement layers decreases, i.e. the stress is more for SASW and less for SADW, Tandem and Tridem axle loading. In case of SASW load, the pavement structures with CTA base and GSB layer (S2, S3, S7 and S8) meets the LLAP criteria as the tensile and compressive strains are below the critical limits of 70 micro strains and 200 micro strains respectively. Fatigue and rutting lives were observed to be improved for pavement sections with CTA base layer compared to sections with GB layer. This may be due to the higher modulus of elasticity of CTA layer. The inclusion of CTA layer in the pavement sections adds some more successful periods to the design life. Sections with modifed binder mixtures in asphalt layers showed lesser stresses and strains compred to unmodifed binder mixtures. This may be due to higher resilient modulus values of modified mixtures, which enabled to obtain required strain values using the layer thickness within the recommended range (by Newcomb et al. 2001, APA 2002, Newcomb et al. 2010). The Maintenance and surface rehabilitation programs must be strictly adhered to ensure maximum performance in these pavement structures.

CHAPTER 7

CONCLUSIONS

This study demonstrates the evaluation of performance characteristic of the Superpave and CTA mixtures for the asphalt intermediate and asphalt base layers, and the base course of the LLAP. Various laboratory tests were conducted to evaluate the performance characteristics of these mixtures. Analysis was performed using laboratory measured modulus values for all mixtures considered, to evaluate the response of different pavement structures proposed in the study. Based on the tests conducted in the laboratory and the analysis, the following conclusions have been drawn:

7.1 SUPERPAVE MIXTURES

1. All the optimum Superpave mixtures satisfied the volumetric requirements of Superpave mix design including TSR, and SP1 mixtures have slightly higher density and 1.23 – 3.25 % less OBC as compared to SP2 mixtures.
2. Mixtures with modified binders have the best volumetric properties, with lesser OBC values.
3. Mixtures with PMB have the highest IDT strength, rut resistance followed by those with CRMB and VG 30. Optimum mixtures are better resistant to rutting and the deformations are 10 – 24 % less than that of rich binder mixtures for all the wheel cycles. This data support the use of the optimum mixture in the intermediate layer.
4. At different dynamic loads, fatigue life of rich binder mixtures is higher than that of optimum mixtures. This confirms the research idea of using rich binder mixture for the asphalt base layer of LLAP structure.
5. All the mixtures exhibit higher resilient modulus at lower temperatures. The resilient modulus values of optimum binder mixtures at all the temperatures are 11 – 33 % higher than those of rich mixtures.

6. For the optimum mixtures, ITS is higher than that of the rich binder mixtures. However, rich mixtures show greater TSR compared to the optimum mixtures, indicating superior performance of the former, in resisting moisture damage.
7. All Superpave mixtures have very good moisture resistance with TSR above 84%. Among all, the rich mixtures with modified asphalt binder have the best moisture resistance.
8. For all mixture types, SP1 gradation is better than SP2, whereas mixtures with both gradations have similar moisture susceptibility.

7.2 CEMENT TREATED AGGREGATE BASE MIXTURES

1. The two gradations CTA1 and CTA2 significantly contribute for improving the strength characteristics of the CTA mixtures. Cement contents also had substantial effect on the strength development of mixtures.
2. The CTA mixtures satisfy the requirements of IRC 37 in terms of compressive and flexural strengths with the minimum cement content of 3%.
3. The benefits of the CTA mixtures proposed to use in the base course of LLAP are experimentally proved in terms of its higher modulus of elasticity values and improved fatigue behaviour.
4. For all mixture types, CTA1 gradation performed better than CTA2.

7.3 KENPAVE ANALYSIS

1. As per the KENPAVE analysis, in SASW load case, the pavement sections S2, S3, S7 and S8 meet the LLAP criteria as the tensile and compressive strains are below the critical limits.
2. The maximum tensile and compressive strain values are observed for pavement sections with GB and GSB layers; whereas it is the minimum for sections with CTA base and GSB layer. This indicates that, the usage of CTA base is a better option to achieve LLAP criteria, than using GB layer.
3. Pavement sections with VG mixtures did not satisfy the tensile strain requirements of LLAP.

Superpave mixes with modified binders and CTA mixes in this study have shown satisfactory results for their use in highway applications. From the entire study it can be concluded that, the improved rutting resistance and stiffness of the optimum mixtures with modified binders make it a better substitute mixture for the intermediate layer of LLAP. Even though the resilient modulus values of rich mixtures are less than optimum mixture, it can resist the fatigue cracking experiencing in the bottom asphalt layer of LLAP to some extent. Further, this study identifies improved functionality of the CTA base mixtures, which helps to achieve stronger base leading to enhancement of pavement service life in terms of reduced strains and increased fatigue and rutting lives.

7.4 SCOPE FOR FURTHER RESEARCH

- Extensive study on Superpave mixtures with waste plastic, recycled asphalt pavement materials etc. can be carried out.
- The work can be extended to field track and evaluated for a period of years.

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APPENDIX I

Calculation of Volumetric Properties for 2-VG (SP2 Mixture with VG 30)

Consider mixture with 4.71% asphalt content by weight of mixture

Weight of Aggregates (Including mineral filler) = 1200g

Weight of Asphalt = 59.31g

Total weight of mixture = 1259.31g

Bulk specific gravity of aggregates, G_{sb} = 2.652

Maximum theoretical density of loose mixture, G_{mm} = 2.463

Bulk Density of Specimen, G_{mb} = 2.339g/cc

Air Voids, Va (in %) $= \frac{G_{mm} - G_{mb}}{G_{mm}} \times 100\ \%$

= 5.04%

Aggregate content (% by total weight of mix), P_s = 100 × [1200 ÷ (1200 + 59.31)]

=95.29

Voids in Mineral Aggregates, VMA (in %) $= 100 - \frac{G_{mb} . P_s}{G_{sb}}$

=15.97%

Voids Filled with Asphalt, VFA (in %) $= \frac{VMA - Va}{VMA} \times 100$

= 68.45%

Effective specific gravity of aggregate, G_{se} = 2.655

Asphalt content, (% by total weight of mix), P_b = 4.71%

Effective asphalt binder content
(% by total weight of mix), P_{be} $= -(P_S \times G_b) \times \left(\frac{G_{se} - G_{sb}}{G_{se} - G_{sb}}\right) + P_b$

= 4.674

Aggregate content passing the 0.075mm sieve,
(% by mass of aggregate), $P_{0.075}$ = 5%

Dust to Binder Ratio, DP $= \frac{P_{0.075}}{P_{be}}$

=1.07

APPENDIX II

Volumetric Properties of SMA Mixtures with Polymer Modified Binder

In this current investigation, SMA mixtures were prepared using PMB 40 asphalt and aggregate gradation having NMAS 13.2mm. The gradation considered for the study was adopted from IRC guidelines (IRC SP 79-2008). The gradation ranges and adopted values are presented in Table A and Figure A.

Table A Aggregate Gradation of SMA Mixtures

IS Sieve Size (mm)	Cumulative % by weight of total aggregate passing
19	100
13.2	90-100
9.5	50-75
4.75	20-28
2.36	16-24
1.18	13-21
0.6	12-18
0.3	10-20
0.075	8-12

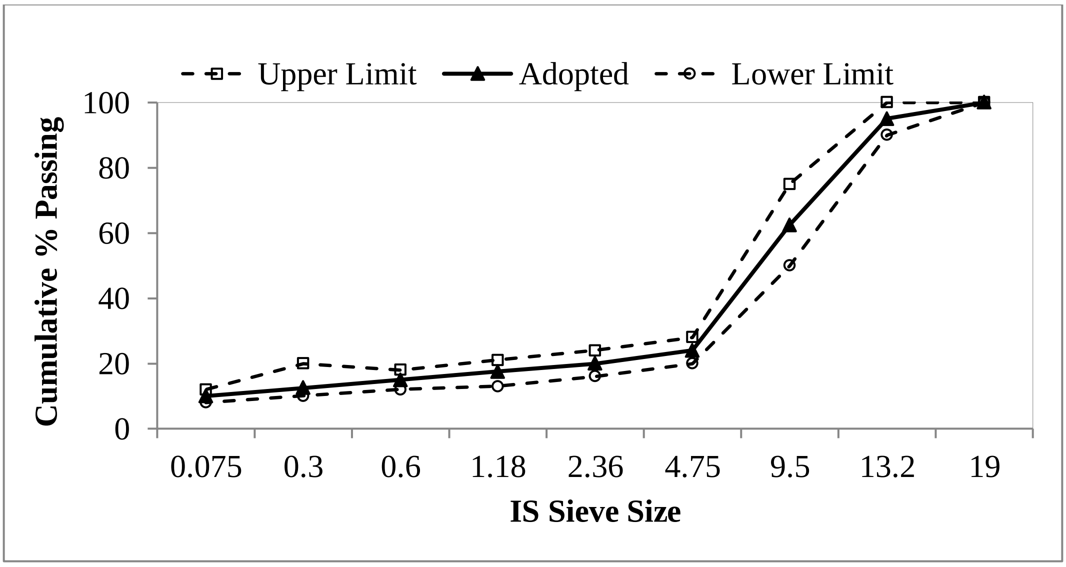

Fig. A Aggregate Gradations for SMA Mixtures

The SMA mixture requirement specified by IRC is presented in Table B. Cylindrical test specimens with 100mm diameter were prepared by adding 5.0, 5.5, 6.0, 6.5, and 7.0 per cent of asphalt by total weight of mixture, and 100 gyrations were provided to compact the specimen to determine volumetric properties of the mixtures. The results are presented in Table C.

Table B SMA Mixture Requirements as per IRC

Mix Design Parameters	Requirement
Air void content, %	4.0
Asphalt content, %	5.8 min.
VMA, %	17 min.
VCA_{MIX}, %	Less than dry rodded VCA (VAC_{DRC})
Asphalt drain down, % (AASHTO T 305)	0.3 max
TSR, %	85 min.

Table C Properties of SMA Mixture with PMB

Property	Asphalt Content by Weight of Mix				
	5.0	**5.5**	**6.0**	**6.5**	**7.0**
G_{mm}	2.501	2.482	2.463	2.444	2.426
G_{mb} (g/cc)	2.344	2.357	2.366	2.353	2.348
Va (%)	6.27	5.01	3.94	3.72	3.22
VMA (%)	17.59	17.55	17.70	18.57	19.18
VFA (%)	64.36	71.47	77.72	79.96	83.24
VCA_{MIX} (%)	36.95	36.93	37.04	37.70	38.17
VCA_{MIX}/ VCA_{DRC}	0.905	0.904	0.907	0.923	0.934
OBC (%)	6.08				

The OBC of the mixture at 4% air voids was determined as 6.08%. At OBC the mixture satisfied all the major requirements suggested for SMA including drain down (0.117%) and TSR (95.23%). For determining the resilient modulus of SMA mixture at OBC the same test procedure as mentioned in Section 3.4.1.6 (chapter 3) was followed. From the test the resilient modulus and Poisson's ratio at 35^oC was obtained as 1372MPa and 0.34 respectively.

Table D Properties of SMA Mixture at OBC

Mixture	**PMB**
OBC (%)	6.08
G_{mm}	2.460
G_{mb} (g/cc)	2.365
Va (%)	3.86
VMA (%)	17.80
VFA (%)	78.33
VCA_{MIX} (%)	37.12
VCA_{MIX}/ VCA_{DRC}	0.909

LIST OF PUBLICATIONS

Journal

- **Priyanka, B.A.**, Sarang, G. and Ravi Shankar, A.U. (2018). "Evaluation of Superpave mixtures for perpetual asphalt pavements." *Road Materials and* Pavement *Design*, Taylor and Francis, Published online on 23 May 2018. (https://www.tandfonline.com/doi/full/10.1080/14680629.2018.1474794)
- **Priyanka, B.A.**, Sarang, G., Kondeti Chiranjeevi and Ravi Shankar, A.U. (2018). "Laboratory investigation of cement treated aggregate base mixtures for flexible pavements." *International Journal of Pavement Research and Technology*, Elsevier. (Under Review)

Conference

- **Priyanka, B.A.**, Sarang, G. and Ravi Shankar, A.U. (2016). "Development of perpetual pavements with cement treated base and rich bottom layer using KENPAVE analysis." *Conference on Sustainable Asphalt Pavement for Developing Countries (CONSAP) 2016*, Conducted by Centre for Road Research Institute (CRRI), New Delhi, India, March 11-12.
- **Priyanka B.A.**, Sarang, G. and Ravi Shankar A.U. (2017). "Laboratory performance of Superpave mixes for perpetual pavements." *International Conference on Highway Pavements and Airfield Technology*, T&DI Congress by ASCE, Philadelphia, USA, August 27-30.

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Ph.D.	Transportation Engineering (Civil Engineering)	National Institute of Technology Karnataka, Surathkal, Karnataka, INDIA.	2014 – 2019
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Higher Secondary	Science (Physics, Chemistry, Mathematics & Biology)	DVS Composite Pre University College Vinobanagar, Shimoga, Karnataka, INDIA.	2005 – 2007
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PROJECTS:

B.E : "Experimental Studies on Influence of Fine Aggregate on Properties of Concrete."

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Undergone subjects in M Tech and Ph.D

1. Pavement Design
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4. Statistical Methods
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6. Pavement Materials and construction
7. Traffic flow Theory
8. Traffic Design and Studio lab
9. Transportation Engineering lab
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Journals:

1. **Priyanka, B.A.**, Sarang, G. and Ravi Shankar, A.U. (2018). "Evaluation of Superpave mixtures for perpetual asphalt pavements." Road Materials and Pavement Design, Taylor and Francis, Published online on 23 May 2018.

2. **Priyanka, B.A**., Sarang, G., Kondeti Chiranjeevi and Ravi Shankar, A.U. (2018). "Laboratory investigation of cement treated aggregate base mixtures for flexible pavements." International Journal of Pavement Research and Technology (Under Review), Elsevier.
3. Ravi Shankar, A.U., **Priyanka, B.A.**, Sarang, G. and Lekha B.M. (2017). "Laboratory investigation of blended lateritic soil for gravel roads." Indian Roads Congress Journals (Under Review).

<u>Conference:</u>

1. **Priyanka B.A.**, Lekha B.M., Sarang, G. and Ravi Shankar A.U. (2015). "KENPAVE analysis for low volume roads with reduced resilient modulus values." *2nd Conference on Transportation Systems Engineering and Management*, NIT Tiruchirappalli, Tamil Nadu, India, May 1-2.

2. **Priyanka, B.A.**, Sarang, G. and Ravi Shankar, A.U. (2016). "Development of perpetual pavements with cement treated base and rich bottom layer using KENPAVE analysis." *Conference on Sustainable Asphalt Pavement for Developing Countries (CONSAP),* Conducted by Centre for Road Research Institute (CRRI), New Delhi, India, March 11-12, 2016.

3. Sarang, G., Lekha B.M., **Priyanka B.A.** and Ravi Shankar A.U. (2016). "Stone Matrix Asphalt mixtures with sisal fiber and Crumb Rubber Modified Binder." *Conference on Sustainable Asphalt Pavement for Developing Countries (CONSAP) 2016*, Conducted by Centre for Road Research Institute (CRRI), New Delhi, India, March 11-12.

4. Ravishankar, A.U. and **Priyanka, B.A**. (2016). "Analysis of Low Volume Roads Under Submerged Condition." *10th International Symposium on LowLand Technology (10th ISLT 2016)*, organized by International Association of Lowland Technology (IALT), National Institute of Technology Karnataka, Mangalore, India and Institute of Lowland and Marine Research (ILMR), Japan, September 15-17.

5. **Priyanka B.A.**, Sarang, G. and Ravi Shankar A.U. (2017). "Laboratory performance of Superpave mixes for perpetual pavements." *International Conference on Highway Pavements and Airfield Technology,* T&DI Congress by ASCE, Philadelphia, USA, August 27-30.
6. Ravishankar, A.U., **Priyanka, B.A**. and Tejaswi S. (2018). "Analysis of high volume roads during heavy monsoon in coastal and low land areas." *International Symposium on LowLand Technology (ISLT 2018)*, organized by International Association of Lowland Technology (IALT), Thuyloi University, Vietnam, September 26-28.
7. Ravishankar, A.U., **Priyanka, B.A** and Avinash (2018). "Experimental studies on laterite soil stabilized with coconut coir, cement and aggregate." *Indian Geotechnical Conference (IGC 2018)*, Indian Institute of Science, Bengaluru, December 13-15.

Conferences/Workshops Attended

1. International Workshop on "Civil Infrastructure and Structure Materials", National Institute of Technology Karnataka (NITK) Surathkal, Karnataka, India, 28 – 29, July 2014.
2. Three-day Workshop on "Mitigation of Road Disasters" organised by Centre for Disaster Risk Reduction", National Institute of Technology Karnataka (NITK) Surathkal, Karnataka, India, 15 – 17, Septmber 2014.
3. 2nd Conference on "Transportation Systems Engineering and Management", NIT Tiruchirappalli, Tamil Nadu, India, 1 – 2, May 2015.
4. Conference on "Sustainable Asphalt Pavement for Developing Countries (CONSAP)", conducted by CRRI, New Delhi, India, 11-12, March 2016.
5. 10th International Symposium on "LowLand Technology (10th ISLT 2016)", National Institute of Technology Karnataka (NITK) Surathkal, Karnataka, India, 15 – 17, september 2016,
6. The short term programme on "Transportation Systems Simulation", National Institute of Technology Calicut (NITC), Kerala, India, 4 – 6, February 2016.

7. Knowledge Dissemination Workshop on "Development of Warrants for use of Modified Binders for Improved Performance of Flexible Pavements", Indian Institute of Technology Madras, Chennai, Tamilnadu, India, 24 – 25, October 2016.
8. 5-day course on "Engineering Analysis and Design of Rigid Pavements", under the Global Initiative of Academic Networks (GIAN), a program of Ministry of Human Resource Development, Government of India, National Institute of Technology Karnataka (NITK), Surathkal, Karnataka, India, 25 – 29, July2016.

Memberships

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