COMPARATIVE PERFORMANCE ANALYSIS OF SELECTED SECTIONS OF LAHORE-ISLAMABAD MOTORWAY (M-2)

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Certified that the contents and form of thesis titled **"Comparative Performance Analysis of Selected Sections of Lahore-Islamabad Motorway (M-2)"** submitted by Humza Ejaz have been found satisfactory for the requirement of the degree.

Supervisor: ______________________________

Associate Professor (Dr. Muhammad Bilal Khurshid, PhD)

DEDICATED

TO

MY FAMILY, TEACHERS AND COLLEAGUES

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(Humza Ejaz)

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LIST OF ACRONYMS

AASHTO	- American Association of State Highway and Transportation Officials
ASTM	- American Society for Testing and Materials
ANOVA	- Analysis of Variance
BS	- British Standard
DFFITS	- Difference in the Fitted Values
ESALs	- Equivalent Single Axle Loads
HMA	- Hot Mix Asphalt
LVDTs	- Linear Variable Differential Transformer
ME	- Mechanistic- Empirical
UTM	- Universal Testing Machine
VMA	- Voids in Mineral Aggregates
VFA	- Voids Filled with Asphalt

ABSTRACT

Pakistan is country which constitutes a huge network of roads and this network is constantly increasing. Due to the lack of the periodic maintenance the condition of the roads are not very impressive. Some of the roads do not even complete their design life due to poor construction methodologies or incompatible design or job mix formula (JMF). The pavement design in Pakistan is based on AASHTO design guide and Marshall Method is generally used for the mix design. Both these methods are empirical in nature and are developed for the environmental conditions of USA, however these design methodologies are used in different parts of the world. In flexible pavements of Pakistan, the two main distresses are Rutting and Fatigue Cracking. Motorway (M-2) has generally performed very well after the construction was completed in 1997. However, some sections of the motorway are subjected to rutting and fatigue cracking and at the same time some sections have performed very well.

This study is based on forensically analyzing the selected good and bad sections of M-2 to compare their results and to find the causes of rutting and fatigue cracking in Bad section. The analysis is based on the destructive testing of the pavement. In addition to the conventional testing, this study includes performance testing of both good and bad sections of the pavement. Stability and flow, moisture susceptibility by tensile strength ratio test (TSR), Indirect tensile fatigue test (IDFT) and resilient modulus test were performed on the cores recovered from site.

The experimental results were analyzed statistically in Minitab to spot the significant differences in values. The resilient modulus values for Asphaltic Wearing Course of selected bad section showed that the material was susceptible to rutting as compared to the material of asphaltic wearing course of selected good section and analysis of variance (ANOVA) proved the same. The indirect tensile fatigue test (ITFT) elaborated that the wearing course of selected bad section failed after significantly less number of load cycles as compared to the wearing course of selected good section. The moisture susceptibility by tensile strength ratio (TSR) showed that both asphaltic wearing course and asphaltic base course of selected bad section were highly moisture susceptible. So the experimental and statistical results showed that the selected bad

section was susceptible to rutting, fatigue cracking and moisture induced damage. The main reasons for the distresses was high percentage of asphalt content, low stiffness and high percentage of air voids.

Chapter 1

INTRODUCTION

1.1 GENERAL

Pavements can be divided into two categories, flexible and rigid pavement. The load distribution pattern to the subgrade in both of these types of pavements are entirely different. In flexible pavements, the load is distributed to the subgrade gradually through the layer system and the high quality material is used in the top layer of the pavement. In rigid pavement type, the load is transferred to the subgrade soil only by the concrete slab.

The structural design of the flexible pavements and asphalt overlay is a process which is based mostly on the experience. This evolutionary process is expanded by empirical relationships based on research and field observations. Several complex and interrelated factor are considered. Interaction of these factors have developed new design models which are based on elastic and viscoelastic theories. In general, the design methodologies for the flexible pavements can be divided in to empirical and mechanistic-empirical approaches. Empirical procedures are very easy to use but they are derived from experience and they lack theoretical background. They are often custom designed, thus limiting their application. However, mechanistic-empirical approaches is based on the theory and is validated by experience but they are still unable to model completely the interactions of different factors like environment, drainage etc. which initiate the distresses in the pavement.

Fatigue cracking is defined as load induced cracks which can be found in the wheel path and are accelerated by both loading and environmental factors. Traffic volumes, material properties, construction quality, layer thicknesses, and the environmental effects have an influence on rutting and fatigue cracking potentials. Therefore, any methodology which is solely based on empirical or mechanistic-empirical approach cannot model the flexible pavement behavior(Mukhtar 1993). In addition to the conventional testing, this study includes performance testing on the samples of both selected good and bad sections of the pavement.

1.2 PROBLEM STATEMENT

The pavement should be designed in such a manner so that it can handle the stresses and strains produced by the moving traffic. Moreover, the design should also withstand the damaging effects of the climatic condition. Performance of asphalt concrete layer depends upon:

1. The compaction of asphalt mix.
2. The volume and type of traffic.
3. Environmental and drainage conditions.
4. Quality control during manufacture of hot mix asphalt (HMA) and pavement construction.
5. Performance of underlying layers.

The factors mentioned above govern the expected life of the compacted mix(Livneh 1990). The flow properties of the bituminous mixtures generally depends upon the temperature. Viscosity is the dominating factor of the asphalt binder and it depends on the viscosity. It is observed through general experience that the resistance to deformation of bituminous materials decreases drastically with increase in the temperature. The temperature of Pakistan increases drastically from May to August. The pavement temperature is generally 10^{o}C to 15^{o}C more that the air temperature. The stiffness modulus of HMA is high when the temperature is low, thus the deformation under loads does not occur. However, when the temperature rises to a very high value the stiffness modulus of the HMA decreases and the chance of deformation under the load increases.

Rutting is caused by the following factors:

1. Heavy vehicle loads and high number of trucks and trailer
2. The existing asphalt mix design practices.
3. The pavement design process.
4. Existing construction and quality control practices

In Pakistan, AASHTO design procedure is used for the pavement structural design. The problem with the design is that no efforts have been made to calibrate the design according to the climatic conditions of the country. Mix design is very critical when it comes to pavement performance. Marshall Mix design is being used for the mix design process. In past it has been observed that some pavements including M-2 have performed extraordinarily well. On the other hand most of the pavements with same thickness designs and same level of traffic loading have failed in their early years of operation. Thus the mix design properties of good performing pavement must be evaluated for the betterment of the overall network of the country.

1.3 RESEARCH OBJECTIVES

- Select representative good and bad test section along the Motorway M-2
- Obtain representative core samples from adjacent good and bad test sections.
- Test the pavement cores to determine their physical and engineering properties.
- Determine the variability of the material and layer thicknesses along the section for each cored pavement section.
- To carry out comparative analysis of selected sections of M-2 on the basis of performance testing for:
    - Rutting
    - Moisture susceptibility
    - Fatigue cracking
- To recommend the remedial measures.

1.4 SCOPE OF THE STUDY

The research includes evaluation/analysis of good and bad sections along Motorway M-2 with emphases on the slow lane. The theme of the analysis is to find out the material variability in different test sections and to find the factors that causes failure or success of a particular sections. In order to conventional testing, this study emphasizes on performance based testing of the on the cores taken from the test section.

1.4 SEQUENCE OF THESIS

There are five chapters in the thesis.

- Chapter 1 is based on introduction and the objectives which were opted for the thesis.
- Chapter 2 throughs light on the review of the past studies and research related literature review.

- Chapter 3 explains the detailed research methodology which was opted for the whole research project. The results of experimentation are also discussed in this chapter
- Chapter 4 includes the analysis of the results which came out as a result of experimentation.
- Chapter 5 includes the conclusions of the research and future work that can be further be done.

Chapter 2

REVIEW OF THE PAST STUDIES

2.1 INTRODUCTION

Load distribution in flexible pavement is done through a layered system in which uses the highest quality material in the upper or top most layer (Yoder 1975). Strength of the Flexible pavement (layered system) is derived from the load distributing characteristics and it yields elastically to loads of traffic (Lenz 2011). The design of the flexible pavement should be in such a manner as to provide successful performance and also serve other functions such as capacity to carry load, friction (skid resistance), drainage throughout its life (sub surface and surface), riding comfort and above all safety (MS-4 Asphalt Handbook). Hence, for a design to be cost effective high strength material should be paced on the top as to take the high magnitude of stresses while the inferior quality and lows strength material should be place at the bottom where the magnitude of the stresses is low. Asphalt pavement is comprised of the following layers. (Lenz 2011).

1. Wearing surface
2. Base course
3. Subbase course
4. Subgrade

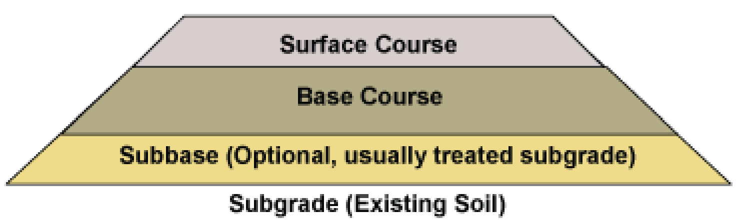

Figure 2.1: Pavement Structural Layers (Adopted from Lenz 2011)

As shown in figure 2.1, the top of the layer is the wearing course and it should be design as to that it strong enough to support the load and provide a smooth ride for comfort to the traffic at the same time. For the surface course generally the dense graded bituminous mixture is provided. Provision of the drainage layer for the infiltration of water is must. Under the wearing course there is the base course that consists of crushed materials or materials that stabilized by the used of Portland cement or and Asphalt.

Forensic studies have been carried out in the past. Kashif Ahmed khan carried out a study on National highway N-5 from Taxila region to Dina region in the year 2000. He first identified and selected successful and failed sections with the help of distress survey. He concluded that the main cause of rutting in the pavement was high percentage of bitumen content in the failed section.

Ahmed (2000) also carried out research on Islamabad Highway. He selected a particular stretch of Islamabad highway and carried out an investigation based on laboratory testing. The causes of failure he ascertained was high bitumen content and low air voids in the asphaltic layer of the pavement that attributed to the failure of the pavement.

Chen, Bilyeu et al. (2003) carried out a forensic study on a specific pavement section in Texas. A field investigation was initiated; the original plan was to cut nine trenches, however, after four trenches were cut, the problematic layer was identified and the trenching operation was terminated. Dynamic cone penetrometer, stiffness gauge, seismic pavement analyzer, and nuclear density gauge tests were then conducted on top of the base and subgrade layers. The trench profiles indicated that the rutting was coming primarily from the top 50-mm (2-inch) asphalt-concrete layer. Asphalt cores were taken from both rutted and non-rutted sections and bag samples of the base were tested in laboratory. The binder was recovered, and the asphalt content and penetration, aggregate gradation, and type were determined. The main cause which attributed to the failure was aggregate screening and excess amount of binder in the top layer of the pavement.

E Denneman & ES Sadzik (2008) carried out a forensic research in order to find out the performance of hot mix asphalt on five road sections in the Gauteng province. The research was based on improving the rutting and fatigue cracking behaviors of pavements in South Africa. The performance of the selected five sections were carefully studied and recommendations and conclusions were presented in order to build the pavements which would be resistant to rutting as well as fatigue cracking.

Nadkarni, Kaloush et al. (2009) carried out a study in which they used dynamic modulus test in order to evaluate the moisture susceptibility of the HMA mixes. In this study it was concluded that the moisture susceptibility test can be carried out in simple performance tester with the help of ESR (E* stiffness ratio) in addition to TSR (tensile strength rate) in indirect tensile mode. In this research the samples were tested with both the methods and it was found out that the difference between the results were not statistically significant.

2.2 FLEXIBLE PAVEMENTS

Flexible Pavement Analysis and Design Methods

There are three basic categories for the design of the flexible pavements. Empirical method, Empirical-Mechanistic method and the Mechanistic method. These methods are discussed below:

- **Empirical Method**

  The results of experience and experimentation provides the basis for Empirical method of pavement design. The design of the pavement using the Empirical method can be done even if the soils strength tests are not available. The design is no longer valid if the conditions are changed and a new method must be developed (Huang 2003).

- **Empirical-Mechanistic Method**

  The mechanics of materials play an important role in the pavement design (Huang 2003).when the traffic load s applied to the pavement deflections, stresses and strain are generated within a pavement structure. The physical causes of failure in a pavement structure are the material properties and loads of the pavement structure.

- **Mechanistic Method**

  Based on the assumed mechanism of pavement-traffic interaction, mechanistic models are built to calculate the pavement responses (stress, strain and deflections) due to traffic loading and environmental changes (Zhao and Arkansas 2007).

2.3 PAVEMENT DESIGN PARAMETERS

The design input values, (such as properties of the material, traffic and the applied load, and the environmental properties) and their selection comes as the major design considerations for the flexible pavement structural design. Another input parameter i-e pavement distresses is taken into account as a design parameter by the pavement design guide (AASHTO 2002).

2.3.1 Traffic and Loading

The vehicle speed is important to the loading due to viscoelastic nature of bituminous mix. With the application of load, the bituminous material exhibit deformation that is time dependent. The applied load duration in the asphalt layer depends on the speed. Loading time of higher speed vehicles is low and it results in smaller deformation and large resilient modulus. Therefore, for accurate determination of resilient modulus of asphalt, the loading duration during test should simulate the field conditions.

2.3.2 Material Properties

The Asphalt Institute procedure characterizes the properties like Resilient Modulus (M_r) and a Poisson's ratio. Due to the viscoelastic nature of the bitumen, variation is caused in the elastic modulus of the bituminous mixture with the loading time. Therefore, for the flexible pavements design and analysis the resilient modulus is selected.

2.3.3 Environment

For the design of the flexible pavements the Asphalt Institute Method takes into account the effects of environmental conditions on elastic modulus of bituminous paving mixes. The major environmental factors are:-

1. Moisture level

   It affects the Resilient Modulus of soil significantly.

2. Temperature

   It effects the resilient modulus of asphalt. Rutting may be caused by high temperature as it makes the asphalt layer viscous. Cracking failure may be caused by low temperature.

3. Frost Penetration

   It has effects on the whole pavement system.

2.3 DISTRESS SURVEY AND EVALUATION

2.3.1 Rutting

Plastic deformation in Asphaltic course, base, subbase or subgrade can be defined as Rutting. Slow moving heavy traffic is a major factor that leads to rutting which can be increased in extent by adverse environmental factors. To reduce the chances of rutting material used in all the pavement layers should be stiffer, design of the pavement should be proper and good construction practices should be adopted. Generally wheel path is the area where rutting is found. (Roberts 1996).

2.3.2 Fatigue Cracking

It is also known as alligator cracking. This distress type is associated to the load applied and usually found along the wheel path and it is accelerated by environmental factors. Fatigue cracking potential o pavement can be minimized by using appropriate pavement materials, proper design procedure, and good construction practices(Willis and Timm 2006). Deformation in the wearing course is mainly the result of lateral distortion due to repeated shear deformation(Morris 1973).

The tensile stresses or strains at the bottom of first layer is the major cause of this distress. Initially the crack develops from the end of the first layer and then it propagate upwards and appears at the surface at last(Willis and Timm 2007). Hence, the fatigue cracks may be present in a pavement for several years and will only be observed when they propagate to the top of the AC layer(Kandhal 1994).

In the laboratory, the fatigue life of a compacted asphalt specimen is defined by load cycles (which are counted in numbers) that can causes the fatigue failure of the sample. The definition of fatigue failure, however, varies from one researcher to another.

For example it can be defined as fatigue failure as the load applications required to reduce the stiffness of the specimen by 60 percent of its initial stiffness measured at 200 load applications(Santucci 1969). On the other hand it is also known as the load cylces at which the cumulative horizontal plastic deformation (measured along the horizontal deformation specimen tested to failure in the indirect tensile test mode)occurs in the sample (Baladi 1988).

It can lead to the separation of pieces of HMA by breaks away from the pavement under the traffic and lead to the pothole (Roberts 1996; Shahin 2005). This leaves behind just one option to reconstruct the pavement.

Figure 2.2: Alligator pattern cracking (Miller and Bellinger 2014)

2.4 HOT BITUMINOUS PAVING MIXES

The bituminous paving mixes or hot mixed asphalt (HMA) contains aggregates uniformly mixed and coated with the bitumen (MS-4 Asphalt Institute). To achieve proper mixing and fluidity both the aggregates and asphalt must be heated initially.

2.4.1 Types of Bituminous Paving Mixes

Depending basically on the aggregate gradation which is used in the HMA mix. The bituminous paving mixes are divided into three different types of mixes. These three types of mixes are (MS-2 Asphalt Institute):

- **Dense Graded Mixes**

Dense graded bituminous mixes are the one that consist mainly of well graded aggregates i.e. course, fine and filler material is mixed with the HMA. These mixes work well for the structural, patching, friction, and leveling needs.

- **Open Graded Mixes**

The open graded bituminous mixes usually consist of small amount of fine aggregates mixed with bitumen and large quantity of coarse aggregates. The use of these mixes is to provide an open surface texture that will allow the water to drain into the mix. The mix design procedure of the open graded mixes is different from dense graded bituminous mixes due of the lack of fines in the mix. Also the quantity of bitumen is less in open graded mixes as compared to dense graded mixes.

- **Gap Graded Mixes**

If the amount of fine aggregates in a mix is greater than that of fine aggregates in the open graded mix then it is known as a Gap Graded Mix. For a gap graded mix the materials required are;

1. Crushed stone and gravel
2. Bitumen
3. Manufactured sand

Aggregates of the #4 and #30 (middle size aggregates) are ether missing or present in a very small quantity in the mix.

2.5 VOLUMETRIC ANALYSIS OF COMPACTED PAVING MIXTURES

Volumetric analysis is carried out with the help of different tests including specific gravity tests for aggregates, bitumen and bituminous mixes. Calculation of the volumetrics of compacted paving samples are the carried out after determination of aggregates and bitumen properties and after mixing and compaction. Generally it includes following important properties:

- Range of acceptable Air Void Contents (V_a)

- Minimum amount of Voids in the Mineral Aggregate (VMA)
- Percent of Voids Filled with Asphalt (VFA)

2.5.1 Voids in the Mineral Aggregate (VMA)

The voids in mineral aggregate, VMA, are the void spaces between the particles of grains in a compacted bituminous paving mixture that also includes the air voids and the effective bitumen content and the values of the mentioned particulars are given in percentage. Bulk specific gravity is the basis for the calculation of the VMA. It is expressed as percentage of the bulk volume of compacted paving mixture. If the volume of aggregate calculated by its bulk specific gravity is subtracted from the bulk volume of the compacted paving mixture, the resultant is the VMA. The method for calculation is illustrated as follow:

$$VMA = 100 - \left[\frac{G_{mb} P_s}{G_{sb}} \right] \tag{2.1}$$

Where,

VMA	=	Voids in mineral aggregate (percent of bulk volume).
Ps	=	Percent of total aggregates in the mix.
G_{mb}	=	Bulk specific gravity of the compacted mix (ASTM D2726)
G_{sb}	=	Combined specific gravity of aggregates.

The value of the VMA has a significant role on the behavior of the mixes, for example the mix will suffer from durability issue if the VMA is too small. If the value is too large it may affect the stability of the mix or result in an uneconomical mixture. In a mixture there is a film of bitumen around every aggregate particle, the thickness of this film is determined by the volume of bitumen in the mix along with the aggregate. If the film thickness is not sufficient it result in faster oxidation of the bitumen, penetration of water, and negative impacts on the tensile strength of the mix.

2.5.2 Air Voids

These are small air spaces between the particles of HMA in a compacted mix. It can be calculated by using the below given equation.

$$V_a = 100\left[\frac{G_{mm} - G_{mb}}{G_{mm}}\right] \tag{2.2}$$

Where,

G_{mb} = Bulk specific gravity of the compacted mix.

G_{mm} = Maximum theoretical specific gravity of the mix.

V_a = Air voids in compacted mixture, percent of total volume.

2.5.3 Voids Filled with Asphalt

The intergranular space, which is also known as VMA, filled with asphaltic particles is known as the voids filled with asphalt. VFA, not including the absorbed asphalt, is determined using following equation:

$$VFA = 100\left[\frac{VMA - V_a}{VMA}\right] \tag{2.3}$$

Where,

VFA = Voids filled with asphalt.

VMA = Void is mineral aggregates.

V_a = Air voids in the compacted mix.

2.6 MARSHALL TEST

This test is used for the evaluation of bituminous mixture and mix design (ASTM D6927). Compression testing machine is used to carry out stability and flow tests. The maximum load that is taken by test sample at the constant loading rate of about 2-inch/minute gives the stability of the sample. Initially the load increases when applied to the specimen then it reaches to its maximum and then it starts to decrease. The maximum load is taken as the stability of the sample. As a result of loading the plastic flow also occurs which is recorded with the help of a gauge which is attached to the specimen.

2.7 STRUCTURAL EVALUATION

a. Resilient Modulus Test

The Modulus of Resilience is a key material stiffness parameter and basis for material characterization in AASHTO design guide 1993(Highway and Officials 1993). It is mechanical property of material and defines stress strain relationship under loading cycle closely simulating the actual traffic under various conditions of confinement. For the roadbed materials, resilient modulus tests are conducted in laboratory in light of AASHTO T 307-2003 on represented samples in stress and moisture conditions simulating those of primary moisture conditions or alternately seasonal resilient modulus can be determined. An effective roadbed soil resilient modulus is then established which is equivalent to cumulative effect of all the values of seasonal modulus. According to the above mentioned standard materials such as natural soils and unbound granular layer (low stiffness material) should be tested. The HMA resilient modulus test, ASTM D4123, is relatively simple and can be carried out on the cores taken from the field. The test is conducted in indirect tension mode on field cores by applying compressive loads and the waveform which is opted for the test is haversine. Detail procedure for this test was developed for HMA, aggregate base and subgrade materials in the study "laboratory determination of resilient modulus for flexible pavement design".

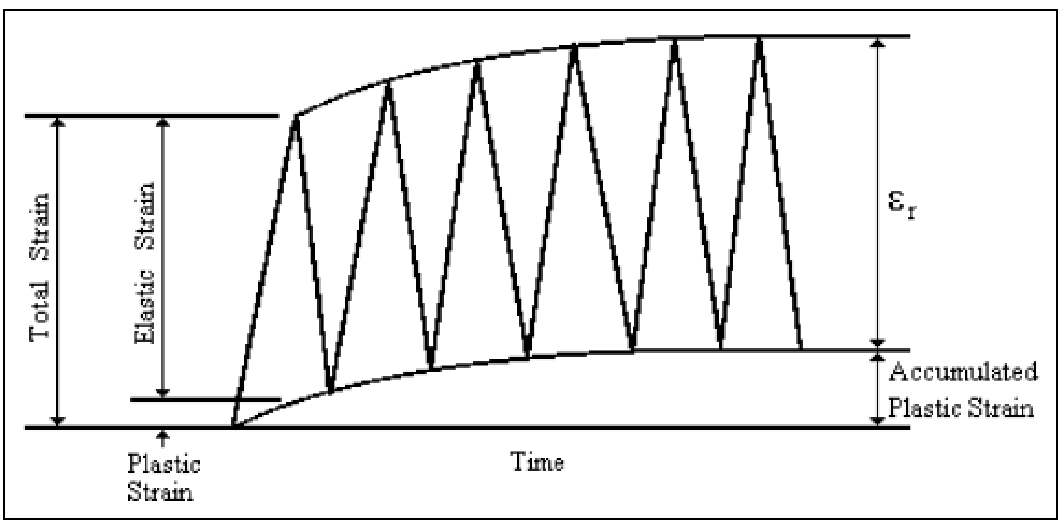

Figure 2.3: Recoverable Strain under Cyclic Load (Huang 1993)

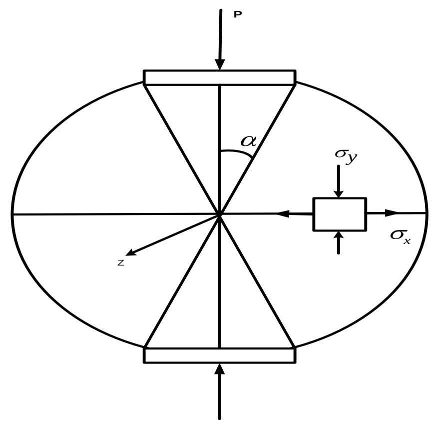

Figure 2.6: Schematic for Indirect Tension Test (Yoder 1975)

b. Moisture Susceptibility Test

Stripping in the HMA pavement is a major distress which is induced by the moisture and it is also known as the moisture susceptibility. The presence of water causes the internal asphalt binder-to-aggregate to weaken and the HMA is said to be moisture susceptible (Kandhal 1994). In order to measure the moisture damage or moisture susceptibility of the HMA mixes, this test can be performed(Kandhal and Association 1992; Aksoy, Şamlioglu et al. 2005).

IDT (Indirect Tensile Test) is carried out using two sets of HMA. Conditioning of one set is carried out by soaking it in water for a complete day (24 hours) along with partial vacuum saturation with water an freeze thaw cycle (optional). As a control the other set is used (Aksoy, Şamlioglu et al. 2005).

$$TSR=S1/S2$$

Where:

TSR = Tensile Strength Ratio

S1 = Average Tensile Strength of Unconditioned Samples

S2 = Average Tensile Strength of Conditioned Samples

c. Indirect Tensile Fatigue Test

This test is quite simple as the cylindrical specimens that are used in the test are easy produced in the laboratory or can be acquired from the field by coring of the pavement. A study was carried out by Adedimila and Kennedy in 1975 to find out the behavior of Indirect Tensile Fatigue Test in when compared to other test methods. The study results showed that Indirect Tensile Fatigue Test presented much shorter fatigue lives to failure, based on log-log relationship, than the bending beam test or the trapezoidal cantilever test method. The Strategic Highway Research Program (SHRP) of United States evaluated the Indirect Tensile Fatigue Test in comparison to four point bending beam and trapezoidal cantilever test, and the conclusions drawn from the evaluation are shown in the Table 2.1. Indirect Tensile Fatigue Testing subjects the HMA materials to repeated line loading along the vertical diameter of the cylinder shaped specimens. The relative stress distribution along the cylindrical specimen are shown in Figure 2.7. Following assumptions can be made to determine the maximum strain at the Centre of the sample:

- Behavior of material is linear elastic and the material is homogeneous.
- Material behaves in isotropic manner
- The material's Poisson's ratio is known
- The force is applied as line loading
- Specimen is subjected to plan stress conditions

Table 2.1: List of Cons and Pros of ITFT

Test	Advantages	Disadvantages
Indirect Tensile Fatigue Test	• Test is relatively simple • Response from the test and field correlation can be used to design the HMA mixture and pavement to resist fatigue • Equipment can be used for other tests also • Failure initiated in region of relative uniform tensile	• Impossible to vary the horizontal and vertical components to replicate the state of stress at critical locations in pavement • Method significantly underestimates the fatigue life is the damage is determined using tensile stress

	stress • Biaxial state of stress better represent field conditions as compared to flexural test • Test can also be performed on field cored samples • Discrimination between mixtures containing different binders can be done on the bases of stiffness and cycles to failure • Repeatability of test for cycles to failure is much better than both trapezoidal or beam flexure test	• Absence of stress reversal and accumulation of permanent deformation • Reliability to measure stiffness in not as good as trapezoidal or beam flexural test

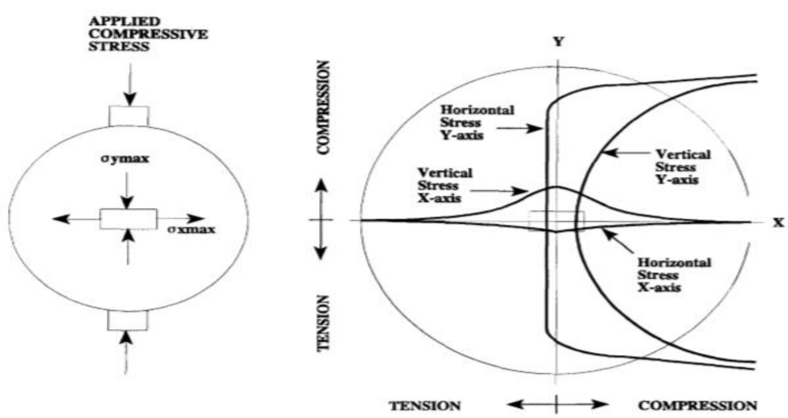

Figure 2.7: Relative stress distribution along the cylindrical specimen (Yoder 1975)

2.12 UNIVERSAL TESTING SYSTEM

The Universal Testing System also called as Materials Testing Machine (System) can be used to test resilient modulus, tensile strength, dynamic modulus, permanent deformation, fatigue etc. of asphalt concrete. The Universal Testing System consists of following parts (Vos 2006).

- A hydraulic or, pneumatic axial and pneumatic confining stress loading system.
- A Control and Data Acquisition System (CDAS).
- A computer (PC) with the Microsoft Windows operating system.
- A suite of UTS software applications and support files.

The CDAS provides both the servo-feedback loading control electronics and transducer data acquisition and timing functionality. Overall system control is managed by the PC under direction of the application software. Also, data gathered by the CDAS during specimen testing is processed, displayed, reported and archived on the PC. A PC-based pendant provides axis jogging operations together with power-pack control for hydraulic loading systems.

2.14 SUMMARY

This chapter first discusses about the different types of the distresses in the asphaltic pavements. For the identification of distresses, the chapter edifies about visual distress survey. After establishing the facts about the distresses, the chapter throws light on non-destructive and destructive testing of the pavement to evaluate its performance. Indirect tensile fatigue test, modulus of resilience test and moisture susceptibility tests were also deliberated in this chapter. Volumetric analysis of HMA samples were also conferred in detail.

Chapter 3

RESEARCH METHODOLOGY

3.1 INTRODUCTION

The detailed methodology of the research is discussed in this chapter in order to fulfil the research objectives. The distress survey for the selection of representative good and bad section and acquisition of cores from site and laboratory testing of cores are discussed in this chapter.

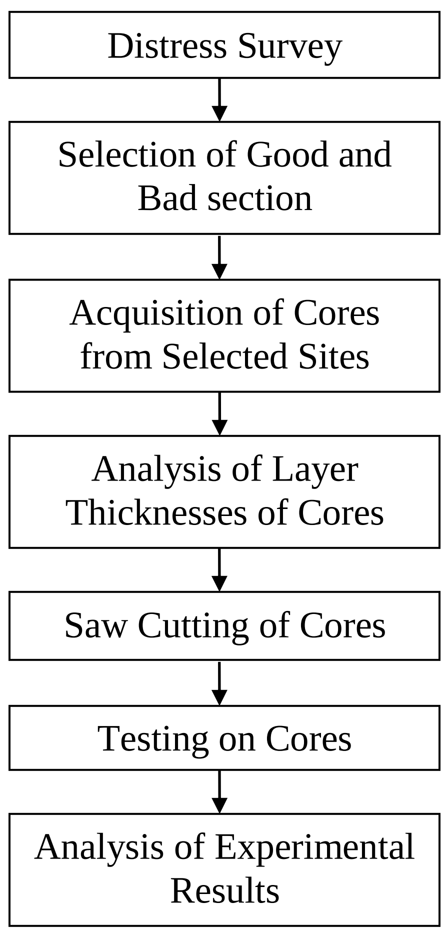

Figure 3.1: Sequence of Research Methodology

3.2 DISTRESS SURVEY

Pavement visual distress survey is done out in order to identify the representative good and bad section on Motorway (M-2). The distress survey was carried out in light of pavement distress manual. The selected bad section constituted rutting of and fatigue cracking and longitudinal and transverse cracks. The identified representative bad section was from RD 308+00 to RD 307+00 south bound. The identified good section which was clear of the distresses was from RD 310+00 to RD 307 south bound. Only slow lane of the pavement was considered for the research. Temperature, climatic conditions and traffic load was same for both representative good and bad sections.

3.2.1 Rutting

Rutting is the depression on the road that can be observed over the whole length of a pavement along the wheel path. The rutting observed in the selected good section on M-2 was of low severity. Whereas the rutting observed at selected bad section was of medium severity. Rutting in a section can be measured either by using a straight edge or it a profilometer. Profilometer is a device used to check the roughness of a surface of a road but special profilometer with additional sensors can also measure the rut depth along a particular section.

Average rut depth in the selected Bad section was recorded as 0.76 inched which comes under the medium severity level of the rutting criteria. Average rut depth encountered in the good section was 0.25 which comes under the low severity.

The severity levels according to Pavement Distress Manual for Long Term Pavement Performance are as follows (John S. Miller and William Y. Bellinger (2013)):

- LOW

  Ruts with a measured depth ≥ 0.20” and ≤ 0.49”
- MED

  Ruts with a measured depth ≥ 0.50” and ≤ 0.99”
- HIGH

  Ruts with a measured depth ≥ 1.00”

Figure 3.2: Rutting Measurement

3.2.2 Fatigue Cracking

The main distress in the asphaltic concrete roads is the series of interconnected cracks that can form a shape of crocodile. It occurs due to the heavy loads which are subjected to the pavement. Fatigue cracking, if not handled properly can lead to raveling and formation of potholes in the pavement. Excessive cracking can also lead to moisture infiltration.

More than 25% of Area of selected bad section was observed to have a medium level of fatigue cracking. Less than 10% area of selected good section had a low severity level of fatigue cracking.

The severity levels according to Pavement Distress Manual for Long Term Pavement Performance (LTPP) are as follows (John S. Miller and William Y. Bellinger (2013)):

- Low if an area of cracks with no or only a few connecting cracks. Cracks are not spalled or sealed. No pumping is evident.
- Moderate if an area of interconnected cracks forming a complete pattern. Cracks may be slightly spalled. Cracks may be sealed. No pumping is evident.

- High if an area of moderately or severely spalled interconnected cracks forming a complete pattern. Pieces may move when subjected to traffic. Cracks may be sealed. Pumping may be evident

Figure 3.3: Fatigue Cracking on M-2

Figure 3.4: Fatigue Cracking (Close View)

3.2.3 Selected Good and Bad Section

The Basic criteria of the selection of good and bad sections was:

- The traffic load on both should be the same.
- The selected sections should have never been subjected to any kind of treatments in the past.
- The sections must have same environmental and traffic conditions.

The selection of good and bad section was majorly based on the pavement distress survey. The rutting and fatigue cracking encountered in the both section are described in the previous paragraph. In short, the section which had minimal distresses was selected as a selected good section and the homogeneous section to that good section which had medium to high severity distresses was selected as selected bad section. The Map below shows the location of Selected good and bad Section on M-2.

Figure 3.5: Map of Selected Sections

3.3 ACQUISITION OF CORES FROM SITE

The electrically driven core cutting machine with 4 inch diameter coring bit was used for taking cores from the selected representative good and bad sections. The core cutting was carried out according to the standard practice.

Figure 3.6: Core Cutting Process

Figure 3.7: Core Taken from Site

The coring plan was to take 3 cores in 100 meters from slow lane of representative good or bad section. One core was taken from left wheel path, one was taken from right wheel path and one was carried out from the middle of both wheel paths. The coring location plan for good and bad section was given below.

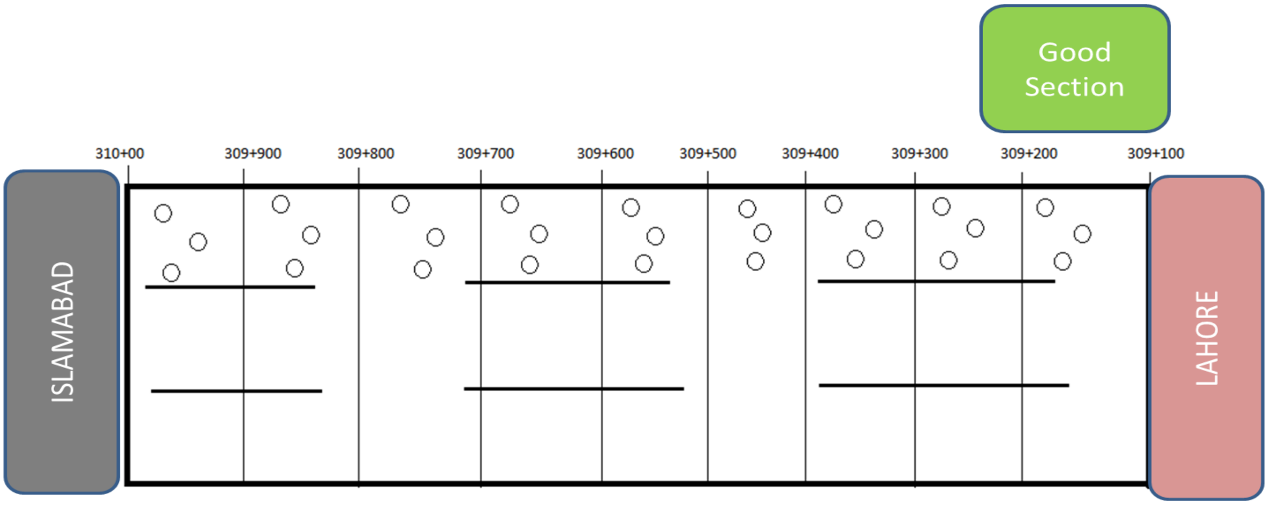

Figure 3.8: Coring Plan for Good Section

- No. of cores obtained from Right wheel path = 9
- No. of cores obtained from Left wheel path = 9
- No. of cores obtained from between the wheel path =9
- Total No. of cores obtained= 27

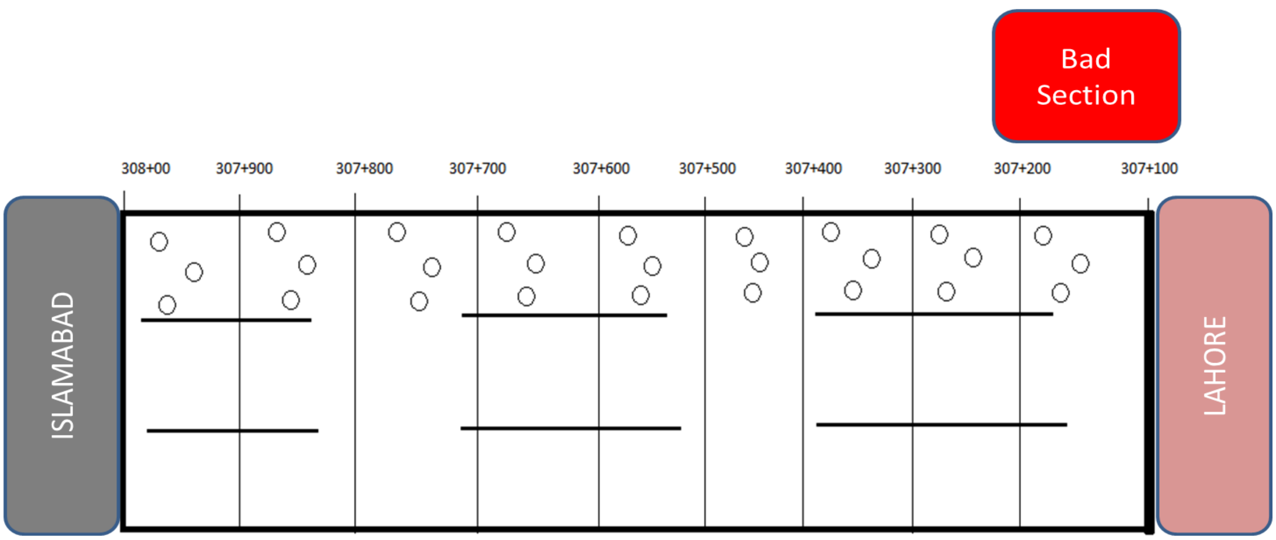

Figure 3.9: Coring Plan for Bad Section

The coring details of the Bad section are as follows:

- No. of cores obtained from Right wheel path = 9
- No. of cores obtained from Left wheel path = 9
- No. of cores obtained from between the wheel path =9
- Total No. of cores obtained= 27

3.4 LAYER THICKNESSES AND SAW CUTTING OF SAMPLES

After acquisition of cores from the site, the layer thicknesses of each core was measured and documented.

Table 3.1: Layer Thickness Values of Good Section

RD	Sample	Thickness (Inches)	Sample	Thickness (Inches)
310+00-309+900	WLP	2.3	BLP	3.5
	WM	2.4	BM	3.6
	WRP	2.3	BRP	3.5
309+900-309+800	WLP	2.4	BLP	3.7
	WM	2.5	BM	3.8
	WRP	2.3	BRP	3.7
309+800-309+700	WLP	2.2	BLP	3.4
	WM	2.4	BM	3.5
	WRP	2.2	BRP	3.4
309+700-309+600	WLP	2.3	BLP	3.2
	WM	2.4	BM	3.4
	WRP	2.2	BRP	3.3
309+600-309+500	WLP	2.3	BLP	3.4
	WM	2.4	BM	3.5
	WRP	2.3	BRP	3.3
309+500-309-400	WLP	2.2	BLP	3.5
	WM	2.3	BM	3.7
	WRP	2.3	BRP	3.6
309+400-309+300	WLP	2.2	BLP	3.4
	WM	2.3	BM	3.6
	WRP	2.2	BRP	3.5
309+300-309+200	WLP	2.3	BLP	3.3
	WM	2.4	BM	3.5
	WRP	2.3	BRP	3.3
309+200-309+100	WLP	2.1	BLP	3.4
	WM	2.3	BM	3.5
	WRP	2.2	BRP	3.4
	Average	2.3	Average	3.47
	W= Wearing Course **B= Asphaltic Base course** **LP= Left Wheel Path** **M= In Between of Wheel paths** **RP= Right Wheel Path**			

Table 3.2: layer Thicknesses Values of Bad Section

RD	Sample	Thickness (Inches)	Sample	Thickness (Inches)
308+00-307+900	WLP*	2.1	BLP*	3.4
	WM*	2.2	BM*	3.5
	WRP*	2.1	BRP*	3.3
307+900-307+800	WLP*	1.9	BLP*	3.6
	WM*	2.2	BM*	3.7
	WRP*	2	BRP*	3.6
307+800-307+700	WLP*	2	BLP*	3.3
	WM*	2.1	BM*	3.4
	WRP*	2	BRP*	3.3
307+700-307+600	WLP*	2.2	BLP*	3.2
	WM*	2.4	BM*	3.4
	WRP*	2.2	BRP*	3.2
307+600-307+500	WLP*	2.1	BLP*	3.4
	WM*	2.3	BM*	3.5
	WRP*	2.2	BRP*	3.3
307+500-307-400	WLP*	2	BLP*	3.4
	WM*	2.2	BM*	3.6
	WRP*	2.1	BRP*	3.5
307+400-307+300	WLP*	2.1	BLP*	3.3
	WM*	2.2	BM*	3.5
	WRP*	1.9	BRP*	3.4
307+300-307+200	WLP*	2	BLP*	3.3
	WM*	2.1	BM*	3.5
	WRP*	2	BRP*	3.3
307+200-307+100	WLP*	2.2	BLP*	3.3
	WM*	2.3	BM*	3.4
	WRP*	2.1	BRP*	3.3
	Average	2.12	Average	3.40
	W= Wearing Course **B= Asphaltic Base course** **LP= Left Wheel Path** **M= In Between of Wheel paths** **RP= Right Wheel Path** *** represents bad section**			

After documenting the layer thicknesses of the samples the samples were cut to separate the wearing and asphaltic base course from each other. Saw cutter was used in order to perform the above mentioned responsibility.

Figure 3.10: Samples from Site

Figure 3.11: Saw Cutting of Samples

3.5 TESTING ON CORES

After recording the thicknesses of cores and subjecting them to saw cutting, the cores taken from sites were evaluated with the help of following tests.

- Marshal Stability and Flow Test
- Bulk Specific Gravity
- Theoretical Maximum Specific Gravity
- Bitumen Extraction Test
- Gradation of Aggregates
- Indirect Tensile strength Test
- Resilient Modulus Test
- Indirect Tensile Fatigue Test
- Moisture Susceptibility by Tensile Strength Ratio

3.5.1 Marshal Stability and Flow Test

Standard for this test is AASHTO T 245. The strength of the samples and also their plastic flow are determined with the help of this test. According to asphalt institute the stability value should be greater than 6.67KN and the flow value must be between 8 to 16 mm. Before the test is carried out the samples are conditioned in water bath at 60^0C for one hour. After it is tested in the Marshall Equipment and stability and flow value is recorded.

Table 3.3: Marshall Stability and Flow for Good Section

MARSHALL TEST (WEARING COURSE)		
SAMPLE	STABILITY (KN)	FLOW (mm)
WLP	13.089	8.117
WM	15.963	8.753
WRP	14.349	8.574
AVG	14.467	8.481
MARSHALL TEST (ASPHALTIC BASE COURSE)		
SAMPLE	STABILITY(KN)	FLOW(mm)
BLP	16.473	10.25
BM	14.793	10.883
BRP	14.697	10.43
AVG	15.321	10.52

Table 3.4: Marshall Stability and Flow for Bad Section

MARSHALL TEST (WEARING COURSE)		
SAMPLE	STABILITY (KN)	FLOW (MM)
WLP*	16.345	10.54
WM*	12.543	9.65
WRP*	13.499	10.68
AVG	14.112	10.290
MARSHALL TEST (ASPHALTIC BASE COURSE)		
SAMPLE	STABILITY(KN)	FLOW(MM)
BLP*	15.973	9.52
BM*	14.513	10.98
BRP*	13.6396	10.43
AVG	14.71	10.31

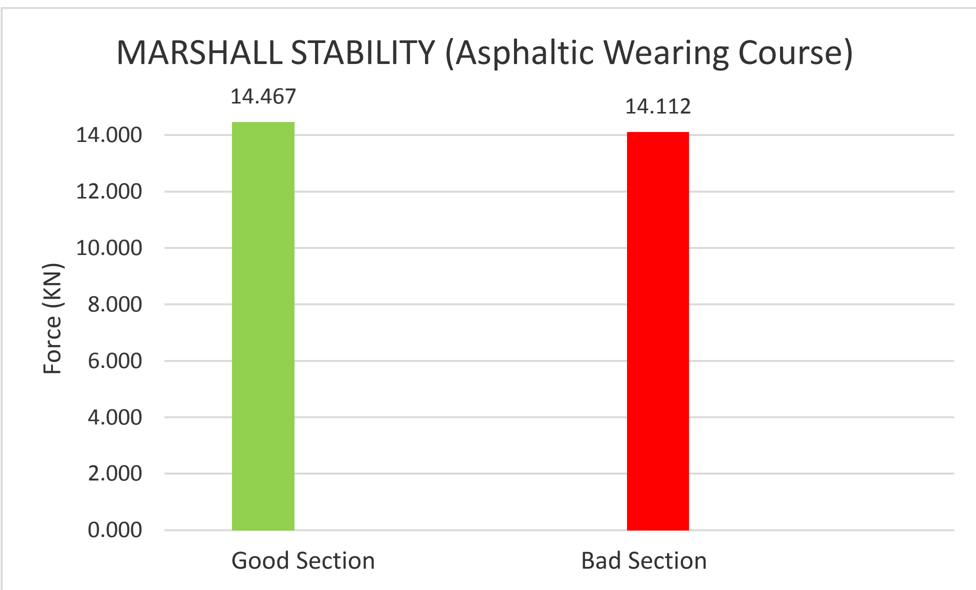

Figure 3.12: Comparative Graph of Marshall Stability of Wearing Course

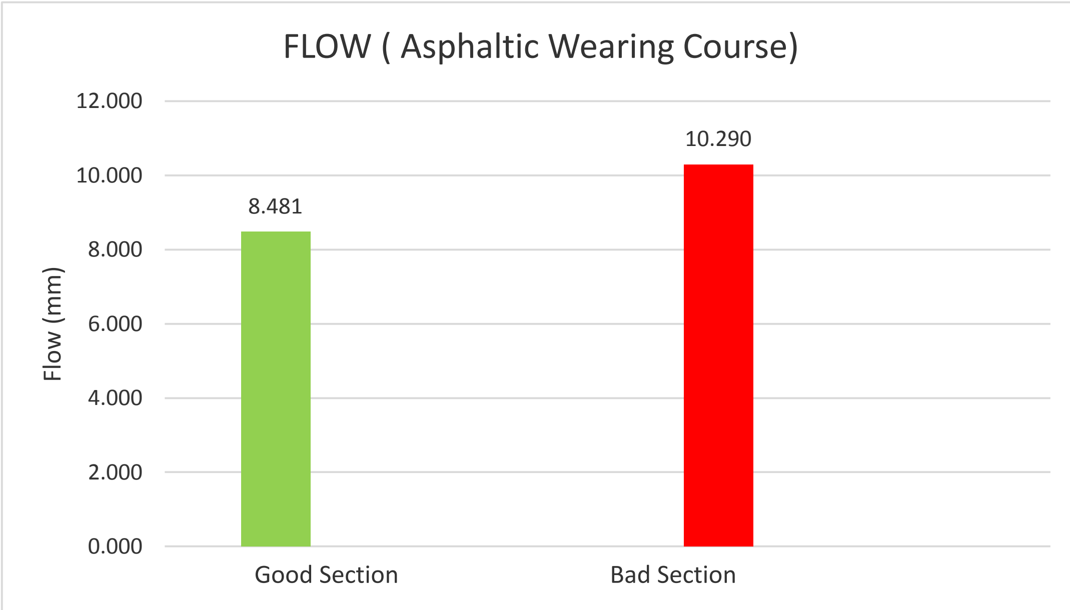

Figure 3.13: Comparative Plot of Flow values for Asphaltic Wearing course

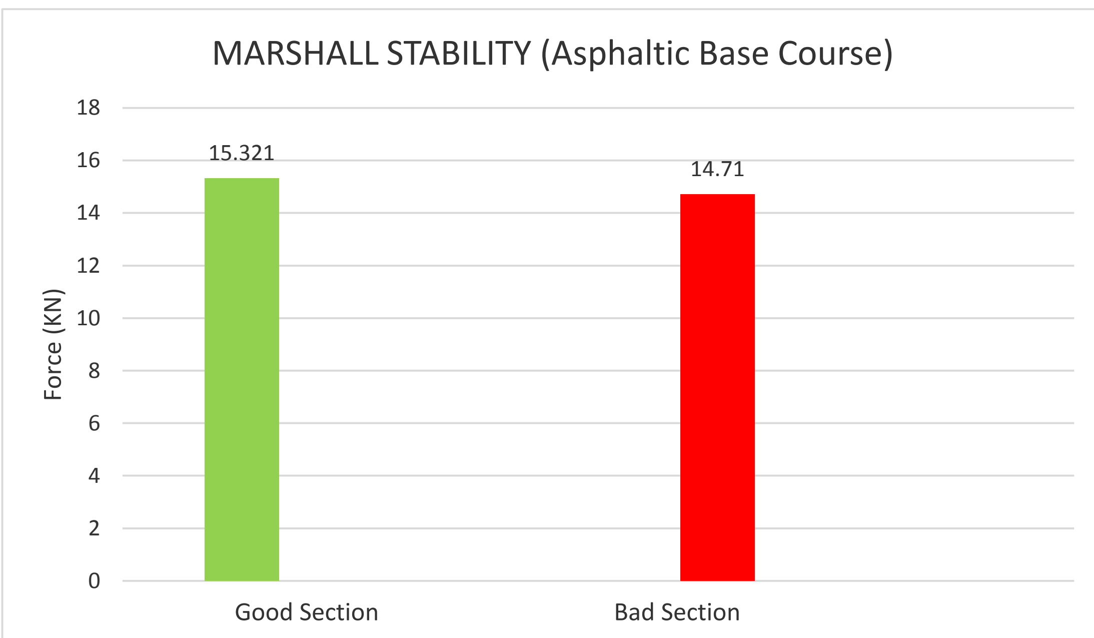

Figure 3.14: Comparative Graph of Marshall Stability of Asphaltic Base Course

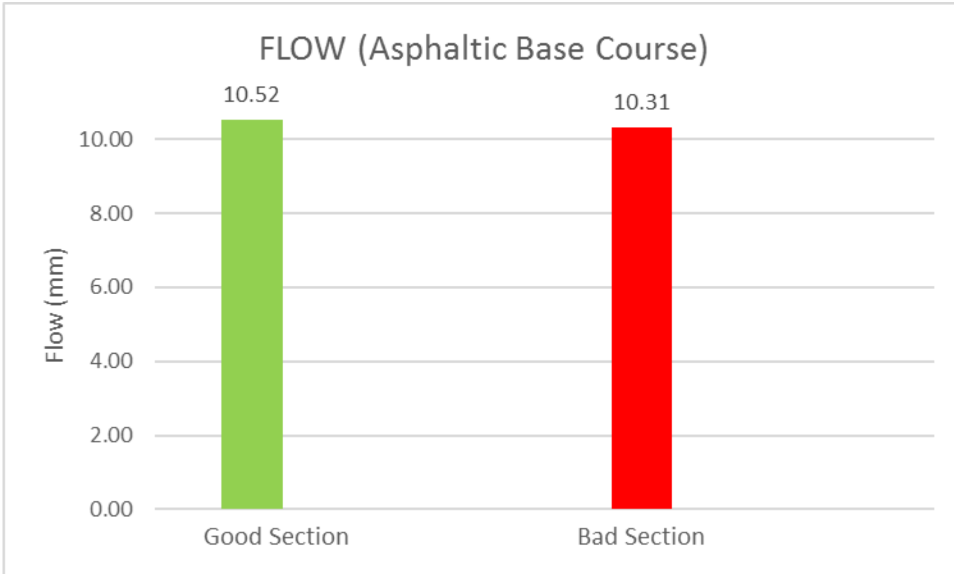

Figure 3.15: Comparative Graph of Flow of Asphaltic Base course

As it is evident from the results and the graphs presented above, the wearing course and asphaltic base course of good section seems to have a better stability value than that of bad section. However, the flow of samples of wearing and asphaltic base course of bad sections seems to have higher values than that of good section and higher flow value can be attributable to the rutting in the bad section.

3.5.2 Bulk Specific Gravity

The standard test procedure for this test is ASTM D 2726. This Test is used to calculate volumetric properties in a HMA mix. The first step of the procedure is to take the weight of the dry sample at room temperature. After that the sample is submerged in water for 5 minutes and its weight n water is recorded. At last the saturated surface dry weight of sample is recorded by taking it out of water bath and then drying its surface.

Another method of finding bulk specific gravity is the gamma ray method. The gamma ray method is based on the scattering and absorption properties of gamma rays with matter. When a gamma ray source of primary energy in the Compton range is placed near a material, and an energy selective gamma ray detector is used for gamma ray counting, the scattered and unscattered gamma rays with energies in the Compton range can be counted exclusively. With proper calibration, the gamma ray count is directly converted to the density or bulk specific gravity of the material.

Table 3.5: Bulk Specific Gravity for Good Section

Gmb (WEARING COURSE)				
SAMPLE	WEIGHT IN AIR	WEIGHT IN WATER	SSD WEIGHT	Gmb
WM	931.4	529.2	933	2.306587
WLP	824.7	473.1	826.1	2.336261
WRP	878.4	501.5	880.2	2.319514
		AVG		2.32
Gmb (ASPHALTIC BASE COURSE)				
SAMPLE	WEIGHT IN AIR	WEIGHT IN WATER	SSD WEIGHT	Gmb
BM	987.4	566.6	989.7	2.333727
BLP	864.8	495.6	866.8	2.329741
BRP	921.6	527.1	923.9	2.322581
		AVG		2.33

Table 3.6: Bulk Specific Gravity for Bad Section

Gmb (WEARING COURSE)				
SAMPLE	WEIGHT IN AIR	WEIGHT IN WATER	SSD WEIGHT	Gmb
WM*	903.4	521.5	904	2.36183
WLP*	790.6	451.4	791.5	2.32461
WRP*	876.5	500.2	877.8	2.321239
		AVG		2.34
Gmb (ASPHALTIC BASE COURSE)				
SAMPLE	WEIGHT IN AIR	WEIGHT IN WATER	SSD WEIGHT	Gmb
BM*	1005.2	581.6	1006.5	2.365733
BLP*	939.2	545.3	940.1	2.378926
BRP*	875.6	499.8	876.9	2.321931
		AVG		2.36

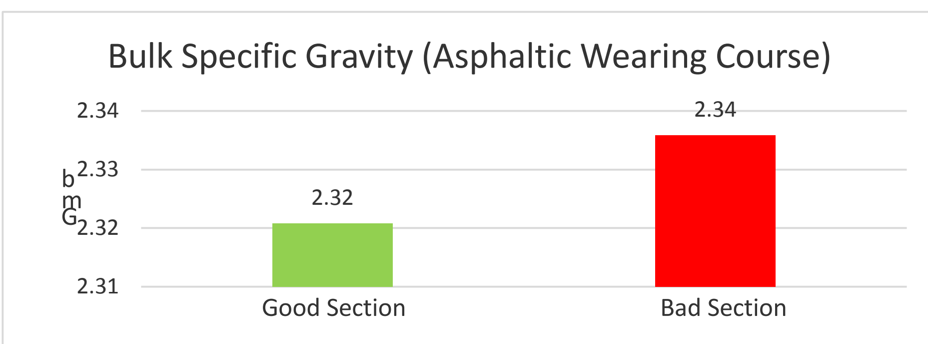

Figure 3.16: Comparative Plot for Gmb for Good and Bad section

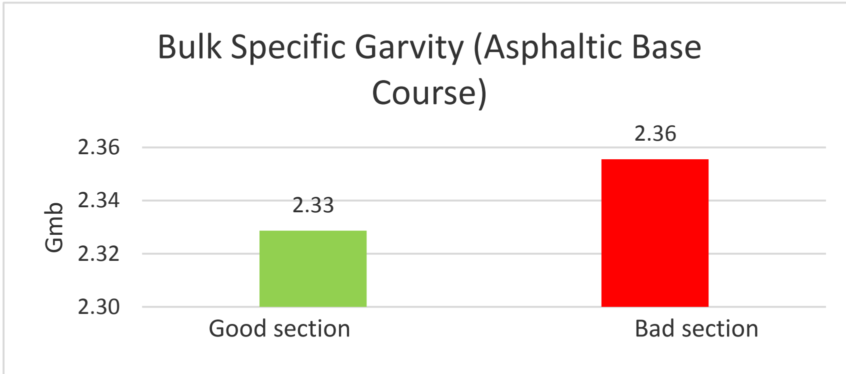

Figure 3.17: Comparative Plot for Gmb for Good and Bad Section

3.5.3 Maximum Theoretical Specific gravity

The Standard Test procedure for the test is given by ASTM 2401. This test is used in calculation of volumetric properties of HMA mixes.

$$\text{Theoretical Maximum Specific Gravity} = G_{mm} = \frac{A}{(A + D - E)}$$

Where:

- A = sample mass in air (g)
- D = mass of flask filled with water (g)
- E = mass of flask and sample filled with water (g)

Table 3.7: Maximum Theoretical Specific Gravity for Good Section

Gmm (WEARING COURSE)				
SAMPLE	WEIGHT IN AIR A	WEIGHT D	WEIGHT E	Gmm
1	1743.4	7451	8483	2.450661
2	1849.3	7451	8542	2.438745
3	1769.5	7451	8490	2.422313
AVG				2.44
Gmm(ASPHALTIC BASE COURSE)				
SAMPLE	WEIGHT IN AIR A	WEIGHT D	WEIGHT E	Gmm
1	1847.3	7451	8550	2.468662
2	1943.4	7452	8611	2.477562
3	1754.9	7453	8489	2.441091
AVG				2.46

Table 3.8: Maximum Theoretical Specific Gravity for Bad Section

Gmm (WEARING COURSE)				
SAMPLE	WEIGHT IN AIR A	WEIGHT D	WEIGHT E	Gmm
1*	1766	7451	8498	2.486189
2*	1876	7451	8576	2.498003
3*	1759	7451	8487	2.512918
AVG				2.49
Gmm(ASPHALTIC BASE COURSE)				
SAMPLE	WEIGHT IN AIR A	WEIGHT D	WEIGHT E	Gmm
1*	1966	7451	8625	2.482323
2*	1947	7452	8612	2.533952
3*	1895	7453	8591	2.503303
AVG				2.51

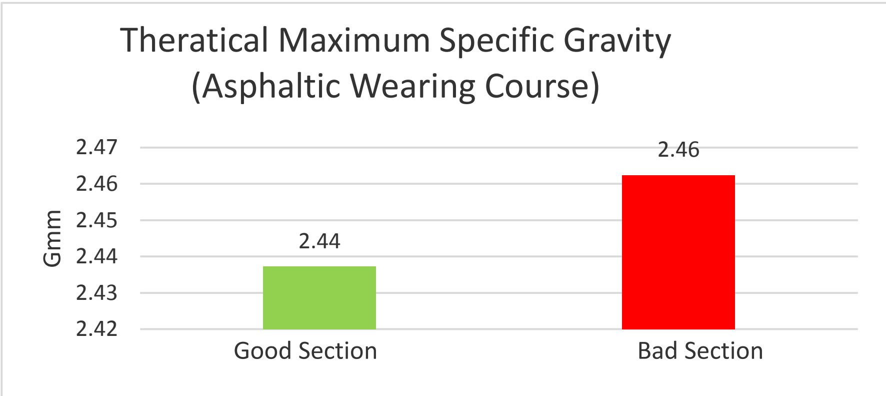

Figure 3.18: Comparative Plot of Gmm for Good and Bad section

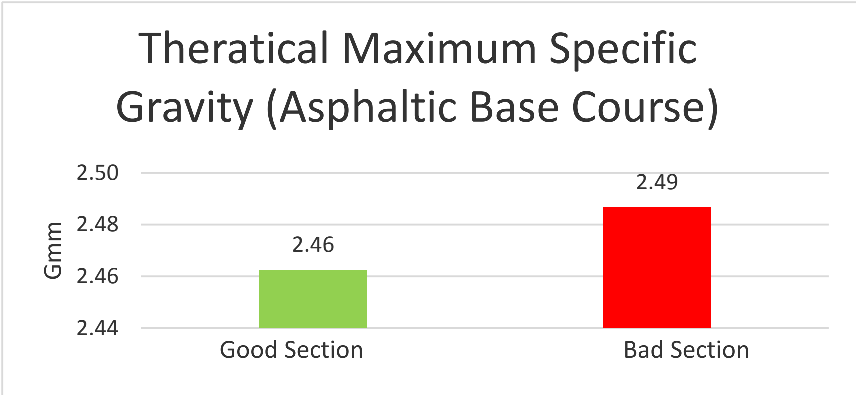

Figure 3.19: Comparative Plot of Gmm for Good and Bad Section

Table 3.9: Air Voids for Good Section

AIRVOIDS (WEARING COURSE)		
Gmb	Gmm	Va
2.32	2.44	4.91
AIRVOIDS (AB COURSE)		
Gmb	Gmm	Va
2.33	2.46	5.28

Table 3.10: Air Voids for Bad Section

AIRVOIDS (WEARING COURSE)		
Gmb	Gmm	Va
2.34	2.49	6.02
AIRVOIDS (AB COURSE)		
Gmb	Gmm	Va
2.36	2.51	5.97

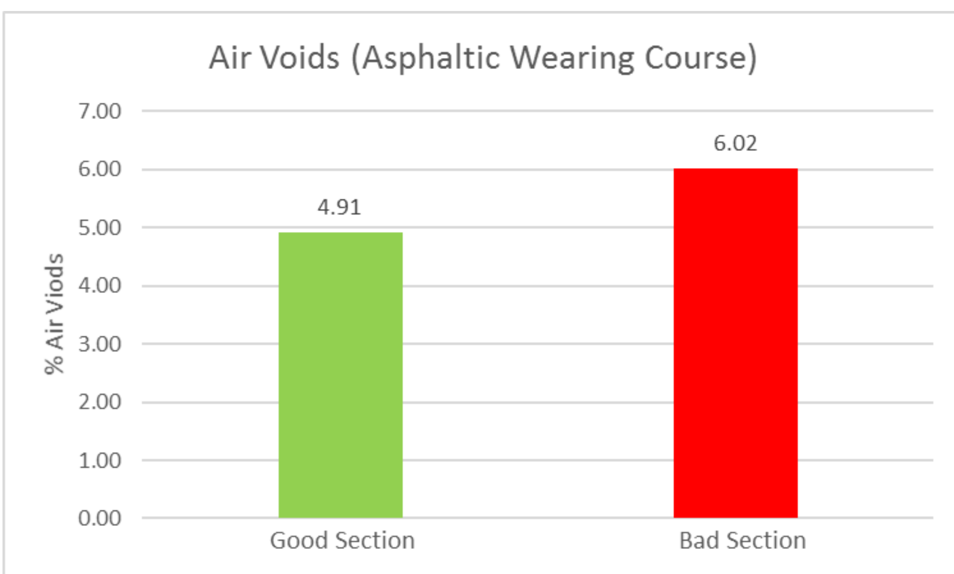

Figure3.20: Comparative Plot of Air Voids for Wearing Course

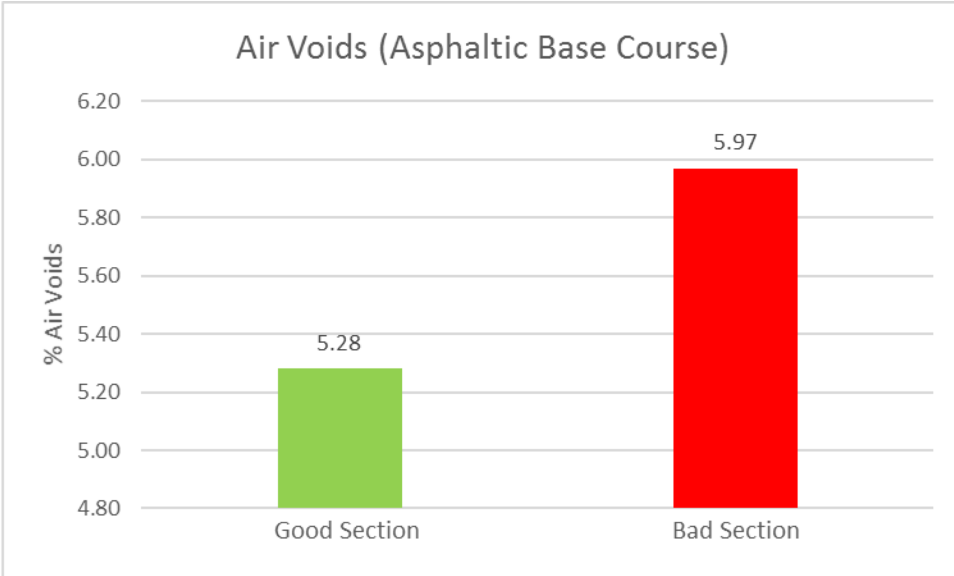

Figure 3.21: Comparative Plot of Air Voids for Asphaltic Base Course

As it can be seen from the calculations, results and graphs above, the asphaltic wearing course and asphaltic base course of bad section consists of more percent air voids than that of good section. These high percent air voids may be attributable to the moisture induced damage in the selected bad section.

3.5.4 Extraction Test by Ignition Method

The bitumen content of the extracted bitumen sample is determined with the help of extraction test method by ignition. The loose mix of HMA is placed in the burner and test is started. The burner burns the asphaltic content and only aggregates remain after the experiment is completed.

Figure 3.22: Extraction by Ignition Method

Table 3.11: Extraction Test for Good Section

EXTRACTION TEST (WEARING COURSE)	
SAMPLE	% bitumen
WM	4.77
WLP	4.57
WRP	4.93
AVG	4.75
EXTRACTION TEST (ASPHALTIC BASE COURSE)	
SAMPLE	%BITUMEN
BM	4.56
BLP	4.74
BRP	4.68
AVG	4.66

Table 3.12: Extraction Test for Bad Section

EXTRACTION TEST (WEARING COURSE)	
SAMPLE	% bitumen
WM*	5.57
WLP*	5.45
WRP*	5.20
AVG	5.40
EXTRACTION TEST (ASPHALTIC BASE COURSE)	
SAMPLE	%BITUMEN
BM*	5.27
BLP*	5.21
BRP*	5.35
AVG	5.27

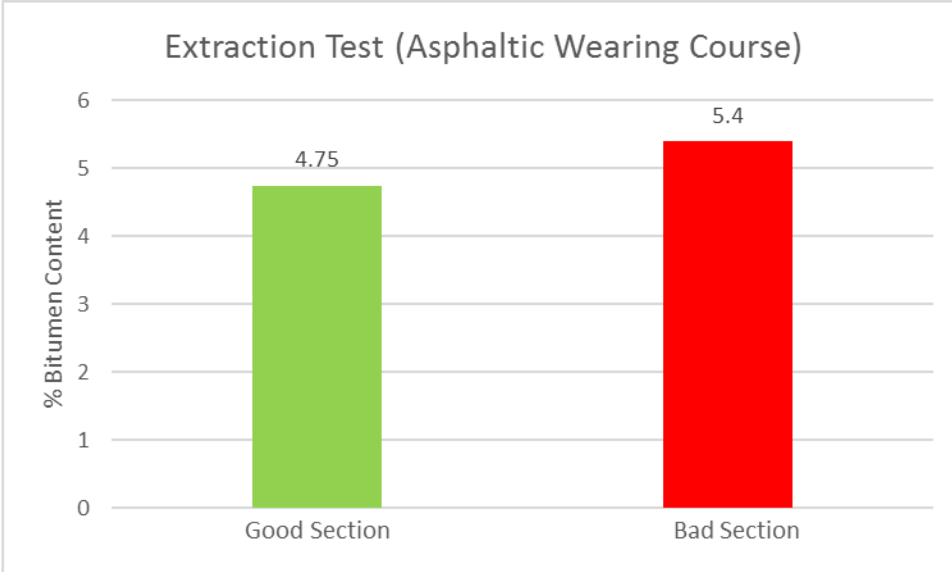

Figure 3.23: Comparative Graph of Extraction Test for Wearing Course

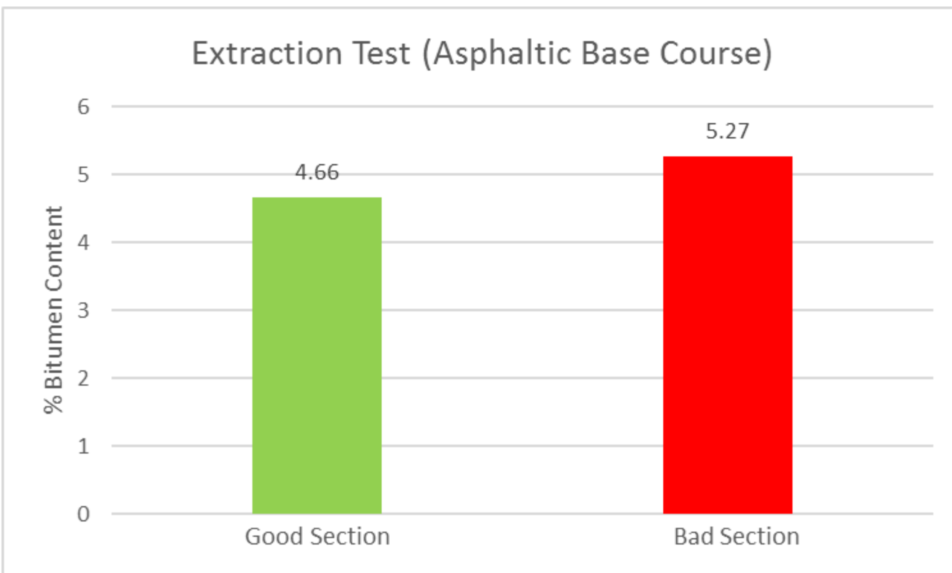

Figure 3.24: Comparative Plot of Extraction Test for Asphaltic Base Course

The results and graphs show that wearing and asphaltic base courses of bad section had high percentage of bitumen content than that of good section.

3.5.5 Gradation of Aggregates

The aggregates remain after the extraction test was analyzed for the gradation. The results of the gradation were plotted and presented by the graphs below. There was very little difference between the gradations of asphaltic wearing and base course of selected good and bad sections.

Table 3.13: Gradation of Wearing Course

Sieve Size	Good	Bad	NHA (A)
N0.200	3.6	2.7	5.5
No.16	21.3	21.3	10
No.8	38.05	34.1	30
No.4	56.6	52.6	50
3/8"	73.2	74.23	70
1/2"	87.3	88.8	82.5
3/4"	99.25	100	100

Table 3.14: Gradation of Asphaltic Base Course

Sieve Size	Good	Bad	NHA (B)
N0.200	2.2	2.5	5
No.16	15	17.02	8.5
No.8	26	25.8	29
No.4	36	38.6	42.5
3/8"	49	51.4	33
3/4"	71.2	73.3	95
1"	90.2	88	100

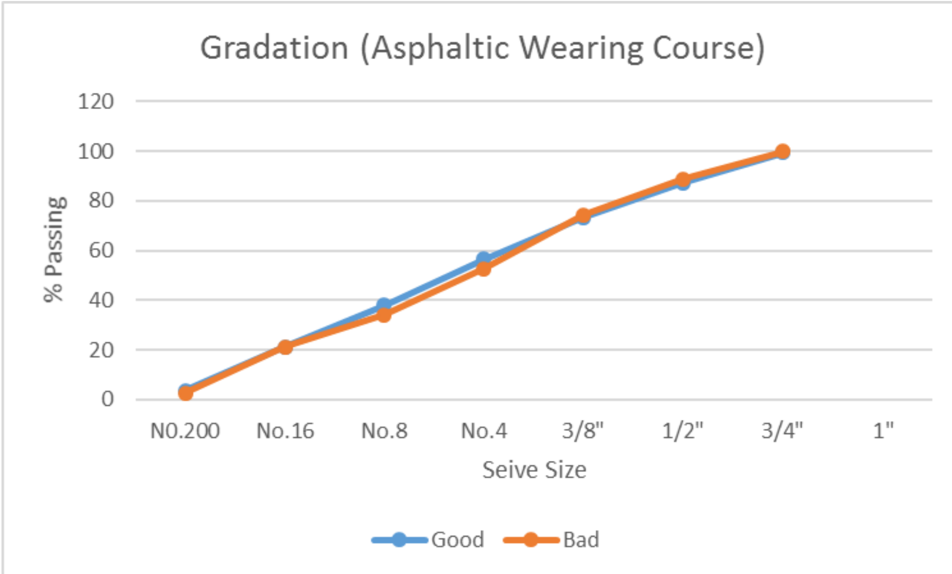

Figure 3.25: Gradation of Wearing Course

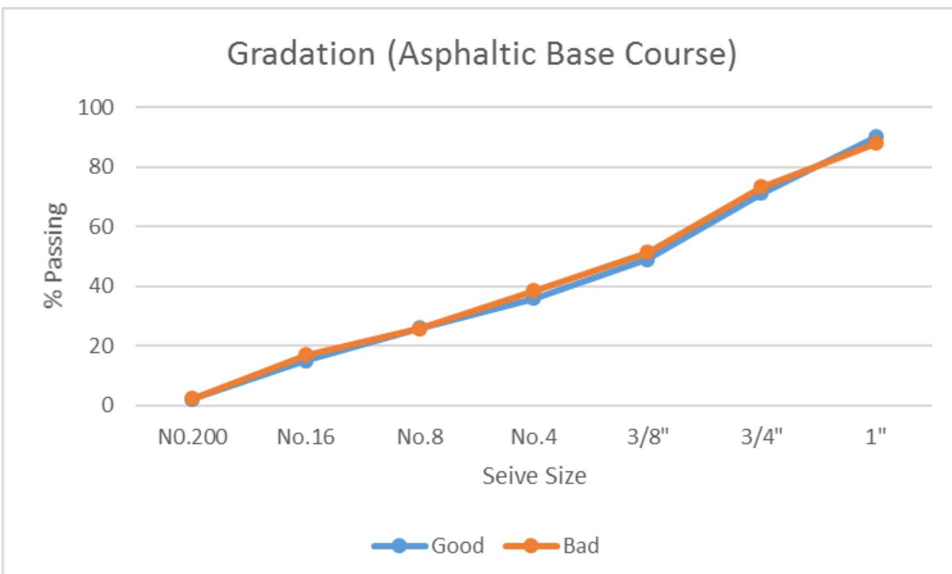

Figure 3.26: Gradation of Asphaltic Base Course

There is no evident difference seen in the gradation of both wearing and asphaltic base course of good and bad sections.

3.5.6 Indirect Tensile Test

ASTM D6931 was followed in order to perform this test. The tensile strength of the HMA samples are calculated with the help of this test. For research thesis this test was used because 20 percent of the value of indirect tensile test is used as an input parameter in the resilient modulus test. The loading rate is 50mm/min at 25°C.

Table 3.15: Indirect Tensile Strength Values for Good Section

INDIRECT TENSILE TEST(WEARING COURSE)	
SAMPLE	FORCE (KN)
WM	8.438
WLP	8.458
WRP	8.6
AVG	8.50
INDIRECT TENSILE TEST(ASPHALTIC BASE COURSE)	
SAMPLE	FORCE (KN)
BM	8.3
BLP	8.217
BRP	8.35
AVG	8.29

Table 3.16: Indirect Tensile Strength Values for Bad Section

INDIRECT TENSILE TEST(WEARING COARSE)	
SAMPLE	FORCE (KN)
WM*	8.3
WLP*	8.52
WRP*	8.625
AVG	8.48
SAMPLE	FORCE (KN)
BM*	8.158
BLP*	8.291
BRP*	8.25
AVG	8.23

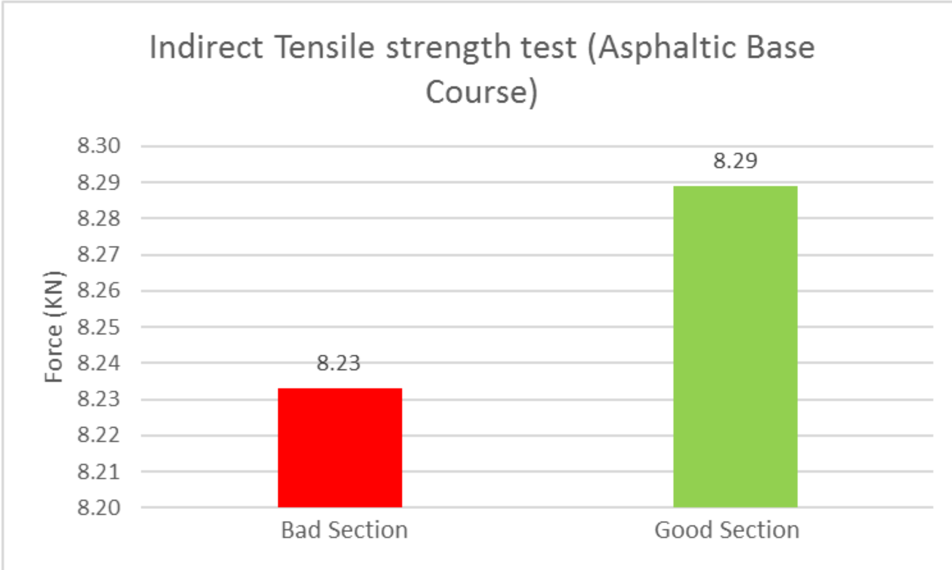

Figure 3.27: Comparative Plot of ITS for Asphaltic Base Course

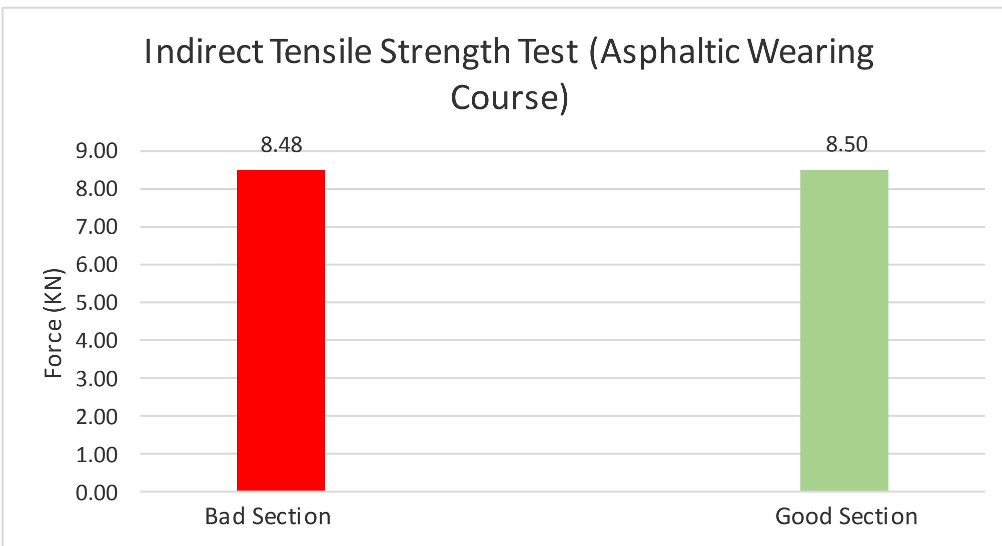

Figure 3.28: Comparative Plot of ITS for Wearing Course

3.5.7 Resilient Modulus Test

This test is performed according to ASTM D4123 in order to check the stiffness of the HMA samples. Modulus of resilience test was performed with the input variable of 20 percent load of indirect tensile test. Figure 3.29 shows the component of jig for resilient modulus testing. LVDTs (linear variable differential transformer) were installed with the help of metallic setup and horizontal displacement of the specimen when load is applied to it was measured with the help of these LVDTs. The sample was then loosely fitted into the jig on the bottom loading platen. The yoke support cross-arm was raised by lifting then turning the support spacers. Top loading platen was then placed and lowered it onto the specimen as shown in Figure 3.30.

The ASTM standard for this Test is D4123. First 100 conditioning pulses were applied to the sample and after that 5 pulses are applied in order to get the resilient modulus value of the sample.

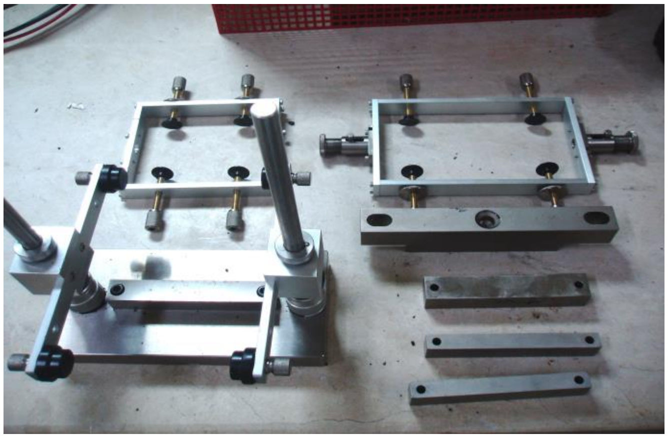

Figure 3.29: Specimen Jig Accessories for Resilient Modulus Test

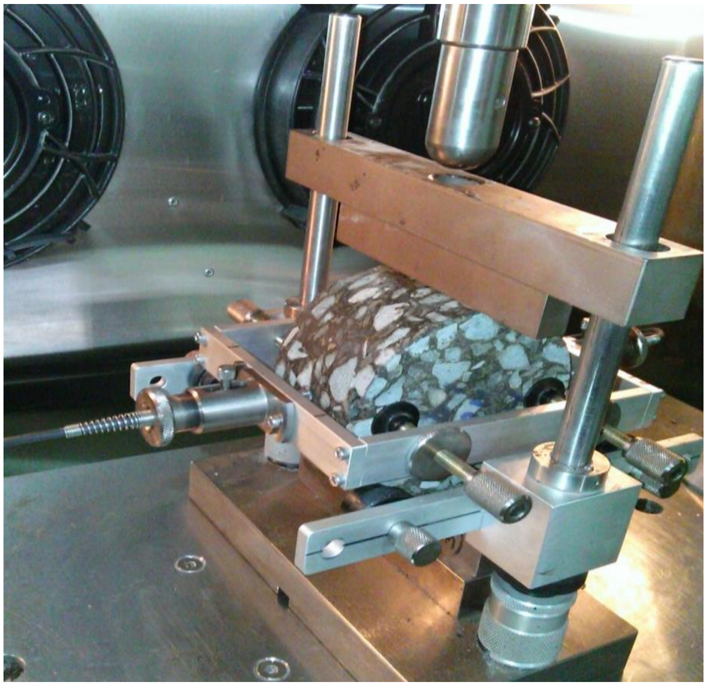

Figure 3.30: Sample in UTM

Table 3.17: Resilient Modulus Test values for Good Section

RESELIENT MODULUS(WEARING COURSE)	
SAMPLE	MR (MPA)
WM	2189
WLP	1856
WRP	1962
AVG	2002.3
RESELIENT MODULUS(BASE COURSE)	
BM	2479
BLP	2186
BRP	2321
AVG	2463.7

Table 3.18: Resilient Modulus Test Values for Bad Section

RESELIENT MODULUS(WEARING COARSE)	
SAMPLE	MR (MPA)
WM*	1517
WLP*	1762
WRP*	1673
AVG	1650
RESELIENT MODULUS(BASE COARSE)	
BM*	2556
BLP*	2478
BRP*	2357
AVG	2328

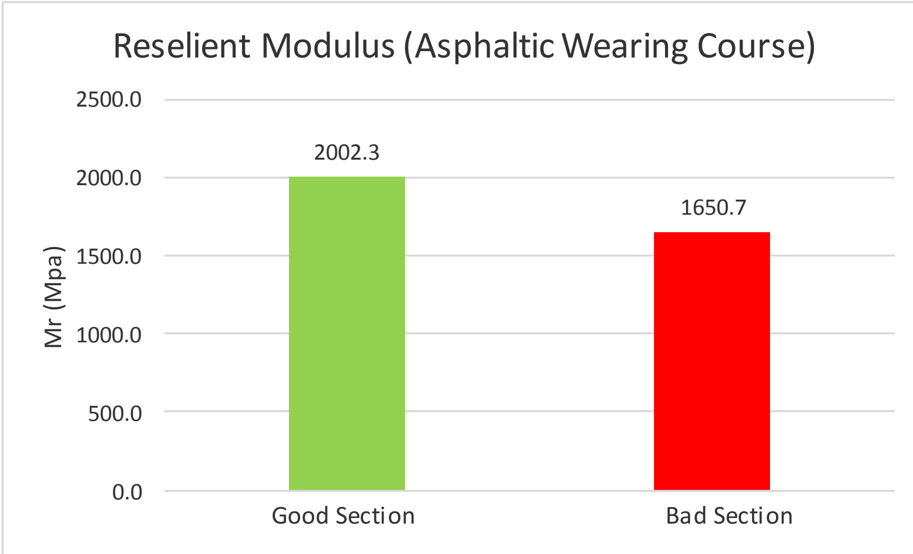

Figure 3.31: Comparative Plot of M_R Values for Asphaltic Wearing Course

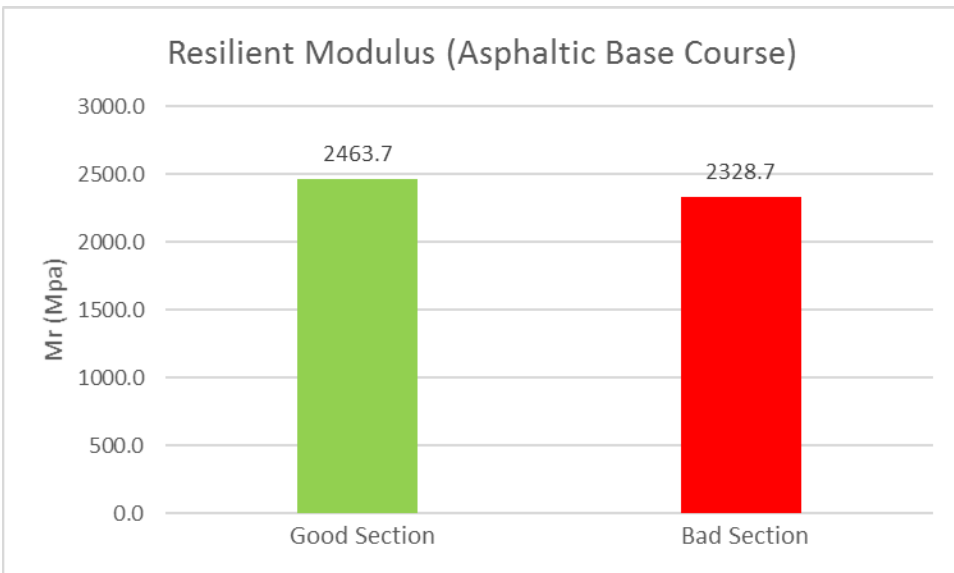

Figure 3.32: Comparative Plot of M_R Values for Asphaltic Base Course

It can be seen from graph that the resilient modulus value of wearing and asphaltic base course of bad section is lower than that of good section. Comparative higher rutting in the selected bad section may be attributed to lower value of M_R.

3.5.8 Moisture Susceptibility Test

This test is performed according to AASHTO T 203 to find out the moisture susceptibility of the HMA samples which can lead to stripping of the pavement. In this test, two set of samples were used. One conditioned and one unconditioned set of the samples were used in this test. The ratio of two tensile strength values gives the TSR (Tensile Strength Ratio) value.

The sample is conditioned by giving them freeze and thaw cycle. First they ae stored with paraffin coating on at -17°C in the freezer for 16 hours. After giving the freeze cycle, the sample is kept in water bath for 24 hours and the temperature of the water bath is set at 60°C. The samples are to be tested at 25°C.

$$TSR = \frac{S_2}{S_1}$$

Where:

TSR = tensile strength ratio

S1 = average tensile strength of unconditioned samples

S2 = average tensile strength of conditioned samples

According to Superpave mix design specification the tensile strength ratio value should be greater than 0.80.

Table 3.19: TSR Values for Good Section

Moisture susceptibility by TSR (Wearing course)			
Trial	S1	S2	TSR
1	971.3	903.29	0.92998
2	986.5	915.45	0.927978
3	954.8	901.6	0.944282
AVG			0.93
Moisture susceptibility by TSR (Asphaltic base course)			
Trial	S1	S2	TSR
1	838.21	743.34	0.886818
2	878.3	796.58	0.906957
3	845.67	748.96	0.885641
AVG			0.89

Table 3.20: TSR Values for Bad Section

Moisture susceptibility by TSR (Wearing course)			
Trial	S1	S2	TSR
1 *	1017.2	493.196	0.484856
2*	958.4	476.8	0.497496
3*	1005.9	654.2	0.650363
AVG			0.54
Moisture susceptibility by TSR (Asphaltic base course)			
Trial	S1	S2	TSR
1 *	996.7	505.77	0.507445
2*	854.78	479.6	0.56108
3*	1024.14	645.9	0.630675
AVG			0.57

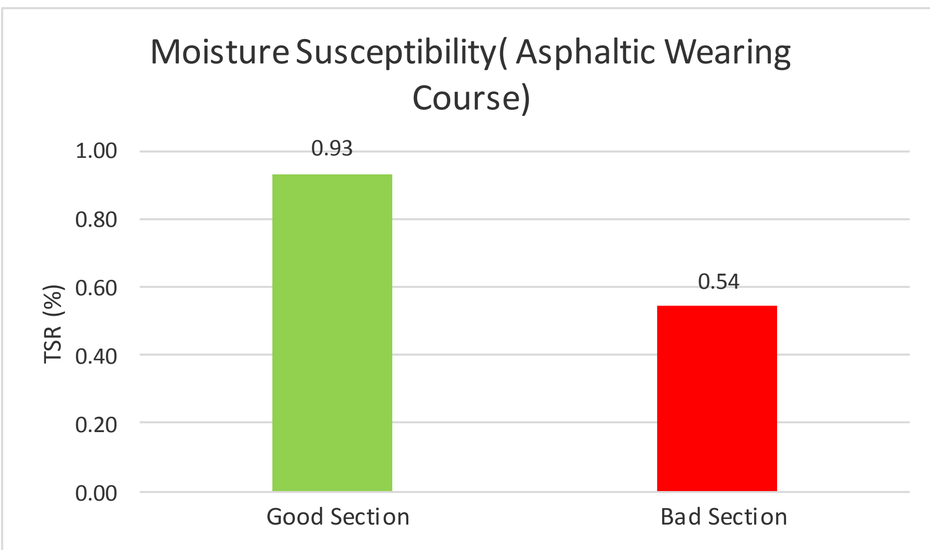

Figure 3.33: Comparative Plot of TSR Values for Wearing Course

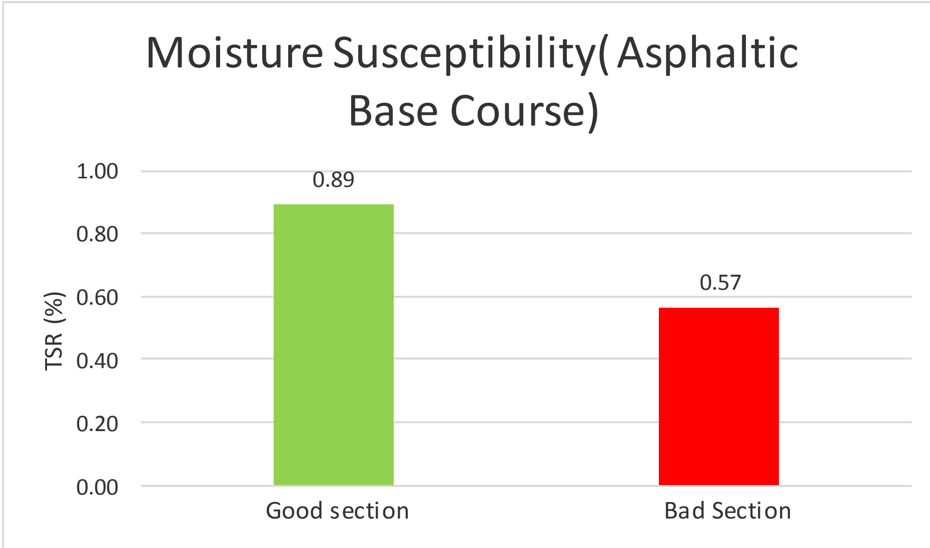

Figure 3.34: Comparative Plot of TSR Values for Asphaltic Base Course

According to the standard if the value of TSR is coming out to be lower than 0.80 then HMA is considered to be moisture susceptible and as it can be seen from the results that the values of TSR for wearing and base course of bad section are lower than 0.80

3.5.9 Indirect Tensile Fatigue Test

The performance testing selected for the research is the indirect tensile fatigue test on cylindrical shaped samples to characterize the different HMA mixes used in the research under repeated load applied with constant load mode. The cylindrical shaped samples prepared in the laboratory are used in the test, cored samples from the field can also be used in the test to give a view of the conditions at site. The cylindrical shaped test samples are subjected to repeated compressive haversine load in the vertical direction. The vertical compressive load produces reasonably uniform tensile stress in the direction which is horizontal to the sample and perpendicular to the load applied on the sample that is why it is known as an indirect tensile test as the tensile load is applied through compressive loading.

The fatigue potential of the sample number of cycles before the sample fractures. The haversine load applied to the sample include a loading time period of 0.1 seconds as well as a rest time period of 0.4 seconds. The testing was performed at 25 °C only and a frequency of 2Hz.

The samples were tested in UTM 25, using the jig assembly shown in Figure 3.7 along with the transducers attached on the diametric plane, at six different stress levels and at least three samples were tested for each level of stress. During the loading process deformation for the first 150 cycles were recorded and the transducer removed after that so that the fractured sample does not harm the sensitive transducers. The deformation reading of the transducers attached to the sample in the jig assembly were used to determine the initial strain at the center of the sample that is the strain developed in the sample at the 100th cycle of loading. The testing is completed once the sample is fractured.

Table 3.21: ITFT for Good Section

Indirect tensile fatigue test (wearing Course)	
Sample	no. of cycle to fatigue failure
WM	4721
WLP	3691
WRP	2691
AVG	3701
Indirect tensile fatigue test (Asphaltic base Course)	
Sample	no. of cycle to fatigue failure
BM	8931
BLP	6351
BRP	5891
AVG	7058

Table 3.22: ITFT for Bad section

Indirect tensile fatigue test (wearing course)	
sample	no. of cycle to fatigue failure
WM*	1561
WLP*	1331
WRP*	1471
AVG	1454
Indirect tensile fatigue test (Asphaltic base course)	
Sample	no. of cycle to fatigue failure
BM*	8161
BLP*	7271
BRP*	5561
AVG	6998

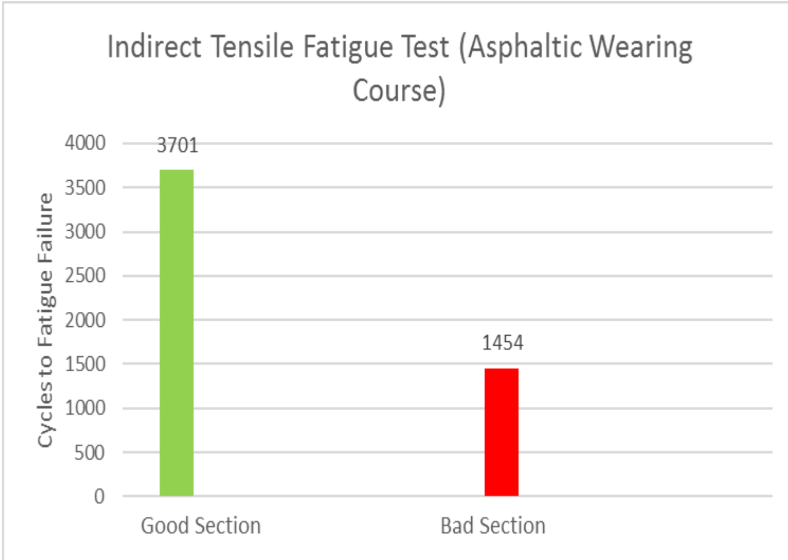

Figure 3.35: Comparative Graph of ITFT for Wearing Course

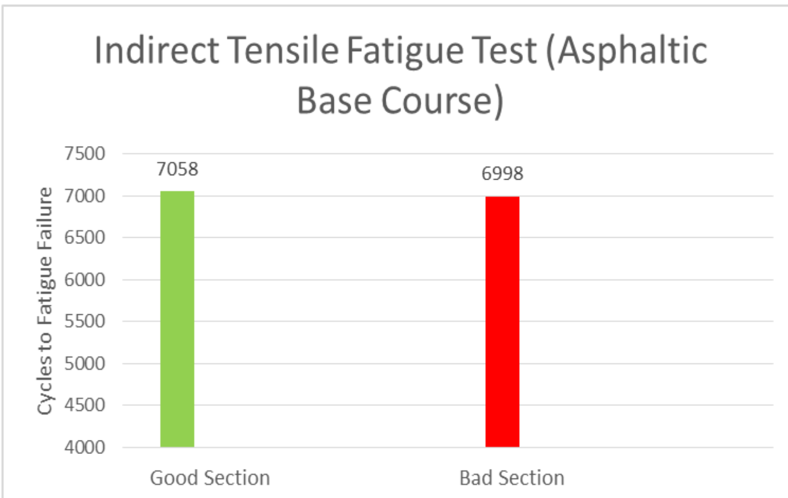

Figure 3.36: Comparative Graph of ITFT for Asphaltic Base Course

The results of ITFT shows that the samples from selected bad section failed after lower number of applied load cycles than that from selected good section.

3.6 SUMMARY

The first part of the chapter discusses the methodology adopted for the research. The second part elaborates all the tests which were carried out during the research. The chapter also contains the results and comparative graphs for all the tests.

Chapter 4

ANALYSIS OF RESULTS

4.1 INTRODUCTION

The statistical analysis method was used for analysis of the results of the experiments and to spot the differences between the results of representative good and bad sections. In this chapter the reasons of any spotted difference between the results of two sections will also be highlighted.

4.2 STATISTICAL ANALYSIS

Statistical analysis is a technique to analyze the results of the experiments. The type of analysis adopted for the research was analysis of Variance also known as ANOVA.

4.2.1 Analysis of Variance (ANOVA)

This technique uses the F test to analyze the values. If F value is coming out to be greater than F critical than the means of the two groups are significantly different but if the F value is less than F critical then the difference will be insignificant.

4.3 ANALYSIS OF THE EXPERIMENTAL RESULTS

4.3.1 Analysis of Marshall Stability

The figure 3.12 show that the stability values of wearing course of good section are greater than that of bad section. The table 4.1 shows the analysis of variance (ANOVA) of Marshall Stability for Asphaltic Wearing Course. As we can see from the table that the F value is less than F critical so it implies that the difference between the means of stability of good and bad section is not significant.

Table 4.1: ANOVA of Stability for Wearing Course

ANOVA (Asphaltic Wearing Course) Stability						
Source	DF	SS	MS	F	P	Fcr
Between Groups	1	0.07	0.07	0.04	0.854	7.71
Within Groups	4	7.07	1.77	Not Significant		
Total	5	7.13				

Figure 3.14 shows the values of the stability for good and bad sections of asphaltic base course. The stability of good section is greater than the bad section as shown by figure 3.14. ANOVA was performed for the Asphaltic base course of representative good and bad section. As we can see in the table 4.2 given below that the F value is less than F critical so it is pertinent to mention that the difference between the means of Marshall Stability for asphaltic base course of good and bad section is not significant.

Table 4.2: ANOVA of Stability for Asphaltic Base Course

ANOVA (Asphaltic Base Course) Stability						
Source	DF	SS	MS	F	P	Fcr
Between Groups	1	0.56	0.56	0.47	0.53	7.71
Within Groups	4	4.77	1.19	Not Significant		
Total	5	5.34				

4.3.2 Analysis of Flow value

The figure 3.13 shows that the flow values of samples from wearing course of bad section are greater than that of good section. This difference can be statistically analyzed. As we can see in table 4.3 that the F value is greater than F critical. In lieu of this it is obvious that the mean of flow values for representative good and bad sections are significantly different from one another. The flow value of the bad sections (wearing course) were greater than that of good section. Hence it can be established as a cause of rutting in the representative bad section.

Table 4.3: ANOVA of Flow Values for Wearing Course

ANOVA (Asphaltic Wearing Course) Flow						
Source	DF	SS	MS	F	P	Fcr
Between Groups	1	4.907	4.907	23.38	0.008	7.71
Within Groups	4	0.839	0.21	Significant		
Total	5	5.746				

Figure 3.15 shows that the difference between the flow values of asphaltic base course of good and bad section is very low with the value for good section slightly on the higher side. However this difference can be statistically analyzed. ANOVA was performed for the flow values of asphaltic base course of representative good and bad sections. It is pertinent to mention that the difference found between the means of good and bad sections was not significant. Table 4.4 shows that the F value is less than F critical which implies the insignificance difference between the means.

Table 4.4: ANOVA of Flow Value for Asphaltic Base Course

ANOVA (Asphaltic Base Coarse) Flow						
Source	DF	SS	MS	F	P	Fcr
Between Groups	1	0.067	0.067	0.21	0.674	7.71
Within Groups	4	1.3	0.325	Not Significant		
Total	5	1.367				

4.3.3 Analysis of Bulk Specific Gravity

ANOVA for bulk specific gravities showed no significant difference between good and bad sections. Table 4.5 shows that the F value is less that F critical value which nullify the significance difference between the means of two sections.

Table 4.5: ANOVA of G_{mb} for Asphaltic Wearing Course

ANOVA (Asphaltic Wearing Course) Bulk Specific Gravity						
Source	DF	SS	MS	F	P	Fcr
Between Groups	1	0.000342	0.000342	0.94	0.387	7.71
Within Groups	4	0.001457	0.000364	Not Significant		
Total	5	0.0018				

For asphaltic base course, the bulk specific gravities values mean for representative good and bad section are not significantly different from one another. The fact that the F value in the table 4.6 is coming out to be less that critical value of F in the same table implies that the means of both good and bad sections are not significantly different from one another.

Table 4.6: ANOVA of G_{mb} for Asphaltic Base Course

ANOVA (Asphaltic Base Course) Bulk Specific Gravity						
Source	DF	SS	MS	F	P	Fcr
Between Groups	1	0.001081	0.001081	2.34	0.2	7.71
Within Groups	4	0.001844	0.000461	Not Significant		
Total	5	0.002925				

4.3.4 Analysis of Maximum Theoretical Specific Gravity

ANOVA was performed to analyze the differences between the means of maximum theoretical specific gravity of representative good and bad sections (Wearing Course) and it was found out that the difference was in significant. The fact that the F value in table 4.7 is less than F critical value strengthens the insignificant variability between the means of two groups.

Table 4.7: ANOVA of Gmm for Wearing Course

ANOVA (Asphaltic Wearing Course) Maximum Theoretical Specific Gravity						
Source	DF	SS	MS	F	P	Fcr
Between Groups	1	0.000947	0.000947	1.47	0.292	7.71
Within Groups	4	0.00258	0.000645	Not Significant		
Total	5	0.003528				

Maximum theoretical specific gravity values for asphaltic base course were analyzed in order to spot significant difference between the means of representative good and bad section but the fact that F value in table 4.8 was less than F critical nullify any chance of significant difference between two groups.

Table 4.8: ANOVA of Gmm for Asphaltic Base Course

ANOVA (Asphaltic Base Course) Maximum Theoretical Specific Gravity						
Source	DF	SS	MS	F	P	Fcr
Between Groups	1	0.00087	0.00087	2.95	0.161	7.71
Within Groups	4	0.00118	0.000295	Not Significant		
Total	5	0.002051				

4.3.5 Analysis of Percent Air Voids

Figure 3.20 shows that the percent air voids in wearing course of bad section was more than that of good section. Statistical analysis was carried out in order to check the significance of difference. The analysis of Variance (ANOVA) when performed for spotting significant difference between the air voids of good and bad sections (wearing course) showed that there was no significant difference. As it is evident from table 4.9 that the F value is less than F critical so hence proved that no significant difference was found between the percent air voids of good and bad sections for asphaltic wearing course.

Table 4.9: ANOVA of PAV for Wearing Course

ANOVA (Asphaltic Wearing Course) PAV						
Source	DF	SS	MS	F	P	Fcr
Between Groups	1	0.18	0.18	0.1	0.764	7.71
Within Groups	4	7.06	1.77	Not Significant		
Total	5	7.24				

Figure 3.21 represents the comparison of the air voids between good and bad section for asphaltic base course. It can be seen that air voids in base course of bad section was higher than that of good section. Percent air voids of asphaltic base course for representative good and bad section were analyzed. It was found that there was no significant different between the means of two groups because the F value in the ANOVA table was coming out to be less than F critical value.

Table 4.10: ANOVA of PAV for Asphaltic Base Course

ANOVA (Asphaltic Base Course) PAV						
Source	DF	SS	MS	F	P	Fcr
Between Groups	1	0.04	0.04	0.02	0.883	7.71
Within Groups	4	6.89	1.72	Not Significant		
Total	5	6.93				

4.3.6 Analysis of Percent Bitumen Extraction

Figure 3.23 shows the comparative plot of percent bitumen content for wearing course of good and bad section. The bitumen content in the wearing course of bad section was higher as compared to that of good section. ANOVA shows that the difference between the extracted bitumen content for wearing course of representative good and bad section is significant. The results showed that the bitumen content of bad section for wearing course was very high. This adds up to the belief that the fatigue characteristic and stiffness of the bad section is highly affected by significantly low amount of bitumen content. The ANOVA table 4.11 shows that the F value is greater than F critical value which concretes the fact that the bitumen content in the bad section is significantly low than bitumen content in the good section.

Table 4.11: ANOVA of Percent Bitumen Content for Wearing Course

ANOVA (Asphaltic Wearing Course) % Bitumen Content						
Source	DF	SS	MS	F	P	Fcr
Between Groups	1	3.8065	3.8065	133.38	0	7.71
Within Groups	4	0.1142	0.0285	Significant		
Total	5	3.9206				

Figure 3.24 shows the comparative plot of percent bitumen content for asphaltic base course of good and bad section. The bitumen content in the asphaltic base course of bad section was higher as compared to that of good section. ANOVA shows that the difference between the extracted bitumen content for asphaltic base course of representative good and bad section is significant. The results showed that the bitumen content of bad section for asphaltic base course was very high. This adds up to the belief

that the fatigue and rutting characteristics of the bad section is highly affected by significantly high amount of bitumen content. The ANOVA table 4.12 shows that the F value is greater than F critical value which concretes the fact that the bitumen content in the bad section is significantly high than bitumen content in the good section.

Table 4.12: ANOVA of Percent Bitumen Content for Asphaltic Base Course

ANOVA (Asphaltic Base Coarse) % Bitumen Content						
Source	DF	SS	MS	F	P	Fcr
Between Groups	1	1.144	1.144	25.76	0.007	7.71
Within Groups	4	0.1777	0.0444	Significant		
Total	5	1.3217				

4.3.7 Analysis of Indirect Tensile strength Values

The Analysis of Variance (ANOVA) was performed to find out the differences between the means of representative good and bad sections for asphaltic wearing course. It was found out the means of two groups were coming out to be insignificant because the F value was coming ou to be less than F critical value in the table.

Table 4.13: ANOVA of ITS Values for Wearing Course

ANOVA (Asphaltic Wearing Course) Indirect Tensile Strength						
Source	DF	SS	MS	F	P	Fcr
Between Groups	1	0.0004	0.0004	0.02	0.883	7.71
Within Groups	4	0.0706	0.0177	Not Significant		
Total	5	0.0711				

The Analysis of Variance (ANOVA) was performed to find out the differences between the means of representative good and bad sections for asphaltic base course. It was found that there was no significant difference between the means of two groups because the F value in the table 4.14 is less than F critical value.

Table 4.14: ANOVA of ITS for Asphaltic Base Course

ANOVA (Asphaltic Base Course) Indirect Tensile Strength						
Source	DF	SS	MS	F	P	Fcr
Between Groups	1	0.0047	0.0047	1.03	0.368	7.71
Within Groups	4	0.0183	0.00458	Not Significant		
Total	5	0.02301				

4.3.8 Analysis of Resilient Modulus Values

Comparative plot of resilient modulus values of good and bad section for wearing course can be seen in figure 3.31. The stiffness or resilient modulus value of bad section is lower than that of good section. Analysis of Variance when performed on the resilient modulus values of good and bad section for the asphaltic wearing course, it was found out that the difference was significant because the F value in the table below was greater than the F critical value. It was observed by the result that the resilient modulus values for bad section was coming out to be comparatively low as mentioned in the previous chapter. It shows that the material has low stiffness and was prone to the rutting in the asphaltic wearing course.

Table 4.15: ANOVA of M_R Values for Wearing Course

ANOVA (Asphaltic Wearing Course) Resilient Modulus						
Source	DF	SS	MS	F	P	Fcr
Between Groups	1	81999460	81999460	10.69	0.031	7.71
Within Groups	4	30669371	7667343	Significant		
Total	5	1.13E+08				

Comparative plot of resilient modulus values of good and bad section for asphaltic base course can be seen in figure 3.32. The stiffness or resilient modulus value of bad section is lower than that of good section. ANOVA was performed for spotting the significant difference between the means of the M_R values of asphaltic base course and it was found that the difference between them was insignificant due to the fact that the F value in the table 4.16 was less than F critical value.

Table 4.16: ANOVA of M_R Values for Asphaltic Base Course

ANOVA (Asphaltic Base Course) Resilient Modulus						
Source	DF	SS	MS	F	P	Fcr
Between Groups	1	735000	735000	0.71	0.447	7.71
Within Groups	4	4150627	1037657	Not Significant		
Total	5	4885627				

4.3.9 Analysis of Moisture Susceptibility

The moisture susceptibility test was performed according to AASHTO T 283 in which tensile strength ratio (TSR) is used for determination of susceptibility of a sample to moisture induced damage. According to the standard if the TSR value is above 0.85 then the mix is not considered to be susceptible to moisture induced damage. But if the value is less than 0.85 then the HMA is considered susceptible to moisture damage. In our case, the TSR value for wearing and asphaltic base course of good section is above 0.85 so they are not susceptible to the moisture induced damage (figure3.33, figure 3.34). However, the wearing and base course of bad section had TSR values below than 0.85 which makes them moisture susceptible.

4.3.10 Analysis of Indirect Tensile Fatigue Values

The comparative plot in figure 3.35 shows that the wearing course of bad section failed after a low number of load cycles as compared to wearing course of good section. The ANOVA when performed for the fatigue resistance of good and bad section for wearing course, it showed that the difference between the means of the two groups was coming out to be significant. The F value in the ANOVA table was greater than F critical which also concretes the fact that the difference between the means of two groups was significant.

This shows that the representative bad section's wearing course had less strength against the fatigue failure as compared to that of good section. The samples from bad sample failed after very less no of cycles before fatigue failure. The significantly low fatigue resistance was the main cause of fatigue cracking in the bad section.

Table 4.17: ANOVA of ITFT Values for Wearing Course

ANOVA (Asphaltic Wearing Course) Indirect Tensile Fatigue Test						
Source	DF	SS	MS	F	P	Fcr
Between Groups	1	7571267	7571267	14.51	0.019	7.71
Within Groups	4	2087467	521867	Significant		
Total	5	9658733				

The comparative plot in figure 3.36 shows that the asphaltic base course of bad section failed after a low number of load cycles as compared to asphaltic base course of good section. The analysis of variance was carried out for spotting the difference between the means of indirect tensile fatigue test values of representative good and bad section for asphaltic base course. It was found out that the difference between the means of two groups was not significant. The table below also show that the F value is less than F critical value which concretes the fact that the different between the mean of two groups is not significant.

Table 4.18: ANOVA of ITFT Values for Asphaltic Base Course

ANOVA (Asphaltic Base Course) Indirect Tensile Fatigue Test						
Source	DF	SS	MS	F	P	Fcr
Between Groups	1	5400	5400	0	0.96	7.71
Within Groups	4	8861933	2215483	Not Significant		
Total	5	8867333				

4.3.11 Analysis of Layer Thicknesses

The layer thickness of good and bad section was shown in table 3.1 and table 3.2 ANOVA was done for the layer thicknesses of good and bad section (Wearing Course). The analysis showed that there is significant difference between the means of two group because the F value was greater than F critical value as shown in table 4.19. The significant difference between the layer thicknesses was due to the fact that the wearing course of bad section was affected by rutting.

Table 4.19: ANOVA of Layer Thicknesses for Wearing Course

ANOVA (Asphaltic Wearing Course) Layer Thicknesses						
Source	DF	SS	MS	F	P	Fcr
Between Groups	1	0.4267	0.4267	37.58	0	4.03
Within Groups	52	0.5904	0.0114	Significant		
Total	53	1.017				

ANOVA was carried out in order to spot the significant difference between the means of layer thicknesses of good and bad sections (Asphaltic Base Course). The F value was less than F critical which shows that difference between the means of two groups is not significant.

ANOVA (Asphaltic Base Coarse) Layer Thicknesses						
Source	DF	SS	MS	F	P	Fcr
Between Groups	1	0.0741	0.0741	3.95	0.052	4.03
Within Groups	52	0.9763	0.0188	Not Significant		
Total	53	1.0504				

4.4 SUMMARY

In this chapter detailed statistical analysis of the experimental results were shown. The results of the experiments were analyzed and were presented and explained with the help of ANOVA tables. The reasons of the significant differences between the means of good and bad sections for different test results were discussed in detail.

Chapter 5

CONCLUSIONS AND RECOMMENDATIONS

5.1 SUMMARY

The research was carried out in order to spot the differences in the material properties and performance of representative good and bad sections. First of all the representative good and bad sections were selected by carrying out distress survey. After the selection of representative good and bad sections, the cores from each were acquired for the lab experimentation. In laboratory the cores obtained from each section was subjected to stability and flow test, Gmm, Gmb, Extraction, and Gradation and the layer thicknesses of the cores from each representative sections were also documented. In addition to conventional testing, performance testing like resilient modulus test, moisture susceptibility and indirect tensile fatigue test was also carried out on the cores. The results of the experiments were documented and were statistically analyzed in MINITAB-15 software.

5.2 CONCLUSIONS

Based on the results obtained from the testing of cores from both representative good and bad section and analysis of experimental results, the following conclusions have been drawn

1. The layer Thicknesses were analyzed and it was found out that the layer thickness of wearing course of bad section was comparatively less than layer thickness of wearing course of good section. This may be attributed to the rutting observed in the wearing course of the bad section.
2. The Stability and flow test results when analyzed, it was found out that there was no significant difference between the Stability values of wearing and asphaltic base course of representative good and bad section. However, the flow value for wearing course of good and bad section were significantly different and the plastic flow in the wearing course of bad section was more than that of good section.

3. The gradation of wearing course and asphaltic base course was almost same for both representative good and bad sections.
4. The percent extracted bitumen content of bad section was comparatively high than that of good section for wearing course. The high amount of the bitumen content in the wearing course of the selected bad section is likely to cause comparatively higher plastic flow, poor resistance against rutting and fatigue cracking.
5. A significant difference was observed for the resilient modulus values between the wearing course of good and bad section. It was noted that the wearing course of the bad section had low stiffness than that of good section. Comparatively higher rutting in bad section may be attributable to their lower stiffness (M_R) value.
6. The wearing course of selected good section was found to have better resistance against moisture induced damage as compared to that of selected bad section. The high moisture damaging probability in wearing course of selected bad section may be attributed to high percentage of air voids encountered.
7. The wearing course of selected bad section was susceptible to comparatively more fatigue cracking because of low fatigue resistance attributable to low stiffness.

5.3 FUTURE WORK AND RECOMMENDATIONS

1. It is recommended that the field construction should be according to the top standards and it should be made sure that the design is strictly followed.
2. To ensure fatigue cracking and rut resistant pavement, appropriate bitumen content and stiffness of HMA should be taken into account during the preparation of mix through quality job mix formula (JMF) and these parameters should be crosschecked during construction.
3. It should be made sure that excessive air voids should be removed to avoid moisture susceptibly by achieving the required compaction.
4. Similar study can be carried out on pavement with the help of other performance tests like Hamburg wheel tracker and dynamic modulus.

5. To ensure the consistency of JMF and good performance of the roads, in addition to onsite collected cores, lab prepared samples should also be used for performance testing

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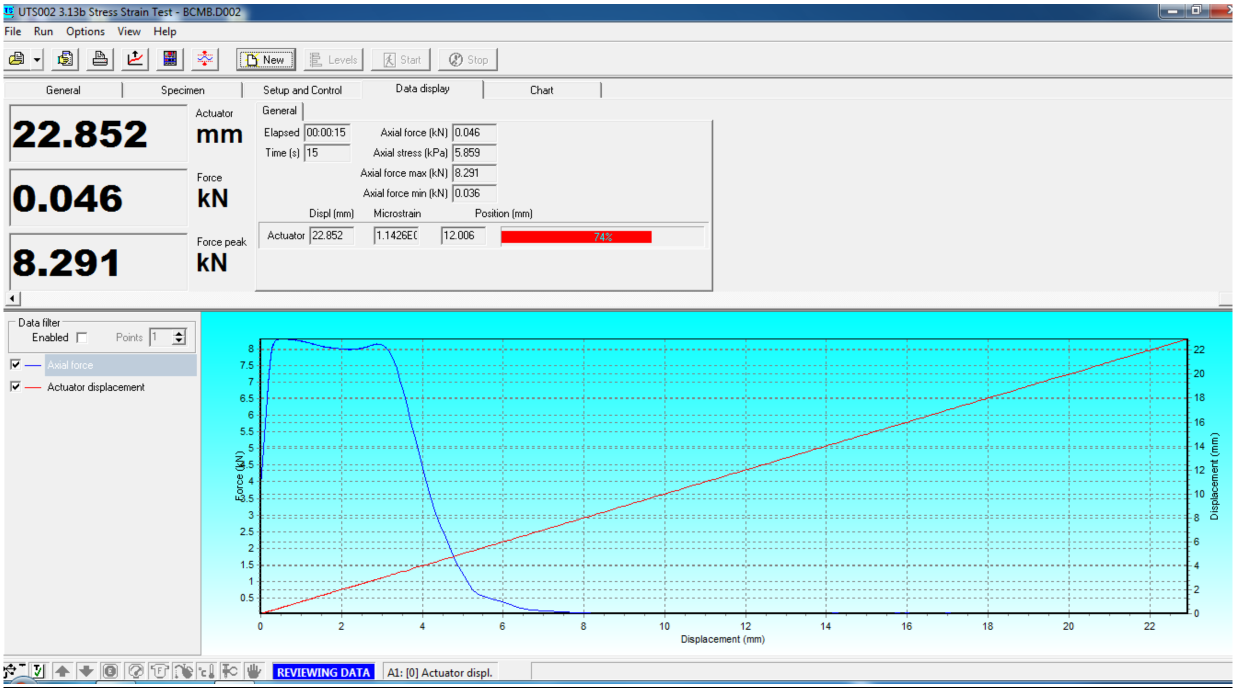
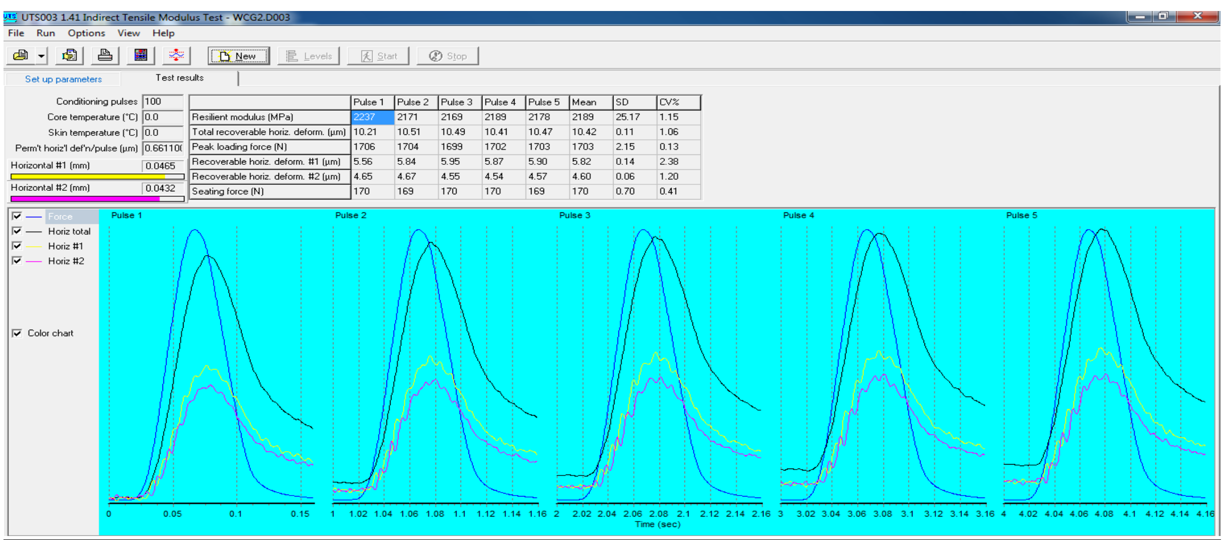
APPENDICES

APPENDIX: I **UTM -25 TEST RESULTS**

Indirect Tensile Test



Resilient Modulus Test



	Pulse 1	Pulse 2	Pulse 3	Pulse 4	Pulse 5	Mean	SD	CV%
Resilient modulus (MPa)	2237	2171	2169	2189	2178	2189	25.17	1.15
Total recoverable horiz. deform. (µm)	10.21	10.51	10.49	10.41	10.47	10.42	0.11	1.06
Peak loading force (N)	1706	1704	1699	1702	1703	1703	2.15	0.13
Recoverable horiz. deform. #1 (µm)	5.56	5.84	5.95	5.87	5.90	5.82	0.14	2.38
Recoverable horiz. deform. #2 (µm)	4.65	4.67	4.55	4.54	4.57	4.60	0.06	1.20
Seating force (N)	170	169	170	170	169	170	0.70	0.41

Indirect Tensile Fatigue Test